EFFECTS OF SALTS ON ROAD EMBANKMENT STABILITY UNDER FREEZING AND THAWING CONDITIONS

FINAL REPORT

By:

Thomas C. Kinney
Institute of Northern Engineering
University of Alaska-Fairbanks
Fairbanks, Alaska 99775-0660

and

Matthew Reckard
State of Alaska
Department of Transportation and Public Facilities
Research Section
2301 Peger Road
Fairbanks, Alaska 99709

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16. Abstract

A frost heave test performed on a base course material as part of an earlier study indicated that the addition of 0.1% calcium chloride reduced heave by 50%, while 0.5% reduced heave by 95%. The gravel was of a type widely used in Interior Alaska for highway base and subbase materials.

In response to this, a more detailed study was undertaken to assess the effects of salt on soil stability, principally frost heaving. Gravels from the original study source and others typical of Interior Alaska were used.

Frost heave tests were performed in the laboratory under various conditions of soil salt content, temperature, temperature gradient, and overburden pressure. CBR tests were also performed to assess thaw weakening.

Field samples were gathered from highway locations where known quantities of calcium chloride had been applied in the past. The dissipation rate of field-applied salts was examined by analyzing the salt content of these samples.

The study found that frost heave effects due to salt varied widely with soil type and freezing conditions and could not be predicted using standard soil tests. It also indicated that little salt remains in a road section several years after application. It was concluded that the use of salt is probably not a practical or economical design alternative for increasing the frost stability of embankment materials.

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EFFECTS OF SALTS ON ROAD EMBANKMENT STABILITY

1.0 INTRODUCTION

Extensive damage to pavements is observed each winter and spring due to frost heaving and subsequent thaw weakening. This problem prompted an interest in the effectiveness of a low-cost calcium chloride treatment designed to decrease the frost susceptibility of granular base and subbase materials.

A frost heave test for a recent Alaska Department of Transportation and Public Facilities (DOT&PF) Research Section project entitled "Economic Aspects of High Speed Gravel Roads," found that the addition of 0.1% calcium chloride in gravel from one Interior Alaskan material source reduced heave by 50%, while the addition of 0.5% reduced heave by 95%. Outside studies have reported that the addition of salts to certain materials may increase their bearing capacity when thawed, at least in the short term, and may reduce the compactive effort required during construction.

Still other reports indicated that salts may be driven downward in advance of the freezing front or leached out of the road profile. If this happens, there may not be a significant long-term change in the properties of the materials. Furthermore movement of the salts might create detrimental side effects such as thawing of underlying permafrost, if present, and contamination of the surrounding ground or groundwater.

Clearly, if salt is added to embankment materials, there will be significant effects. What these effects might be, their magnitude, and their desirability in any given situation are not well understood. Our research addressed some of these questions: namely, the change in the frost susceptibility of various commonly used materials as a function of the salt content in the pore water and the permanence of the salt content remaining in the soil.

1.1 Objectives

The objectives of this study were:

1. To determine if salt treatment of frost susceptible, low quality base and subbase materials will reduce frost heaving and subsequent thaw weakening, which lead to pavement damage.

2. To determine how long salt treated bases and subbases can be expected to retain significant levels of salt under field conditions.

3. To review other beneficial or detrimental aspects of salt treatment of soils, such as increased compactability or increased thawing activity in permafrost foundation soils.

1.2 Scope

The proposed work plan contained the following steps. The work was divided between the Department of Transportation and Public Facilities (DOT&PF) and the University of Alaska-Fairbanks (UAF). The task performed by each is indicated in the listing.

1. Literature search (UAF).

2. Sample materials from the Parks Highway which were salt treated during construction in 1973-74 and the Alaska Highway where surface treatment with calcium chloride has been done for several years (DOT&PF).

3. Determine salt content, gradations and Atterberg Limits of soils collected in the field (DOT&PF).

4. Sample embankment materials commonly used in Alaskan highway construction (DOT&PF).

5. Determine gradation, Atterberg Limits, moisture density relationships, frost susceptibility, and thaw weakening of the embankment materials sampled (UAF).

6. Analyze the results of the literature search, the field tests and the laboratory tests, and prepare a final report (DOT&PF and UAF).

1.3 Acknowledgements

The study was funded by the Federal Highway Administration under the title "Effects of Salts on Road Embankment Facilities." The University of Alaska contracted to the Alaska Department of Transportation and Public Facilities for part of the work. The University of Alaska portion of the research was substantially completed by Rachael Mays as her master's thesis. The University work presented herein is a summary of her thesis, "The Use of Calcium in Alleviating Frost Heave of Alaskan Granular Soils," May 1986.

2.0 BACKGROUND

Frost heave is the increase in thickness of a soil column caused by the migration of moisture to the freezing front. This moisture may form ice lenses up to several inches thick or may merely fill the voids to the point where the grains are forced apart, causing expansion of the system. The volume change caused by freezing the pore water without any moisture migration is not considered frost heave.

Studies concerning the mechanisms involved in frost heave are being performed at an ever increasing rate in response to more population and construction activity in the northern latitudes. In spite of the research that is being done, there are still some very basic unanswered questions. Our ability to predict the amount of frost heave under a given set of circumstances is not good. Several discoveries have been made in the last few years that have led some investigators to believe that we are getting close to developing a predictive model. There are still many phenomena observed that do not seem to fit any model developed to date.

This section has been divided into two main parts. The first deals with frost heave in general, while the second deals specifically with how salt in the pore water affects frost heave. We have attempted to summarize the most pertinent information available that is widely accepted.

2.1 Mechanics of Frost Heave

The freezing of a soil system is a complex multicomponent system where heat transfer is associated with moisture migration through a porous medium. We will first describe the physical processes that are known to affect frost heave. Following this we will discuss the effects of the various major intrinsic and extrinsic factors found in the field that have an effect on frost heave.

2.1.1 Physical Processes Controlling Frost Heave

As energy is taken out of the soil at the surface, it becomes colder and a freezing front develops. The freezing front is the isotherm at which water in the soil first freezes. The factors controlling the freezing front temperature are described in section 2.1.2. Here, it is sufficient to say that the freezing front temperature is usually below 32^{0} F and that not all of the water will freeze at the freezing front temperature.

