FINAL REPORT

FHWA-WY-14/01F

State of Wyoming
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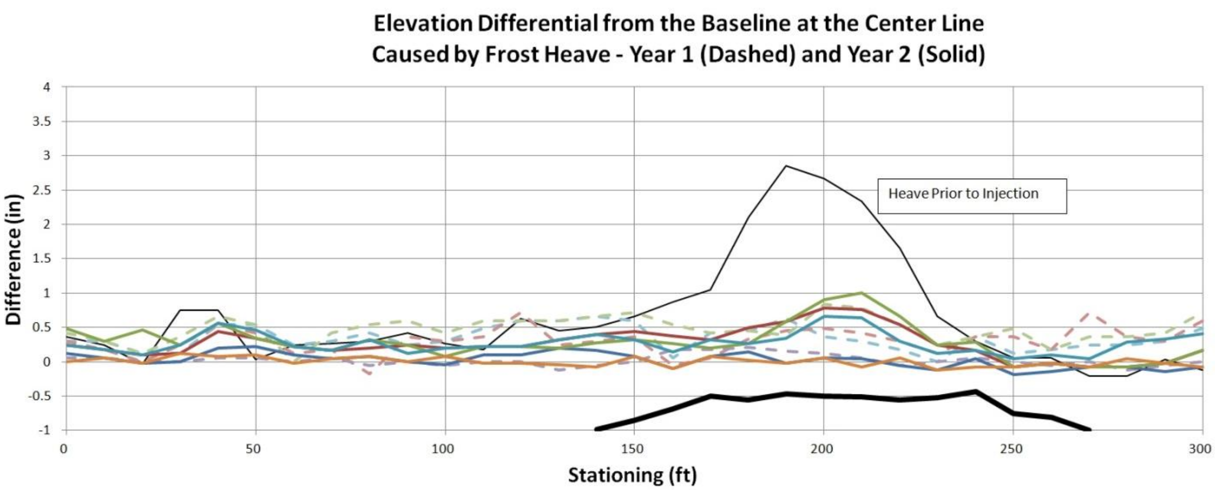
Federal Highway Administration

Instrumentation and Analysis of Frost Heave Mitigation on WY-70, Encampment, WY



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FORWARD

Frost heave in roadbed subsoils is a common occurrence in northern regions worldwide. Ice lenses are formed when the pavement is subjected to consistent freezing temperatures for the duration of a winter, when water is available at the freezing front and when the subgrade soil is comprised of fine sands and silts. The most common remediation technique for heaving roads is to excavate the fine cohesionless soil to below the depth of freezing and replace the full depth with non-freezing aggregate. An alternative procedure is to place layers of polyurethane foam insulation panels above the subgrade and cover with a thick base layer.

This project investigated a novel procedure to reduce or prevent subgrade freezing and heaving non-destructively by injecting a two-part polymer foam at the top of the subgrade. Controlled injection of Uretek Star, an expanding structural polymer foam, created a continuous three-inch thick layer of insulation that significantly reduced the heat loss from the deeper soil and almost totally eliminated frost heave at a site on highway WY-70, four and one-half miles west of Encampment, WY. The foam layer also prevented the upward movement of water from the warmer regime under the foam to the upper frozen regime above the foam. This prevented any segregational freezing in the upper zone.

The two-year research project consisted of measuring pavement elevation changes along five 300-foot long lines over the heave area and monitoring subsurface temperatures at six locations inside and outside of the injection zone. The construction time for the 100-foot section was one week for injection and milling the surface. Construction was contained in one lane, leaving a lane open for the entire duration without a detour, increasing safety and minimizing impact for the driving public.

Additionally, a procedure is developed for estimating the thickness of the foam layer required for other sites with different average temperatures.

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Abstract
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SI* (MODERN METRIC) CONVERSION FACTORS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
	NOTE: volumes greater than 1000 L shall be shown in m^3			
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised March 2003)

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
CL	Centerline of Highway
CLEB	Centerline of the east bound lane
CLWB	Centerline of West Bound Lane
CST	Concrete Stabilization Technologies, Inc.
MBE	Modified Berggren Equation
MNDOT	Minnesota Department of Transportation
NEWB	North Edge West Bound Lane
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources and Conservation Service
NWS	National Weather Service
PI	Plasticity Index
PMP	Plant Mix Pavement
PVC	Polyvinyl Chloride
PZ	Piezometer
SEEB	South Edge East Bound
SNOTEL	Snow Telemetry Station
STA	Station
STAS	Stations
TH	Test Hole
UW	University of Wyoming

LIST OF SYMBOLS

A_o	Amplitude of one-half the difference between the highest and lowest temperatures over a given time period, such as 24 hours or 365 days
C	Volumetric Heat Capacity
°C	Degree Celsius
d	Damping depth
°F	Degree Fahrenheit
FI	Freezing Index
FIR	Required Freezing Index, the *FI* required to freeze a soil layer
h_i	Thickness of material layer
k_t	Thermal Conductivity of soil or aggregate
k_{foam}	Thermal Conductivity of foam
L	Volumetric Heat of Latent Fusion
n	Ratio of Ground Surface Freezing Index to Air Freezing Index
q_h	Heat Flux
R value	Thermal Resistance
T, $T(z,t)$	Temperature at any location z and time t
T_{ave}	Average Temperature at depth in soil profile
t	Time
w	Water content
z	Vertical Distance
γ_d	Dry unit weight of material
λ	Multiplier Coefficient in MBE accounting for Initial Ground Temperature and thermal properties of the soil
ω	Radial frequency of time, such as 24 hours or 365 days

CHAPTER 1. PROBLEM DESCRIPTION

SITE CONSIDERATIONS

Frost heave at milepost 51.8 of Wyoming Highway WY-70, the Battle Mountain Highway, has been great enough to create dangerous driving conditions for vehicles and trailers. The highway is closed during the winter at the Medicine Bow Forest Service Boundary 1.6 km (1 mi) to the east and that location has become the trailhead for a popular snowmobile run. Vehicles pulling snowmobile trailers accelerate rapidly on the downhill slope from the parking lot and are often traveling much faster than the posted speed. When the vehicles hit the sharp hump, and especially the one-inch deep dip in the center of it, they can become airborne and poorly restrained snowmobiles have been thrown from their trailers. The site is about 7.2 km (4.5 mi) west of Encampment, WY as shown in figure 1.

During construction in the early 1990s, the contractor encountered problems excavating a tough crystalline bedrock outcrop running perpendicular to the highway alignment. After trying unsuccessfully to remove the rock above subgrade, it was agreed that the contractor could alter the vertical alignment by placing approximately 1.0 m (3 ft) of subgrade soil over the rock and smoothing the vertical grade over several hundred feet to the east and west of the problem location. The soil texture, a silty sand, is highly susceptible to frost heave.

Frost heave developed during the first winters of operation. A variety of remediation techniques have been attempted at the site. Two drainage pipes were installed across the road at different times during the 1990s. More recently, two 30 m (100 ft) long lateral drains were installed in the north and south ditches running parallel to the pavement. None of these techniques have substantially reduced the effects of the heaving on the road surface.

