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Performance of the Alaska Highway Beaver Creek Area

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Yukon Highways and Public Works

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February 16, 2005.

Robin Walsh P.Eng.
Director, Transportation Engineering
Yukon Highways and Public Works

Re: Research on the performance of the Beaver Creek section of the Alaska Highway

Dear Mr. Walsh:

I am pleased to submit the following final report for your consideration. This project involved a detailed literature review, an analysis of the thermal regime in the Alaska Highway and the development of adaptation scenarios for Yukon highways.

As you know, the development of northern natural resources will involve the construction of new transportation infrastructures in the future decades. During that same period, significant increase in ground temperature is likely to occur causing important damage to existing infrastructures.

This project has proven that the Alaska Highway in the Beaver Creek area constitutes an excellent field laboratory to study permafrost degradation and its impact on transportation infrastructures. Yukon Highways and Public Works, Public Works Canada and the Federal Highway Administration should take advantage of this unique situation to keep improving the current knowledge on permafrost degradation and its impact on transportation infrastructures and begin preparing adaptation scenarios.

Don’t hesitate to contact me for additional information on the report.

Sincerely,

Guy Doré, Ph.D., ing.
Associate Professor
Département de génie civil
Université Laval

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1 Introduction

The construction of transportation infrastructures in permafrost conditions affects the thermal regime of frozen ground and may cause deterioration of the permafrost. If the pavement foundation is constituted of ice-rich permafrost, the structural and the functional capacity of the infrastructure will be affected. Moreover, pavements that have always been stable over permafrost now begin to become unstable as a result of the observed trend toward a global warming of the planet. This is becoming an important engineering problem for northern transportation infrastructures. Several techniques have been developed and experimented to protect pavements from permafrost degradation. However, few of these techniques have been widely adopted in current construction practice.

This project is intended to provide guidance to Yukon Highways and Public Works and other interested Canadian highway agencies on possible adaptation scenarios.

The project objectives was to propose and document cost-effective adaptation scenarios for Alaska Highway as well as other Yukon highways to permafrost degradation resulting from global warming

2 Literature review

Several researchers have studied the behaviour and the performance problems of transportation infrastructures built on unstable permafrost. Several solutions have been proposed and experimented to mitigate the problems. Few have been implemented at a large scale level in road and airstrip construction practice.

The specific objectives of this research activity was to document deterioration mechanisms affecting pavements built over unstable permafrost and to identify potential cost-effective solutions for the typical problems encountered in the Beaver Creek area. In order to achieve these objectives, relevant literature dealing with pavement deterioration and adaptation in degrading permafrost conditions has been reviewed. Potential cost effective solutions are identified and documented in this section and are further discussed in section 3 of the report.

INTRODUCTION

According to the 2004 Cold Regions Conference, there is 1.2 billion people living in cold regions and global warming has become an important issue to the different infrastructures throughout the world such as buildings, roads and airfields due to the thawing of permafrost. Permafrost is a thermal condition and its formation, persistence or disappearance is highly dependent on climate. Its thickness, distribution or temperature respond to natural environmental changes and anthropogenic disturbances that cause an alteration to the ground thermal regime (Geological Survey of Canada, 2003). The presence of modifiers can determine also the permafrost existence and it can lead to the aggradation or degradation of permafrost. The modifiers such as snow or vegetation cover can change the stable thermal regime. Not all the permafrost existence today is in equilibrium with the present climate (Shur, class handouts, 2003).

The report will identify and describe problems related to thawing permafrost under transportation infrastructures like access roads and airfields. In order to understand the problems with thawing permafrost, an introduction that covers the influence of global warming and the economic impacts on infrastructures will be done. According to the first topic, the paper is also proposing a review of the permafrost degradation processes and its geotechnical impacts such as loss of bearing capacity, thaw settlement, thaw consolidation, thermal regime, profile distorsion and drainage. It also includes a review of technical and operational solutions to the problem in order to recommend realistic adaptation scenarios that will assure safe transportation services to the infrastructure users for the next twenty-five years. Three different kinds of applicable solutions will be discussed to mitigate the observed or expected problems: methods preventing heat intake underneath the embankment, methods allowing extracting heat from underneath the embankment and methods aiming at the reinforcement of the embankment. Finally, feasibility and cost effectiveness of these approaches will be discussed.

To achieve the present state of the art, a review of different documents has been done. Technical papers from ASCE monographs, Permafrost Conferences, Cold Regions Conferences, Geological Survey of Canada, the Alaska Department of Transportation and Public Facilitities (ADOTPF) and Canadian Geotechnical Journal were consulted. Some reliable authors who have written reference books about permafrost were also consulted such as Dysli, Ladanyi and Andersland. A book on permafrost published by the National Research Council of Canada (1981) and edited by G.H. Johnston was also used. Finally, interview transcripts are available on Appendix A at the end of the present paper. Interviews were conducted with several experts in arctic engineering such as Billy Connor

(ADOTPF), Douglas Goering (University of Alaska – Fairbanks or UAF), Yuri Shur (UAF) and John Zarling (UAF).

IDENTIFICATION AND DESCRIPTION OF PROBLEMS RELATED TO PERMAFROST THAWING DEGRADATION

a) Global warming

Since last glacial period in Wisconsinien (6000 years ago), the northern hemisphere is now free of persisting ice and a post-glaciation warming is now prevailing as a part of the glaciation cycles (Bourque, 2001). The question which should be raised now is to verify if the climatic warming is a consequence of the post-glacial period or if it is a climatic warming caused by anthropogenic activities. Many experts have detected a fulgurating trend in the last three decades that seems not to be linked with to the post-glaciation.

The global warming can have a great influence on the existence of permafrost. Air temperatures and snow cover especially influence the temperature of permafrost. Changes in snow cover thickness will reduce the reflective effect of the snow and will increase the absorption of solar radiation into darker surfaces. If the snow cover increases (providing insulation) with no change in air temperature, the permafrost will warm. On the other hand, if there is an increase in the air temperatures and if snow cover remains constant, then the permafrost warming will be magnified (Cole, Colonell, Esch, 1999). Smith (1975) and Zhang & al. (1997) (cited in Burn's paper, 1998) demonstrated that warming of permafrost could occur as a result of warmer summers and/or warmer winters. Winters with thicker or more persistent snow cover could also have warmed the permafrost. Burn (1998) concluded that increases in permafrost temperatures are not necessarily associated with the increase of mean annual air temperature (MAAT) or the depth of the active layer, and vice-versa.

For example, in areas of discontinuous permafrost, permafrost can be reached in Alaska at a depth of about one to fifty meters in the ground. Ice or frozen ice lenses in ice-rich permafrost can occur in the top of permafrost, but the ice can be deeper (Cole, Colonell, Esch, 1999). A layer of vegetation on the ground, especially peat, serves as an insulation layer during the summer which prevents it from extended thawing of permafrost (Cole, Colonell, Esch, 1999). A degradation (by forest fires) or a removal of the vegetation layer can lead to a degradation of the permafrost. However, natural thawing of the permafrost layer following disturbance is an extremely slow process, often requiring decades to complete unless thawing is carried out during construction (Cole, Colonell, Esch, 1999).

According to Geological Survey of Canada, general circulation models predict that, for a doubling of atmospheric concentrations of carbon dioxide due to anthropogenic sources, MAAT may rise up to several degrees over much of the Arctic. In the discontinuous permafrost region, where the ground temperatures are within 1-2 degrees of melting, permafrost will ultimately disappear as a result of ground thermal changes associated with global warming climate. Based on Smith and Burgess experiments (1999), the area of the northern hemisphere occupied by permafrost could be reduced by 12-22% of its current extent and Canadian permafrost could eventually disappear from half of the present-day permafrost region under climate warming.

As it is possible to see in Canada (see figures 2.1 and 2.2), the highest impacts of a global climate warming will occur over the areas where the permafrost is discontinuous (Geological Survey of Canada, 2003). As a result of the global warming, the southern boundary of discontinuous permafrost in northwest Canada will move northwards. The thermal response of the permafrost warming will occur in the discontinuous permafrost. Some areas have already a permafrost temperature closed to 0^{o}C and permafrost is already in a state closed to melting (Ladanyi, 1995 cited by Instanes, 2003). Further warming may be extremely serious causing damage to infrastructures and even possible threat to human lives. However, the impacts of global warming in continuous permafrost will not be an immediate issue since there is an important permafrost thickness that could persist for centuries.

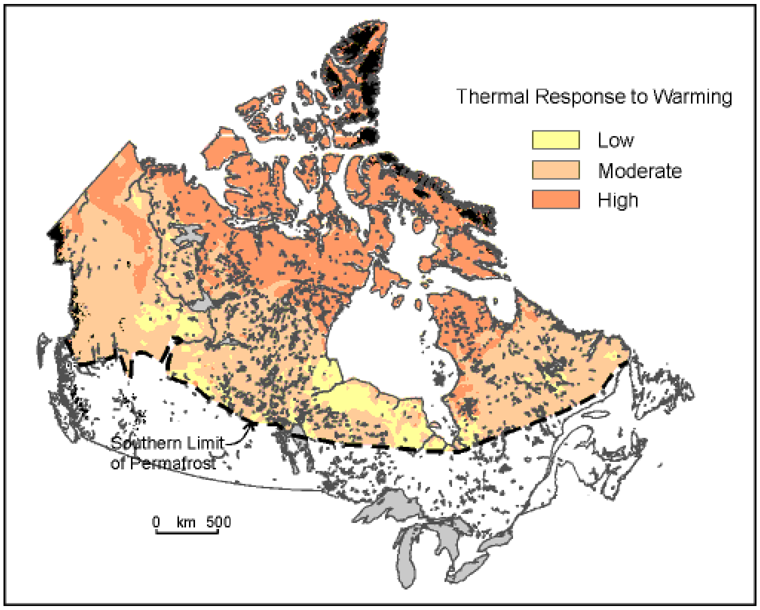


Figure 2-1: Canadian thermal response to warming (Geological Survey of Canada, 2003)

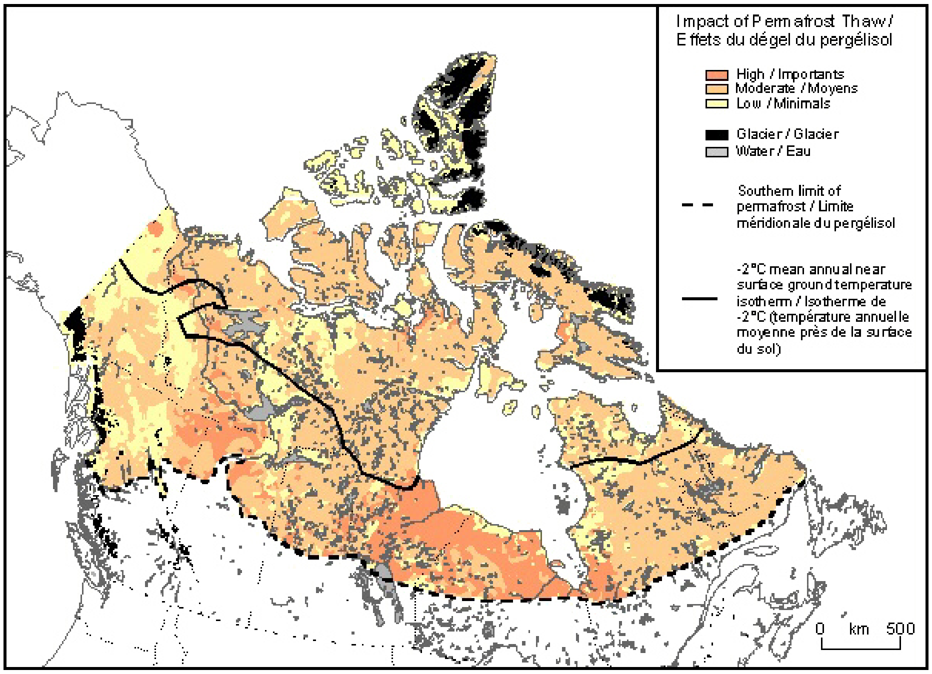


Figure 2-2: Canadian impacts of thawing permafrost (Geological Survey of Canada, 2003)

According to Burn (1998), field evidences from the continuous permafrost indicate that the considerable persistence of permafrost following climate change is due to the huge latent heat contained in ground ice. Global warming impacts may increase the active layer thickness and the permafrost temperature, but the most rapid response will still be changes in surface conditions. Many experts throughout the world (Osterkamp & Romanovsky (1999) and Khrustalev (2000, 2001) cited in Instanes, 2003) have already found indication of climate changes such as increased of active layer, warming permafrost, increased mass movement, thawing of ground ice, coastal erosion and damage to infrastructures. Finally, it is possible to conclude that climate scenarios do not pose immediate threat to infrastructures in continuous permafrost compared to discontinuous permafrost.

Another example of the global climate warming influence is the Hudson Bay slope in Northern Quebec (ex. Kuujuraapik and Inukjuak). During the second half of 20th century, these areas had a relatively stable climate with a slight increase of the summer temperatures. As a result, a regular and slow degradation of permafrost has been observed especially in peat bogs and along littoral. During that period, the Ungava Bay slope experienced a slight cooling. Between 1995 and 2002, the entire Nunavik territory experienced a clear climatic change trend : a rising warming now prevails (see figures 2.1 and 2.2). This warming is beginning to have significant impacts on infrastructures and it has already involved very expensive maintenance operations.

In addition, Nidowicz and Shur (1998) has also demonstrated how the global warming could affect the state of Alaska by calculating the ratio of absolute value of freezing index to thawing index (I_R). Two maps of Alaska were divided into zones of typical thermal regime under asphalt (see figures 2.3). The first map is showing four different zones at the present climate : zone of permanent thawing of permafrost (I), zone of periodical thawing of permafrost (II), zone of short-term retreat of permafrost table with formation of a thin residual layer (III) and zone of constant merge of bottom of the active layer and the permafrost table (IV). The second map is showing how the different zones will change due to global climate warming. It is possible to observe that the first zone has moved upwards and has taken most of the place in Alaska. The second zone is also moving upwards and has become smaller. The third zone has completely disappeared. As a result, the global climate warming will not affect immediately the continuous permafrost (zone 4).

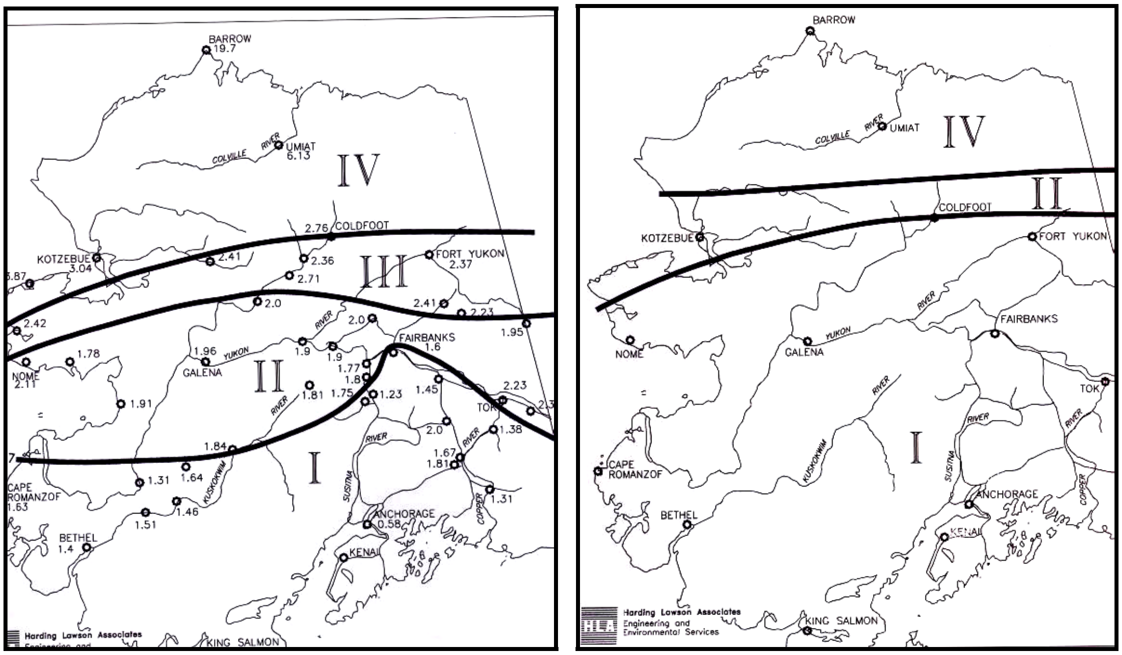


Figure 2-3: Zones of typical permafrost thermal regime under asphalt pavement in Alaska at this present climate and under global climate warming (Nidowicz, Shur, 1998)

A study on the economic impact and consequences of a global climate change on Alaska's infrastructure in 1999 has been done by the University of Alaska-Faibanks (UAF) and the Department of Transportation and Public Facilities of Alaska (ADOTPF). Infrastructures, especially roads and airfields, are important factors of the economic life and they influence many of the population activities. Among northern regions, Alaska is the area where most of the discontinuous permafrost can be found.

The ADOTPF maintains 2415 km of paved roads and 3220 km of gravel roads. In ADOTPF, the state-wide budget is $300M for road design and construction projects. Approximately $75M of this amount is spent for the Northern Regions, where the risks due to discontinuous permafrost are most severe (Cole, Colonell, Esch, 1999). ADOTPF has various repair options : to reconstruct completely a road, to perform remediation that may last for seven years or to repair a road that may last 3 or 5 years. For example, the reconstruction of the seven miles Chena Hot Springs Road east of Fairbanks costs more than $1M per mile (Cole, Colonell, Esch, 1999). The use of two mitigation methods (thermosyphons and geomembrane) increased the project costs to $12M. This construction is expected to last 10-15 years. Annual cost of road repair in the Interior of Alaska (areas with discontinuous permafrost) is budgeted at $15M. Travel restrictions due to bad roads have been calculated to cost to the trucking industry $0.5M per year (Cole, Colonell, Esch, 1999). It is possible to see now how important it is to plan the cost of a potential global warming on roads in discontinuous permafrost areas. Unfortunately in Alaska, many of the engineering solutions for permafrost shown workable through research at the University of Alaska, such as snowsheds, thermosyphons, geotextiles or polystyrene board, are used in less than 1% of the road miles (Cole, Colonell, Esch, 1999). Finally, the ADOTPF is estimating the global warming impact to cost up to $12M yearly for permafrost effects on road building and maintenance. Accelerated permafrost thawing will lead to costly increases in road damage and maintenance. However, eventual disappearance in some permafrost areas reduces construction problems (Cole, Colonell, Esch, 1999).

Some statistics on the mean annual permafrost temperature (MAPT) around the world support the theory of global warming. As it is possible to see in figure 2.4, Alaska has experienced the largest regional warming in United States of America with a rise in the MAAT of about 3^{o}C since 1960s and 4.5^{o}C in winter (Alaska Regional Assessment Group, 1999). From this figure, it is possible to see a quite sudden warming in the 1970s of 1.5-2^{o}C.

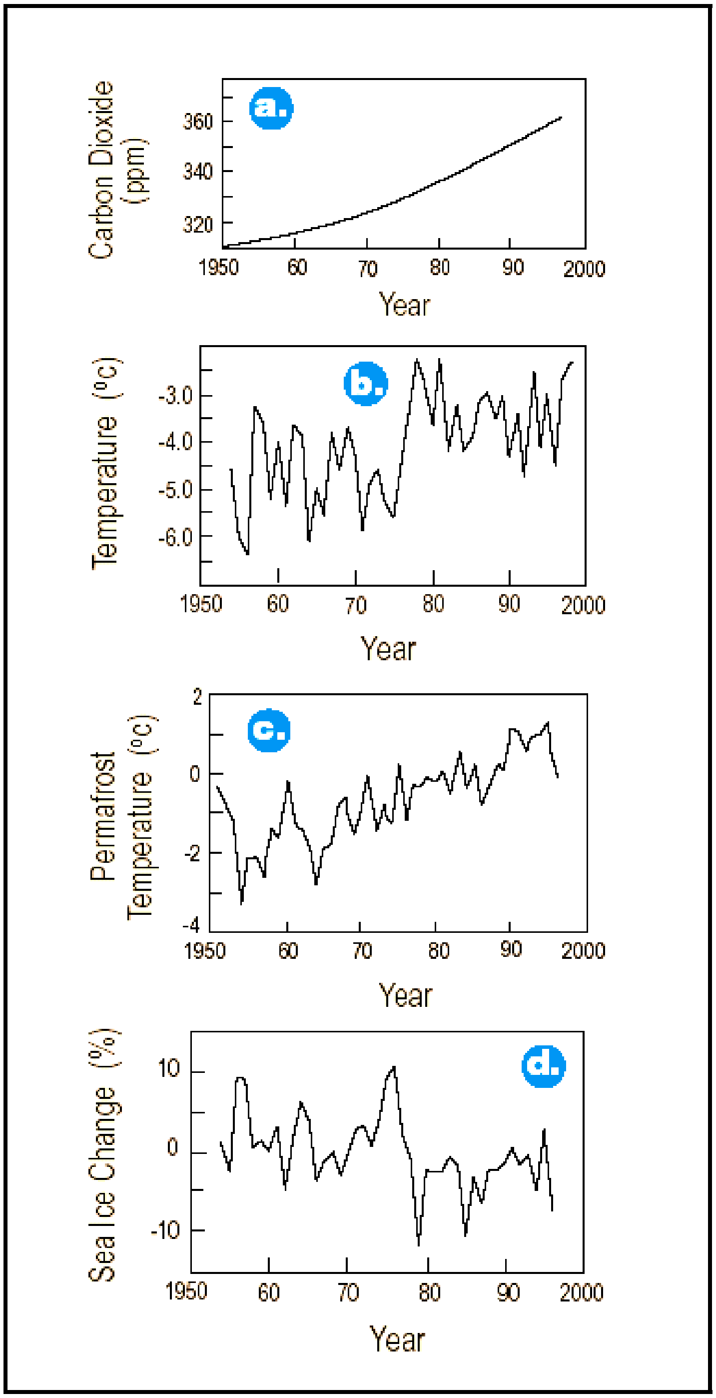


Figure 2-4: Climate and related trends in Alaska A) CO_2 at Barrow, Alaska (smoothed), B) MAAT in Anchorage, Fairbanks, Nome and Barrow, C) permafrost temperatures at Fairbanks, d) Change in sea ice extent (%) in the Bering sea (Alaska Regional Assessment Group, 1999)

Figure 2.5 is showing the changes in permafrost temperatures in Fairbanks, Alaska (Alaska Regional Assessment Group, 1999). It is possible to see an increasing trend at different depths over the years. According to the Alaska Regional Assessment Group, a warming of approximately 1.5-3 ^{o}C is projected by 2030 with 5-10 ^{o}C warming by 2100. In the continuous permafrost of Alaska (south of Brook Range), the permafrost has warmed by 4^{o}C and in the discontinuous permafrost region further the south of Brook Range, the warming has been less than 1-2 ^{o}C. Some of the continuous permafrost in Alaska has warmed continuously since 1980s. This kind of warming usually occurs in the slopes facing south where the active layer may be thicker (Cole, Colonell, Esch, 1999). In the same way, Osterkamp (2003) has a similar opinion on the discontinuous permafrost temperature warming. The warming is 0.5 to 1.5^{o}C and the discontinuous permafrost is thawing at the base at a rate of 0.04 m/year.