As further energy is removed from the soil, moisture migrates through the soil towards the freezing front. When the moisture reaches the freezing zone, some of it is frozen in the pores as pore ice and some of it migrates past the freezing front and forms ice lenses apart from the soil. The volume of soil between the freezing front and the location of ice lens formation is called the frozen fringe, which is typically a few hundredths of an inch thick.

The lens ice pushes the soil apart in a direction perpendicular to the freezing front. The stress that was passed through the soil matrix is then carried by the ice. The work involved in lifting the soil column and any load that is on the soil column must be added to the system during the freezing process.

Virtually all frost heave theories rely on four basic assumptions.

1. A water source is present at some location beyond the freezing front.

2. There is a continuous water film along the surfaces of the soil particles between the point of formation of the ice and the free water source.

3. Fluid and heat flow through the porous medium is laminar and unidirectional.

4. The amount of moisture that moves in the vapor state is negligible.

These assumptions do not appear to be overly restrictive, and they significantly simplify the theoretical analysis of frost heaving.

Several driving forces cause water to move toward the freezing front, and several resisting forces slow this migration of water. These forces control the rate of frost heave. The following paragraphs outline the major driving and resisting forces involved in the fluid flow. Notice that there is no mention of osmotic forces or electrochemical forces. These forces may play an important part in frost heaving in some soils, especially soils with saline pore water. Little has been done to quantify these effects so they have been left out of our discussion.

Driving Forces. - The driving forces that cause the frost heaving have not been fully accounted for, and the magnitudes of these forces are the subject of considerable controversy. In general, no single transfer mechanism proposed to date has been adequately supported by experimental evidence and accepted by the scientific community. There are a variety of mechanisms proposed, but all investigators use at least the following two components.

1. Surface tension developed within the interfaces of the various elements due to drying (freezing the water) within the voids.

2. Suction developed in the frozen fringe due to the change in energy as the water changes to ice.

Resisting Forces. -1 Several forces tend to resist water migration and hence frost heaving. The following list highlights the major forces:

1. Gravity, if the freezing front is moving downward.

2. The resistance to flow within the porous media. Water is being drawn through an unsaturated medium of soil and ice. It must pass through the unfrozen water film that surrounds the soil and ice particles. The film has a low to very low coefficient of permeability.

3. The weight of overlying material that must be lifted to form the ice lens.

2.1.2 Intrinsic Factors Controlling Frost Heave

Several factors relate to the soil, regardless of its environment, are important to its frost heave potential. These factors include the physical properties of the soil matrix, the mineralogical composition of the fines in the soil matrix, and solutes in the pore water.

The physical properties of the soil matrix include such things as the grain size, the uniformity of the grain size, the soil density, and the grain shape. These factors relate primarily to the size and shape of the pore spaces, and therefore to the thickness and distribution of the water film that contributes to the driving and resisting forces. Large grain sizes, poorly graded materials, rounded particles, and low densities tend to produce large voids and thin film thicknesses. Materials with these properties tend to be less frost susceptible. On the other hand, if the voids are very small, the permeability decreases to the point where frost heaving is not a major problem in typical frost heaving environments. Silts tend to be the most frost susceptible, while sands and gravels without fines tend not to be frost susceptible. Clays tend to be less frost susceptible than silts under road freezing conditions.

The mineral composition of the soil grains -- in particular the size and shape of the grains, and the edge charges on the grains -- will have an effect on the boundary layer of water that is next to the grain. Some soils will have a thick layer of water that is bound (adsorbed) to the particle. This water is at a higher energy state than the free water in the pours and is more viscous. The adsorbed water will freeze at a lower temperature, leaving a permeable layer along the particle boundaries at temperatures below the freezing temperature of the free water in the large voids. However, because this water is more viscous, it will demonstrate a lower effective permeability than a similar thickness of unbound water. The implications of these factors are complex, and it is difficult to quantify the net result. This is particularly so because the various factors affect both the driving and the resisting forces.

The solutes in the pore water cause several changes in the material response during frost heaving. There are osmotic and electrochemical forces produced by the solutes. No one has quantified these effects to date. Solutes in the water depress the initial freezing temperature, making the response to any given set of circumstances significantly different. For instance, a soil with freshwater in the pores may be highly frost susceptible under one set of thermal conditions, while those same thermal conditions may not be sufficient to start freezing in the same soil with salt in the pore water. This is accentuated as freezing progresses because the salts are excluded from the frozen water, thus concentrating salts in the remaining unfrozen water. On the other hand, the depressed freezing temperature may leave a thick layer of unfrozen water, resulting in higher permeability and therefore more frost heaving under other thermal conditions. The valence and the size of the ions in the water also affect the thickness and physical properties of the adsorbed water layer and, therefore, the frost susceptibility of the soil.

2.1.3 Extrinsic Factors Controlling Frost Heave

The primary extrinsic factors are temperature, pressure, and position of the free water. The rate of heat removal from the system is a critical factor in the rate of frost heaving and the amount of frost heaving that will occur. A slow rate of heat removal provides more time for the driving forces to suck the water to the freezing front which tends to increase the total heave however it also produces a thicker frozen fringe, which tends to slow the heave rate and therefore decrease total heave. A fast heat removal rate will produce more but thinner ice lenses than will a slow heat removal rate. The net result of a fast heat removal rate of heat removal is a fast heave rate but less total heave. It is, therefore, critical to choose standard and reasonable freezing conditions in the laboratory when evaluating the frost heave potential of a soil.

It takes energy to cause frost heaving in a soil that is under pressure. A higher pressure will require more energy until a pressure is reach where the energy requirement cannot be met by the driving forces. This is called the shut-off pressure. The shut-off pressure is typically quite high, on the order of several tons of psi. However, the rate of frost heave may be significantly reduced at much lower pressures.

The depth of the free water is directly proportional to the gravimetric resisting force. At some height above the free water the continuous film of water will be broken and frost heaving will not occur above that point. Therefore, one would expect more frost heaving when the free water is close to the freezing front. Frost heaving will continue when the freezing front progresses below the phreatic surface because of the negative pressure developed at the interface between the water and ice.