PROPOSED REMEDIATION

The process of installing panels of extruded polyurethane foam insulation has been used for a number of years. By the mid-1960s, several states including Wyoming and Canadian provinces had ongoing research projects to investigate the suitability of the process (Novak and Mainfort, 1969, DOW, 2008a). The procedure requires a complete reconstruction of the highway, with its attendant costs, time, delays and safety issues.

Tim McGary with the Wyoming Department of Transportation (WYDOT), District 1 Maintenance Engineer, and Roy Mathis, Concrete Stabilization Technologies, Inc. (CST), recommended a novel procedure of injecting a structural polymer foam into the sub base or subgrade of the road to level the surface and to provide a thermal barrier to reduce heat loss from the lower subgrade. An average of 75 mm (3 in.) of CST Uretek 486 STAR #3 was injected at a depth of 500 mm (18 in.) through the worst sections of the heave and then tapered out away from that zone to provide a smooth transition for the drivers.

The University of Wyoming was contacted to perform installation of data acquisition instrumentation and provide two seasons of monitoring and analysis. This report represents the completion of this project.

Figure 1. Map. Location of Study Area

STUDY OBJECTIVES

The primary objective of this research was to determine whether the controlled injection of a structural polymer foam into the subgrade soil would reduce and stabilize the frost heave occurring at milepost 51.8 on WY-70. This can be broken into three questions.

- Can injection of the foam nondestructively level the road surface and make the road safer to drive without full reconstruction of the site?
- Will the foam provide a sufficient thermal barrier to reduce or eliminate the frost heave caused by penetration at the site?
- Will the continuous blanket of foam create a barrier to moisture during the spring thaw?

REPORT FORMAT

The first chapter provides a brief description of the problem statement, the proposed remediation and the study objectives. The second chapter presents a review of the mechanisms that cause frost heaving and how the proposed remediation should be successful in this application. Additionally, a procedure to predict the depth of frost penetration into the soil profile is defined. The site is described in chapter three. Chapter four presents the construction sequence and the details of the instrumentation installation. Chapter five presents the data collected over the two-year duration of the project. Using the data collected in this research, chapter six develops a procedure for computing frost depth, which is compared to this site and shown to provide realistic results for cases both with and without the injected foam. The project conclusions and recommendations are presented in chapter seven. Additional data, figures and drawings are presented in the appendices.

CHAPTER 2. GROUND FREEZING MECHANISMS

Soil heave has been studied for many years (Casagrande, 1931). In general, three factors must be present for heaving to occur:

- The soil must be susceptible to heaving, typically having silt or fine sand.
- Surface temperatures must be cold enough for a freezing front to develop in the soil.
- Water must be available that will allow ice lenses to develop.

This chapter will discuss how these factors interact to produce heaving and how the injection of a foam layer can alter the last two of these factors.

FROST SUSCEPTIBLE SOILS

Frost heave in roadbed soils is a common occurrence in the northern climes around the world. (ACPA, 2008) There are two types of heave associated with freezing, interstitial water freezing and segregational frost heave. Interstitial freezing occurs in the void volume between the soil particles. Segregational freezing causes ice lenses to develop and usually creates much greater deformation (Lay, 2005). Rapid advance of the freezing front from the surface favors interstitial freezing while segregational freezing is more significant when the heat flux from below essentially balances the heat loss to the surface. The ice lens development is most pronounced when the freezing front is stagnant. An example of this kind of ice lens is given in figure 2.

The process of water freezing in a natural environment is quite complex. Significant effects include:

- While it is well known that pure water will freeze at 0^{o} C (32° F) and one atmosphere of pressure, salts and other impurities in the water can reduce the freezing temperature, in a process known as “freezing point depression”. For example, surface seawater freezes at a temperature of -1.9^{o} C (28.6° F). Similarly, road salts are applied to a road surface to melt the ice. A 10 percent salt solution lowers the freezing point to -6^{o}C (20^{o}F). (Georgia State University, 2005). Salts occurring in the soil cause a similar freezing point depression. This allows the water to flow at temperatures below 0^{o}C (32° F).
- The density of ice is less than that of liquid water because a crystal lattice structure is formed by the ice. The spacing of the molecules in the ice lattice is somewhat greater than the spacing in liquid, hence the density is lower and ice floats on the water surface.
- As the water molecules bond onto the surface of the crystal, it lowers the pressure around the molecule and creates a negative pressure or suction on the particle face. This suction causes capillary water to flow towards the crystal.
- Because water molecules create a regular crystalline lattice structure when they freeze due to hydrogen bonding, ice crystals contain almost pure water. (Cary and Mayland, 1972). Consequently, the salinity of the water around the crystals increases, further dropping the freezing point.
- Water molecules adjacent to a solid surface become bonded to the surface, which alters its thermodynamic properties, including its specific entropy. This prevents the water from freezing immediately at the surface and keeps it in a fluid state.

Figure 2. Photo. Ice lens in Montana silt. (Lay, 2005).

Figure 3 shows a general relationship between the hydraulic properties of the soil and its reaction to frost heaving. (ACPA, 2008). In general, coarse-grained soils have high permeability (or hydraulic conductivity) and low capillarity while clays are the opposite. Silts to very fine sands have the properties that appear to balance between conductivity and capillarity. Coarse soils with deep water are less affected while shallow water depths make the coarse soils more susceptible.

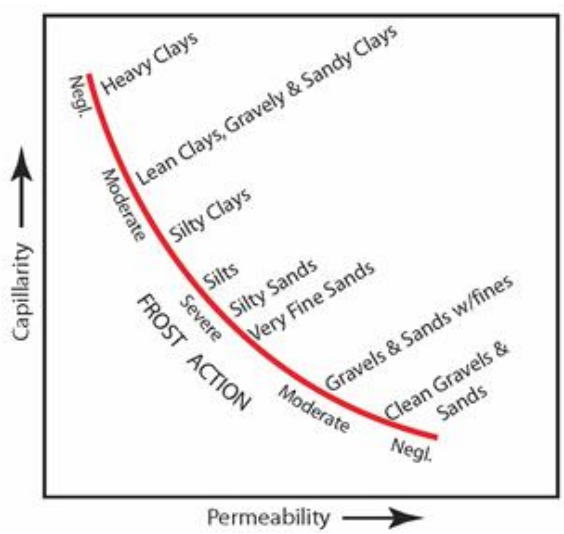


Figure 3. Graph. Relationship between frost action and hydraulic properties of soils. (ACPA, 2008).

In general, interstitial freezing in sands and gravels is not significant because the free water can flow out of the voids as the water expands during freezing. Figure 4 shows an ice crystal forming as the freezing front moves down into the soil. As the ice freezes, it expands and forces some of the water around it to flow out of the void space. Because the water below the freezing front is not frozen, the water can flow downward unimpeded.