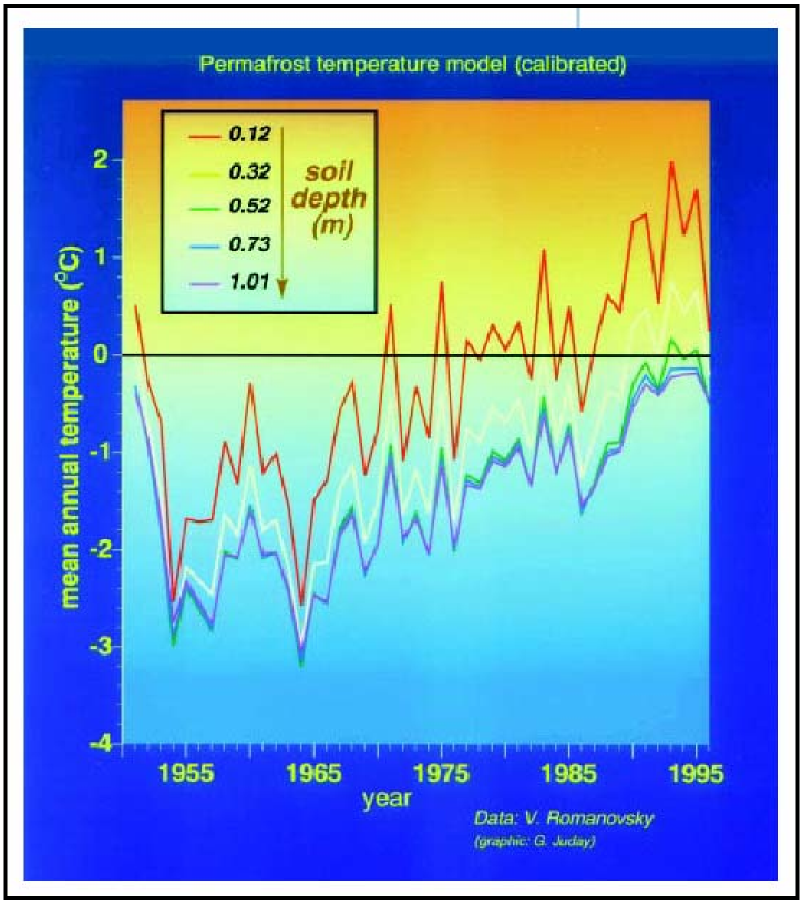


Figure 2-5: Change in permafrost temperatures at various depths in Fairbanks, Alaska (Alaska Regional Assessment Group, 1999)

According to Overpeck et al. (1997) cited in Burn (1998), the climate is warming faster in the Arctic than at lower latitudes of the Northern Hemisphere and warming has been greater in the 20th century than in the previous 400 years. In Canada, similar warming is predicted by the year 2030 (Alaska Regional Assessment Group, 1999). During the recent decades, the general tendency of climatic change in China is similar to the Northern hemisphere. In the northeastern part of China, the southern limit of permafrost is moving northwards at a rate of 1500-3000 m/yr and the permafrost table is decreasing at a rate of 9-11 cm/yr (Yongjian, 1998). In Western Siberia, (Pavlov (1997) cited in An & Devyatkin (1998)), the southern limit of continuous permafrost will move northwards by 200-400 km in the 21st century and permafrost near-surface will be preserved only in northern parts of the Yamal and Gydan peninsulas. In addition, recent changes have occurred in the permafrost of Mongolia (Sharkhuu, 2003). The active layer and the permafrost temperature have increased respectively to 0.1-0.6 cm/year and by 0.05-0.15°C.

On the other hand, because of a terrestrial geothermal gradient (4°C per 100 m), permafrost will tend to warm naturally with depth. Heat flow from deeper layers will eventually thaw any permafrost which is not being cooled by heat loss to the air each winter (Esch, 1996). Talik formation prevents heat loss and is a good indication of permafrost degradation. On the other hand, disturbance of the surface due to the construction of a roadway or runway embankment often increases the mean annual surface temperature (MAST) leading to permafrost degradation (Goering, 1998).

Interpreting global warming is a real challenge to deal with because the climate change implies three environmental systems such as the atmospheric climate, the ground surface and active layer and permafrost (Lachenbrunch et al., 1988 cited by Burn). These three environments will react differently to any change from one to another. In conclusion, a global warming is prevailing now according to many experts and some measures must be taken to ensure the quality of the infrastructures.

b) <u>Description of the phenomena related to permafrost degradation</u>

Thermal reactions and thermal properties of thawed soils

According to the work of Mühll and Romanovsky (2003), thermal reaction of permafrost degradation can be described in general over time as an immediate response (years), as an intermediate response (years and decades) and as a final stage (decades, centuries and millenia). The first response will be a change in the active layer thickness as a result of the ice thawing near the permafrost table. A thaw settlement and frost heave will also occur in the ice-supersaturated material at the permafrost table. Over years and decades, disturbances of temperature profiles within permafrost will begin to occur between permafrost table and permafrost base. After many decades, vertical displacement will occur on the permafrost base.

Many effects of the climate warming can be easily monitored such as the increasing of the active layer thickness overlying permafrost, the rise of ground temperatures and the diminution of the permafrost thickness. Changes in permafrost thermal regime are not only dependent on increased air temperature but also on the ground moisture content that includes water and ice content, the thermal conductivity and the specific heat capacity of frozen and unfrozen ground, the geothermal gradient below the permafrost base, the annual range of air temperature changes in local regions and on the ground surface conditions (Shuxun, Ruijie, 1998).

Thawing of frozen soils is mainly a consequence of conductive heat transfer (Lunardini, 1998). Also, convective heat transport due to ground water flow can lead to permafrost degradation. Convection causes a positive heat flow into the system. The positive heat flow can slow down cooling or can accelerate the thaw depth (Rooney and Vinson, 1996). The thermal conductibility in frozen ground is higher than in unfrozen ground and the volumetric specific heat is higher in unfrozen ground than in frozen ground. Thermal conductivity depends on the type of soil, water moisture content and dry density. Volumetric specific heat is defined by the energy required to change the temperature of a unit of soil by one degree. In fact, unfrozen water does not require a lot of energy compared to frozen water to change its temperature. Due to latent heat effects, the temperature of discontinuous permafrost may be subjected to immediate changes rather than continuous permafrost for the same increase in heat flux.

Thaw settlement

In permafrost, ice can be distributed in soil in different ways. In sand and gravel, ice fills generally the voids while in silt and clays, ice appears in ice lenses and fills vertical cracks to create a network of veins filled with ice. Thaw settlement depends on the distribution of ice in the frozen soil and the volume increase of the thawed soil particle. The distribution of ice and the stressed-deformation behaviour of the thawed soils depend on different factors such as stress, thermal and moisture changes (National Research Council of Canada, 1981).

When the thaw front penetrates the permafrost, the moisture content of the thawing soil will be greater than the soil thawed void volume exceeding the moisture content needed in normal consolidation. As a result, the thawed soil will begin to settle under its own weight tending to the normal consolidation even if any outside load is applied (Ladanyi, 1996). In permeable granular soils, the rate of settlement follows closely the rate of penetration of the thaw front. In other words, settlement at every moment is proportionnal to the thickness of the thawed layer under the foundation. Settlement will increase with time and will follow a square root function of time. Settlement will stop when the thawed front reaches a non-compressible layer. For impermeable fine-grained soils like silt and clays, water pore pressure cannot be dissipated during penetration of the thawed front. As a result, final settlement and final consolidation will be reached only a long time after the frozen front had stopped (Ladanyi, 1996). In ice-rich soils, settlement can be roughly estimated by the visible thickness of ice lenses. Although, this method can result in significant errors (Andersland and Ladanyi,1994).

Under undrained conditions, thawed soil will return under its original volume. On the other hand, under drained conditions, thawed soil will be sujected to extra volume changes that depend on the degree of consolidation and the structural change that occurred during the previous cycle (National Research Council of

Canada, 1981). Hence, more settlements are occurring when drainage is permitted.

While the compressing rate of frozen ground subjected to thawing depends on the ground thermal properties, its final compression depends mainly on its initial frozen density and final density after a total thawing under load. To predict settlement during thawing, it is possible to perform triaxial test to get a thawing settlement graph (figure 2.6). On figure 2.6, it is possible to observe that the void ratio is in equilibrium with each load as the load increases on the soil specimen (Andersland and Ladanyi, 1994).

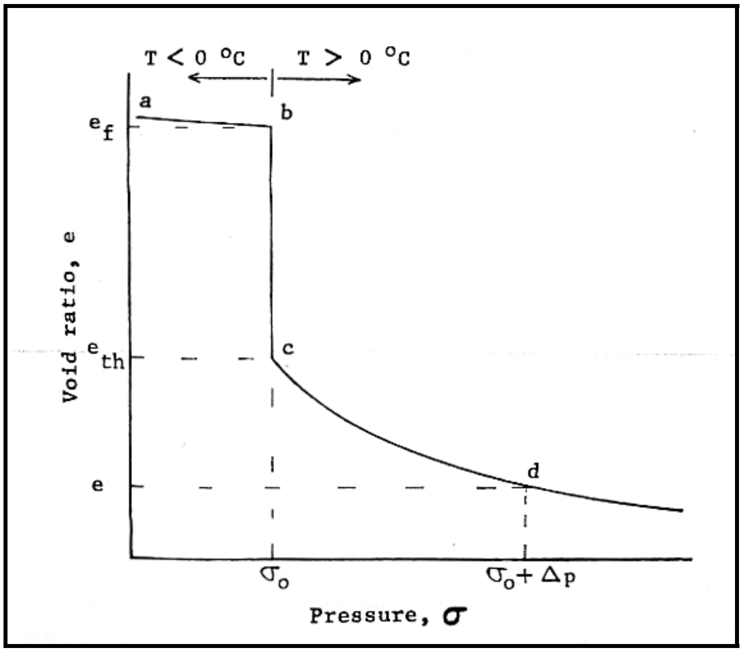


Figure 2-6: Thaw settlement expressed in function of void ration and pressure (Ladanyi and Andersland, 1994)

According to Andersland et Ladanyi (1994), between a and b on figure 2.6, soil is still frozen and between c and d, soil is unfrozen. A decrease of void ratio can be observed as the pressure increases. The change of volume in frozen soils is small compared to the change of volume that will occur in thawed soils. In this case, settlement in frozen soils (between a and b) may be neglected. Between b and c, the sample is allowed to thaw at a pressure σ_0. A large change in void ratio (equal to $e_F - e_{TH}$) occurs due to the change of phase and the drainage of excess water. The pressure σ_0 is usually selected on the basis of effective overburden pressure for the field sample. A thawed-strain parameter A_0 can be now defined (see equation 5 below). If the effective stress is increased of an amount of Δp (or $\Delta\sigma$), consolidation will occur until a new equilibrium void ratio e is attained at point d (Andersland and Ladanyi, 1994). Accordingly, the vertical strain $\Delta H/H$ of a thawed element under a stress σ_0 and loaded to $(\sigma_0 + \Delta p)$ or $(\sigma_0 + \Delta\sigma)$ can be described as (Andersland and Ladanyi, 1994 and National Research Council of Canada, 1981) :

$$(1) \qquad \Delta H / H = A_0 + m_v \Delta\sigma \,,$$

where H the element layer height, A_0 is the thawed-strain parameter and m_v is the volume compressibility coefficient.

Considering that the major thaw settlement is occuring in the thawing period (Ladanyi, 1996), total settlement *s* (see equation 2) can be defined as the summation of the thawing settlement ΔH_t and the settlement that is due to subsequent consolidation ΔH_c. Equations can be defined for a layer of frozen soil H_F or a layer of unfrozen soil H_{TH} on which effective stress acts (National Research Council of Canada, 1981) :

$$(2) \qquad \Delta H = \Delta H_t + \Delta H_c \,,$$

$$(3) \qquad \Delta H_t = A_0 H_F \,,$$

$$(4) \qquad \Delta H_c = m_v \Delta\sigma H_{TH} \,.$$

The use of H_F for H_{TH} in equation 4 will introduce a very small error that is usually assumed negligible (Andersland and Ladanyi, 1994).

A_0 can be defined as the void ratios of the frozen (e_f) and unfrozen ground (e_t) when $\sigma'_0 \leq \sigma'$ or can be determined in term of dry density before and after thawing. m_v can be defined for an increasing on effective stress from σ'_0 to σ' associated with a diminution of the frozen void ration e_f to e (Ladanyi, 1996). These equations are (National Research Council of Canada 1981 and Ladanyi, 1996) :

$$(5) \qquad A_0 = \frac{\Delta e}{1 + e_F} = \frac{e_F - e_{TH}}{1 + e_F},$$

$$(6) \qquad m_v = \frac{(e_F - e)/(\sigma' - \sigma'_0)}{1 + e_F}.$$

Total thaw settlement for *n* strata, each with its own properties, can be evaluated by a summation (Andersland and Ladanyi, 1994) :

$$(7) \qquad \Delta H = \sum_1^n A_{oi} H_{fi} + \sum_1^n m_{vi} \Delta\sigma_i H_{THi},$$

where *i* has values of 1 to *n*. Values of A_0 for each strata should be determined under loading conditions similar to those in the field for best results (Andersland and Ladanyi, 1994).

Crory (1973 cited in Andersland and Ladanyi, 1994) developed a quick evaluation of the possible thaw settlement without the need of a thaw-consolidation test. The relationship to predict thaw settlement is based on the frozen and thawed dry densities of soil, ρ_{df} and $\rho_{d, TH}$:

$$(8) \qquad \Delta H = (1 - \frac{\rho_{dF}}{\rho_{dTH}}) H_F.$$

Johnson (1984 cited in Johnson and Kinney, 1988) used a different approach to calculate thaw settlement. His approach is based on thaw settlement lab data. This technique was first seen in the Russian literature. One-dimensional thaw settlement equation from lab results can be expressed :

$$(9) \qquad S = A_0 X + m_V \int_0^X (P + \gamma' X)\, dx ,$$

where S is total settlement, A_0 is the thaw settlement parameter, X is the depth from the original surface to the thaw front, m_v is the average compressibility coefficient, P is the surcharge load, γ‘ is the submerged unit weight of thawed soil. In equation 9, the water table is assumed to be at the ground surface. In figure 2.7 (Johnson and Kinney, 1988), the settlement is expressed in function of thaw strain versus pressure. A_0 is the value of thaw train when a best fit line is extrapolated to the ordinate and m_v is defined by the slope of the line. Each layer of soil that is suspected to thaw has to be included into the settlement calculations. Since any ice lenses in soil will dissipate completely upon thawing, it is important to include in the calculations an amount of settlement equal to ice thickness.

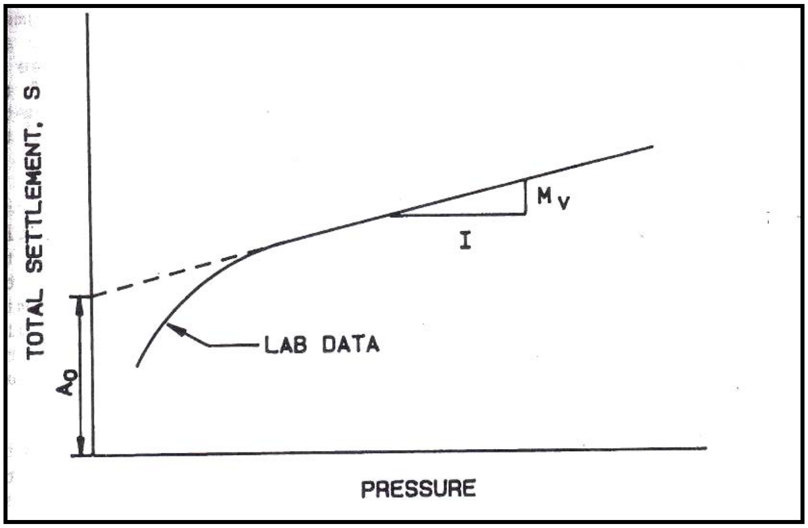


Figure 2-7: Thaw settlement from lab data (Johnson and Kinney, 1988)

Major differential settlement occurs in the upper 10 m of permafrost areas where there are high water content and massive ice deposits (Esch, 1996). This particular layer is the most thaw susceptible layer and is the one that causes main problems (Shur, personal communication, 2003). Pre-thawing can reduce rate of permafrost thawing and settlement.

In order to predict thaw settlement, it is important to perform a soil characterization in the preliminary design phase to identify anticipated thaw strain, time dependent thaw penetration depths and acceptable tolerance for total and differential settlement (Rooney and Johnson, 1988).

Thaw consolidation

In 1971, Morgenstern and Nixon (cited by National Research Council of Canada, 1981) developed a simple linear theory of thaw consolidation. A one-dimensional configuration (see figure 2.8) is considered when a step increase in temperature at the surface is imposed at the semi-infinite homogeneous mass of frozen soil (National Research Council of Canada, 1981). Settlement with time will be governed by the thawed plane location if the thaw plane is moving downwards according to a function X(t) and if water flow soil is unimpeded from the thawed plane location (Andersland and Ladanyi, 1994). Movement of the thaw interface with time indicates that consolidation of thawed soil above interface is controlled by a moving boundary condition. Thawed soils are exposed to self-weight load and to applied surface load (Andersland and Ladanyi, 1994).

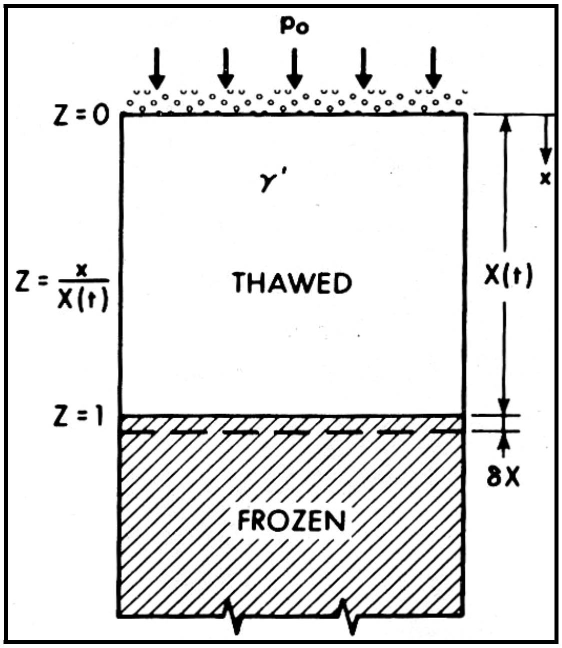


Figure 2-8: One-dimensional thaw consolidation (Morgenstern and Nixon, 1971)

The equation for these conditions of the thaw plane is :

$$\tilde{.}! \qquad X(t) = a\sqrt{t}.$$

where X is the distance to the thaw plane from the soil surface and t is time. *a* is a constant equal to (Jonhson and Kinney, 1988) :

$$(11) \qquad a = \sqrt{\frac{2k_u T_s}{L}}$$

where k_u is the thermal conductivity of thawed soil, T_s is the surface temperature and L is the latent heat of the soil. As a result, thawing was assumed to progress into the frozen soil following Stephan equation.

Morgenstern (1971, cited in Ladanyi, 1996) developed a thaw-consolidation ratio that can be used to predict soil behaviour :

$$(12) \quad R = \frac{\alpha}{2\sqrt{C_v}}$$

where α is equal to factor $\sqrt{t}$ in Modified Bergren Equation and $C_v = k/m_v\gamma_w$ is consolidation factor. The thaw consolidation ratio R is ratio between the rate at which water is liberated at the freeze-thaw interface and the rate at which water can be squeeze out of the soil pores (Johnson and Kinney, 1988). It is influenced by excess pore pressures and the degree of consolidation in thawing soil (National Research Council of Canada, 1981).

If R is less than 1, more than 70% of the settlement will have occurred by the end of the thawed period (National Research Council of Canada, 1981). Consequently, there is no existed excess pore pressures. If R is geater than 1, the pore water carries the entire overburden pressure and the effective stresses between the soil particles approach zero reducing the effective strength of the soil to or near zero (Johnson and Kinney, 1988).

Water pore pressure and drainage

When a load is suddenly applied to a saturated fine-grained soil, an increase in water pressure equal to the applied load is generated. And, when an ice-rich permafrost thaws, excess water is generated at the freeze-thaw interface (Ladanyi, 1996).