2.2 Frost Heave of Soils with Saline Pore water

Studies to date are inconclusive and contradictory. From an engineering standpoint, the presence of salt in the pore water may be beneficial or harmful, depending upon the conditions and the application. Salt in the pore water always depresses the freezing temperature and always produces a weaker ice-soil matrix. All other engineering aspects of the effects of salt on frost heave depend upon the individual soils and conditions of freezing.

2.2.1 Description of Freezing Saline Soil

The addition of salts radically alters the way soil freezes. This is due primarily to the wide range of salt concentrations present during freezing which occur because of brine exclusion, diffusion, and convection. The wide range of salt concentrations causes freezing to occur over a wide temperature range and along an irregular path which might not be continuous. Under slowly freezing conditions, the salts tend to be excluded downward and ahead of the freezing front. Under rapid freezing conditions, the salts tend to become trapped in pockets within the frozen mass. When an ice lens starts to form, it excludes the salts and forms a higher brine concentration on the warm side, requiring a colder temperature to cause more freezing. At some point, a new ice lens will form below the brine, causing a layering effect of ice and brine. The layers are not necessarily planar features. This phenomenon effectively shuts off frost heaving in jumps which may result in less total frost heave -- particularly in those situations where the soil is frozen rapidly. However, the rate of frost heave may be greater under some circumstances due to the greater permeability in the frozen fringe caused by the increased thickness of the unfrozen water films.

Aside from the above-mentioned phenomenon, the salts may cause some soils (particularly high plasticity clays) to disperse and in effect change their grain size. This may increase the frost heave potential of those soils.

The freezing front frequently advances more rapidly in a soil with saline pore water because the higher unfrozen moisture content requires less energy to depress the temperature. This could cause freezing and hence frost heaving to occur at a lower depth than would have otherwise been expected. A secondary effect reinforcing this behavior is that the thermal conductivity is frequently greater in a soil with more unfrozen water.

2.2.2 Laboratory Experiments Using Salt Treatment

Numerous laboratory studies have determined the effects of salt on the behavior of soils with saline pore water during freezing and thawing conditions. These studies have had various degrees of sophistication, and the results are a mixture of general conclusions and results under a specific set of circumstances. The results of the testing are often inconclusive or conflicting. Some studies indicate significant benefits, while others indicate primarily negative effects. The following statements with regard to adding salt generally apply to the types of materials that may be used as base or subbase materials on Alaskan highways when good quality NFS material is not available.

1. The optimum moisture for compaction is decreased.

2. The permeability is decreased.

3. The unfrozen compacted strength is usually lower with increasing salinity in the pore water.

4. The decrease in strength due to freeze-thaw action is usually less with increasing salinity in the pore water.

5. The frost heave shut-off pressure usually decreased with increasing salinity in the pore water.

2.2.3 Field Experiments Using Salt Treatment

Several field studies have attempted to determine the effects of salinity on pavement performance in freezing and thawing conditions. In general, the control has been less than desired, and the results have been inconclusive. Some test results were very encouraging and indicated a nearly complete elimination of frost damage, while others indicated that there was no noticeable improvement and perhaps a decrease in performance with the addition of salt. In our opinion, the differences lie in the test conditions and not in the general suitability of using salts to reduce frost effects in road construction.

3.0 LABORATORY STUDY

We conducted a series of tests to determine the effects of salt treatment on the frost heave behavior of three soils commonly used in base and subbase construction in Alaska. Specifically, the tests were designed to determine the effects on total frost heave, heave rate and thawed strength. Frost heave tests were performed in which the temperature and the temperature gradient were regulated, with four different salt concentrations on three different soils.

3.1 Materials, Equipment and Procedures

Soils were taken from three different borrow pits in interior Alaska. The borrow pits are referred to as the Ester Pit, the Parks Pit, and the Northway Pit. Each sample was separated into three sizes: smaller than 0.074 mm; between 0.074 mm and 9.6 mm; and larger than 9.6 mm. The larger materials were discarded and the other two were reconstituted to give 12% smaller than 0.074 mm. The percentage of the material passing the 0.02 mm size ranged between 8.0 and 8.6 for each of the samples tested. The mineralogy and the grain size within the two fractions were not changed.

The Ester Pit is in dredge tailings about 1 mile west of the town of Ester. The material is derived from the Fairbanks schist which is a muscovite quartz schist with biotite, garnets and feldspars. The clay minerals are predominantly chlorite, disordered kaolinite, sericite and illite.

Material from the Parks Pit is a muscovite quartzite derived from the Cleary sequence. The clay minerals include illite and poorly crystallized kaolinite.

Material from the Northway Pit was derived from an igneous source. Plagioclase, quartz, biotite, and hornblend were identified in the coarse fraction. Clay minerals include predominantly montmorillonite with some kaolinite and illite.

The final gradation of grain size the samples tested is shown in Table 1 and other pertinent engineering properties appear in Table 2.

TABLE 1. Gradation of laboratory study samples

Size	Percent Passing Ester Pit	 Parks Pit	 Northway Pit
3/8 in	100	100	100
1/4 in	78	100	66
#8	52	70	38
#30	30	42	18
#200	12.0	12.0	12.0
02 mm	8.0	8.7	8.6

TABLE 2. Other properties of laboratory study samples

Property	Ester Pit	Parks Pit	Northway Pit
Permeability (cm/sec)	$7.5 * 10^{-7}$	$5.9 * 10^{-6}$	0
Max. Density (pcf) (AASHTO T-180)	135.0	135.0	137.0
Opt. WC (%)	6.5	6.5	7.2
CBR (%)	64.0	67.0	60.0

The frost heave tests were run four at a time in a frost heave cabinet made for DOT&PF by Ed Chamberland at The U. S. Army Cold Regions Research and Engineering Laboratory (CRREL). The samples were 6 inches high and 6 inches in diameter. They were contained in 1 inch high plastic rings that were free to move relative to each other. The sides of the cells were insulated. Samples were frozen from the top down. Both upper and lower temperatures were controlled by circulating glycol through the top and bottom caps of the cells. A 0.5 psi surcharge was placed on the top of each sample during testing and water with the desired salinity was allowed free access to the bottom of each cell at all times. The heave and settlement were measured with an LVDT and the temperatures at 35 locations were measured with thermistors. All data were collected automatically and stored on magnetic tape for machine processing.