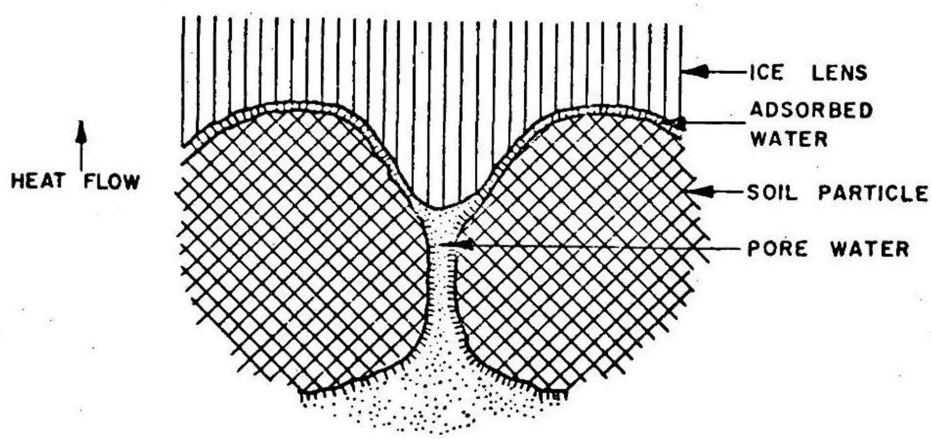


Figure 4. Diagram. Section of an ice lens with soil particles and pore space (Chamberlain, 1981).

Water freezing in clays generally cannot escape because of the low conductivity and will expand by approximately nine percent of the volume of the pore space. The total volume of swell associated with this freezing is usually significantly less than nine percent and in unsaturated clays, a common value is approximately three percent vertical heave. Additionally, clay layers often extend over large areas and whatever heaving may occur tends to be fairly uniform.

Segregational frost heave is typically much more significant than interstitial freezing and can continue over long periods of time. Studies in the mechanism of frost heave have taken place since the 1930s. Three items are necessary for a soil to have frost heave potential:

1. The soil must have substantial amounts of silt or fine sand. Silty and fine sandy soils have the combination of characteristics that allow the formation of ice lenses. The silts are coarse enough that the hydraulic conductivity can be many times that of clay. The air voids in the silt may be large enough for the water vapor to diffuse over a relatively large distance.
2. Temperatures at the ground surface must be significantly colder than freezing. This condition is obviously met in Wyoming and the northern states. The problem is made worse in road surfaces since they are normally cleared of snow, which acts to expose the surface to greater temperature fluctuations than the snow covered unplowed ground surface around it.
3. A source of water must exist below the freeze zone. The source of water does not need to be shallow, and may be very deep. Studies have indicated water at a depth of 6 m (20 ft) can be the source for ice lens water. The soil may be saturated or unsaturated and the water may flow due to positive total head gradient or to suction. In unsaturated soil, the flow can be in thin films caused by absorption or capillary forces surrounding the particles or by vapor flow from the warmer lower water to the colder lenses of ice.

When these three conditions are met, ice lenses can develop in the soil. As the ice freezes, it creates a negative partial pressure that is satisfied by the adsorption of pure water on the ice surface, causing the ice lens to expand. As the ice lens grows, it lifts the soil above it and causes the heave. Osmotic effects are also significant since the increased salt concentration in the unfrozen water causes freezing point depression and the water remains mobile at temperatures well below 0° C (32^{o} F).

Water from the surface generally does not affect the heave as much as water from below because it is likely to be frozen and has little tendency to sublimate downward to the ice lenses. It can be a contributor during the transition seasons of fall and spring when the large temperature swings of freeze/thaw can occur.

DETERMINING FROST SUSCEPTIBILITY OF SOILS

E. J. Chamberlain (1981) of the Cold Regions Research and Engineering Laboratory studied over one hundred criteria that had been published internationally to predict the frost susceptibility of soils. Many techniques have been developed to predict freezing behavior based on particle size percentage, grain size distribution, uniformity coefficients and Atterberg limits, i.e., the soil index tests. He also reviewed techniques that used pore size distribution, moisture-tension, hydraulic conductivity, heave-stress and frost heave tests.

The most common criteria dealt with the percentage of fines (-#200, > 0.074 mm) or percent less than 0.02 mm (in the middle of the silt sizes) and the grain sized distribution of the soil. For example, the United States Army Corps of Engineers has developed a Frost Susceptibility Classification system, presented in table 1, based on “Percentage finer than 0.02 mm by weight”. The data set on which it was based is presented in figure 5 on remolded soils frozen under controlled laboratory conditions. A number of agencies worldwide have developed similar tests and there is an ASTM Standard D5918-13 “Standard Test Methods for Frost Heave and Thaw Weakening Susceptibility of Soils” (ASTM, 2013) covering the procedure and its analysis.

The Frost Susceptibility Classification analysis requires performing a hydrometer analysis to determine the percentage smaller than 0.02 mm in the soil. Other tests have correlated heave with the percentage minus 0.075 mm (-#200) which is commonly performed for classification. For example, Croney and Jacobs (1967, cited in Janoo, etal 1997) performed controlled heave tests on non-cohesive samples 100 mm in diameter and 150 mm tall. Their data indicates heaves up to 40mm (27 percent) on a sample having 26 percent smaller than 0.075 mm (figure 6). They have also correlated their results to the Plasticity Index, PI, for the soil in figure 7. It has the interesting result that soils with lower PI’s (usually silts) have greater potential to heave than soils with higher PI’s (clay).

Table 1. Corps of Engineers (COE) Frost Susceptibility Classification.

Frost group	*Soil*	*Percentage finer than 0.02 mm by weight*	*Typical soil types under Unified Soil Classification System*
NFS*	(a) Gravel Crushed stone Crushed rock	0–1.5	GW, GP
	(b) Sands	0–3	SW, SP
PFS†	(a) Gravel Crushed stone Crushed rock	1.5–3	GW, GP
	(b) Sands	3–10	SW, SP
S1	Gravely soils	3–6	GW, GP, GW-GM, GP-GM
S2	Sandy soils	3–6	SW, SP, SW-SM, SP-SM
F1	Gravely soils	6–10	GM, GW-GM, GP-GM
F2	(a) Gravely soils	10–20	GM, GW-GM, GP-GM
	(b) Sands	6–15	SM, SW-SM, SP-SM
F3	(a) Gravely soils	over 20	GM, GC
	(b) Sands, except very fine silty sands	over 15	SM, SC
	(c) Clays, PI > 12	—	CL, CH
F4	(a) Silts	—	ML, MH
	(b) Very fine silty sands	Over 15	SM
	(c) Clays, PI < 12	—	CL, CL–ML
	(d) Varved clays and other fine-grained, banded sediments	—	CL, ML and SM, CL, CH and ML, CL, CH, ML and SM

* Non-frost-susceptible.

† Possibly frost-susceptible, requires lab test to determine frost design soil classification.