In the beginning, there is no change in void ratio and in shear strength (National Research Council of Canada, 1981). With the action of time, water is forced out of soil and settlements occur as the applied stress in soil particles increases. The shear strength increases as the pore water pressures decrease. The decrease of pore water pressure with time is followed by the Terzaghi theory of consolidation National Research Council of Canada, 1981). If the thawing rate is fast enough, water will be discharged at a rate superior to the one it can flow from the soil. So, pore pressure in excess will be generated. If maintained, these excess pore pressures will cause severe problems (National Research Council of Canada, 1981). If the soil permeability is sufficiently low, the pore water cannot flow away from the interface during thawing, the overburden pressure is supported partly by the pore water and excess pore pressure developed (Johnson and Kinney, 1988). According to Dysli (1993), during the ice-lenses thaw, water is flowing downwards. It is possible to assume that when frozen soils are degrading, water is also flowing downwards.

Where ground ice contents are high, the permafrost degradation will lead to possible instability (slope failure) and potential thaw settlement. The thawing of ice-rich permafrost can also produce subsidence of the ground surface as the ice volume turns into slurry (Cole, Colonell, Esch, 1999). As a result of the subsidence, a thermokarst will be probably developed. Development of thermokarst in relatively warm, discontinuous permafrost in central Alaska has transformed some upland forests into extensive wetlands (Osterkamp et al., 2000 in Climate Change Report, 2001).

When water is flowing out of the embankment, it can lead to excess thawing more than the expected thawing from heat conduction of the road surface. Convection and conduction due to movement of water can significantly influence the local ground thermal regime (Rooney and Johnson, 1988). Roadways are usually designed to avoid ponding and longitudinal flow by using drainage culverts through the embankment at appropriate locations (Esch, 1988). Culverts must be installed above the water table to avoid problems. When culverts are used under the embankment to provide drainage, special precautions regarding frost heave and jacking forces have to be taken. Usually, designers neglect the heat thermal gain caused from water flowing since some observations indicate that flowing water has generally a minor impact in poor embankment performance (Esch, 1988). There are cases in which flowing water can greatly increase the thaw depth beneath embankments and cause thaw settlement.

Creep, bearing capacity and problems in warming soils

With warming, the mechanical strength of permafrost decreases, especially in compressive and shear strength. In addition, the creep rate increases for frozen ice-rich soils (Cole, Colonell, Esch, 1999). Problems with embankments overlying ice can be encountered. The creep rate of ice under shear stress is much higher than the creep rate of frozen soils. So, creep movements can lead to excessive settlements even if there is no thawing of the ice (Esch, 1988). Creep rates depend on ice content, temperature, loading conditions, soil grain size and pore fluid impurities (Johnson and Kinney, 1988). A leading international expert, in reology of frozen soils Vyaloy (cited in Johnson and Kinney, 1988) has attributed the creep settlement to the melting ice under pressure in the soil at soil-grain contact points, to the migration of unfrozen water to region of lower stress, to the breakdown of the ice bonds in the soil grains, to the pore ice plastic deformation and to a readjustment of soil particles.

As a result, the embankment will lose its integrity and will require additional material at each repaving cycle for paved roads and airfields (Esch, 1996). Creep movements also damage drainage structures like culverts and make the water flowing out of the embankment harder.

Some design solutions can be considered to reduce creep settlement. Insulation or wood chips and lowering grade can be used to reduce embankment loading (Rooney and Johnson, 1988). Wood chip fill has been used to reduce loading and resulting creep settlement. Some experiments using wood chips, in Alder Creek, Alaska by McHattie (Rooney and Johnson, 1988), have demonstrated a reduction of the previous soil foundation overburden load by 20%, while bringing the road surface up to 3 m and back to pre-creep grade. At Alder Creek, a wood chip layer of 5.8 m was capped on the top and on the sides by 1.1 m and 1.2 m of filled material (McHattie and Esch, 1988).

In 1999, Weller and Lange (Climate Change Report, 2001) note that the bearing capacity of permafrost had decreased with warming, resulting in failure of pilings of buildings as well as pipelines and roadbeds.

As a result, permafrost problems can result in different modes of distress in roads and airfields such as distorsion, faulting, cracking, disintegration and wear (Rooney and Vinson, 1996). The first problem is caused by frost heave, thaw weakening in the active layer and permafrost degradation. The second is associated with the change of temperatures over a year. The last one is generated by thaw-freeze cycles and traffic loads.

IDENTIFICATION, DESCRIPTION AND CLASSIFICATION OF APPLICABLE MITIGATION METHODS

The main thermal impact on roadways or airfields after their construction is the great increase of the seasonal variations in surface temperatures, which results in a much thicker active layer beneath the road or the runway (Esch, 1988). Concretely, it is possible to know that the active layer is increasing when the mean annual surface temperature (MAST) is significantly over 0°C. As a result, the depth of thaw will be higher than the depth of freezing and residual thaw zones called taliks will form beneath the roadway or the runway under side slopes. In other words, if air temperatures continue to increase over the years, taliks will continue to grow under embankment and more settlements will take place. Taliks form particularly beneath the slopes where permafrost thawing is more intense in the lower slope areas where the thickness of the embankment material is at its minimum (Esch, 1996). Maximum settlement problems generally occur beneath the slopes and exceed the settlement problems on the top surfaces of the embankments (Esch, 1996).

Some other thermal effects have to be taken into account and understood to be able to protect accurately embankments from permafrost thaw degradation such as heat gains from flowing water, net surface warming, side-slope warming, massive subsurface ice and inadequate thermal resistance (Esch, 1988). When seeking a solution for permafrost degradation problems, it is important to identify the contribution related to conductive versus convective heat transport processes (Rooney and Vinson, 1996).

Since the 1960's, numerous methods have been used to counter the permafrost thawing effects. Many of them have been developed and tested by ADOTPF. Some have been abandoned for many reasons and others are still used in construction. Many methods have been developed to protect the side slopes. Three different kinds of applicable solutions will be discussed to mitigate the observed or expected problems : methods preventing heat intake underneath the embankment, methods allowing to extract heat from underneath the embankment and methods aiming at the reinforcement of the embankment. Each method will be described and discussed at the same time.

The chosen mitigation method choice will be based on different factors such as continuous/discontinuous permafrost, cost, material and machinery availability and users road safety. Also, methods can be combined with some other methods to give a better protection in mitigating problems.

The main objective of this section is to review the possible technical and operational solutions in order to recommend realistic adaptation scenarios that will assure safe transportation services to the infrastructure users for the next twenty-five years.

a) Methods preventing heat intake underneath the embankment

Embankment thickening and excavation of unsuitable material

In cold permafrost area, the most common method used to protect embankments against thaw settlement is to use a gravel fill layer that is thick enough to contain the active layer (Zarling, Braley, Esch, 1988). If MAST increases, the required fill thickness will also increase. When MAST is around the freezing point, the use of fill is becoming uneconomical. To reduce the fill thickness, insulation like polystyrene can be used (Esch, 1973 and 1983 cited in Zarling, Braley, Esch, 1988). Non frost-susceptible (NFS) material can also be placed on top of the vegetation layer to retain the insulating latent effect and the latent heat retention of the near-surface organics (Crory, 1988).

Molmann et al. (1998) also think that an addition of thaw stable material will act as a thermal insulation that will prevent permafrost degradation. An estimated 1.5 m of thaw stable material was suggested for the Svalbard Airport, Norway (Molmann et al., 1998). Insulation can also be used. For example, at Svalbard Airport, a 50 mm thickness of extruded polystyrene placed immediately on the existing pavement could have reduced the amount of NFS material required by 0.5 m. Finally, this alternative requires an adjustment and replacement of the runway lights and navigational aids Molmann et al., 1998).

According to Crory in his paper about Airfields in Alaska (1988), fills can be placed in summer and in winter. To install the fill material in summer, it is better to wait until its end to avoid vegetation and permafrost disturbances, particularly along shoulders. The depth of gravel fills for airfields normally ranges from 1.5 m to 2.0 m although some runway fills have been as shallow as 1.0 (Crory, 1988). These kinds of runways can be graded and recompacted during summer. Adding filled material is often required to level the runway. Insulation can also be used in runways. A high-density polystyrene is placed directly on frozen cut or fill section of sand. The thickness of insulation can vary from 4.0 to 7.5 m (Crory, 1988). To provide the required bearing capacity and to protect the insulation from being crushed, a gravel gradable surface of a minimum of 30 cm is required on top of the insulation. If granular material is not available in the area, quarried rocks can be used for runways (Crory, 1988).

Remove and replace can also be a solution. In ice-rich permafrost, a thin ice-rich layer can be removed completely down to the underlying frozen gravel layer (Molmann et al., 1998). After, the cut is filled with NFS material. This option eliminates the formation of differential ice. Insulation can also be placed, but will require more NFS filled material increasing the construction costs. In an airfield, the remove and replace option will require the runway to be closed for at least one summer (Molmann et al., 1998).

According to the work of Tremblay and Doré in Kangirsuk Airport, Québec (1988), cuts should be performed carefully in thaw susceptible soils. The subbase thickness has to be thick enough to minimize the thaw penetration in subgrade and differential movements related to thaw-freeze cycles. Laing suggested a total granular thickness of 1.5 m (1983 cited Tremblay and Doré, 1988) as being a proper foundation in the presence of thaw susceptible soils. Also, excavation, preparation of the subgrade and placing the subbase should be performed early in the season while placing top layers of the foundation should be done at the end of the thaw season. Such method will result in a deeper thaw penetration in the first year. Adding the base layer later in the season would help to correct the first year settlement and reduce thaw penetration during the following years (Tremblay and Doré, 1988).

Embankment insulation

Polystyrene insulation

The use of insulation in embankment can slow the heat flow into the permafrost and hence permafrost degradation. Polystyrene protects the permafrost from the summer warm air temperatures. Therefore, permafrost temperatures during summer are decreasing if the soil is insulated instead of increasing if no insulation has been installed. During winter, insulation does not allow permafrost to cool. Permafrost temperatures seem to increase during winter because insulation does not permit evacuation of the heat that is kept inside the ground. Depending of the thermal balance over the year and the MAST, the increase of permafrost temperature during winter can be ignored if the MAST is keeping under 0^{o}C. On the other hand, if the MAST is significantly positive, insulation cannot be used alone and some other methods have to be integrated in the embankment. Often, insulation is used with a granular pad that is thick enough to avoid the effects of a positive MAST.

As a result, insulation has to be used carefully on warm permafrost due to a positive MAST and to the snow cover. The snow cover increases temperatures into the embankment. In the case of discontinuous permafrost, insulation only slows the permafrost degradation process. Settlement and thaw depth are continuing to increase over years causing problems. To summarize, the roadway or airway side-slope surfaces have been found to become generally warmer than either the travelled way surface or the undisturbed ground surfaces (Esch, 1988). Usually, slope surfaces become significantly warm. As a result, progressively deeper annual thawing of the permafrost occurs and taliks develop beneath slope and ditch areas causing ultimately a loss of lateral and vertical support (Esch, 1983 and Mchattie, 1983 cited in Esch, 1988). Cracks may also form on shoulders due to formation of taliks in slope-slide embankments where there is no insulation to protect the slopes and ditches (Esch, 1988). For continuous permafrost, smaller problems can be encountered with insulation because MAST is less than 0^{o}C.

In embankments, two kinds of polystyrene can be used : expanded polystyrene and extruded polystyrene. Expanded polystyrene is a rigid type of insulation formed by expanding polystyrene beads in a mold (Zarling, Arctic Engineering, class handouts, 2003). Thermal resistance R is approximately 4 $HrFt^2F/Btu$ for each inch of material thickness. Expanding polystyrene is the least expensive of polystyrene type insulations. Expanded polystyrene is not losing its R-value with time due to diffusion of gas, however it can deteriorate due to water absorption and to freeze-thaw cycles (Zarling, class handouts, 2003).

Extruded polystyrene is formed using an extrusion process. This kind of insulation has a R-value of approximately 5 $HrFt^2F/Btu$ per inch of material thickness (Zarling, class handouts, 2003). Extruded polystyrene has a good R-value per inch of thickness although has a high cost per R-value. Some degradation of R-value due to diffusion of gas can be encountered. Water will be absorbed into insulation under extreme cases (Zarling, class handouts, 2003).

In 1986, from different test sites in Alaska, Esch (1996) noticed that extruded polystyrene was found to have absorbed not more than 2% water by volume while expanded polystyrene had absorbed less than 5% water by volume. In spite of a slightly increase tendency toward water long term absorption when compared to extruded polystyrene, expanded polystyrene has been proved for soil burial under less severe moisture conditions (Esch, 1996).

Generally, the use of polystyrene is well established in roads and runways over the years and has shown long-term moisture and creep compression resistance. Insulation will be mostly effective if placed as close as possible to the surface, but the depth of cover above has to be thick enough to prevent crushing of the insulation from cyclic wheel loadings (Esch, 1996). The depth of insulation depends on the kind of vehicle or aircraft wheel loadings, the type and thickness pavement and the polystyrene compressive strength (Esch, 1996). The cover depths will range from 0.4 m to 1 m (Esch, 1996). Another advantage to burry deeper insulation is to prevent frost formation. According to ADOTPF, polystyrene has to be installed under roads at a minimal depth of 0.9 m. Also, ADOTPF avoids the use of insulation around sharp curves and on steep grades (Esch, 1996). The thickness of insulation that should be used depends on permafrost ice content, depths of cover layers, thermal properties, layer thickness, local climate and n-factors (Esch, 1996). In fact, the use of polystyrene should reduce the NFS filled material. And, if the insulation thickness placed in the embankment is well designed, thaw settlement can be negligible and filled material be reduced (Johnston, 1983).

In any case, insulation should be placed immediately on top on permafrost. A NFS pad should be placed before installing any kind of insulation (Esch, 1996). In addition, care must be taken at transition zones between non-insulated and insulated areas to reduce differential heat flux through the surface of road or runway (Molmann et al., 1998). According to Molmann et al., it has been determined that transition zones extending 8.5 m in length which the thickness of insulation is systematically reduced

In warm permafrost, polystyrene should be installed at the end of winter during March and April, when the snow cover disappears to make sure polystyrene installation will not result in increasing permafrost temperatures. In cold permafrost, insulation should be installed when MAST is below 0^oC. To proceed to the polystyrene installation in early spring, dry materials should be available from excavated borrow pits or placed in advance in stockpiles (Johnston (1983)). Sometimes, filled material can be hard to get at this time of year. Johnston (1983) has shown that if insulation is placed during summer and fall, the thawed layer does not refreeze for several months and the full advantage of insulation is lost the first summer.

Polyurethane insulation

Polyurethane is a kind of insulation to be foamed in place. The biggest advantage to use polyurethane is the reduction of the shipping costs. Only the drums of reactants that have to be shipped while mixing and foaming will occur on site (Esch, 1996). Problems encountered with polyurethane are water absorption and compression with time of the insulation foamed in place. An overabundance of water increases the thermal conductivity (Esch, 1996). As a result, polyurethane will lose quickly its R-value (3 HrFt^{2}F/Btu per inch of thickness). Initially, polyurethane has a good R-value of about 6 HrFt^{2}F/Btu per inch of thickness (Zarling, class handouts, 2003). As the extruded polystyrene, the polyurethane R-value may decrease due to a diffusion of gasses with time (Zarling, class handouts, 2003).

Peat insulation

Sometimes, peat can be used as insulation. Embankments can be built directly on the peat layer where trees were removed with precaution to make sure vegetation will not be altered and will keep its isolating effects. Occasionally, the peat layer can be pre-thawed and consolisated prior construction (Esch, 1996).

In other cases, a peat layer of 1.2 to 1.5 m thick can be added after excavation on permafrost as a thermal underlayer where peat is available (McHattie, 1983 cited in Esch, 1988). Using peat is a huge advantage. Its frozen thermal conductivity may be twice as its thawed thermal conductivity (McHattie, 1983 cited in Esch, 1988). The presence of peat increases the heat flow out of the ground during freezing and decreases the heat flow into the ground during summer (Esch, 1988). The difference of peat conductivity and its high latent heat reduce the thaw depth and cause the mean annual temperature of the overlying permafrost to be significantly lower than that the overlying road surface (Esch, 1988).

As a result, the presence of peat have caused a net long-term heat removal and have prevented talik development under embankment. Peat has reduced thaw settlement beneath pavement of 1 m and has lowered the permafrost temperature at depth by 0.5°C compared to an unsulated road (Reckard et al., 1988 cited in Esch, 1996). However, peat has not reduced thaw under the side slopes and the ditches.

For maximum effects, peat has to be installed high into the active layer, as close as possible to the surface. Because of the low peat elastic modulus and in order to carry wheel loadings, one meter of fill material has to be put over the peat layer. Precautions should be taken to make sure that peat does not contain old polygonal cracking patterns with large ice-wedges that would create irregular settlement if the ice-wedges are left in place (Esch, 1996).

Reflective surface

The warming of the soil beneath paved roads is attributable to several causes such as removal of vegetation, reduction in evaporation due to the presence of pavement, loss of shading due to clearing for a read and reduction of albedo (ability to reflect short-wave solar radiation) caused by dark surfaces (ADOTPF Report no. FHWA-AK-RD-85-16, 1985).

To reduce summer n-factor and to increase albedo, ADOTF has applied white paint on roads in attempt to reduce thaw settlement problems. For example on Peger Road in Fairbanks, AK, a white-painted section had a MAST of – 0.5°C compared to a normal pavement temperature of 1.1°C for a mean annual air temperature (MAAT) of – 3.3°C (Esch, 1988). In other words, paint applications has reduced average pavement temperatures by 1°C and settlement (Esch, 1996). On figure 2.9, it is possible to see workmen applying paint.

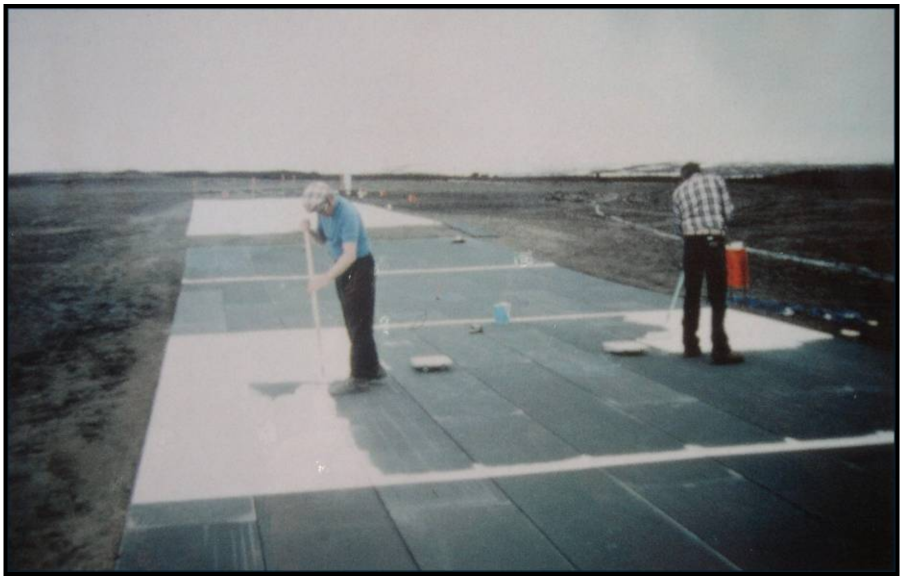


Figure 2-9: Workmen applying white paint on a test section (courtesy of Cold Region Reasearch and Engineering Lab, CREEL)

In Svalbard airport pre-design, in Norway (Molmann et al., 1998), the use of white-painted surface would have shown that the thaw front would have been reduced of 0.4 m. As well as ADOTPF conclusions, this alternative would not solve the problem, but would only slow permafrost degradation. The use of painted surfaces will introduce new maintenance issues to make sure the runway will remain white all the time.

Many problems with white-painted surfaces were observed. Firstly, very high costs are associated with the application of paint. White painted surfaces can create localized frost formation causing the roads and runways to be slippery. Also, roads and runways can be slippery after rain approaching curves, intersections and braking zones. It is also hard to follow its way when it is snowing and when it is sunny due to the sun reflection. Another problem is that the paint is wearing away easily under high traffic and under studded tires, so that annual repainting will be required (Esch, 1988). Heavy traffic generates high average wind speeds, which in turn acts to increase summer warming of paved surfaces (Esch, 1996). It was also discovered that white paint is less effective near road shoulders than at centreline due to heat input from the unpainted embankment slopes (ADOTPF Report no. FHWA-AK-RD-85-16, 1985). This approach is less effective on narrow embankments than on wider paved areas such as runways (ADOTPF Report no. FHWA-AK-RD-85-16, 1985). Where ice-rich soils are limited to a shallow surface layer, painted surfaces will not reduce the ultimate amount of settlement, but merely slow it down (ADOTPF Report no. FHWA-AK-RD-85-16, 1985). In this case, painted surface should not be used and excavation or pre-thawing should be performed. Another solution is to let the road unpaved (gravel road) for few years before pavement installation to let the settlement occur.

Some experiments at Thule, Alaska were performed in 1959. It was possible to observe that a pavement with new white paint application absorbed about 16% of the incoming solar radiation. The absorptivity rose to about 42% after one year due to weathering and traffic (ADOTPF Report no. FHWA-AK-RD-85-16, 1985).

Otherwise, this method would be effective on low-volume roads and where stopping and turning are not required. This treatment has shown good results in reducing thaw depths and lowering surface and subsurface temperatures (Esch, 1988). Benefits of painted surfaces will depend on latitude, cloud cover, wind, traffic speeds and abrasion (reported by Berg cited in Esch, 1996). Because of costs, reflective surfaces should be used in designated areas where painting would be advantageous.