The samples were oven dried, separated into the three groupings by grain size and then reconstituted to give the desired gradation, 100% passing the 3/8 inch and 12% passing the #200 sieve. The amount of sample needed for a given test was then raised to its optimum moisture content with water mixed to one of the salinities shown in Table 3.

TABLE 3. Salinity of Pore Water

Treatment No.	Parts per Thousand (ppt)	Moles (M)
1	0	0
2	15	0.135
3	37.5	0.338
4	70.0	0.631

The wetted soil was cured for 24 hours and then compacted into 6 inch diameter steel molds with 1/2 inch high plastic rings using a hand held vibratory compactor. A standard level of compactive effort was used to give about 93% of the dry density obtained using the AASHTO T-180 (Modified Proctor) procedure. The steel molds were removed and the samples in the rings were then soaked in water (with the same salinity with which they were compacted) for 24 hours prior to mounting in the frost heave cell. Two entirely different temperature boundary conditions were applied. They are called two-cycle tests and ramped tests.

Two-Cycle Tests. - The two-cycle test temperatures were controlled in a more or less standard fashion. The tests consisted of 136 hours of controlled temperature testing including an 8 hour stabilization period, followed by a 48 hour rapid freeze, a 16 hour thaw, a 48 hour slow freeze and, finally, a 16 hour thaw period. After the final thaw, the samples were taken out of the heave cabinet, and hose clamps were used on each of the plastic rings to support them during CBR testing. The warm and cold plate temperatures were controlled as shown in TABLE 4.

TABLE 4. Two cycle test boundary temperatures

Hours into Test	Top Plate Temperature Deg. (^{o}C)	Bottom Plate Temperature Deg. (^{o}C)
0	+ 1	+ 1
8	- 15	- 2
56	+ 5	+ 5
72	- 5	- 0.5
120	+ 5	+ 5

Ramp Tests. - In this series of tests, temperatures were adjusted continuously to give the wide range of temperatures and temperature gradients expected in the base and subbase materials in interior Alaska. The thermal gradients were maintained between 14 and 6^{o} C/m and the absolute temperatures were maintained between 0 and -3.5^{o} C. Each of these tests ran in 30 hour increments, following the 16 hour curing period and the 48 hour quick freeze and a 16 hour thaw as just described. The salinity of the pore water in each sample was determined after the tests were thawed for the final time.

3.2 Test Results

The results of the laboratory tests consist primarily of heave rates, total heave, CBR tests, and measurements of the net salinity redistribution in the samples due to freezing and thawing. The following descriptions are in the form of conclusions. Data are only presented they are needed for clarity or explanation. All other data are presented in the thesis.

In the tests performed for this study, the initial rate of heave of the entire sample always decreased as the salinity increased, Figures 1 and 2. The samples with lower salinities would freeze completely at a relatively high warm-side temperature. Therefore, the samples with lower salinities would stop heaving while the higher salinity materials would not be completely frozen and would still continue to heave. The net result is that the higher salinity samples would continue to heave when the lower salinity samples had stopped, so the higher salinity samples would sometimes undergo more total heave.

In order to determine the relative frost susceptibility of two soils, it is necessary to compare them under field freezing conditions. In central Alaska, field freezing conditions means that the samples will eventually get very cold whereas, in warmer parts of the state, the bottom of the subbase materials may not even get below the initial freezing temperature.

One commonly used parameter to make such comparisons is the specific heave which is defined as the total heave divided by the thickness of material that is below the initial freezing temperature. The concept of specific heave is not completely appropriate for comparing samples with different salinities, but it is much more appropriate than total heave in situations where the samples are not completely frozen. Table 5 shows the specific heave for the two-cycle frost heave tests.

TABLE 5. Results of two-cycle frost heave test

Soil	Treatment Level % CaCl2	Total Heave (in)	Freezing Thickness (in)	Specific Heave
Ester	0.00	0.46	5.4	8.5
	1.50	0.49	5.3	9.1
	3.75	0.19	1.9	10.0
	7.00	----	---	----
Northway	0	0.20	6.0	0.3
	1.50	0.09	6.0	1.4
	3.75	0.14	5.4	2.6
	7.00	0.01	---	---
Parks	0	0.28	6.0	4.7
	1.50	0.25	6.0	4.2
	3.75	0.22	4.2	5.2
	7.00	0.11	0.1	7.7

From Table 5, it is obvious that the specific heave generally increases with increasing salinity. The effect would be further accentuated if only the thickness of the material that had stopped frost heaving was used in the calculations, as opposed to the full thickness of the material below the initial freezing temperature of the pore water.

In summary, high salinities appear to decrease initial heaving rates and decrease total heave, if the temperatures are not low enough to cause the saline pore water to freeze. If freezing is continued to completion, it appears that the materials with saline pore water may heave significantly more than those with freshwater.

Table 6 shows the CBR values measured after the final thawing in the two-cycle tests.