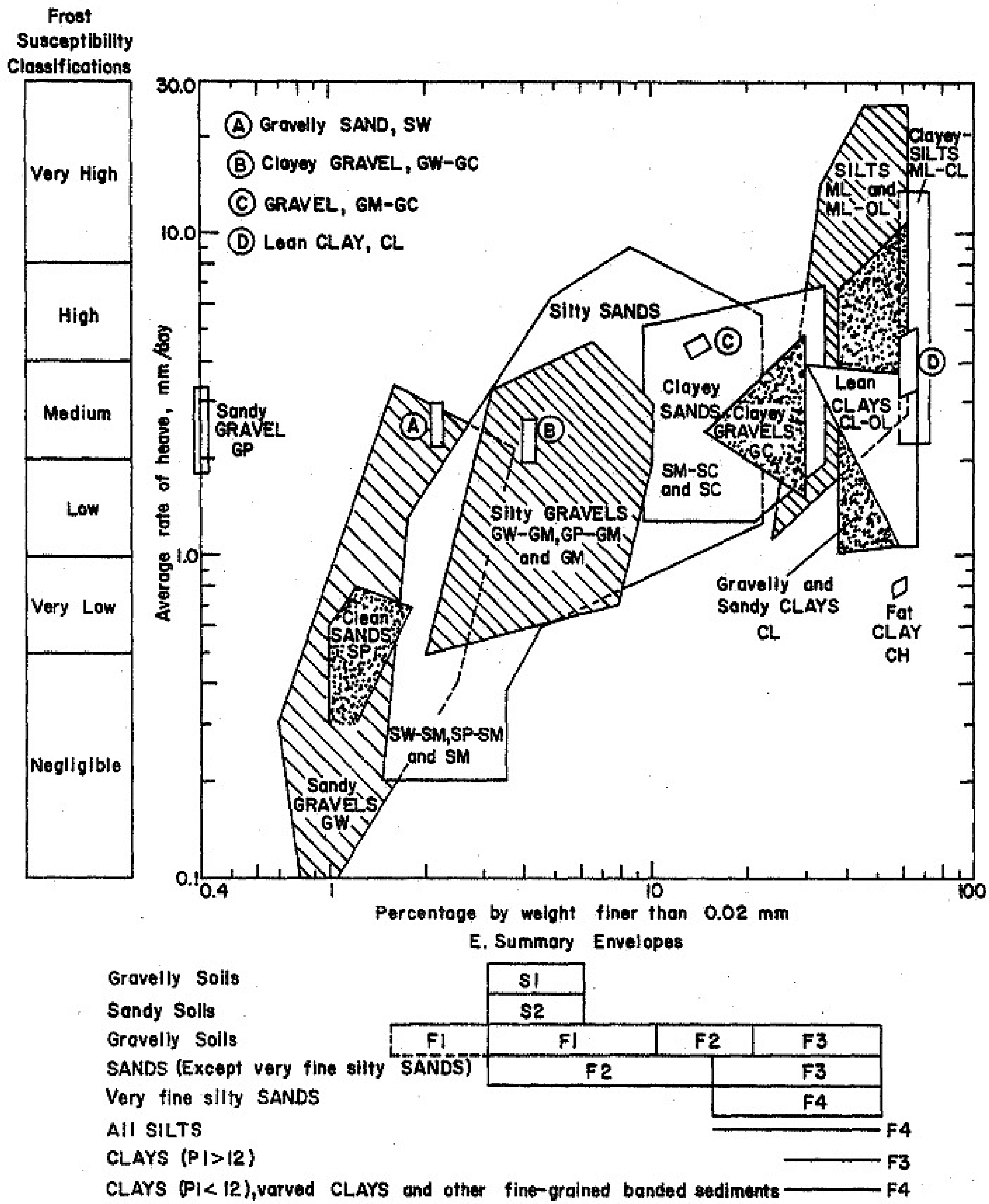


Figure 5. Graph. Rates of heave in laboratory freezing tests on remolded soils.

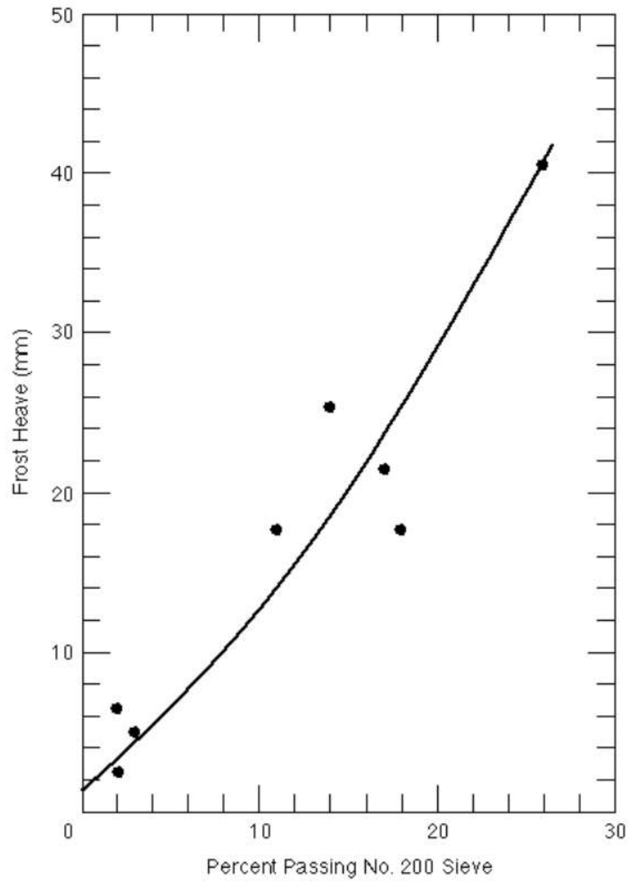


Figure 6. Graph. Heave related to percent fines in non-cohesive soils (Croney and Jacobs, 1967).

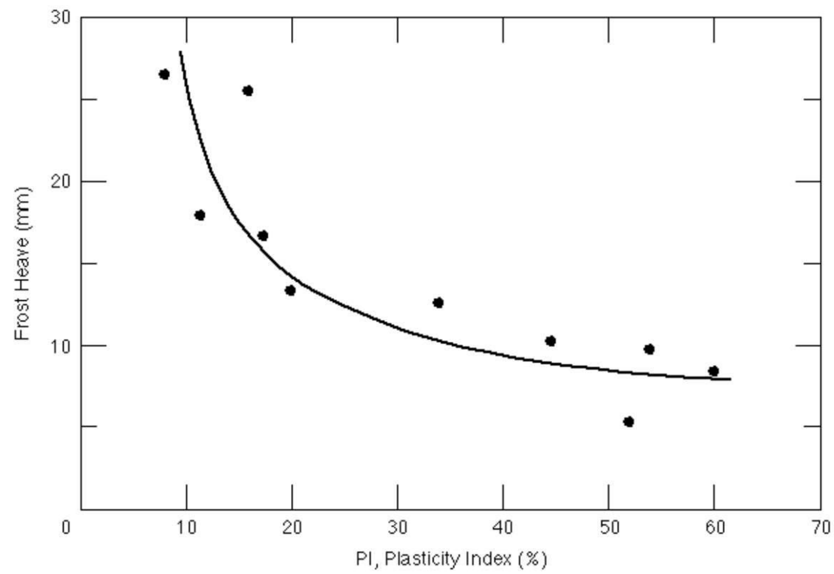


Figure 7. Graph. Heave related to the Plasticity Index, PI (Croney and Jacobs, 1967)

Other organizations have related heave to soil gradations. The Canadian Department of Transportation has developed a grain sized distribution chart correlating size ranges to frost susceptibility, Figure 8. An important feature of this chart is that the really fine-grained soils, i.e., the clays, have reduced susceptibility compared to the silts.