Also, light coloured-aggregate has been used in the asphalt composition to reduce n-factor. As well as for white-painted surfaces, problems have been encountered. Light-coloured soft aggregates (limestone and marble) do not resist to wearing under high traffic and studded tires. To decrease wearing, hard rocks like granite and quartzite should be tried. Sometimes, these kinds of rocks are difficult to find in some areas. After a certain amount of time, the use of pale surfaces do not prevent heat intake underneath the embankment because the road is becoming darker due to dust and to tire passage.

According to Zarling, a light-coloured aggregate surface is a good alternative to painted surfaces because maintenance is reduced (personal communication, 2003).

Sunsheds or snowsheds

ADOTPF developed a method that can protect embankment slopes during winter and summer. Sunsheds or snowsheds let the cold air circulates during winter to protect embankment slopes from snow insulation. In summer, they can reduce solar radiation on the embankment slopes.

At Bonanza Creek, Alaska, an experimental embankment was built with snowsheds along the slopes. Zarling and Braley (1987 cited in Esch, 1988) monitored temperatures, thaw depths, covered and normal embankment slopes which were studied for two years. The mean annual slope surface temperatures were reduced from a normal slope value of 3.9^{o}C to a value of – 2.3^{o}C beneath the sheds (Esch, 1988). The presence of sheds along the embankment had shown that they were able to control and reverse the progressive thawing trends.

For example, at Bonanza Creek, each shed was 9.8 m long and 3.7 m wide and they were constructed adjacent to one another. The sheds were built with a roof trusses with 3.7 m span, a 1 : 4 pitch and a 0.6 m overhang (eave) from a nominal 5 x 10 cm lumber (see figure 2.10 from Zarling, Braley, Esch, 1988).

Figure 2-10: Snowsheds in Bonanza Creek, Alaska (Courtesy of John Zarling)

The trusses were erected on 0.6 m centers and supported on sleepers placed on the ground running down the runway side slope. Then, the trusses were decked with 1.27 m thick plywood to form the roof (Zarling, Braley, Esch, 1988). Plywood was also nailed on the upper and lower ends, and the longitudinal wall of the snowsheds. A space was left open at the top of the longitudinal wall to allow air circulation and to protect from snow accumulation within the sheds. To prevent plows from throwing snow into the sheds, the sheds were oriented with the ventilation space facing away the direction of traffic (Zarling, Braley, Esch, 1988). Finally, sheds were painted white to reflect solar radiation during summer.

In Qinghai-Tibet roadway on Tibetain Plateau, sunsheds were installed to protect from the solar radiation in summer and the snow accumulation in winter (see figure 2.11).

Figure 2-11: Sunsheds, Qinghai-Tibet Roadway, Tibetain Plateau (Courtesy of Professor Ma, Lanzhou, China)

The utilisation of sheds in Alaska was stopped for safety reasons. It is necessary to install guardrails along the embankment to prevent hazards for vehicles accidentally leaving the road (Esch, 1996). Also, sheds have a high-maintenance cost and lacked durability, but are a low cost deployable system compared to some other methods. On the other hand, berms have the same functions as snowsheds and are less expensive. Berms should be used rather than snowsheds (Zarling, personal communication, 2003).

b) Methods based on heat extraction from the embankment

Air ducts

The air duct is a system that allows heat extraction beneath the embankment by natural convection. As a result, air duct can cool the side slope areas during winter. This device is not working during summer and a protection has to be installed at the pipe exit to stop warm air input and to avoid snow blockage in winter (Esch, 1996).

There are two kinds of duct system. One of the systems is to install open-ended ducts in the pad or roadbed oriented in the direction of the prevailing wind (Nixon, 1978 cited by Niu et al., 2003). The second system is based on the chimney effect to induce air movement in stacks connected to the ventilation ducts (Tobiasson, 1973 cited by Niu et al., 2003).

For example, on Alaska Highway near Gardiner Creek in Alaska, a 0.6 m diameter corrugated metal pipe is placed 0.30 m to 0.60 m above the permafrost table on a sand pad to avoid pipe water filling problems. The duct system inlets are elevated just above the maximum snow level leading to nearly horizontal 22 to 45 m heat exchanges parallel to the roadway in embankment toe berms (Esch, 1988). The warmed air exits from vertical exhaust stacks up to 3.5 in height (Esch, 1988).

Zarling (1988 cited in Esch, 1996) tried a larger air duct system with a diameter of 62 cm and a pipe length up to 50 m. In the first years of installation, such system had appeared to function better than a smaller system. According to Zarling, even if there is a windy area, natural convection is not enough to cool the ground (personal communication, 2003).

When ducts are placed under a layer of polystyrene insulation, ducts seem to reduce even more the thaw depths (Esch, 1996). However, the ducts have not eliminated the embankment spreading and cracking. Ducts can also be combined with lateral berms to cool the side slopes.

Water ponding within the ducts can be a real problem such as settlements. Water ponding will impede air flow to circulate through the pipe and hence, to reduce the heat extraction and the refreezing. To have the best chance of long-term survival and freedom from water ponding, Esch (1996) recommended that culverts might be used as an air duct system. This method was not tested by

ADOTPF. He had discovered that culverts with diameter less than 1 m will have a net warming and that culverts with a diameter greater than 1 m will have a net cooling if they are used like culverts. This choice of culvert diameter is based on presence of snow. Snow can block the opening of the pipe and stop heat exchanges. With a bigger culvert, air flow into the pipe will be possible. If there is no air flow into the pipe, the soil will begin to settle. The culverts will be placed through the embankment from side to side and spaced at longitudinal intervals of 8 to 12 m.

Air ducts or culverts are not a method that is currently used by ADOTPF at this time. However, a laboratory study for the new construction of Qinghai-Tibet railway is expected to use ventilation duct system (Niu et al., 2003) to reduce permafrost degradation and thawing settlement of the roadbed. Because of the presence of huge underground ice layers in the Quinghai-Tibet plateau, ventilated ducts have been chosen to extract heat from the ambient soil in order to keep stable the permafrost table. Using the remove and replace alternative or insulation would be too costly. The laboratory ventilated-duct test had shown a reduction of soil temperature by 11.3-11.8°C compared to 5.9-6.3°C in unventilated roadbeds. The ducts will be placed through the embankment with an open-ended ventilation duct because the Qinghai-Tibet plateau is very windy. During summer, the open-ended ventilation ducts should be closed to keep the warm air outside the embankment in order to avoid thaw settlement. In Qinghai-Tibet roadway, the ventilated embankment has already been used (see figure 2.12).

Figure 2-12: Ventilated embankment with cuverts, Qinghai-Tibet Roadway, Tibetain Plateau (Courtesy of Professor Ma, Lanzhou, China)

Thermosyphons

A thermosyphon, also called heat pipe, is composed of a corrugated pipe partially filled with liquid having low temperature of evaporation such as ammonia, carbon-dioxide or propane. When the air is colder than the ground, heat from the ground causes the liquid to vaporize (Sorensen et al., 2003). The vapor flows upwards from the evaporator section (below ground) to the condenser section (above ground) where the vapor condensates back to a liquid because of cooling by ambient air. The condensate flows downwards by gravity on the internal wall surface of the thermosyphons where it absorbs heat from the ground and is re-evaporated to continue the process (Sorensen et al., 2003). When air temperature is warmer than the saturation temperature of the chosen liquid, the thermosyphon is not working (Sorensen et al., 2003).

When heat is flowing into the pipe, there is an increase of pressure and when the pipe is cooled, there is a decrease in pressure and vapor can condensate.

Thermosyphons keep working because of the low pressure gradients. Under optimum winter conditions of heat transfer, the entire thermosyphon will be cooled to the temperature of the ambient air, causing the freezing and the cooling of the soils surrounding the burried portion (Esch, 1988). In fact, preservation of permafrost can be possible by using cooling pipes which used the phase changes of a single fluid-gas system inside a pressurized tube (Esch, 1996).

In road construction practice, the use of thermosyphons involve pipes buried in parallel trenches near the base of embankment and inclined upwards toward the radiator section (Esch, 1996). An inclination greater than 6% is recommended for proper functioning because the condensate must flow down from above the ground (radiator and condenser) to the end of the pipe buried into ground (evaporator) (Esch, 1996). However, thermosyphons have also been installed flat on the ground.

Thermosyphons can reduce thaw depths and settlements in areas where the surface temperature regime changes due to buildings or roads (DenHatog, 1988). These thermosyphons are two-phase thermosyphons and require a positive slope of at least one in ten so the condensate can flow downward. When a loop of liquid-filled present in pipe is subjected to different temperatures at the oppsite end of the pipe, the fluid will begin to move around the loop (DenHatog, 1988). Consequently, the fluid for a two-phase thermosyphon is chosen to have temperature above boiling point at the pipe warm end while to have temperatures below condensing point at the pipe cold end (DenHatog, 1988). According to Zarling, thermosyphons for horizontal applications are really expensive because of the high costs involved to manufacture them (personal communication, 2003). In addition, inclined thermosyphons are easier to ship at the construction site. A picture was taken on Chena Hot Springs where horizontal thermosyphons were used (see figure 2.13).

Figure 2-13: Horizontal thermosyphons on Chena Hot Springs Road, Alaska

It is possible to see the condenser becoming larger toward the top to allow the liquid moving and not have the condensate trapped on the top (Zarling, personal communication, 2003). On the other hand, Connor (personal communication, 2003) thinks that flat thermosyphons are easier to install on roads because they do not need too much space. Inclined thermosyphons require much more filling material to be able to bury the pipe which increase construction costs.

A 50 to 100 mm standardized radiator with dual or triple radiators should be used to get higher cooling rates (Esch, 1996). The finned radiator sections are exposed to the air and especially to any available wind to increase heat tranfers. To have a system more efficient, the radiator has to be installed 1 to 2 m above the snow surface and the evaporator has to be up to 40 m long with a lateral spacing of about 2 m (Esch, 1996). To maximize the length of pipes, the pipes should be installed at some horizontal angle to the embankment centerline instead of perpendicular to it (Esch, 1996). To keep efficient thermosyphons, the surfaces of the pipes have to be cleaned to avoid reactions that can generate other gases.

Thermosyphons have to be placed on compacted to high relative density NFS pad above the top of the permafrost table. By compacting the NFS materials, the critical contact between soil and pipes will be reduced what will increased the thermosyphons efficiency (McFadden and Bennett, 1991 cited in (Esch, 1996). Thermosyphons will freeze NFS material during winter and will thaw stable during summer. To protect the subsurface from thawing in summer, a 50 to 100 mm polystyrene layer can be installed at depth of about 1 m (depth required by ADOTPF) or as close as possible to the surface. The insulation depth depends on the wheel loadings and frost formation on the roads or the runways (Esch, 1996).

The main problem with thermosyphons is their high cost. They typically cost 1500$ or more per unit. For this reason, thermosyphons are only used in severe permafrost degradation areas. The other problem is to protect the condensers that are outside along the roads and runways to make sure an automobilist or an aircraft will not hit them. A solution to avoid problems is to bury the condensers. In addition of being protected, condensers can help to warm the pavement during winter in order to melt ice that is forming on the surface. Another solution is to use horizontal radiators or protective guard raikings (Esch, 1996) or to use hairpin thermosyphons. Hairpin thermosyphons are more esthetic and facilitate maintenance of the road.

The refrigerated gas has to be chosen carefully. Many thermosyphons on the Trans Alyeska Pipeline System (TAPS) have suffered from a cold topping problem that was due to an accumulation of non-condensable gases, a by-product of corrosion or chemical dissociation of anhydrous ammonia (Sorensen et al., 2003). Now, carbon-dioxide is used on many of the TAPS thermosyphons.

Sorensen et al. have suggested that the non-condensable gases cold topping level depends on ambient air and evaporator temperatures.

Air Convection Embankments (ACE)

This relatively new technique (1995) was tested by Douglas Goering, professor in Mechanical Engineering at the University of Alaska (UAF). ACE is a method that is based on formation of convective cells in embankment using large poorly-graded porous rocks with a low fine content. It is possible to create such convective cells if the voids in a rock embankment are large and interconnected. In winter, the air present in voids is cooled at the top of rock layer and is then settled downwards because of its higher relative density. This cold air movement will cause the warm air to move upwards. The creation of convective cells (see figure 2.14) will speed the cooling and the refreezing of the permafrost underneath (Esch, 1996).

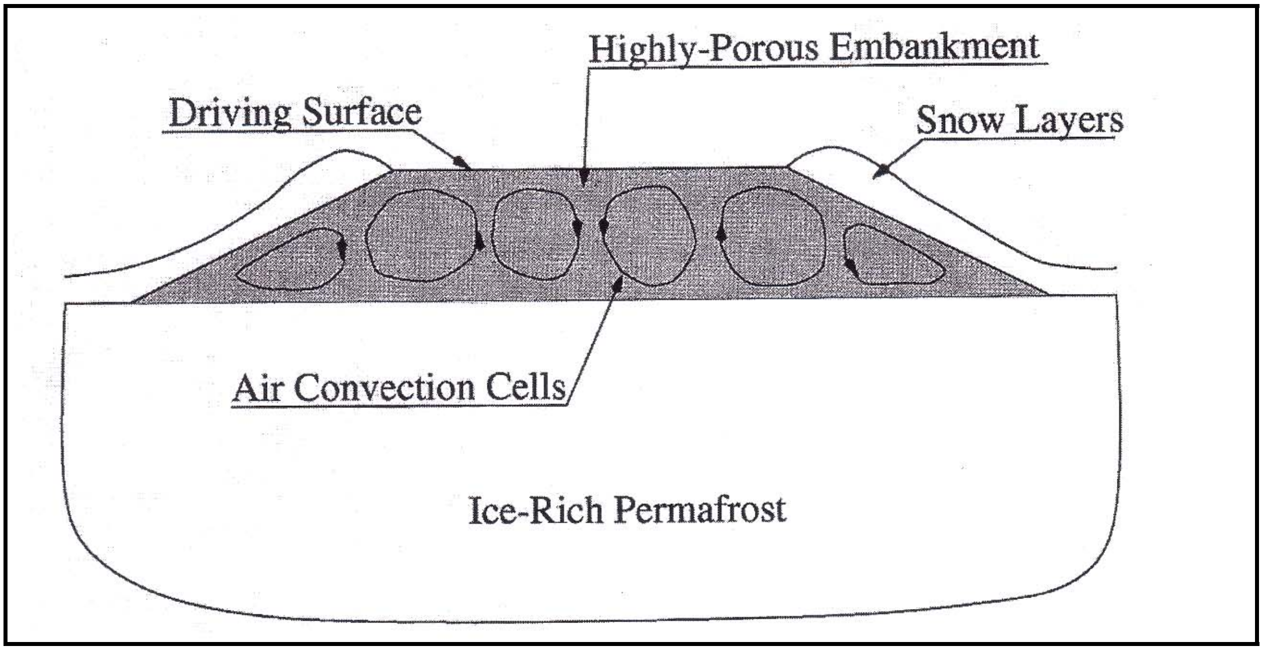


Figure 2-14: Air Convective Embankment (Goering, 1998)

In summer, the warm air will stay on the top layer and the air present in the voids will stay thermally stable. In this case, the rock layer can act as a layer of insulation in summer protecting the ground from solar radiation. As a result, ACE will increase wintertime cooling rates and will decrease summertime warming rates (Esch, 1996). It seems that convection can transfer heat upwards out of the embankment at a rate that may be more than an order of magnitude larger than conduction, resulting in a greater winter cooling (Goering, 2003).

A test was performed in Brown Hill Querry near Fairbanks in 1993 (Goering, 1998). In this case, the embankment was not built on permafrost. The embankment was 2.5 m high and 16 m wide from toe to toe with 2H:1V side slopes. The rocks used in the ACE had a size of 5 to 8 cm and were manufactured at the quarry by crushing larger rock and then screening to remove all material outside the desired gradation range. Placement of the rocks with a front-end loader was accomplished by building 0.6 m at the same time. Compaction was performed after the placement of every 0.6 m layer with a loader. No other compaction was done. After two years of monitoring, it was found that the mean annual temperatures at the upper surfaces of the embankment were about 2^{o}C compared to –1.2 to –3.6^{o}C at the subgrade surfaces (Goering, 1998). Mean annual base temperature of the embankment lowered to –1.5^{o}C. After temperature modeling, the mean annual permafrost temperature could be lowered of up to 5^{o}C (Esch, 1996). As a conclusion, ACE can have a large cooling influence due to air convection within the embankment during winter (Goering, 1998).

Another project was conducted in 2001 by Douglas Goering for the Loftus Road extension. Approximately 80% of the road needed some sort of cooling system to prevent permafrost degradation. The other 20% of the road was pre-thawed many years before construction by vegetation removal, so settlement problems are not anticipated (Goering, 2001). Two different sections were designed to use ACE. The first one was designed with hairpin thermosyphons and insulation. An

ACE berm was only built on the right hand side of the embankment in order to aid the elimination of potential shoulder rotation due to thaw settlement beneath the right side slope (Goering, 2001). The second section consisted of an ACE core combined with ACE side slopes. A layer of 2.5 m of rocks was placed on the embankment (Goering, 2001). The last section consisted only of hairpin thermosyphons and insulation. The evaporators were placed beneath the roadway and the condensers were not placed inside ACE berms (Goering, 2001). All the ACE were composed of poorly graded crushed rocks having a dimension of 15 to 30 cm with low fine content. The ACE had a porosity of about 40% and a low thermal conductivity due to low particle to particle contact. In order to predict the behaviour of such systems, numerical simulations were carried.

If possible, rocks should be sorted in the same particle range to be able to obtain single-sized particles in the convective layer (Esch, 1996). Having similar rock dimension will assure highest voids, highest convection and cooling rates in winter (Esch, 1996). The dimensions of rocks do not need to be as big as 15 to 30 cm. In order to have an extra safety margin, bigger rocks were used to make sure they will not break and become smaller during construction (Goering, personal communication, 2003). According to Goering, smaller rocks are also easier to find than bigger ones (personal communication, 2003).

Angular rocks rather than round rocks should be used to stabilize the embankment. Angular rocks will act as a lock in the embankment to avoid creep and sliding. In a heat transfer point of view, round rocks are better for natural convection, but they will tend to move on each other (Goering, personal communication, 2003). The height of the embankment is based on the Rayleigh number that is a function layer height and material properties (Saboundjian and Goering, 2003). The required height of embankment should be around 1.5 to 2.5 m (Esch, 1996).

In order, to protect the ACE from infiltration of fine particles and from fine filling in voids, a geosynthetic should be placed on top of the embankment. Finally, to prevent thawing of the underlying foundation soils during construction, ACE should be built during winter by placing the core material in a frozen state (Goering, 2001).

The problem with ACE is to find good competent coarse rocks that are big enough to allow the creation of convective cells. ACE is an alternative that can become really costly. In order to get the wanted size rocks, crushing in the quarry is the most expensive part of an ACE. Rocks can also be found in mine tailings (Zarling, personal communication, 2003).

c) Methods based on the reinforcement of the embankment

Geogrids and geosynthetics

The basic roles of a geosynthetic are separation, reinforcement and to a less extent filtration and drainage (Instanes et al., 1998). Installation of geotextile in discontinuous permafrost areas can reduce lateral spreading, cracking and dips on thaw-weakened foundations (Esch, 1988).

Tests were performed by Kinney and Connor in 1984 on the ability of various fabrics and synthetics strapping materials to span artificially created voids (Esch, 1996). It was learned from these tests that voids cavities up to 3 m in width might be spanned by the use of geotextile (Esch, 1996). Geotextiles have to be used carefully to prevent failure of the embankment. If failure occurs under load, high settlement problems are expected. As a result, short term repaving operation might be needed in order to put the geotextile back in tension and to maintain the level embankment surface (Esch, 1996). Over the years, ADOTPF have used geotextile in order to reduce future settlement and cracking problems in embankments up to 5 m in thickness (Esch, 1996).

There are different ways to reinforce an embankment with geosynthetics. Firstly, the embankment can be reinforced with a single layer of geosynthetic to help solve toe and side slopes thaw settlement and formation of grabens (see figure 2.15). Since the geotextile offers poor reinforcement in the vertical direction and much of the lack of shoulder support is caused by the vertical movement of the toe, the horizontal tension in the geosynthetic will not prevent cracks and grabens from occurring in the shoulder (Rooney and Johnson, 1988). This kind of reinforcement will help to prevent the widening of cracks and grabens. As a result, geotextiles can help to reinforce the embankment when it is sufficiently strained.

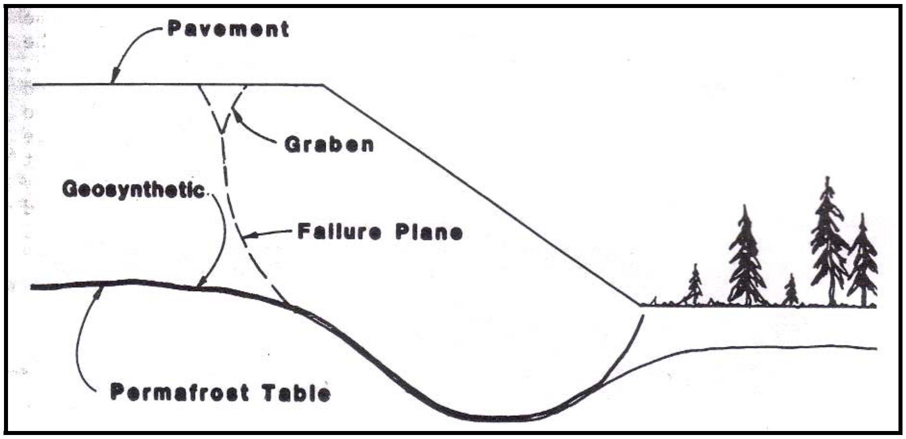


Figure 2-15: Embankment reinforced with a single layer of geotextile (Rooney and Johnson, 1988)

Another method is to build with geosynthetic retained core (see figure 2.16). The embankment is constructed with multiple layers of geotextiles, so as to form two geosynthetics walls tied together through the embankment (Rooney and Johnson, 1988). This method is called a pillow embankment method. The walls are built using the conventional geosynthetic design. To make sure that the embankment core is thermally stable, a sufficient gravel layer or insulation should be placed under the geotextile. This method has given promising results and no cracks and grabens had appeared at the roadway surface. The edges of the walls are located in a way that they will not be thermally affected by the thermal degradation of permafrost under slopes (Rooney and Johnson, 1988).