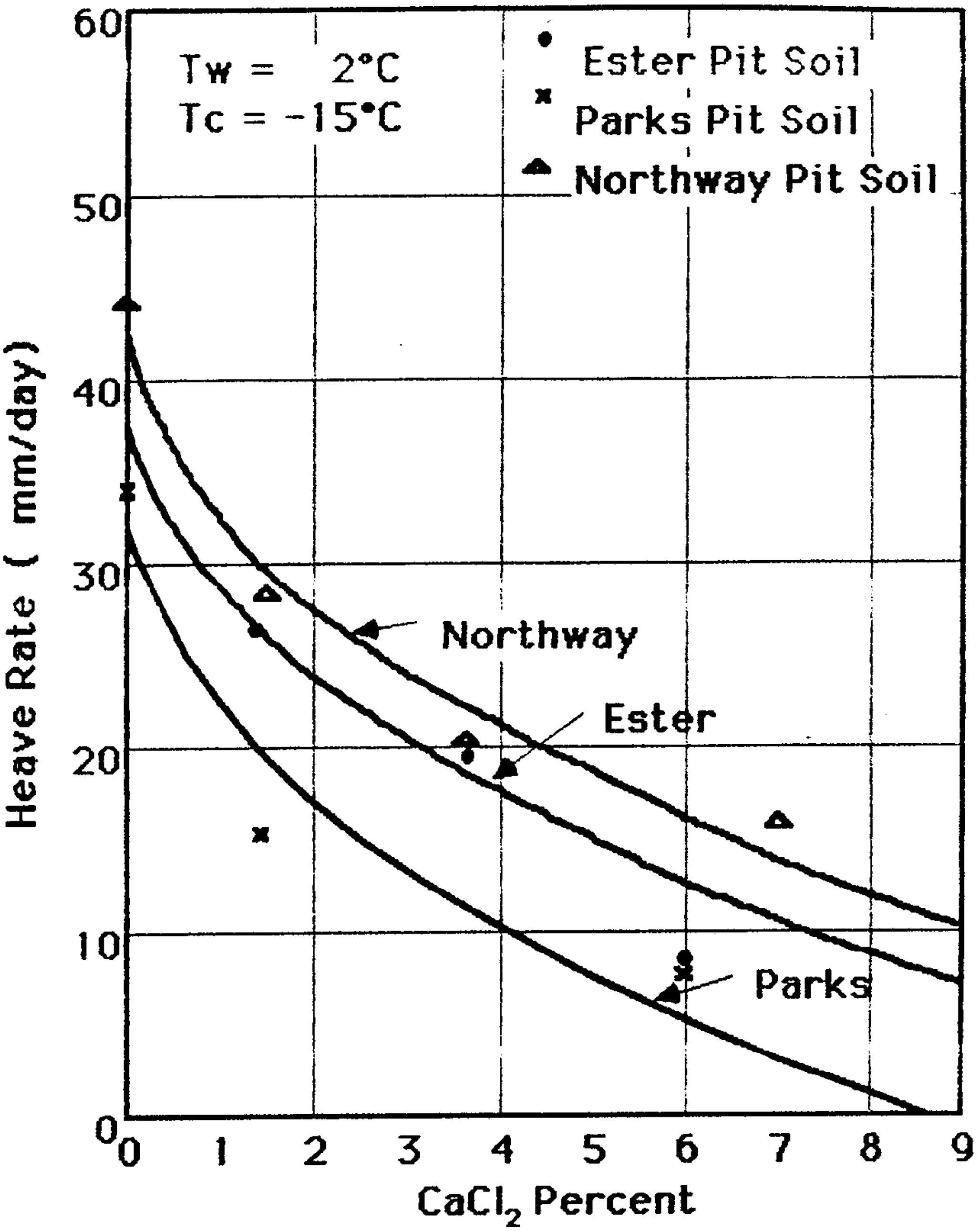

Figure 1. First cycle heave rate versus $CaCl_2$ concentration.

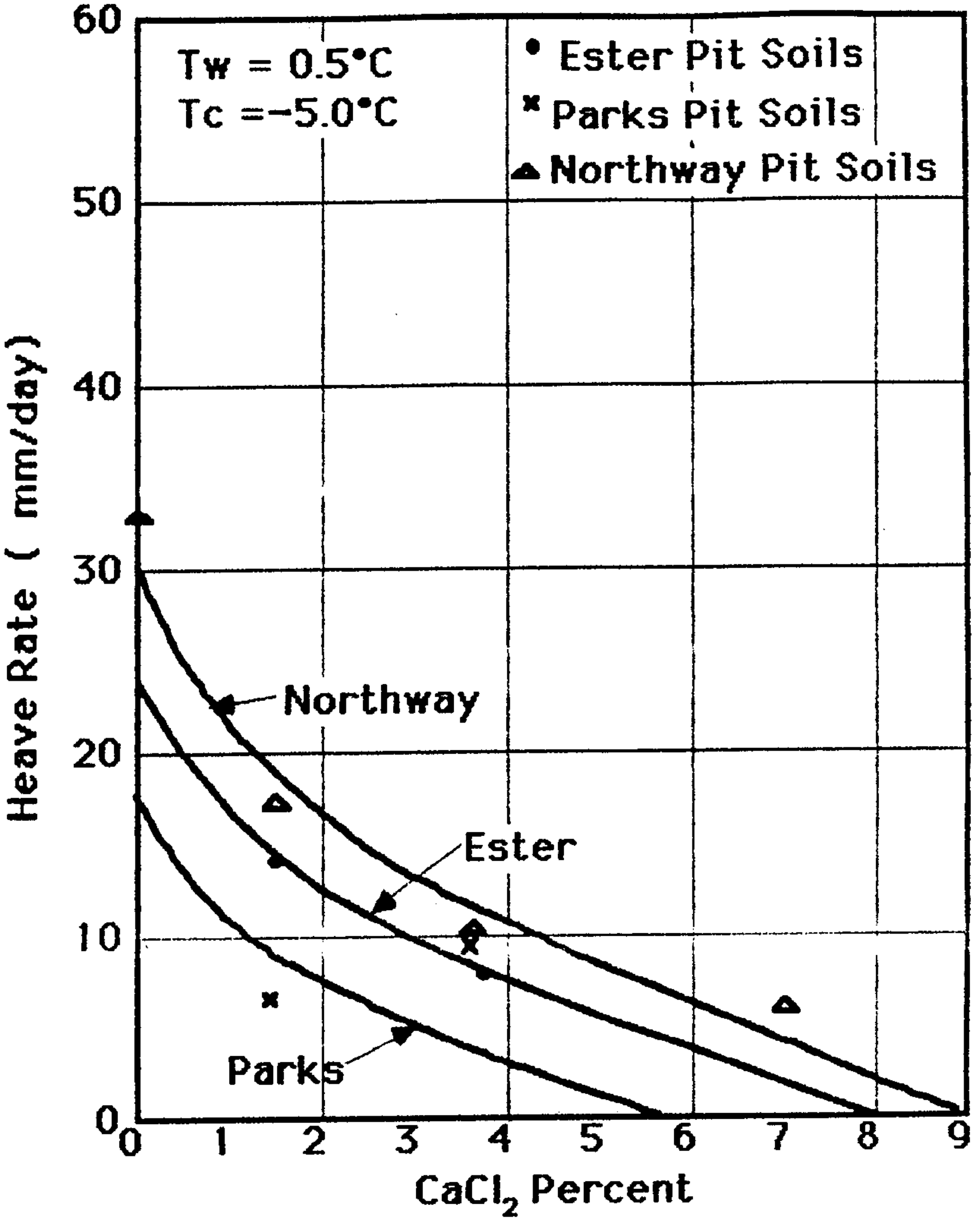

Figure 2. Second cycle heave rate versus $CaCl_2$ concentration.