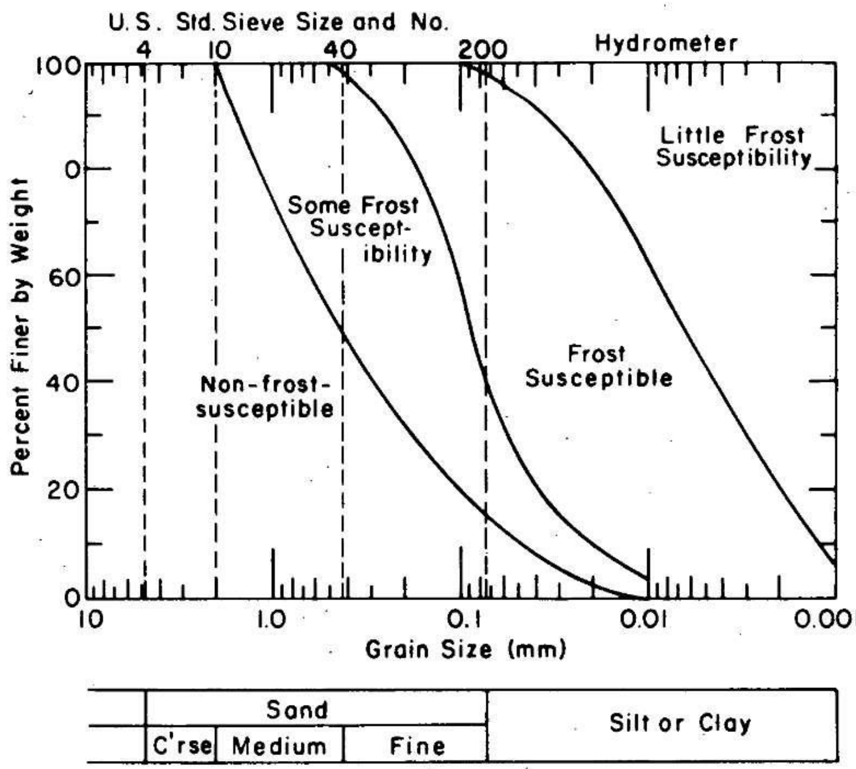


Figure 8. Graph. Canadian Department of Transportation Frost Susceptibility criteria (Janoo, et al., 1997)

FROST HEAVE MITIGATION TECHNIQUES

There are several common strategies for mitigating the problem of frost heave damage in roads (Freitag and McFadden 1997; McCarthy 2007; and McFadden and Bennett 1991). The most common techniques are:

- Remove the frost-susceptible soil and replace it with non-frost-susceptible soil.
- Remove or cut off the source of water that feeds the growth of the ice lenses. This can often be done by putting drains in the soil to remove the water. Sometimes a barrier layer can be installed in the soil to prevent the capillary action from below. A similar technique has been to wrap the frost susceptible soil in the subgrade in a layer of geomembrane, so that water cannot flow into the soil. The base courses are placed on the wrapped soil. The soil may still freeze, but it is only interstitial freezing and not segregational freezing.
- Protect the soil from the freezing temperatures. This can be done in a number of ways, although the most common is to insulate the soil from the source of the freezing temperatures and trap heat from the soil below.

Some combination of these factors is often used to mitigate the frost heave.

Insulation Method in Soil

The most common method used to insulate the soil is to excavate the upper layers of the subgrade and place insulation on the soil surface, cover with the base layers, and repave the road surface, thus requiring a full construction from the subgrade up. (Freitag and McFadden 1997; McCarthy 2007; and McFadden and Bennett 1991). An insulation material commonly used is extruded polystyrene foam, often called "blue board" or "pink board" because of its color, although other materials can be used also. Dow Chemical (2008a) lists over 50 projects using its product since 1963, including Togwotee Pass in Dubois in 1965. This was also used at Bondurant, WY. Blue board has an R-value of around 35 $m^2 \cdot K/W \cdot m$ (5 $hr \cdot ft^2 \cdot °F/Btu \cdot in.$ or R5 per in.) (Dow 2008b). The R-value of a material is a measure of the materials resistance to heat flow. It is inversely related to the thermal conductivity of the material. A high R-value of material will correspond to a low conductivity. Soil has an R-value of 7 to 21 $m^2 \cdot K/W \cdot m$ (R1 to R3 per foot of thickness or R0.08 to R0.25 per in. of thickness).

Effects of Insulation Layers on Temperature Distribution

The purpose of insulation under a roadway is to trap heat from below and prevent heat loss to the cold air above. By preventing heat loss, the temperature of the frost-susceptible soils may stay warm enough to remain above freezing. An insulation layer reduces heat flow by having a low thermal conductivity that reduces the heat flow across the material. For instance, when constructing a house, insulation can be installed in the walls to prevent heat flow. Since heat flows from areas of high temperature to low temperature, insulation traps heat inside a house during the winter to keep it warm. The reverse is also true; insulation reduces heat flow from the outside to the inside of a house during the summer, which keeps the house cooler. The process is described in Weller and Youle (1981).

The same principle works in soils in which the intent is usually to prevent the temperature in the soil from decreasing below freezing. Based on Fourier's Law (figure 9), the heat flux is proportional to the temperature differential and the thermal conductivity and is inversely proportional to the thickness of the material of interest, where q_h is the heat flux, k_t is the thermal conductivity, T is the temperature and z is the distance in the vertical direction (Potter, 2012).

$$q_h = -k_t \frac{dT}{dz}$$

Figure 9. Equation. Heat Flux Equation

The law of Energy Conservation requires that the difference between the heat flow into and out of the element must equal the rate of change of heat stored inside the element, which can be represented by the product of the volumetric heat capacity, C and the change in temperature inside the element, dT, producing the equation given in figure 10.

$$C\frac{\partial T}{\partial t} = -\frac{\partial\left(-k_t \frac{dT}{dz}\right)}{\partial z} = k_t \frac{\partial^2 T}{\partial z^2}$$

Figure 10. Equation. Conservation of Heat Energy.

This equation describes the heat flow in conduction in a simple homogeneous material with constant C and k_t and without phase changes. While this relationship neglects the effects of water freezing in the soil, it provides insight into the thermal regime beneath the surface. In particular, if it can be assumed that the temperature at the ground surface varies sinusoidally, a solution is given by the following equation (figure 11).

$$T(z,t) = T_{ave} + A_o * e^{\frac{-z}{d}} * \sin\left(\omega t - \frac{z}{d}\right)$$

Figure 11. Equation. Sinusoidal Ground Surface Temperature Fluctuation.