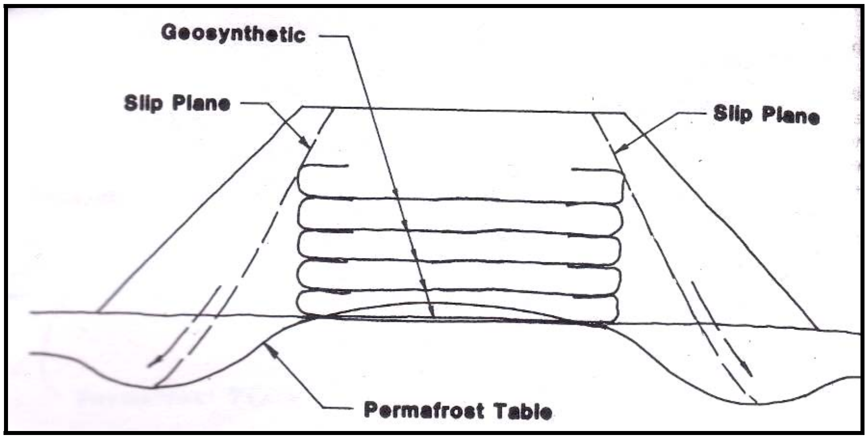


Figure 2-16: Core embankment reinforced with geotextiles (pillow) (Rooney and Johnson, 1988)

Kinney (1981, 1985 cited Rooney and Johnson, 1988) had developed a method to bridge a graben or a thermokarst (thawed ice-wedge) or with geosynthetics (see figures 2.17 and 2.18). In this case, the geotextile is placed at the bottom of the embankment. The geosynthetic layer has to be stiff and strong enough not to creep under a constant load. To make sure the reinforcement system will work, some settlement must be allowed to occur in order to put the geotextile in tension. Once the geosynthetic is put in tension, it will support the embankment material over the voids as ice melts (Rooney and Johnson, 1988). In Kinney's experimentations, voids up to 2.4 m can be spanned by these materials (Rooney and Johnson, 1988). To conclude, engineers do not all agree on this method.

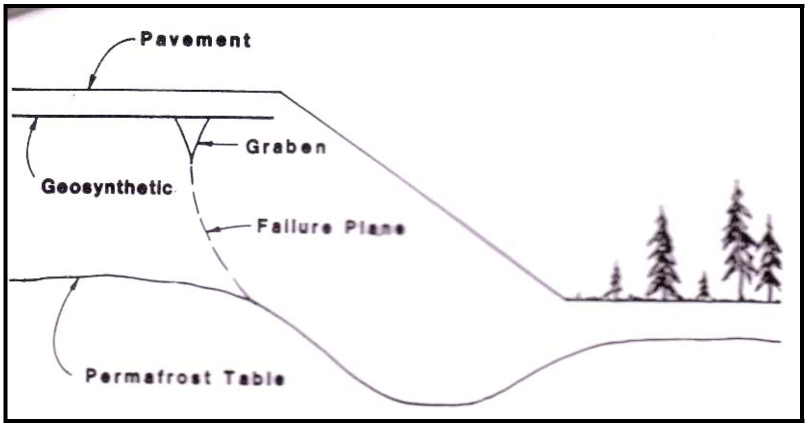


Figure 2-17: Geotextile bridging a graben (Rooney and Johnson, 1988)

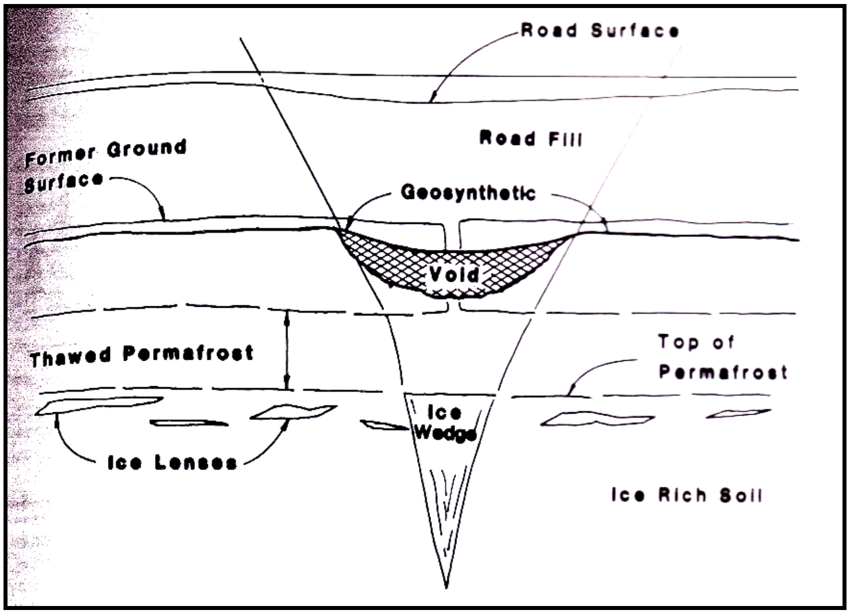


Figure 2-18: Geotextile bridging an ice-wedge (Rooney and Johnson, 1988)

As geotextiles, geogrids can be used to reinforce the embankment. However, it is not a material that is easy to work with. Geosynthetics are easier to use in construction. In Russia, geocell (see figure 2.19) is used to reinforce the embankment in order to stabilize the fill material and the cut side slopes (Grechishchev et al., 2003).

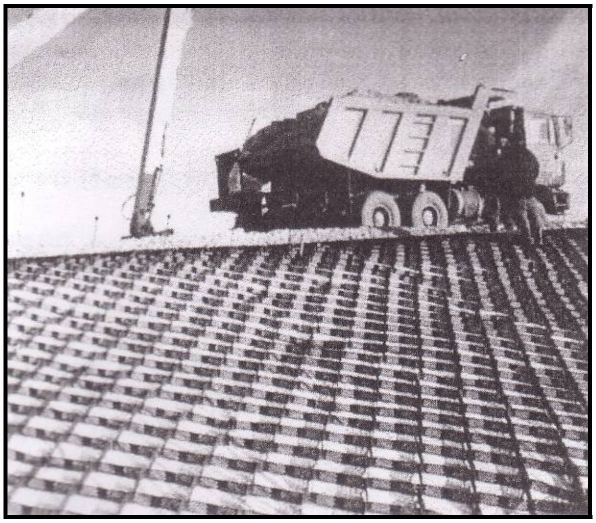


Figure 2-19: Geocell in Russia (Grechishchev et al., 2003)

According to Kinney (1996), geosynthetics cannot be used to reduce undulations, to reduce vertical settlement of the subgrade and to reduce longitudinal cracking. However, geogrids can significantly reduce maintenance required to correct longitudinal cracks (Kinney, 1996). As a result, geosynthetics can be used to reduce lateral spreading, to reduce erosion and to reinforce weak subgrades (Kinney, 1996). According to Zarling, installation of geotextile in embankments can reduce maintenance (personal communication, 2003).

Berms

Berms are used to reduce n-factor and to protect the lower embankment slopes from excessive thawing. Two kinds of berms that can be designed. The first kind is built with silty soils. These kinds of berm have very minor impacts on slowing thawing along the slopes and hence, increase settlements. In fact, such berms act much of the same as the roadway side slopes and also generate additional surface areas at temperatures averaging well above 0^{o}C (Esch, 1988). The use of polystyrene has also been tried to stop movements, but have not shown noticeable effects. In this case, it is not cost-effective to add polystyrene in the embankment (Esch, 1996). Removal of snow along the berms would have helped to decrease depth of thaw (Esch, 1996). Zarling also agrees on this alternative (personal communication, 2003). Snow should not be pushed on the shoulders and along the side slopes. To reduce the water streaming on the roadway in spring, snow is blew out further than the toe embankment base. In winter, the same snow maintenance should be done to reduce side slopes warming.

For example, a study was conducted at Bonanza Creek, Alaska using a 1.8 m high silty-soil berm (Esch, 1983 cited in Esch, 1996). In conclusion of this study, the silty-soil berm had offered only a temporary reduction of 1 to 3 m in thaw depths. In long term, these berms were detrimental to the embankment because their presence had relatively increased the warm surface areas of slopes. After 20 years of evaluation, the berm areas had settled as much as 2.0 m and water ponding was expected to eventually occur (Esch, 1983 cited in Esch, 1996). Such berms had not stopped the creation of cracks and lateral spread. To reduce the sun exposition in summer, this berm had been seeded and vegetated.

The second kind of berms uses big gravel rocks to initiate convective cells into the embankment during winter to extract the heat beneath the embankment in winter. The presence of a rock berm in the summer permits shading of the side-slopes and hence, reduces thaw settlement. These kinds of berms can be also called air convective berms. They were used in Loftus Road described in section 3.2.3. In Yukon, Canada, on the Alaska Highway, rock berms have been built along the road embankment (see figure 2.20).

Figure 2-20: Rock Berm on Alaska Highway, Yukon, Canada (Photo by Guy Doré, Université Laval)

On the other hand, Esch (1996) disadvises rock berm utilisation because gravel layers will not support dense vegetation growth to protect berm sides from sun exposure. However, rock berms can be employed beneficially if topsoil layers and seeding are placed on the gravel layer to encourage regrowth of vegetation (Esch, 1996). Sometimes, berms are built with vegetated waste materials. These kinds of berms also showed high temperatures along the side-slopes and an entire embankment warming.

Another solution is to increase the width of the embankment slopes to move further the thaw and settlement problems of the relatively warmer side slopes from the paved surfaces by use of gentle slopes or flatted top soil berms covering the lower slope (Esch, 1996). According to Esch (1996), the problem with this alternative is the creation of the warming effect of the entire embankment. Also, no investigations have yet taken place to study the influence of snow accumulation on berms during winter (Zarling, personal communication, 2003). In conclusion, berms or other side slope variations increase the roadside warming increase effects by increasing the size of warmer surface areas (Esch, 1996).

d) Other methods

Pre-thawing

This method is usually used to achieve thawing and consolidation prior construction for unstable permafrost layers. Performing pre-thawing prior construction during one or two thawing seasons can reduce significantly thaw settlements. Pre-thawing can be performed in shallow ice-rich permafrost layers, but has to be avoided when deeper ice is present (Esch, 1988).

To accelerate pre-thawing, the vegetation can be stripped to expose the soil to the sun in summer. In winter, it is recommended to insulate the soil to continue the thawing. The main reason to put insulation is to protect what has been so hard to thaw the previous summer. Another method to increase thawing is to place clear plastic coverings on the ground. This greenhouse type of cover will impede evaporation and convection to cool the surface (Esch, 1996). On the other hand, clear coverings will expose the ground to solar radiation. Black coverings were also used, but did not show good results. The plastic film was warmed by the sun, but heat transfer to the soil was reduced. Also, a thin gravel pad (0.3 m) can be applied after the removal of vegetation to provide light traffic and to load the surface slightly to consolidate the thawing soils while retarding erosion and vegetation regrowth (Esch, 1996). A layer of hot asphalt was also applied to get darkener surfaces to maximize surface heat gains from solar radiation (Esch, 1996). Another technique to pre-thaw the ground is to allow injection and circulation of steam, hot water or cold water. Holes are drilled or driven in an equilateral triangle to increase thawing (Rooney and Johnson, 1988). Steam pre-thawing can be hard to control and can be expensive. A practical method to allow pre-thawing is to strip vegetation in summer and use the area as a snow dump in winter (Zarling, personal communication, 2003).

In 1980, at the Cold Regions Research and Engineering Lab (CRREL) near Fairbanks, Alaska, a pre-thawing study was performed. After the removal of vegetation and the application of herbicides, the permafrost melted to a depth of 1.4 m in one summer and of 2.3 m after four years (Esch, 1996). After placing a blackened surface gravel layer and a polyethylene covering, the depths of thaw increased to 1.5 m in one summer and 3.1 m in four years (Esch, 1996). After four years, the ground had settled of 0.33 m.

Pre-thawing of Bethel Airport, Alaska arrived accidentally (Rooney and Johnson, 1988). A three year pavement delay allowed thaw degradation and active layer deepening. As a result, little surface deformation occurred within the initial runway over the next fourteen years.

Soil preparation for pre-thawing should be performed when air temperatures are slightly below freezing to avoid problems with thawed water and bulldozers traction.

Pre-thawing is an excellent alternative that do not involve high costs. However, designers and engineers need time to perform it. The major disadvantage to perform pre-thawing consists in the difficulty of accurately anticipating the thermal regime or thaw bulb position that will be stable particularly if permafrost is too thick to be completely thawed (Arctic and Subarctic Construction for Structures, TM 5-852-4, 1983). Continuing thaw of permafrost could result in settlement and refreezing at the boundaries can produce heave. Excavation and replacement can be used to replace shallow ice-rich foundation soils by NFS soils when time is not available to proceed to pre-thawing. This method can be really expensive, but it is one of the best methods to reduce thaw settlements.

Regardless of all these methods, the most frequently used method is to build a structurally but thermally inadequate embankment (Esch, 1988). Designers simply accept that excessive embankment movements are going to occur and hence, accept to the embankment will require maintenance and repair when problems occur (Connor, personal communication, 2003). The road or airfield is simply patched and leveled over the years. Sometimes, it is a cost-effective method.

An ideal method would be to remove snow along the embankment slopes to reduce soil surface temperatures (Zarling, Braley, Esch, 1988). This method is expensive and will require skilled operators to remove the snow along the embankment. Some operators might destroy the vegetation or break illumination poles. Sometimes, it will be unfeasible to remove snow due to the steepness of the slope and the presence of large rocks. Zarling has also another alternative to remove the snow in order to reduce the warming of side slopes (personal communication, 2003). Snow should not be pushed on the shoulders and along the side slopes. To reduce water streaming on the roadway in spring, the snow is blew out further than the toe embankment base. In winter, the same snow maitenance should be done to reduce side slopes warming. If costs are not a construction factor, another ideal method will be to build an embankment with nearly vertical slopes which do not accumulate snow and which be protected from the solar radiation (Esch, 1988).

Another way to reduce problems with permafrost is to unpave roads and let them as gravel road for few years to let the settlement occur before paving them. The big advantage to use gravel roads compared to paved roads is to decrease thaw depths and hence, the annual settlement. Gravel roads reduce sun radiation absorption compared to black asphalt paved roads. However, gravel roads have many negative effects such as visibility for drivers due to dust, car high cost maintenance and time transportation. Usually, gravel roads are built for traffic below 300 vehicles per day. Over the years, it is cheaper to pave roads than build gravel roads (Connor and Zarling, personal communications, 2003). Concerning airfields, the runways are paved according to the served population in the villages (Zarling, personal communication, 2003). Only bigger locations have paved airports. To pave or not to pave a runway, the decision can also be based on the kind of aircrafts that are landing. The main problems with gravel runways are visibility and the presence of rocks that can damage the propellers. Unpaved runways and roadways are frequently graded.

FEASIBILITY AND COST EFFECTIVENESS OF THESE APPROACHES

Feasibility

Many methods described above only slow permafrost degradation and are not long-term solutions. Settlement, cracking and lateral spreading still occur over the years. For many reasons such as effectiveness, cost, safety, high maintenance, these methods are not actually used. The main objective here is to find methods that are effective, economical, secure for users and with low maintenance. Some methods can have high construction costs, but will not require any further maintenance.

Some methods can be either used in continuous permafrost or in discontinuous permafrost. Embankment thickening, remove and replace alternative, insulation can be performed in continuous and discontinuous permafrost areas. All the other methods such as reflective surfaces, sunsheds or snowsheds, air ducts, thermosyphons, ACE, geosynthetics, berms and pre-thawing are used in discontinuous permafrost in order to prevent permafrost degradation and thaw settlement. In fact, mitigation methods in discontinuous permafrost are the most important to avoid permafrost degradation problems.

In discontinuous permafrost, the remove and replace alternative can be expensive if the ice-rich layer is thick. In addition, the quantity of NFS material required will increase the costs. In continuous permafrost, this alternative is often used. The only problem with this method is to find NFS material in northern areas.

After a long-term evaluation of insulated roads and airfields in Alaska, Esch (1994) has rejected the use of polyurethane for subgrades while extruded polystyrene is preferred based on its superior performance longevity. Expanded or molded polystyrene can give acceptable performance if placed at a thickness 30 to 50% greater than the extruded polystyrenes to provide comparable performance. As a result, the use of expanded polystyrene will require more filling material and will increase the construction costs (Esch, 1994).

It is economically advantageous in colder permafrost areas to use polystyrene where frozen state can be preserved. Insulation costs can be compared to those using thicker fills or sub-excavation to remove permafrost layer (Esch, 1996). In warmer permafrost regions, roads and runways are always subject to permafrost degradation even if the road is insulated. Therefore, polystyrene insulation is only used in severe problem areas and is extended laterally beneath the slopes nearly to the toe of fill location to retard the permafrost warming. For airports, installation of insulation can be costly and a disruption of air traffic has to be planned for at least one summer. To sum up, polystyrene insulation seems to be a good cost-effective alternative. Polystyrene insulation can be also combined with other methods such as thermosyphons to increase the cooling rate of the ground.

Methods designed to provide shade on the side-slopes to avoid talik formation and to increase the albedo are not cost-effective. Sunsheds or snowsheds require high maintenance and can cause safety hazards to road users and aircrafts. Painted surfaces can also cause safety hazards and require repainting which increases costs over the years. Light-couloured aggregate surfaces were also a solution. The problem with this alternative is to find competent light aggregates that will resist to wearing. Where with hard rocks such as quartzite and diorite can be found, an attempt should be made to try light aggregate surfaces. This last alternative can reduce maintenance and then, costs.

Berms can retard the formation of taliks under side-slopes, but are not a long-term alternative. This method can have better results if combined with other methods. For example, in Loftus road extension, berms were used with ACE to prevent rotation due to movements of embankment side slopes. Silty berms should be avoided but rock berms, that can be compared to ACE berms, should be used. The only problem with those berms is to find competent rocks. This method is less expensive than thermosyphons.

Thermosyphons are an excellent alternative, but they are really costly. For example, the use of thermosyphons in a highway near Bethel, Alaska costs 500000$/km (Esch, 1996). Thermosyphons have to be installed only in potential severe permafrost degradation areas.

Air ducts is another good solution that is not often used. It is a cost-effective solution compared to thermosyphons. Air ducts can cover much longer a distance with a pipe of 30 to 100 m in length compared thermosyphons of 15 to 40 m in length. Also, ducts are buried under the ground, so they are not subjected to safety hazards and are easier to install. The main problem with air ducts is that the water can seep into it. An attempt should also been done to try culverts as an air duct system such as the Qinghai-Tibet railway.

Pre-thawing is a cost-effective method if time is available. Because settlements have occurred, it is possible to increase the life of the embankment. Pre-thawing is a permanent treatment that do not involved high construction costs and maintenance after construction.

ACE, a new technique, seems to give good results. The only problem is to find competent rocks. ACE are costly because of the crushing in quarry needed to get the rocks the wanted size. On the other hand, ACE is also a technique that not do require maintenance after construction. It increases the life of the embankment. Also, ACE takes less time to build than a conventionnal embankment. Rocks are brought to the site by loaders and placed in layer of 0.6 m at the same time. No other compaction is required.

Thus, the end of winter seems to be the best time to establish mitigation methods. The soil is not too frozen and prevents the degradation of vegetation. Also, air temperatures are not too warm and construction at this time will help to prevent permafrost degradation.

Cost effectiveness

ADOTPF has various repair options : to reconstruct completely a road, to perform remediation that may last for seven years, to repair a road that may last 3 or 5 years (Cole, Colonell, Esch, 1999).. For example, the reconstruction of Chena Hot Springs Road east of Fairbanks for 7 miles costs more than $1M per mile for about $7M. The use of two mitigation methods (thermosyphons and geomembrane) had increased the project costs of $5M. This construction is expected to last 10-15 years. Annual cost of road repair in the Interior of Alaska (area with discontinuous permafrost) is budgeted at $15M. Travel restrictions due to bad roads have been calculated to cost to the trucking industry $0.5M per year (Cole, Colonell, Esch, 1999). It is possible to see now how it is important to plan the cost of a potential global warming on roads in discontinuous permafrost area.

A last example is the Loftus Road extension that is now on construction. According to ADOTPF (Saboundjian, personal communication, 2003), 150 thermosyphons (75 on each side) were used. Each thermosyphon costs 3000$/piece for a total of 450 000$. Space between the thermosyphons was 4.8 m between each of them. So, the thermosyphons cover 360 m. There are 75 thermosyphons multiplied by a space of 4.8 m equal 360 m. For the ACE, 4400 tons (2200 m^3) of rocks were placed. The cost was 25$/ton. ACE were used over a length of about 500 m. The estimated final project cost is about 11M$ (Goering, personal communication, 2003).

The next table is showing mitigation methods costs. It was possible to obtain these costs from the ADOTPF. These costs are sometimes estimation costs. The main goal here with these costs is to figure out the different mitigation methods costs. Some data and prices are missing. It was difficult to obtain the costs of the different methods discussed above. ADOTPF do not collect these data (Connor, personal communication, 2003). It is possible to see that thermosyphons are really expensive and that geogrids are more expensive than geotextile. Also, the material required to build the ACE was costly.