TABLE 6. CBR VALUES BEFORE AND AFTER TESTING

	Before Testing		During Two Cycle Frost Heave Testing		
Soil	Maximum Density (pcf)	CBR (%)	Treatment Level (%)	Initial Density (pcf)	Final CBR (%)
Ester	135	64.0	0.00	128.5	16
			1.50	126.2	23
			3.75	128.7	28
			7.00	130.7	28
Northway	135	59.7	0	122.6	7
			1.50	122.0	10
			3.75	125.1	10
			7.00	127.4	23
Parks	137	66.7	0	124.9	45
			1.50	119.3	4
			3.75	130.1	50
			7.00	120.9	26

The data for the Parks soil tend to be erratic, perhaps because of the large variation in original density of the samples. The other two soils demonstrate a significantly higher CBR after freezing and thawing with increasing salinity. It is difficult to draw firm conclusions from these data because of the extreme variation in the water content of the samples after freezing, and because of the variations that the salinity may make in the drainage and subsequent consolidation after frost heaving. The freshwater samples should have been at a more or less uniform moisture content after being frozen to the bottom. Only a portion of the materials with high salinity pore water froze and heaved, leaving an uneven distribution of density and water throughout the sample. It is not at all clear how uniform the density was after thawing, and it was not possible to measure the density in layers and still do the CBR test with the equipment on hand.

The salinity of the pore water measured in the samples after the ramped tests were thawed indicated a significant decrease during the test as shown in Table 7.

TABLE 7. Soil Pore Water Salinity Before and After Testing

Test	Soil	Treatment Level	CaCl2 Weight Added	CaCl2 Weight Remaining
		% $CaCl_2$	%	%
Ramp 1	Ester	0	0	0.004
		1.5	0.22	0.107
	Parks	0	0	0.002
		1.5	0.18	0.135
Ramp 2	Parks	0	0	0.004
		3.75	0.23	0.208
	Northway	0	0	0.003
		3.75	0.28	0.213

Several mechanisms could cause the decreased salinity we noted. As salt is excluded from the frozen water, the salt concentration in the remaining water becomes greater. In general, this mechanism alone would cause the salts to migrate downward with each freeze-thaw cycle. In addition, the salinity of the unfrozen water in the frozen fringe is greater than the water below, which sets up a density gradient causing the higher salinity water to settle. This also sets up a condition for diffusion and a downward migration of the ions in the water. Under field conditions, groundwater flow and vertical drainage would also cause the salt to migrate out of the soil.

We observed significant differences in the frost heave performance of the three samples but the reasons for the differences is not apparent. The gradation of the coarse fraction may make a difference in the void sizes and shapes, even though the fraction that passed through the #200 sieve was the same, and the maximum density was about the same. The mineralogy of the clay fraction may cause some differences due to the interaction between the salts and the mineral surfaces. There were slight differences in the compacted density that may have affected the frost heave results, especially the first cycle of freeze thaw. All soils are inhomogeneous, and those differences alone can have a significant effect on frost heave.

4.0 FIELD TESTING OF EMBANKMENT SALINITY

Salts may have little or no long-term effect on road performance if they leach rapidly from the embankment where they have been applied. In order to gain insight possible leaching, we sampled embankment materials at locations where known quantities of calcium chloride had been applied in the past. These materials were then analyzed for both calcium and chloride content.

4.1 Sampling and Testing

Two sets of samples were obtained. One set consisted of material from 17 holes drilled at eight locations on the Alaska Highway between Beaver Creek and the Alaskan border with the Yukon Territory. Calcium chloride had been regularly applied to the gravel surface of this road for 10 years prior to sampling. The second set consisted of material from seven holes bored at three locations on the Parks Highway a few miles north of Cantwell, Alaska. Calcium chloride was applied to the subbase of this road in 1974 when it was built. The following summer, the subbase was covered with a base course and asphalt pavement, which has been maintained since then.

4.1.1 Alaska Highway Sampling

Records of calcium chloride applications to the study area were obtained from the Yukon Department of Highways and Transportation (Table 6). As the table shows, applications of five tons per mile were generally made twice a year over a 10 year period. The total application of 95 tons per mile is equivalent to 13.5 pounds per square yard of driving surface. Salt applications were discontinued after 1982, and the road received a bituminous surfacing in July 1983.

TABLE 8. Salt Application History, Alaska Highway Section

Year	Surfacing Applied yd^3/mile	Calcium Chloride Applied tons/mile	Date
1971	2,400	0	summer
1973		5	fall
1974		5	spring
		5	fall
1975	650	0	summer
		5	fall
1976		5	spring
		5	fall
1977		5	spring
	700	0	spring
		5	spring
		5	fall
1978*		5	spring
	760	0	summer
		5	fall
1979		5	spring
	900	0	summer
		5	fall
1980		5	spring
		5	fall
1981		5	spring
	300	0	summer
		5	fall
1982		5	spring
		5	fall

Some grade raises and minor realignments were added at various spots on this section of the Alaska Highway in 1978 from Yukon Department of Highways.

Drilling was conducted during the first week of October 1983. Eight sampling sites were selected to represent a variety of drainage conditions. In general, we obtained continuous samples from two holes at each site (one hole in the outer wheelpath of each lane) using a split spoon to a depth of 4-1/2 feet. A few samples were also obtained from greater depths and from inner wheelpaths.