$T(z,t)$ is the temperature at any depth and time, T_{ave} is the average temperature of the soil at great depth, A_o is the amplitude of the sine wave describing the temperature fluctuation at the surface, and ω is the radial frequency of time and would typically represent a daily 24-hour variation or an annual 365-day variation. The damping depth d is a characteristic depth given by the equation given in figure 12 and relates the reduction in temperature fluctuation, A_o to the depth.

$$d = \left(\frac{2k_t}{C\omega}\right)^{\frac{1}{2}}$$

Figure 12. Equation. Damping Factor.

Figure 13 shows an example in which the average surface and soil temperature are initially 5°C (41°F) and the amplitude of temperature fluctuation at the surface A_o is ±8°C (46.4°F). Four solutions are shown which are determined at six-hour intervals. The dashed lines indicate the maximum range of temperatures that will occur at any depth.

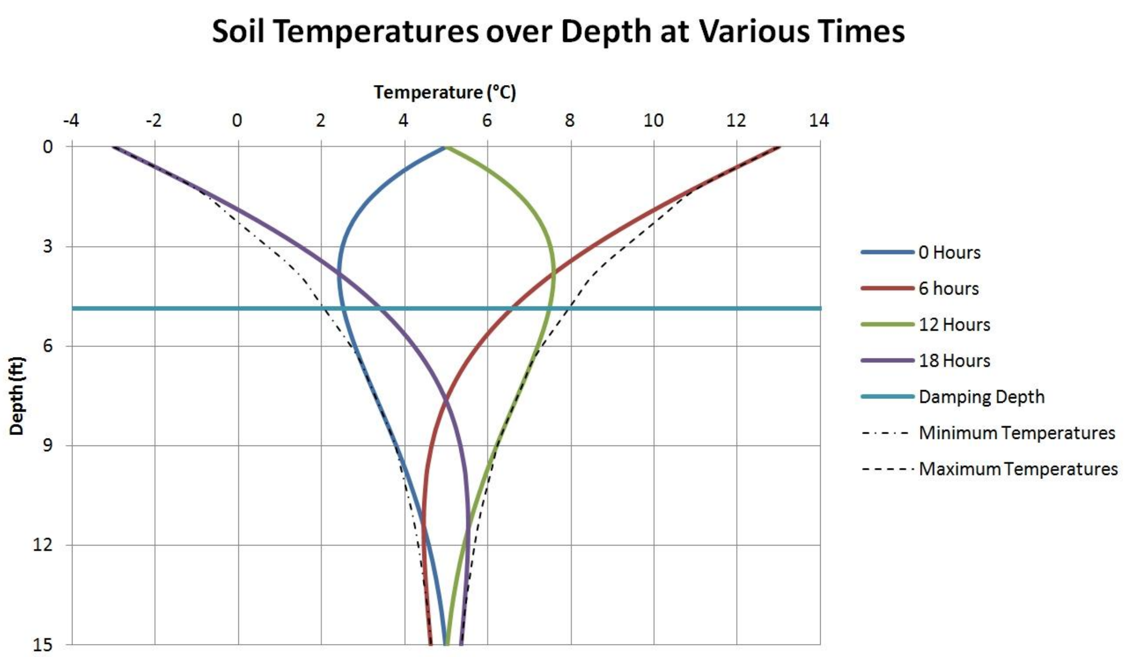


Figure 13. Graph. Example of soil temperature variation with depth in a homogeneous soil.

This temperature pattern will change greatly if a low thermal conductivity layer, such as a layer of rigid foam, is embedded into the profile. Adding a layer of low conductivity foam has the same effect as adding additional soil into the profile by an amount equal to the square root of the ratio of the soil conductivity to the foam conductivity (figure 14).

$$Depth_{soil} = \sqrt{\frac{k_t}{k_{foam}}} Depth_{foam}$$

Figure 14. Equation. Scaled depth of soil caused by insulation layer.

A typical value of soil thermal conductivity is 15 BTU/ft^2-hr-^{o}F/inch while that of a polyurethane foam is 0.18 BTU/ft^2-hr-^{o}F/inch. This would indicate that 6 inches (152.4 mm) of foam would act like 55 inches (1.4 m) of soil (figure 15).

$$Depth_{soil} = \sqrt{\frac{k_t}{k_{foam}}} Depth_{foam} = \sqrt{\frac{15}{0.18}} 6" = 9.1 * 6" = 54.6"$$

Figure 15. Equation. Scaled depth of six inches of foam.

Conversely, the temperature changes that occur over 55 inches (1.4 m) of soil would occur in just 6 inches (152.4 mm) of foam. Figure 16 shows this effect for a 6-inch (152.4 mm) layer of foam at a depth of 18 inches (0.5 m). The temperature at the bottom of the foam layer at a depth of 2 feet (0.6 m) is the same as the temperature at a depth of 18+55 = 73 inches (1.9 m), or about a depth of 6 feet (1.8 m) in figure 13.

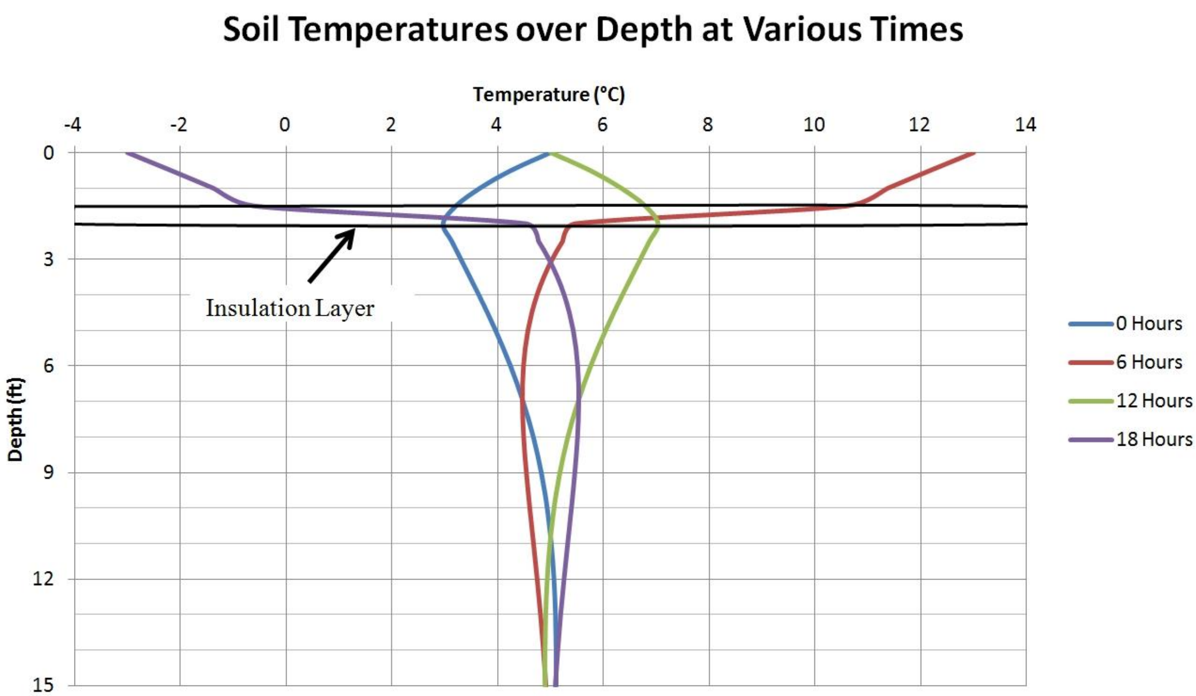


Figure 16. Graph. Soil temperature variation with depth in homogeneous soil with an insulation layer.