Table 2-1: Mitigation Methods Cost Comparison

	Year	Project	Total Distance of Repairs	Distance used	Quantity	Unit Price	Unit Total Cost	Project Total Cost
Embankement thickenning								
Insulation (extruded polystyrene)	Bid Year 1997	Chena Hot Spring Road	7.65 km		88 MBM	380 $/MBM	33 400 $	≈ 10M $
Sunshed								
Reflective surface								
Air ducts								
Thermosyphons	Bid Year 2002	Geist Road			150	3000 $/piece	450 000 $	≈ 11M $
Thermosyphons	In service 2004	Loftus Road		360 m	150	3000 $/piece	450 000 $	Estimated Cost ≈ 11M $
ACE	In service 2004	Loftus Road		500 m	4 440 tons (US)	25 $/ton (US)	110 000 $	Estimated Cost ≈ 11M $
Geotextile	Bid year 2002	Glenn Highway Caribou Creek	15 km		28 904 m^2	1.00 $\$/m^2$	28 904 $	≈ 35M $
Geogrid	Bid year 2002	Glenn Highway Caribou Creek	15 km		92 230 m^2	2.50 $\$/m^2$	230 575 $	≈ 35M $
Berm								
Pre-thawing								

* All the costs are in American Dollar

CONCLUSION

Since thirty years, it is possible to conclude from many experts around the world that a global climate warming is happening. It seems that continuous permafrost will not undergo under immediate changes, but the discontinuous permafrost boundary zone will move northward causing engineering construction problems. In order to prevent permafrost degradation and settlement problems in these areas, several alternatives were reviewed. As a result, some methods seem to be better than others based on effectiveness, cost, safety, high maintenance. Polystyrene insulation, geotextile, pre-thawing, ACE and air ducts seem to be the best methods according to the previous enumerated factors. Pre-thawing is definitely a cost effective method. A good way to perform pre-thawing is to build a gravel road or a gravel runway to let the settlement occur. After few years, the road or the runway can be paved. Thermosyphons are also an excellent mitigation method, but they should be used only in severe permafrost degradation areas. Two or three methods should be tried out such as ventilated embankments with parallel culverts, hard light-colored aggregate surfaces and the removal of snow along the shoulder and side slopes during winter and spring. Finally, cold regions have to expect engineering construction problems due to a global climate warming over the next years.

3 Analysis of the thermal regime at the Beaver Creek station

Thermal monitoring of the embankment and underlining permafrost has been performed at the Beaver Creek Station since 1996. The station is located at km 1928 near Beaver Creek (figure 3.1). Temperature probes were installed at variable spacing down to a depth of 10 m under the center of the pavement, under the berm and in the natural ground at the toe of the embankment slope. Details on the installation of the thermistor strings as well as information on boreholes and soil analysis can be found in the installation report provided in appendix 1. Temperature data have been recorded at 60 min. interval by an automatic data acquisition system located near the road embankment.

Figure 3-1: Temperature monitoring station on the Alaska Highway near Beaver Creek

The specific objective of this research activity was to assess the thermal regime of the road embankment at Beaver Creek and it's evolution over the 7 year monitoring period. Data collected at the site by Yukon Highways and Public Works has been analysed to assess the depth of the active layer and its evolution since the installation of the station in the road embankment considered to be typical of the Alaska Highway.

The following analysis was done for each of the three thermistor strings:

- Assessment of the range of temperature variations with depth during a year (whiplash curves)
- Assessment of temperature variations (yearly minimum, maximum and average) over the monitoring periods for selected depths
- Assessment of the evolution of thaw depth over the analysis period at the three monitoring locations

3.1 Climatic data

In order to fully assess the significance of temperature variations into the embankment and into the subgrade soil, climatic data from the Beaver-Creek station of Environment Canada were compiled for the monitoring period (1997-2003). Figure 3.2 and 3.3 illustrate the variations in mean annual temperature as well as in freezing and thawing indices over the seven-year monitoring period. The figures also include 30-years normals for the parameters considered.

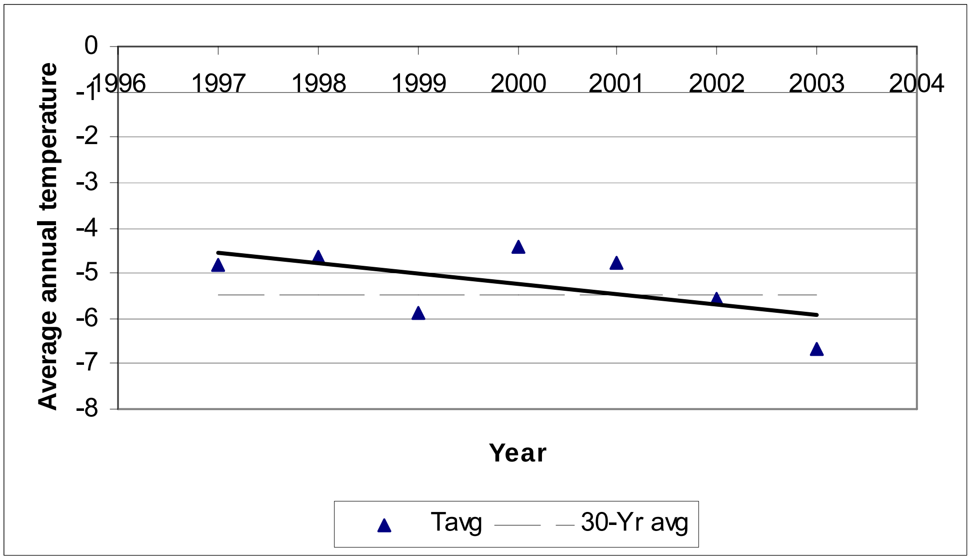


Figure 3-2: Variations of mean annual temperature with time at the Beaver Creek climatic station

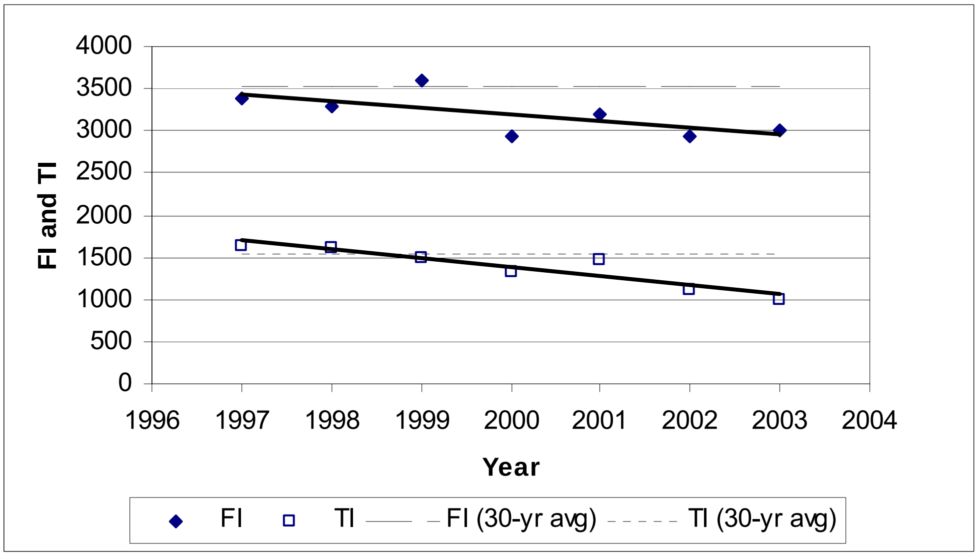
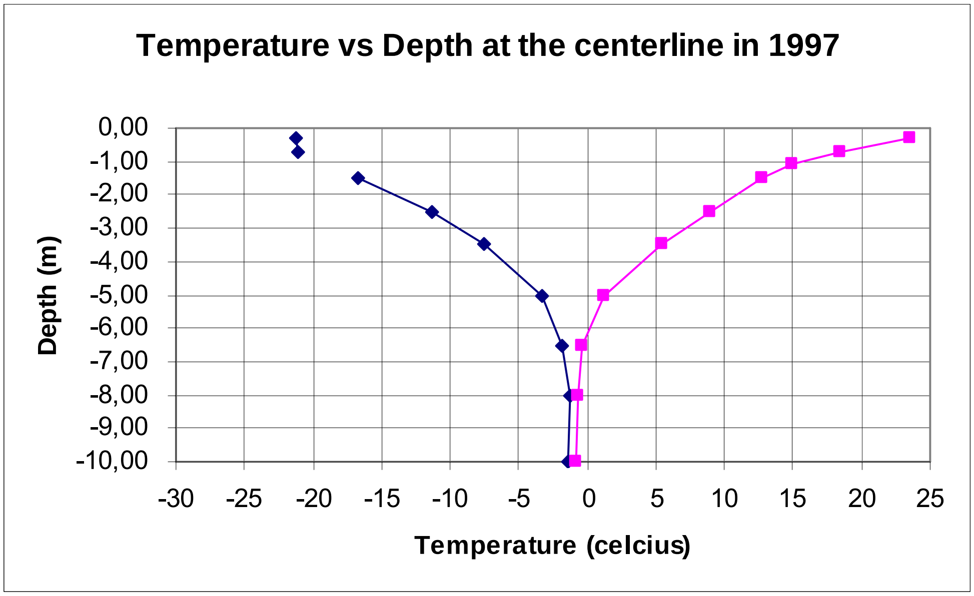


Figure 3-3: Variations of freezing and thawing indices with time at the Beaver Creek climatic station

As shown on figure 2.2, mean annual temperature is in accordance with the 30-years normal but is clearly decreasing over the monitoring period at an average rate of 0,23°C/year. The freezing and the thawing indices are defined as the summation of mean daily temperatures below 0°C (FI) and above 0°C (TI) respectively. Since only monthly averages were available for the Beaver Creek station, Freezing and Thawing indices were estimated from these data. Both indices are decreasing during the monitoring period. The thawing index is consistent with the 30-year normal but is decreasing at an average rate of 105°Cday/year. The decrease appear to be consistent with the decrease of the mean annual temperature. The freezing index is slightly below the 30-year normal and is decreasing at an average rate of 79°Cday/year over the monitoring period. This decrease is not consistent with the decrease in the mean annual temperature. It can probably be explained by a reduction of the amplitude of the annual temperature variation during the monitoring period.

3.2 Assessment of the temperature variations with depth

The domain of temperature variations with depth for each thermistor string and each year of the monitoring period was established by plotting the minimum and the maximum temperatures recorded at each thermistor for a given year. The resulting figures, known as “Wiplash curves” provide useful information on the temperature regimes prevailing at the different monitoring locations and their evolution during the monitoring period. These curves are illustrated in figures 3.4 to 3.6.



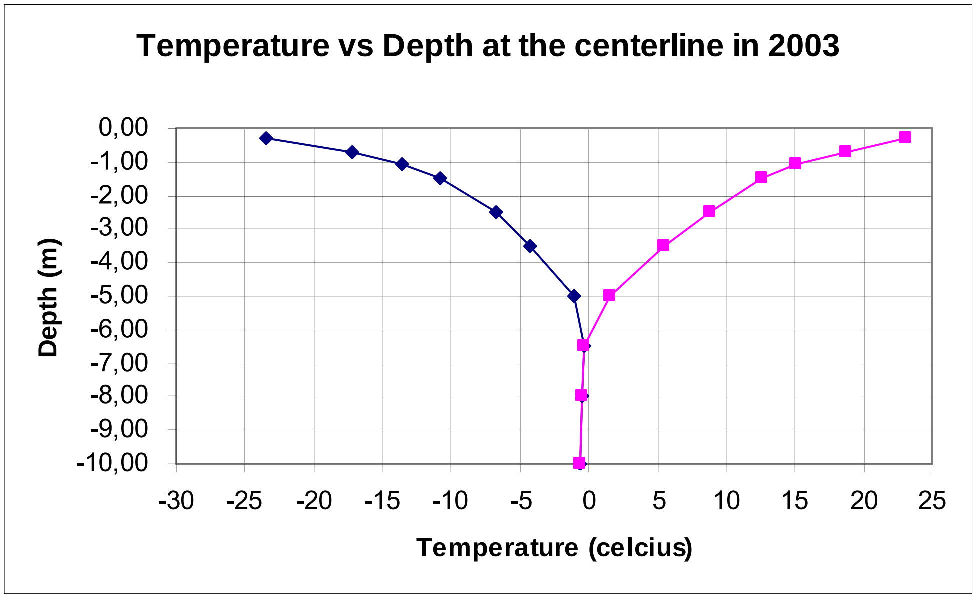
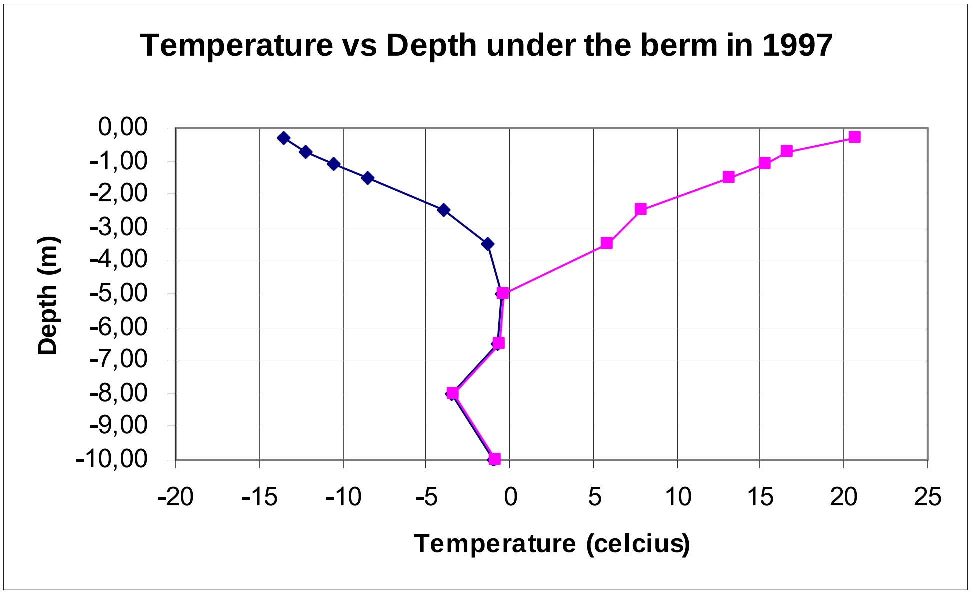


Figure 3-4: Temperature variation with depth at the centerline of the embankment at the beginning and at the end of the monitoring period

Figure 3.4 shows that the temperature swing at the surface of the embankment is relatively constant over the analysis period. It is possible to note a slight increase of the thaw depth and a slight increase of the temperatures deep under the surface (8-10m) in 2003 as compared 1997.



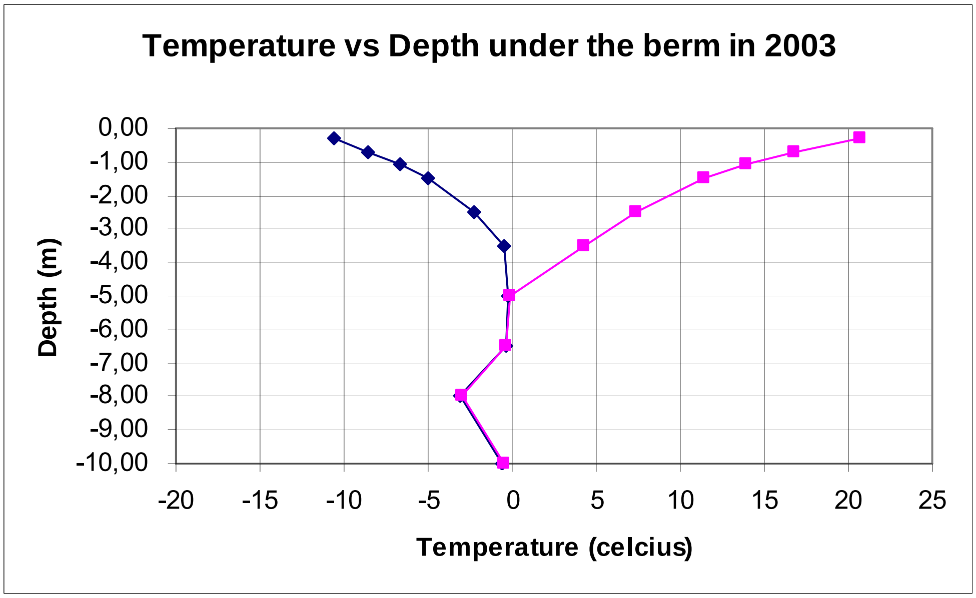


Figure 3-5: Temperature variation with depth under the berm of the embankment at the beginning and at the end of the monitoring period

Conditions in terms of temperature variations appear to be relatively stable under the berm. Despite the fact that the cold temperature swing is significantly higher in 1997 as compared to 2003, thaw depth appear to be stable. A slight increase of temperatures at large depths under the berm (6,5m and 10m) surface can however be noted between 1997 and 2003. The figures also indicate an offset in temperature measurements of approximately -3°C at 8m. It is clear that the thermistor at the depth 8m is not reliable.

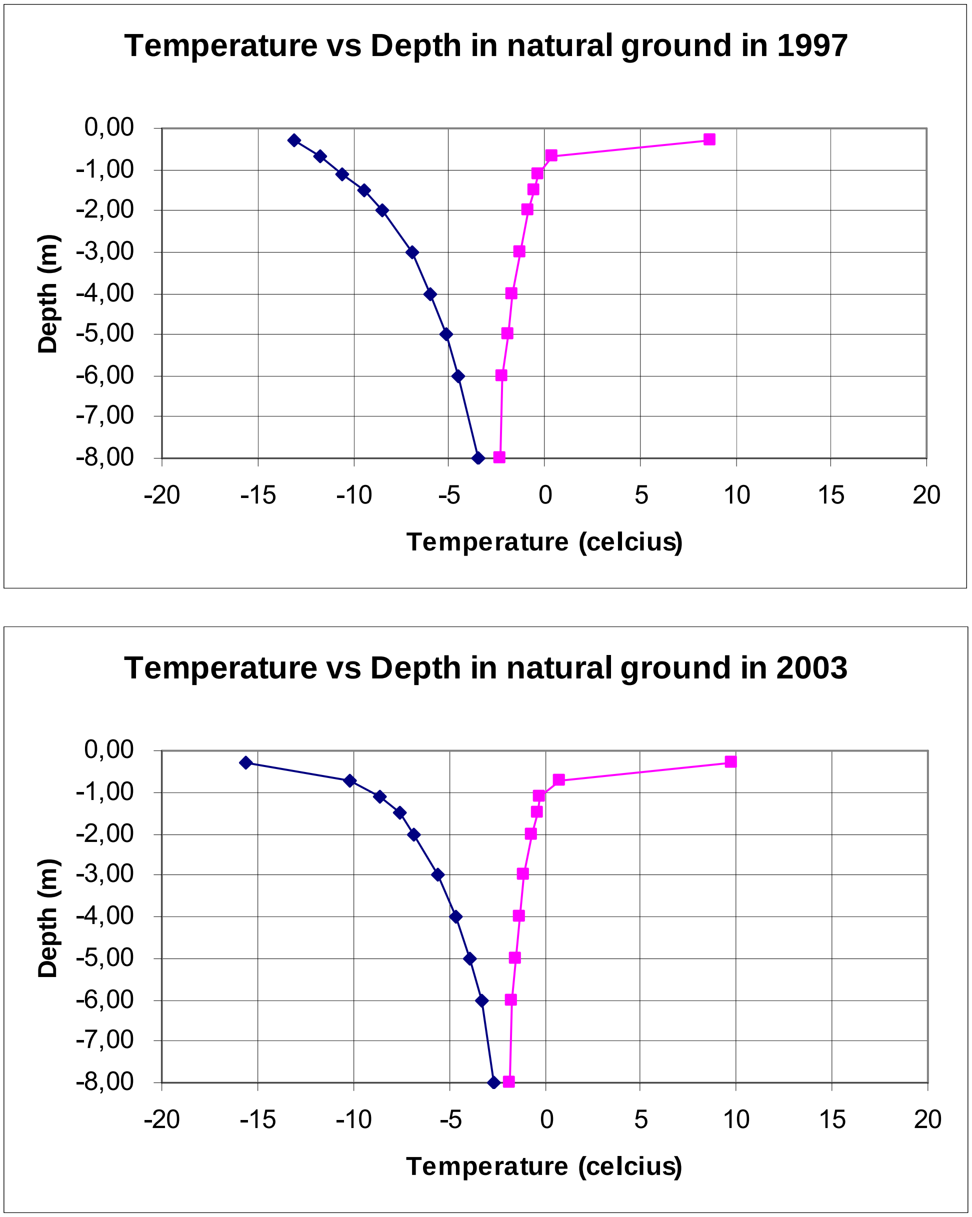


Figure 3-6: Temperature variation with depth in the natural ground at the beginning and at the end of the monitoring period

It is interesting to note that in the natural ground, the cold temperature swing near the surface is reaching colder temperatures in 2003. A slight increase in thaw depth and in temperatures at 8m depth can also be noted in the natural ground between 1997 and 2003.

When comparing thermal regimes at the three monitoring locations, it is interesting to note the decrease of the surface temperature swing going from the center of the embankment towards the natural ground. The presence of a snow cover over the berm and the natural ground as well as the presence of moss over the natural ground are responsible for the decrease. It is also interesting to note that the depth to stable temperature conditions are much more important in the natural ground (>8m) than in the road embankment (5-6m). The damping effect of snow and moss cover over the natural ground is probably responsible for that phenomena.

3.3 Assessment of temperature variations over the analysis periods for selected depths

In order to assess the variations of temperatures over the 7-years observation period, three types of analysis were conducted using data from thermistors located near the surface, at the interface between the embankment and natural soil and 2m-deep into the subgrade soil for all three locations (figure 3.7). The first analysis was done using the average annual temperature, the second using the maximum yearly temperature and the third one using the minimum yearly temperature.