4.1.2 Parks Highway Sampling

Construction records were obtained for the area where sampling was performed. The road in this area has an asphalt pavement 40 feet wide placed over six inches each of crushed base course and subbase. Select material and unclassified borrow materials lie below these layers. Calcium chloride was applied to the 44 foot wide subbase surface in the sampled areas at the rate of one pound per square yard (equivalent to 12.9 tons per mile) during construction in 1974.

The samples were obtained in October 1983 from three sites that were representative of the drainage conditions in the area. Continuous samples were obtained to a depth of 5 feet.

4.1.3 Testing Program

The following information was obtained.

1. field boring logs, including standard penetration information (blow counts).
2. moisture content (Alaska Highway samples only).
3. and fraction of material passing a No. 4 sieve.

The material that passed the # 4 sieve was then tested for soluble calcium and chloride content. This was done by agitating a portion of the sample in distilled water overnight and then testing the filtrate. Calcium was measured with an atomic absorption spectrophotometer. Chloride content was measured with an ion analyzer equipped with a chloride-specific electrode.

The chloride in the samples could be assumed to come almost entirely from the applied salt, as since very little soluble chloride is found in most natural soils. This was confirmed (for the Alaska Highway samples) by testing material taken from eight gravel pits alongside the road. Chloride in these materials averaged less than ten parts per million (ppm); the highest level recorded was 12.8 ppm.

Calcium is more abundant in natural soils than chloride. Typical non-calcareous unfertilized soils contain between 10 and 50 ppm of calcium ions. These concentrations are normally about 10 times higher in calcareous soils (e.g. limestone).[1]

This makes interpretation of the test results more difficult. "Background" levels of calcium could not be measured, since there was no way of determining the material source (gravel pit) from which that a particular sample came. We obtained qualitative assessment of limestone content was made, however, by observing the reaction of the samples to a 10% solution of hydrochloric acid.

4.2 Test Results

Tests results for calcium and chloride content are presented in Table 9 as concentration in the material passing the No. 4 sieve (as tested) and as concentration in the total sample. Ninety percent confidence intervals for the average concentrations in the road (as opposed to the average in the samples) are also shown in the table.

4.2.1 Dissipation Rates

If it is assumed that salt content in the soils dissipates exponentially to background levels after application, the equation for this dissipation can be computed given sufficient information. One must know the time and concentration of salt applications, the concentration at a known time after application, and the background level. Background calcium levels could not be accurately determined, as previously discussed. Dissipation can be calculated for chloride levels, however.

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[1] Steve Sparrow, Asst. Professor of Agronomy, Agricultural and Forestry Experiment Station, University of Alaska at Fairbanks, personal communication.

TABLE 9. Average Field Calcium and Chlorine Content (ppm by weight)

	#4 SIEVE FRACTION		TOTAL SOIL	
	SAMPLE MEAN	TRUE MEAN RANGE* (90% CONFIDENCE INT.)	SAMPLE MEAN	TRUE MEAN RANGE* (90% CONFIDENCE INT.)
ALASKA HWY.				
CALCIUM	392	369-415	214	192 - 236
CHLORINE	142	110-174	83	71 - 95
PARKS HWY.				
CALCIUM	216	173-259	113	87 - 139
CHLORINE	22.2	145- 29.9	11.9	6.9 - 16.8

* Normal distribution assumed.

The information on the Alaska Highway (including all 19 salt applications) yields equation (1) for the dissipation of chloride content following application.

$$Cl_t = Cl_o e^{-0.887t} \tag{1}$$

where Cl_t = chloride content (above background) at time t

Cl_o = chloride content (above background) at time of application (t = 0)

t = time (in years)

This equation is based on the average chloride content (83 ppm) in the samples. Application concentrations were based on the application rate of five tons per mile, dispersed through a critical embankment section assumed to be 5 feet deep and an average of 30 feet wide, with an average dry density of 120 pcf.

The equation implies that half of the chloride applied to the road would no longer be in the critical section (chloride half-life) after 0.78 years. Seventeen percent of the applied chloride would remain after two years while 1% would remain by the fifth year.

If the confidence interval limits listed in Table 9 are used to derive the equation instead of the sample mean, exponents of -0.814 and -0.976 are found instead of -0.887. The corresponding chloride half-life fall is between 0.71 and 0.85 years.

It should be noted that this dissipation rate is based on several assumptions, and it represents only an average for the 20 mile section of unpaved road from which samples were taken. There is considerable variation from this average within the tested section, as there would undoubtedly be in other unpaved road sections.

Similar dissipation of chloride rate calculations can be made from the Parks Highway Test results. A two-step process is needed in this case, since the road section was unpaved for the first ten months after salt application and paved thereafter.

The salt applied to the Parks Highway was equivalent to 96 ppm chloride in a critical embankment section assumed to be 5 feet deep and an average of 50 feet wide, with an average dry density of 130 pcf. This would have dissipated to about 46 ppm by the time the road was paved, if this dissipation was at the average rate found for the Alaska Highway. Assuming that this is true, the equation for the dissipation of chloride in the Parks Highway after paving is:

$$Cl_t = Cl_o e^{-0.180t}$$

based on the average sample concentration, and assumed background levels of 11.9 and 3 ppm, respectively. This corresponds to a chloride half-life of about 3.9 years. If the confidence interval limits are used, the exponent in the equation varies between -0.131 and -0.269, which corresponds to a chloride half-life of between 2.6 and 5.3 years. Again, it should be noted that this is an average for this paved road section, and that variations occur within the section as they undoubtedly would in other road sections.

Dissipation for calcium cannot be computed in this way due to the uncertainty of background levels, as previously mentioned. If dissipation rates are assumed, however, the calcium concentration data can be used to calculate background levels of calcium. If the dissipation rates found for chloride content are assumed to be applicable to the calcium content, the implied background calcium levels are about 170 ppm for the Alaska Highway samples and 110 ppm for the Parks Highway samples.