EXPANDING STRUCTURAL POLYMER

The solution being explored in this project is to inject an expanding structural polymer produced by Uretek to act as insulation. The Uretek material is a two part expanding structural polymer that is mixed and injected directly into the soil. The chemical reaction causes the mix to expand and the resulting material fills nearly all voids in the soil and compacts the soil around the injection area (www.uretekusa.com, nd).

The polymer does not react with water, so it displaces the water in the voids that it enters without contaminating the ground water. The material is durable and chemical resistant so it can remain in the ground for years without breaking down. The polymer is lightweight so it does not contribute to additional soil settling. The material is strong so it increases the bearing capacity of the soil and can distribute a surface load over a greater area by creating a bridging effect. The polymer is inert and odor free so it is environmentally friendly. The deep injection process can fix problems in hours that would normally take days and can fix many issues in a non-destructive manner that would have needed to be removed if repaired with conventional methods.

The Uretek expanding structural polymer is typically used for a variety of purposes. The material can be injected under a slab, curb or roadway that is settling to lift it back into place. The polymer can be injected in order to stabilize soils that have been weakened by frost heave action or other causes or that was improperly compacted by filling voids and strengthening the soil structure. The polymer can also be injected around leaking water pipes to seal the leaks and fill any voids created by leaking water.

Uretek is just now beginning to test the expanding structural polymer as an insulation material. Tests conducted on one of Uretek's chemical grouts by Testing, Engineering & Consulting Services, Inc. (TEC Services) showed that the grout had a thermal conductivity of 0.016 BTU/ft-hr °F (0.192 BTU/ft^2-hr °F/in) and an R-value of 35 $m^{2\cdot}$K/W·m (5 hr·ft^2·°F/Btu·in. or R5 per in.)of thickness of pure material (McCants and McCormick 2011). The R-value for a mix of soil and the polymer was shown to be 1.12 $m^{2\cdot}$K/W·m (0.16 hr·ft^2·°F/Btu·in. or R0.16 per in.) of thickness, which is the same value as the soil without any grout. This shows that the material has the potential to provide insulation if a thickness of pure material can be achieved, but it may not work as well if the material is just distributed amongst the soil. These results do not appear to be consistent with mixing theory and more research should be performed to verify or alter that number.

DEPTH OF FROST PENETRATION

The Minnesota Department of Transportation (MNDOT, 2007) has a procedure in its Pavement Design Manual for predicting the depth of frost penetration under a road surface. It is based on the Modified Berggren Equation (MBE) developed by Aldrich and Paynter (1953) under contract to the U. S. Army Corps of Engineers and modified by Berg (1997) (figure 17). While the original relationship was developed for a homogeneous material by Stefan (1889) who was determining the thickness of arctic ice floes, their development extended the procedure to layered soils.

$$z = \lambda\sqrt{\frac{48k_t nFI}{L}}$$

Figure 17. Equation. The MBE Equation for Frost Depth in a Single Layered Soil.

The depth of frost penetration in a homogeneous soil is given by MNDOT (2007) in a form of the Stefan equation (figure 18).

$$z = \sqrt{\frac{48k_t FI}{L}}$$

where
z is the depth of freezing (feet),
k_t is the thermal conductivity of the layer (BTU/ft-hr.-^{o}F),
FI is the Air Freezing Index at the site (^{0}F-days),
n is the ratio of the Ground Surface Freezing Index to the Air Freezing Index,
λ is a function of the initial ground temperature and the thermal capacities of the soil, and
L is the volumetric heat of latent fusion (BTU/ft^3),

Figure 18. Equation. MNDOT Form of the Stefan Equation.

The volumetric heat of latent fusion is given by the following equation (figure 19).

$$L = 1.43 w \gamma_d$$

where w is the water content (percentage), and
γ_d is the dry unit weight of the layer (pcf).

Figure 19. Equation. Volumetric Heat of Latent Fusion (BTU/ft^3).

The terms λ and n are reduction factors that reduce the depth projected by the Stefan equation. Their combined effect would commonly reduce the depth by 15 to 30 percent. As a design aid, it is appropriate to over-estimate the frost penetration depth when determining thickness that must be removed and replaced with non-heaving fill. It will be shown in chapter 6 that the uncorrected equation is appropriate for this project.

The amount of energy available to remove heat from the road soil profile is given by the Freezing Index, FI. It is the average daily temperature below freezing summed over the winter season. Values of the Freezing Index for a number of communities in Wyoming are given in table 2 (NOAA. 2012). The depth of freezing is controlled by the FI. The energy required to freeze a layer of soil of thickness d_i is given by the equation in figure 20.

$$FIR_i = \frac{d_i^{\,2} L_i}{48k_{t-i}}$$

Figure 20. Equation. Energy Required to Freeze a Layer of Soil.

FIR_i denotes the required FI to freeze an individual material layer. That quantity is subtracted from the previous value of FI until a layer indicates it requires more FIR_i than is available. The equation given in figure 14 is then solved with the remaining FI to find the final depth of freezing, z.

Example:

Six inches of PMP concrete overlays 10 inches of base course that is over a silty sand. The PMP has a dry unit weight of 144 pcf and a water content of 3 percent. The base course has a dry unit weight of 120 pcf and a water content of 7 percent. The silty sand has a dry unit weight of 110 pcf and a water content of 7.5 percent. According to MNDOT (2007), a typical value of thermal conductivity for asphaltic concrete is 10 BTU/ft^2-hr-°F/inch (0.83 BTU/ft$^-$hr-^{o}F).

From table 2 for Encampment, the Freezing Index at the 2-year recurrence interval is 1046 ^{o}F-Days. Using the equation given in figure 19:

$$L_{PMP} = 1.43(144\,pcf)(3\%) = 618\frac{BTU}{ft^3}$$

$$L_{Base} = 1.43(120\,pcf)(7\%) = 1200\frac{BTU}{ft^3}$$

$$L_{sand} = 1.43(110\,pcf)(7.5\%) = 1180\frac{BTU}{ft^3}$$

The thermal conductivity, k_t, for the base course and silty sand are determined from charts developed by Kersten, 1952 (figures 21-24). In the freezing depth analysis, it is appropriate to use the conductivity values for frozen soils. Thawing analysis would use the unfrozen values. The conductivity of the silty sand is the average of the two frozen charts (figure 21 and 22).