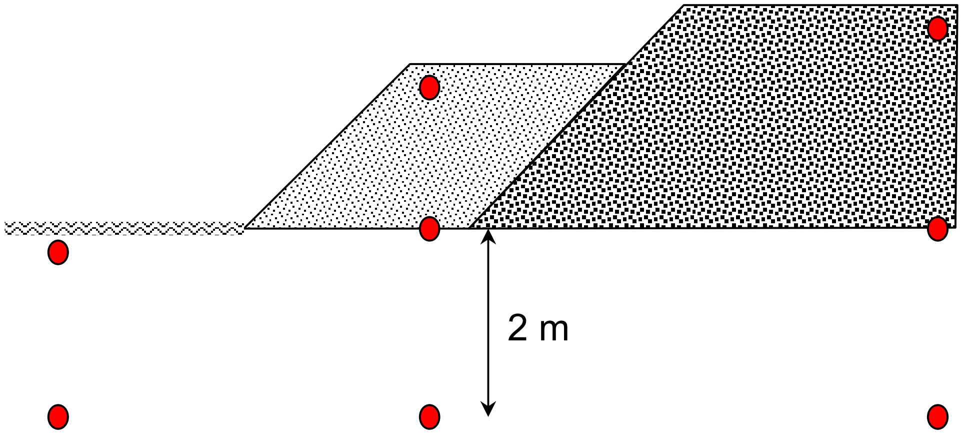


Figure 3-7: Thermistors used for the long-term temperature analysis

Figure 3.8 shows the temperature variations at the centerline of the embankment. Data collected over the last 7 years clearly indicate an increase in the embankment temperature. The average yearly increase in temperature is 0,29°C near the surface, 0,10°C at the interface between the embankment and the natural soil and 0,07°C at a depth of 2 m under the interface.

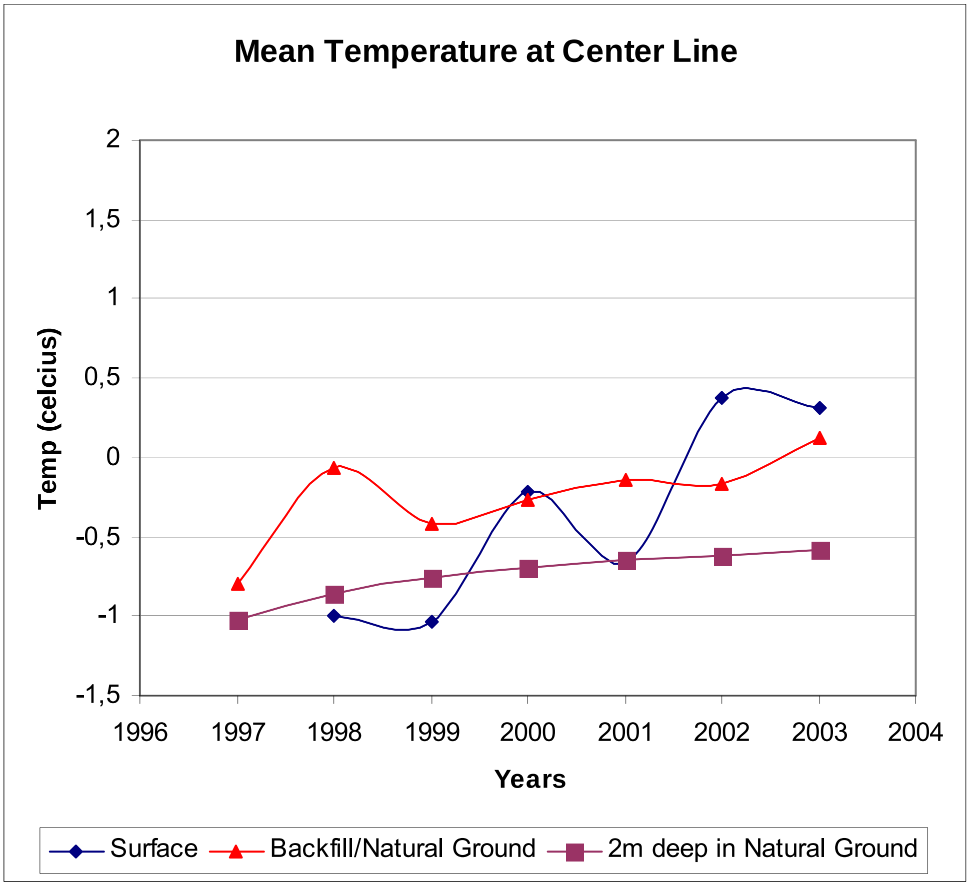


Figure 3-8: Mean annual temperature under the center of the embankment

The same trend can be observed underneath the berm. As indicated on figure 3.9, temperature near the surface of the berm are generally increasing at an average rate of 0,38°C/year. The trend at the interface between the berm and the subgrade soil is not as clear. Over the 7 years of the monitoring period, the temperatures appear to be stable. However, since 1999, there is a clear increase of temperatures at an average rate of 0,28°C/year. The subgrade soils at 2m depth are also warming up at a constant rate of about 0,04°C/year approaching the critical temperature of 0°C at the end of the monitoring period.

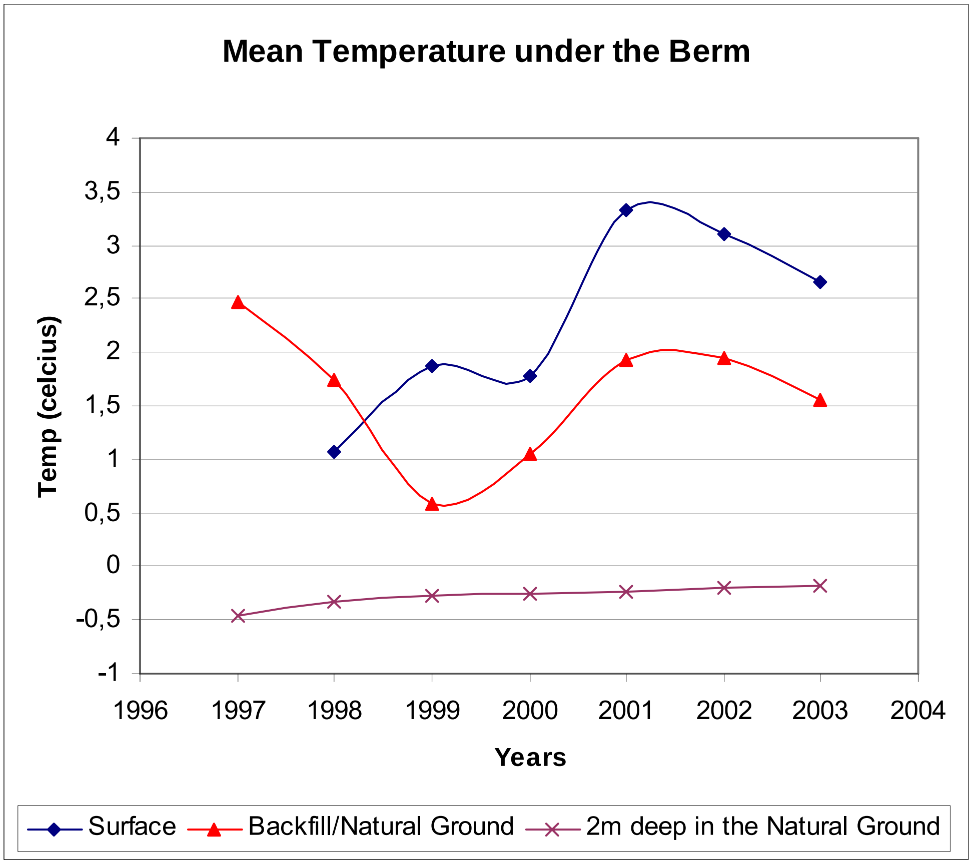


Figure 3-9: Mean annual temperature under the berm

As shown in figure 3.10, temperatures in the natural soil appear to be somewhat stable over the monitoring period. In fact, temperatures near the surface seem to be decreasing at an average rate of -0,11°C/year while temperatures at 2m-depth in the soil are stable with a slight average increase of 0,01°C/year.

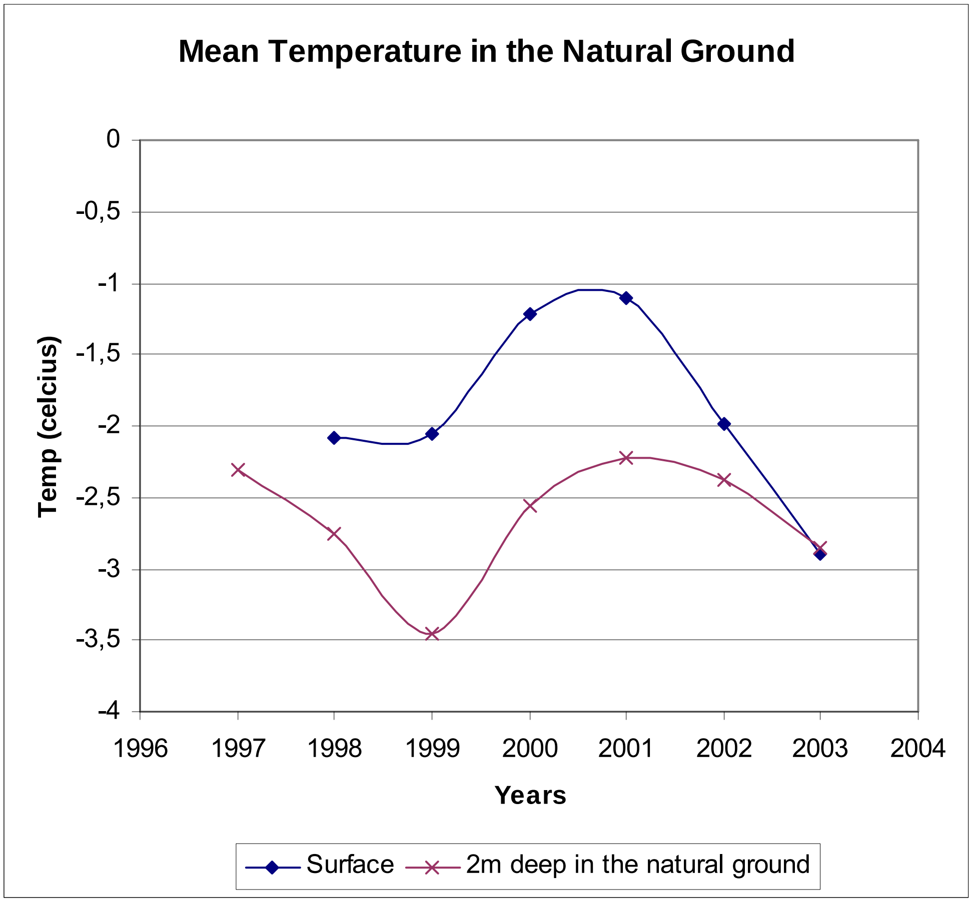


Figure 3-10: Mean annual temperature in the natural soil

Table 3.1 summarizes the trends observed at the different analysis points.

Table 3-1: Average temperature increase at selected locations in the embankment and in the natural ground beside the embankment (re: figure 3.7)

	Average temperature increase in °C/year		
Level	Centerline	Berm	Natural ground
Near surface	0,29	0,38	-0,11
Interface Embankment-soil	0,10	0,28*	
2m-deep in natural soil	0,07	0,04	0,01

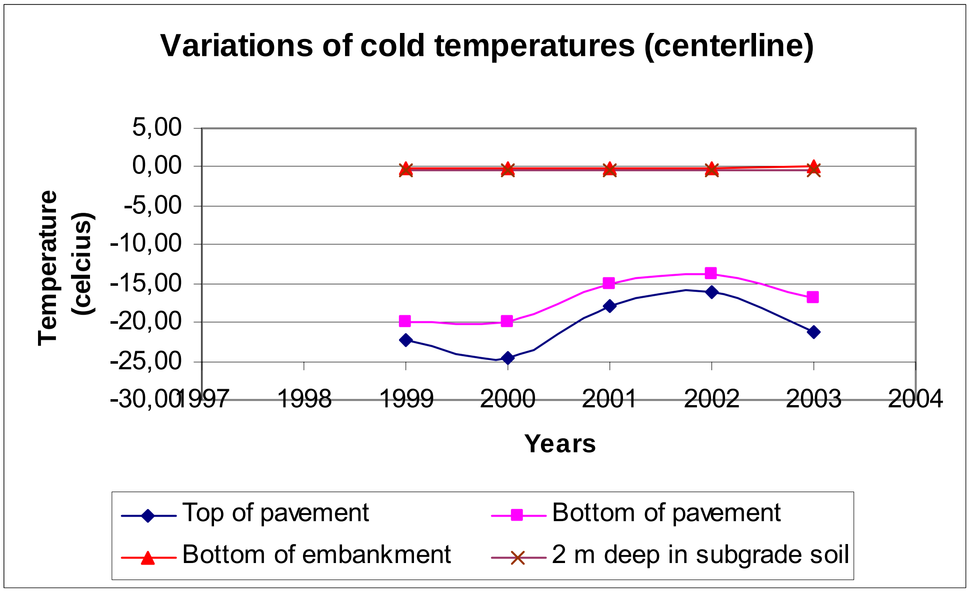
* Considering data from 1999 to 2003

From the analysis done on yearly average temperature data over the monitoring period, the following observations can be made:

- Temperatures appear to be stable in the natural ground over the monitoring period.
- Temperatures at the centerline and under the berm of the embankment are increasing steadily over the monitoring period. Mean annual temperature at the interface fill-natural ground beneath a berm is above 0C which shows that permafrost is degrading beneath the embankment.
- Temperature increase trends vary from 0,04 and 0,38°C/year in the road embankment.
- Temperature increase trends are more important near the surface of the embankment.
- Temperatures at 2-m depth in the natural soil under the embankment are increasing between 0,04 and 0,07°C/year.

- The monitoring period has been characterized by decrease in the mean annual air temperature. Permafrost degradation will increase with time if the short-term trend in air temperature changes.

A similar analysis has been done for the maximum and the minimum temperatures recorded every year of the monitoring period. This analysis highlights the evolution of warm and temperatures in the subgrade soil relative to the critical 0°C temperature. The results are shown in figures 3.11 to 3.13



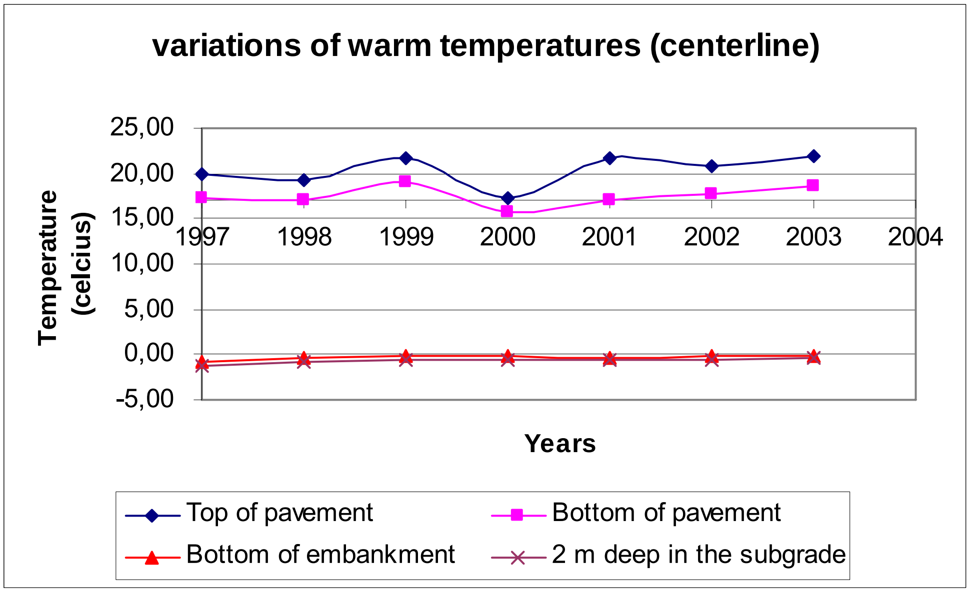
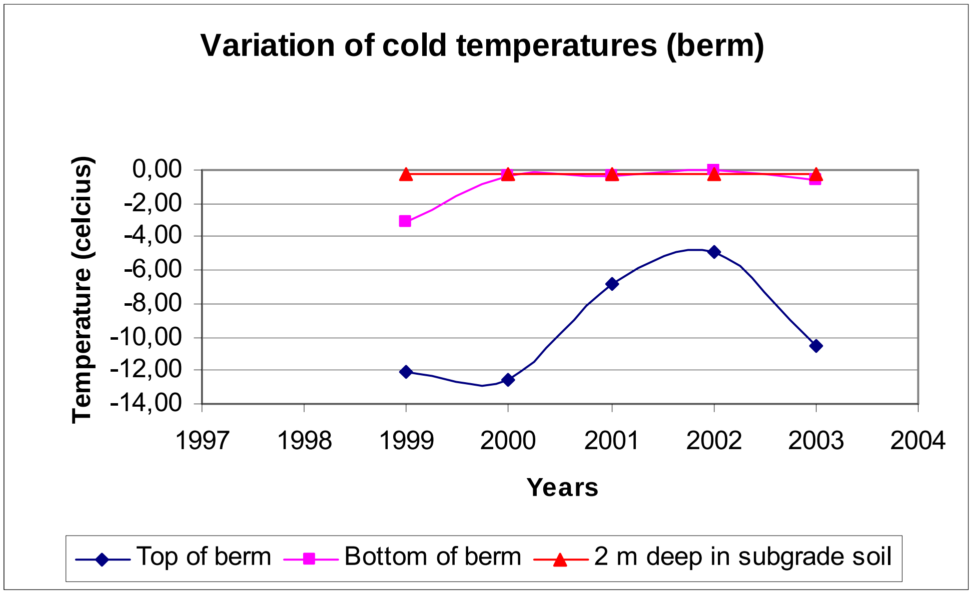


Figure 3-11: Variations of cold and warm temperatures at the centerline of the embankment over the monitoring period



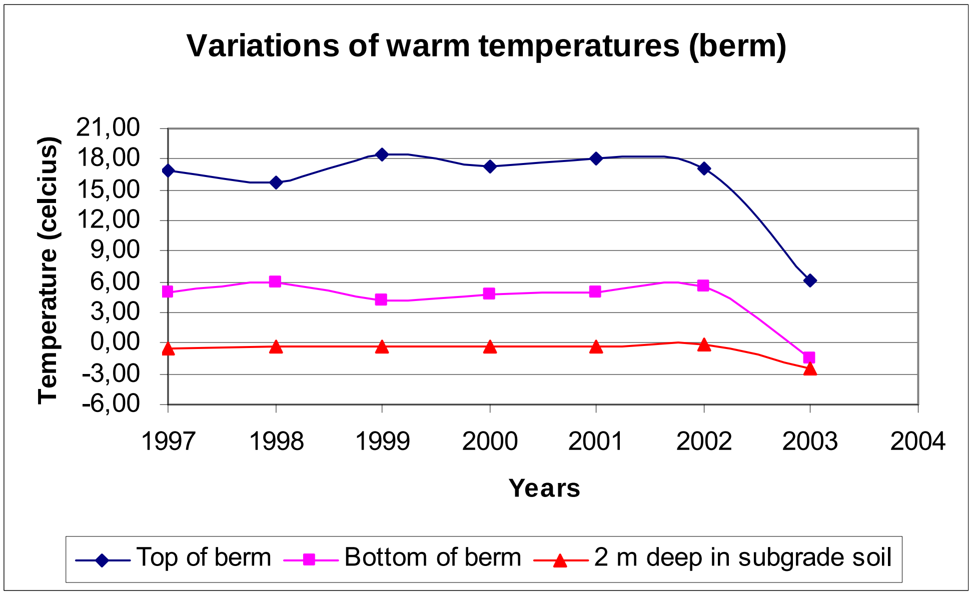
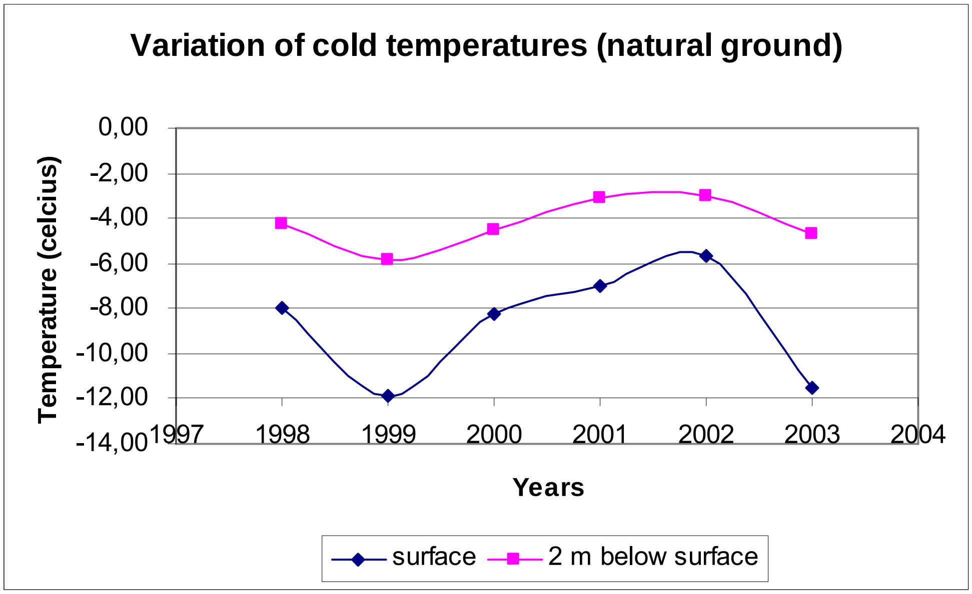


Figure 3-12: Variations of cold and warm temperatures under the berm of the embankment over the monitoring period



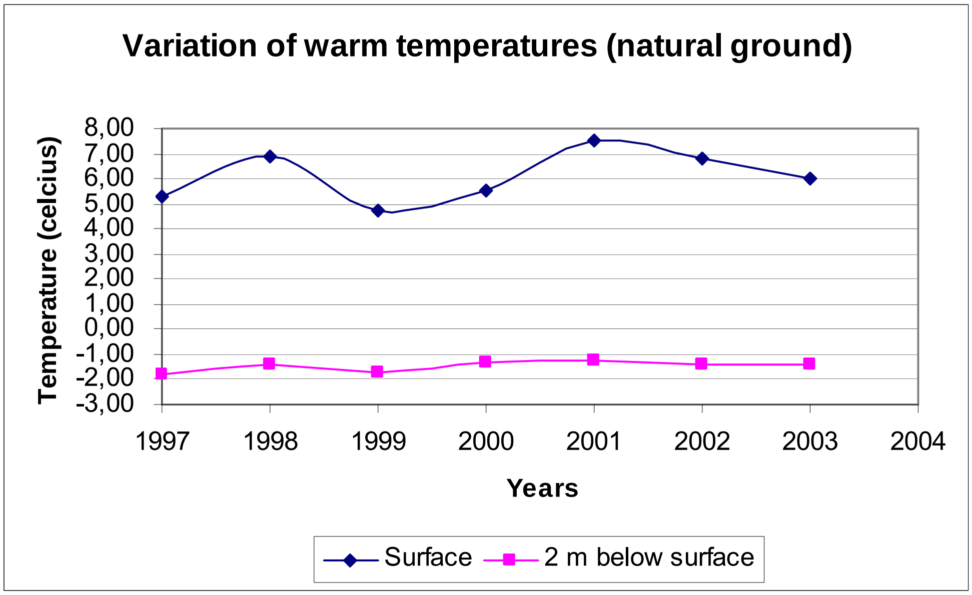


Figure 3-13: Variations of cold and warm temperatures in the natural ground over the monitoring period

The trends observed in figures 3.11 to 3.13 are summarized in table 3.2.