The hydrochloric acid tests indicated that the Alaska Highway samples contained, on the average, a "medium" limestone content. The Parks Highway samples contained, on the average, a "low-to-medium" limestone content. The computed background calcium levels seem reasonable. These levels may be taken as an indication (although certainly not proof) that calcium decay rates are of roughly the same magnitude as those for chlorides.

4.2.2 Other Correlations

Attempts were made, with little success, to correlate calcium and chloride content with other road characteristics such as local drainage conditions, soil gradation and limestone content.

We thought that drainage conditions might significantly affect the amount of leaching of salts from the road embankment. Calcium and chloride contents were, therefore, grouped by drainage category and averaged, as shown in Table 10. No significant correlations were found.

TABLE 10. ALASKA HIGHWAY TEST RESULTS
(Summary by Site Drainage Conditions)

	Chloride Content (ppm by weight)		Calcium Content (ppm by weight)	
Poor Drainage				
Site 1	27		223	
Site 6	147	Avg. 75	236	Avg. 203
Site 8	51		151	
Fair Drainage				
Site 3	173		203	
Site 5	51	Avg. 93	171	Avg. 175
Site 7	54		151	
Good Drainage				
Site 2	135		302	
Site 4	21	Avg. 78	148	Avg. 225

TABLE 11. Field Test Results for Calcium
(Summary by Sample Limestone Content)

Rating	Calcium (ppm by weight)	Sample Set standard deviation	No. of Samples
Limestone	386	138	8
Very high	344	109	14
High to very high	236	86	5
High	220	113	4
Medium-high	445	N/A	1
Medium	237	138	6
Low-medium	330	151	5
Low	322	138	13
Very low to low	304	59	2
Very low	374	137	14
None	379	196	6

Samples were grouped by their limestone content and by the amount of fine material they contained (Tables 11 and 12). We expected that highly calcareous soil samples would show high values of soluble calcium. Samples which were very high in limestone did indeed have higher average calcium levels than those with moderate limestone content, as can be seen in Table 11. Curiously, however, calcium levels in samples with low limestone content were also greater than samples with moderate content.

A possible explanation of this stems from the fact that most samples which were low in limestone content were made up mostly or entirely of fines. Samples with low fines contents tended to have higher calcium content than other samples, as can be seen in Table 12. Calcium ions may be adsorbed onto the surfaces of clay particles. It may be, therefore, that either high clay content or high limestone content results in high calcium levels.

While this explanation is plausible, a multiple linear regression of the data did not reveal a strong statistical relationship of this sort (less than 90% confidence level). Given the large standard deviation of the data in each group, this is not surprising.

Average chloride contents, like calcium contents, tended to be higher for those samples with high fines (see Table 12). A statistical analysis of variance test (ANOVA) on the data, however, revealed less than a 90% confidence level in this relationship. Thus, while there appears to be a trend in the data, there is a significant possibility that it is due merely to chance.

TABLE 12. ALASKA HIGHWAY TEST RESULTS
(Summary by sample fines content)[1]

"Silt" Content (from boring log soils callouts)[2]	Number of Samples	Average Limestone Content	Chlorides		Calcium	
			ppm by weight	Sample Std. deviation	ppm by weight	Sample Std. deviation
Silt or Silt w/trace (x)	11-12	very low	98	113	328	121
Silt w/some (x)	8	very low	168	220	274	157
Silt w/(x)	3	high to very high	13	9	167	96
(x) w/silt	2	medium	12	6	104	52
(x) w/some silt	46-49	high	79	97	199	97
(x) w/trace silt	8	medium	45	39	128	65

[1] Drilling log soils callouts list all fines as "silt"; some may be clays (no hydrometer tests were run on samples).

[2] (x) = other material

5.0 CONCLUSIONS

There were two primary objectives to this study to determine the effect of salinity on frost heave and to determine the permanence of the salt when placed in a road. These aspects are mutually exclusive and are discussed in the following sections.

5.1 Effect of Saline Pore water on Frost Heave

The mechanisms involved in frost heaving are complicated and not fully understood. Adding salt to the pore water greatly complicates the phenomenon and has not been rigorously considered in the literature. The tests and analyses done as part of our study were neither extensive nor conclusive but we can draw several tentative but significant conclusions.

1. Salt greatly affects the mechanisms involved in the frost heaving process. Salt significantly affects the rate of heave and the amount of total heave under any given set of freezing conditions.

2. The effectiveness of the treatment is a function of the salinity and the intrinsic characteristics of the soil such as grain size and mineralogy. It is unlikely that standard soil mechanics tests alone will be enough to identify the important parameters for an accurate evaluation of the potential effectiveness of a salt treatment.

3. Higher salinity in the pore water will prevent frost heaving at temperatures below 32° F. Increased salinity tends to reduce the rate of frost heave at temperatures below the initial freezing temperature of the saline pore water but near 32° F.

4. An increase in the salinity of the pore water may increase or decrease the total frost heave, depending upon the soils and the freezing conditions.

5. The addition of salt may make a soil easier to compact and may decrease a soils optimum moisture content for compaction during construction. It may also increase a soils permeability and, therefore, the rate at which the soil will regain strength in the spring. But salt may decrease a soils CBR and its strength when frozen.

5.2 Permanence of Salts in Road Embankments

Our field tests evaluated existing facilities that were not originally set up with controls to establish accurate initial conditions. Therefore, these data are inconclusive. In general, the field data suggest that probably little, if any, salt remains in a road section several years after construction. Our laboratory data suggest that the freezing and thawing process tends to extrude salt from freezing soils.

6.0 IMPLEMENTATION

The data from this study indicate that the use of salt treatment in a conventional embankment is probably not permanent, and so the use of salt is probably not an economical design tool. If, however, the design either prevented the migration of the salts from the treatment area or allowed the reinjection of salts, then the concept could have merit under some circumstances. Salt could completely eliminate frost heaving if the treated soil never cooled below the initial freezing temperature of the pore water. The addition of salt could also result in less total frost heave under slightly colder conditions. In cold environments where the bottom of the layer being treated is expected to get several degrees below the initial freezing temperature of the pore water, the addition of salt will probably increase the total heave.