Table 3-2: Cold and warm temperatures increase at selected locations in the embankment and in the natural ground beside the embankment (re: figure 3.7)

	Average temperature increase in °C/year					
Level	Centerline		Berm		Natural ground	
	Cold	Warm	Cold	Warm	Cold	Warm
Near surface	1,06	0,33	1,08	-1,08 (0,18)*	0,21	0,17
Interface Embankment-soil	1,22	0,08	0,54	-0,68 (0,02)*		
2m-deep in natural soil	0,02	0,12	0,03	-0,20 (0,05)*	0,05	0,06

* Excluding 2003 data

From the analysis done on cold and warm temperature data over the monitoring period, the following observations can be made:

- Warm temperatures measured under the berm in 2003 are suspect. The variations observed between 2002 and 2003 appear to be much too large to be considered normal variations. Considering 2003 data in the trend analysis, an important reduction of warm temperatures under the berm is observed over the monitoring period. Excluding 2003 data, a slight increase is observed.
- Excluding 2003 data for warm temperatures under the berm, all cases analysed show an increase of temperature over the monitoring period.

- Temperatures at 2-m depth in the natural soil are increasing between 0,02 and 0,12°C/year.

3.4 Assessment of the evolution of thaw depth over the analysis period at the three monitoring locations

Thaw depth was estimated by linear interpolation of temperature data near 0°C using the warm temperature swing from the analysis described in section 3.2. The result of the analysis is shown in figure 3.14.

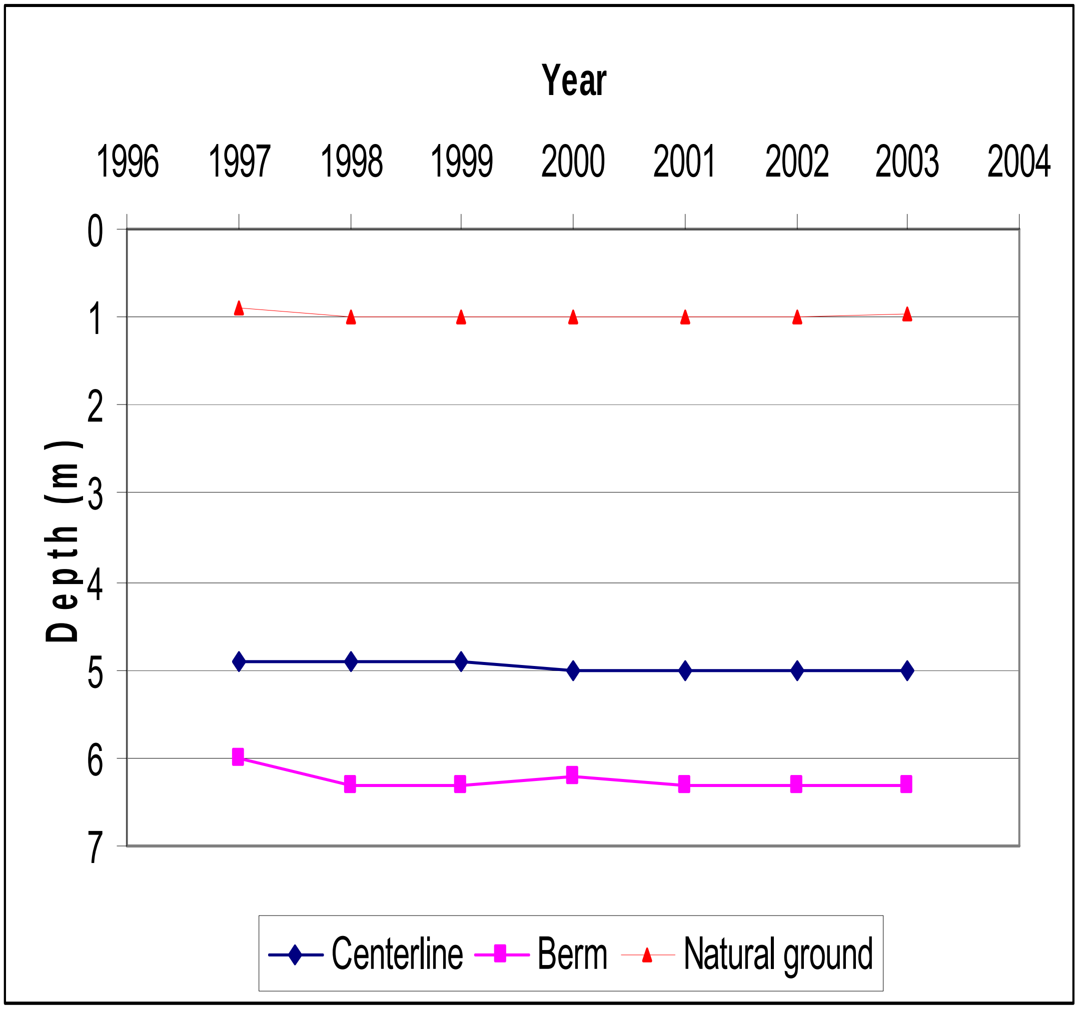


Figure 3-14: Evolution of thaw depth during the monitoring period

The analysis shows a significant increase in thaw depth between 1997 and 2003 at all monitoring locations. The increase trends are 5 mm/year in the natural ground, 21 mm/year at the centerline of the embankment and 32 mm/year under the berm. The following additional observations can be made from the analysis:

- Thaw depth is significantly greater under the berm as compared to the centerline of the pavement. Considering that the surface of the berm is approximately 3 m lower than the surface of the centerline, thaw depth seem to be 4 m deeper under the berm as compared to the centerline of the embankment.
- Thaw penetration rate is 50% higher under the berm as compared to the centerline of the embankment.

3.5 Conclusion

The thermal analysis of the Beaver Creek embankment leads to the following conclusions:

- Despite a cooling trend of average air temperatures over the 7-year monitoring period, all thermal indicators investigated show a degradation of the thermal regime in the road embankment at the Beaver Creek station.
- Thermal degradation is occurring under road embankments in the conditions prevailing in Beaver Creek without the effect of global warming. Construction of roads on warm permafrost using current construction practice in Yukon appears to induce thawing of the underlying ice-rich ground. The degradation is likely to be amplified by a temperature warming trend.

- Thermal degradation appear to be more extensive underneath the berm suggesting that gravel berms such as the one used at Beaver Creek do not provide adequate thermal protection to the road embankment.

4 Development of adaptation scenarios

The literature review has allowed for the identification of deterioration mechanisms prevailing in pavements built over unstable permafrost. It has also helped identifying possible cost effective adaptation techniques to prevent further degradation of the permafrost under transportation infrastructures. The thermal analysis of the Alaska Highway monitoring site at Beaver Creek has shown that despite a clear trend of decreasing air temperature over the monitoring period, soil temperatures beneath the embankment and the berm of Alaska Highway have increased steadily and significantly. A global strategy for the protection of new and existing pavements built on instable permafrost is therefore proposed in this section of the report.

Based on the available information and on the current state of practice in Yukon, cost effective adaptation scenarios including pavement design, construction and management practice, will be developed. The technical and cost implications of the recommended actions as well as the expected benefit will be documented.

The leading principle of the proposed strategy is thermal stabilization. This principle can be compared to the mechanical stabilization of an embankment built over compressible soils. In that specific case, mechanical stabilization is achieved by overconsolidation; in other words by subjecting the soil to a level of stress higher than the level of stress expected over the life of the embankment. The proposed strategy is thus based on the following approach:

a) Design highway embankments in order to achieve a net positive heat balance at the interface between the embankment and the subgrade soil. This will cause the thickness of the active layer to increase (pre-thawing) inducing thus consolidation and mechanical stabililization of the thawed soil (path “a” on figure 4.1)

b) Identify and maintain sections built on instable permafrost for a period of 3 to 5 years

c) Stabilize thermally the unstable sections using appropriate protection techniques (path “b” in figure 4.1). Properly designed thermal stabilization will reduce the thickness of the active layer at a level such that the maximum thickness reached during the pre-thawing step will never be reached during the life of the pavement.

When applied to existing roads, the approach can be modified as follows:

a) Identify and maintain sections built on instable permafrost for a period of 3 to 5 years. Maintenance techniques should include the use of surfacing materials favouring heat intake into the pavement embankment (dark mixes or surface treatments).

b) Stabilize thermally unstable sections.

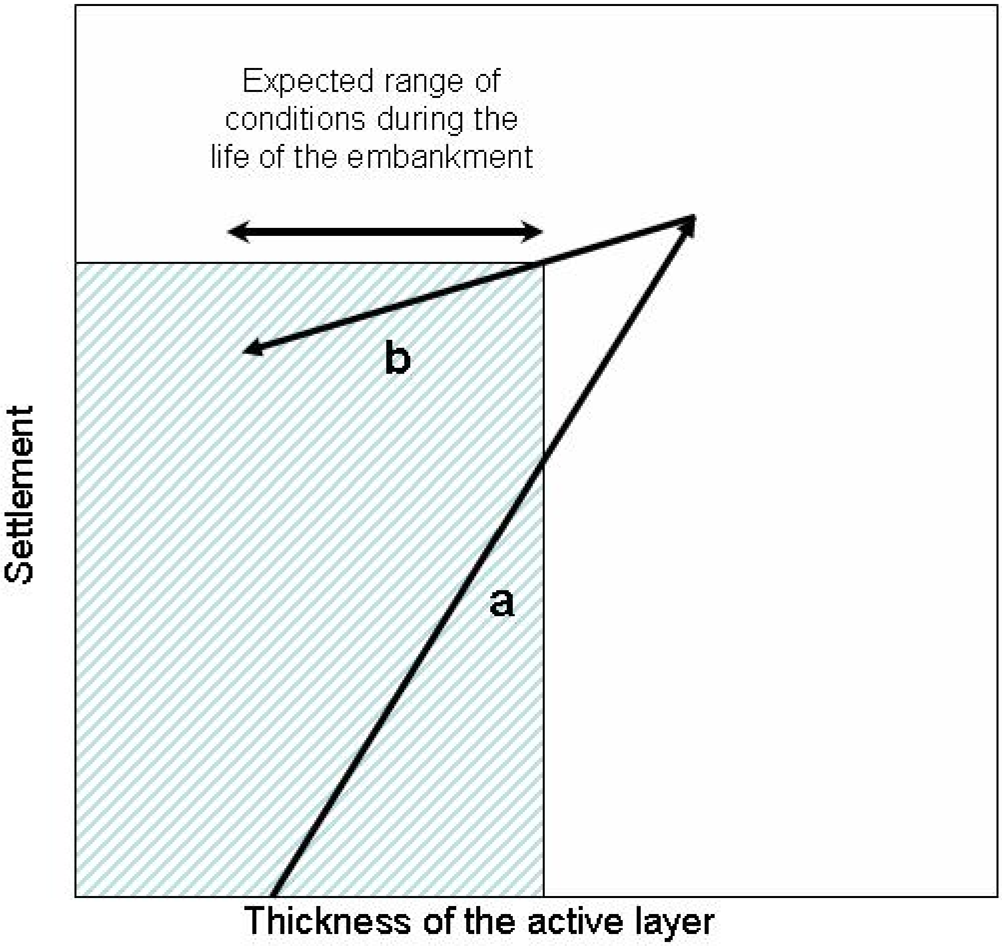


Figure 4-1: Principle of the thermal stabilization approach

The following techniques are proposed for consideration for the thermal stabilization of highway embankments :

- Air convection embankments
- Heat drains
- Reflective surfaces

Air convection embankments (ACE)

As described in section 2 of the report, the air convection embankment is an effective technique for heat extraction from a road embankment. ACE can either be used as a layer of the embankment in order to extract heat from underneath the whole embankment or it can also be used underneath the shoulder (figure 4.2) in order to extract heat from beneath the embankment slope. In the first case, ACE can only be used in new or completely reconstructed embankments.

ACE has proven to be effective and is relatively inexpensive in areas where competent rock is available. ACE materials can readily be found in quarries or gravel pits where retained materials in screening operations are rejected. Ideal ACE material is composed of 150-300mm angular rocks. Among other advantages of the ACE are the easiness of construction and the low level of maintenance required. When used in the embankment slopes, however, ACE can cause some safety concerns.

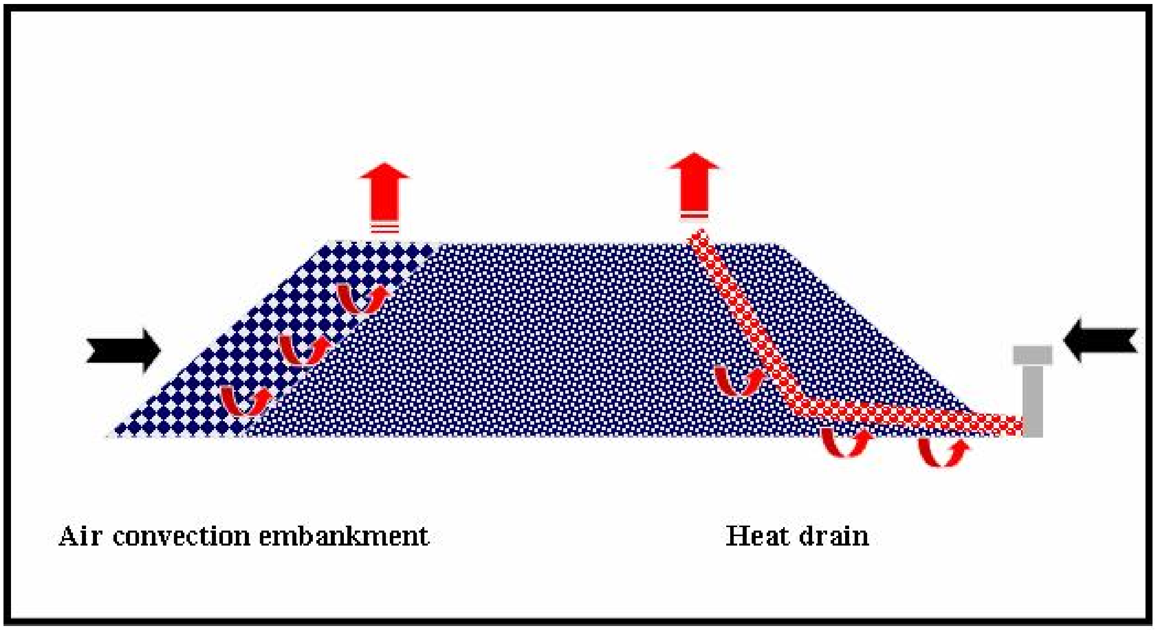


Figure 4-2: Air convection embankment and heat drains

Heat drain

Heat drain is a new technique currently being developed at Laval University. The technique allows for heat extraction from the embankment during winter. The heat drain technique is based on the use of a high permeability geocomposite placed underneath the embankment slope (Figure 4.2). It is expected that the heat drain will create a chimney effect which will facilitate heat extraction by convection. The effectiveness of the system is currently being tested on a small scale model at Laval University. The technique is easy to install and requires little maintenance. It will require transportation of the geocomposite to the site which will increase the cost of the technique and might make it less cost effective than ACE in areas where rock is available. Heat drains do not cause any safety concerns.

Reflective surfaces

This technique can be used to reduce heat intake by absorption of solar radiation. Reflective surfaces based on the use of white painting applied on pavement surfaces have been tested in Alaska. Figure 4.3 shows an example of the effectiveness of a reflective surface tested by CRREL in Fairbanks some 30 years ago (see figure 2.10). On the picture, it can be observed that the reference section (asphalt concrete without protection) located beyond the treated surface has experienced considerable settlement and is now transformed into a swamp. The treated surface is still in relatively good condition after 30 years of exposition to solar radiations during summer.

Figure 4-3: Reflective surface test section after 30 years at the CRREL, Farmer’s Loop experimental site, Fairbanks, Alaska

The technique has been tested in several experimental projects and has proven to be effective in reduction the depth of thaw penetration. It is however not currently used due to safety concerns (low skid resistance) and intensive maintenance required.

It is thus suggested to consider the use of a reflective surface using recently developed pale asphaltic materials in combination with light colored aggregates. One commercially available product has been developed by the Shell cie. in response to an increasing demand in Europe for coloured asphalt products (figure 4.4). These products are used, among other applications, to differentiate pedestrian and bicycle paths and for aesthetic urban designs. The product is rather expensive (typically about 10 times more expensive for the asphaltic binder), but considering a targeted application on limited road surfaces, it might prove to be cost effective on a long-term basis.

Figure 4-4: Coloured asphalt products used in Europe (Mexphalte C, patented by Shell)

This type of pigmented asphalt can be used in hot-mix asphalts, stone-mastic asphalts, cold mixes and surface treatments.

Practical considerations

The adaptation approach and techniques proposed in this section are believed to have a high potential of success in the Yukon environment. Overall, the general approach will lead to cost effective investments on sections affected by degrading permafrost.

By constructing “thermally underdesigned” pavements, considerable savings will be made at the construction stage. The fact that the approach will facilitate the identification of problem area will also lead to savings with respect to site investigation. More investments will however be required for the maintenance and the thermal stabilization of the problem areas.

Research and development work is however required to establish criteria and guidelines for thermally underdesigned and thermally stable pavements in the Yukon context.

5 Conclusion

The analysis of temperature data from the Beaver Creek test section is clearly indicating that the embankment is thermally unstable. Despite a general cooling of air temperature during the monitoring period, clear trends of temperature increase have been observed under the centerline and under the berm of the embankment. Thermal degradation appears to be occurring faster underneath the berm than under the centerline. Considering the fact that the problem will worsen with the global warming trend observed in the north, more effective protection techniques are required to protect Yukon highway infrastructures.

The literature review has allowed identifying several protection techniques that have proven to be effective in Alaska and in other areas where transportation infrastructures are affected by degrading permafrost. These techniques are however rarely used in current practice due to cost, safety, maintenance and other practical considerations. A few promising techniques have however been identified for further consideration by Yukon Highways and Public Works. These techniques are incorporated into a general approach of thermal stabilization of highway embankments.

6 Project continuation

<u>If the results of the first phase of the project are found to be conclusive and promising</u>, Yukon Highways and Public Works might want to consider the following continuation phases:

Phase 2: Development of criteria for thaw consolidation and thermal stabilization and further investigation of the Beaver Creek test site

The phase 1 of the project has been focused on the monitoring of the thermal regime of the embankment and permafrost beneath it. Data show that permafrost slowly degrades beneath the highway. With continuous degradation ice-rich permafrost finally will be affected with sufficient thaw settlement. Our analysis of the limited borehole information show that permafrost in the area is the Pleistocene syngenetic permafrost with ice-wedges and extremely ice-rich soil between ice-wedges. Such permafrost is widely occurs in areas which were not affected by glaciations during Late Pleistocene. Similar permafrost has been extensively studied in Russia and Alaska. Participation of Alaskan knowledgeable in such permafrost trough border partnership with Alaska Department of Transportation can be productive in understanding of permafrost geological and engineering properties.

The phase 2 project could include some or all of the following activities:

- Analysis of structure and properties of the Pleistocene syngenetic permafrost underlying the highway.
- Evaluation of the potential thaw settlement.
- Development of criteria for thaw-consolidation and thermal stabilization through detailed two-dimensional simulation of the effect of proposed adaptation techniques on the thermal regime and consolidation behaviour of the road embankment
- Assessment of the performance of the Alaska Highway in the Beaver Creek area including analysis of performance data, longitudinal profile, and maintenance activities.
- Investigation of thermokarst phenomena at the Dry Creek site using teledetection techniques, geophysical techniques, boring and sampling and other relevant techniques
- Instrumentation of the Dry Creek site (thermistors and other relevant instrumentation techniques)
- Analysis of the performance of rock berms
- Laboratory study to determine the thaw-consolidation behaviour of local soils
- Long term cost-benefit analysis of adaptation scenarios

Phase 3: Field experimentation of the adaptation technique

If phase 2 project is conclusive, the following Phase 3 project should be considered:

- Construction and monitoring of an experimental road embankment to validate Phase 1 and Phase 2 research findings

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Appendix 1

Installation report