$k_{t\text{-}Base}$ = 1.3 BTU/ft$^-$hr-^{o}F.

$k_{t\text{-}Silty\ Sand}$ = (1.0 + 0.6)/2 = 0.80 BTU/ft$^-$hr-^{o}F

FI = 1046 ^{o}F-Days

$$FIR_{PMP} = \frac{(6/12)^2(618)}{48(0.83)} = 46^{\circ}\mathrm{F-Days}$$

$$FIR_{Base} = \frac{(10/12)^2(1200)}{48(1.3)} = 156^{\circ}\mathrm{F-Days}$$

The amount of Freezing Index available for the silty sand is

$$FI_{silty\ sand} = FI - \sum_{i=1}^{n} FIR_i =$$
$$= 1046 - (46 + 156) = 844^{\circ}\mathrm{F-Days}$$

Using that FI, the depth of freezing in the silty sand is given by

$$z_{SiltySand} = \sqrt{\frac{48(0.80)(844)}{1180}} = 1.76\,ft$$

Frost Depth = 0.50 ft + 0.83 ft + 1.76 ft = 3.1 feet which agrees well with the 3.3 foot frost depth in Hole #1.

Table 2. Air Freezing Index for Selected Wyoming Communities (NOAA, 2012)

Air Freezing Index - USA Method (Base 32° Fahrenheit)

Location	Sta. No.	Lat. (ddmm)	Long. (dddmm)	Elev. (ft)	Mean Annual Temp (F)	Air Freezing Index Return Periods (°F-Days) & Associated Probabilities (%)										
						1.1 yr (10%)	1.2 yr (20%)	2 yr (50%)	2.5 yr (60%)	3.3 yr (70%)	5 yr (80%)	10 yr (90%)	20 yr (95%)	25 yr (96%)	50 yr (98%)	100 yr (99%)
AFTON	480027	N4244	W11056	6210	38.7	1101	1289	1637	1736	1839	1955	2108	2228	2262	2357	2439
ALBIN	480080	N4125	W10406	5345	46.9	268	351	528	584	645	716	814	895	918	985	1045
ALTA 1 NNW	480140	N4346	W11102	6431	39.7	975	1125	1396	1472	1551	1639	1755	1845	1871	1942	2003
ALVA	480200	N4439	W10421	4390	41.8	725	880	1179	1267	1360	1466	1608	1721	1753	1843	1923
ARCHER	480270	N4109	W10439	6010	45.9	273	359	546	605	669	744	849	936	961	1032	1096
BASIN	480540	N4423	W10803	3837	45.9	568	766	1207	1349	1505	1690	1951	2167	2230	2411	2574
BONDURANT 3 NW	480865	N4314	W11026	6504	33.3	2069	2287	2660	2761	2864	2977	3122	3234	3265	3351	3425
BORDER 3 N	480915	N4215	W11102	6120	38.2	1302	1518	1916	2029	2146	2278	2451	2587	2626	2733	2826
BOYSEN DAM	481000	N4325	W10811	4642	47.6	429	604	1015	1153	1307	1492	1757	1982	2048	2239	2413
BUFFALO BILL DAM	481175	N4430	W10911	5156	46.8	108	174	356	425	505	606	760	897	939	1062	1177
CARPENTER 3 E	481547	N4102	W10418	5390	47	232	308	469	521	577	642	734	810	832	894	950
CASPER 2 E	481565	N4251	W10616	5195	47.4	230	309	484	541	603	676	779	864	889	960	1024
CASPER WSO//	481570	N4255	W10628	5338	45.2	441	569	833	916	1004	1107	1249	1365	1398	1493	1578
CHUGWATER	481730	N4145	W10449	5282	46.7	200	270	426	476	531	597	689	766	788	852	909
CODY	481840	N4433	W10904	4990	46	256	370	647	743	850	980	1170	1331	1379	1518	1645
COLONY	481905	N4456	W10412	3553	46.2	504	660	992	1097	1210	1343	1528	1680	1724	1849	1961
CRANDALL CREEK	482135	N4454	W10940	6600	38.2	1040	1200	1489	1570	1654	1748	1872	1968	1995	2071	2136
DEAVER	482415	N4453	W10836	4105	44.7	580	760	1143	1263	1394	1547	1760	1934	1985	2129	2258
DILLINGER	482580	N4407	W10507	4306	43.8	674	843	1179	1281	1389	1514	1683	1820	1859	1969	2067
DIVERSION DAM	482595	N4314	W10856	5574	44.3	427	579	916	1026	1146	1289	1490	1658	1707	1847	1973
DIXON	482610	N4102	W10732	6360	41.3	812	999	1366	1475	1590	1723	1902	2045	2086	2201	2303
DULL CENTER 1 SE	482725	N4325	W10457	4415	46.6	351	472	739	825	919	1031	1188	1318	1356	1465	1562
ELK MOUNTAIN	482995	N4141	W10625	7265	41.5	632	755	986	1053	1122	1202	1308	1391	1415	1481	1539
ENCAMPMENT 10 ESE	483045	N4111	W10637	7387	41.7	703	823	1046	1109	1175	1249	1346	1423	1445	1506	1558
EVANSTON 1 E	483100	N4116	W11057	6780	39.8	838	1027	1397	1506	1622	1755	1934	2077	2118	2233	2334
FARSON	483170	N4207	W10927	6591	37.3	1410	1701	2258	2422	2593	2788	3049	3256	3315	3481	3626
GILLETTE 2 E	483855	N4417	W10528	4556	45	502	647	948	1041	1141	1259	1420	1552	1590	1698	1794
GILLETTE 18 SW	483865	N4405	W10543	4903	44.2	531	677	976	1069	1167	1282	1439	1567	1604	1709	1801
GLENROCK 5 ESE	483950	N4250	W10547	4948	47.8	209	295	495	563	638	729	858	968	1000	1094	1178
GREEN RIVER	484065	N4132	W10929	6089	42.7	713	899	1274	1388	1510	1651	1843	1999	2044	2170	2282
HEART MOUNTAIN	484411	N4441	W10857	4790	44.4	459	618	968	1081	1204	1350	1555	1726	1775	1918	2045
JACKSON	484910	N4328	W11046	6244	38	1142	1338	1699	1801	1908	2028	2187	2312	2347	2445	2531

Table 2 (cont.). Air Freezing Index for Selected Wyoming Communities (NOAA, 2012)

Air Freezing Index - USA Method (Base 32^{o} Fahrenheit)

Location	Sta. No.	Lat. (ddmm)	Long. (dddmm)	Elev. (ft)	Mean Annual Temp (F)	Air Freezing Index Return Periods (^{o}F-Days) & Associated Probabilities (%)										
						1.1 yr (10%)	1.2 yr (20%)	2 yr (50%)	2.5 yr (60%)	3.3 yr (70%)	5 yr (80%)	10 yr (90%)	20 yr (95%)	25 yr (96%)	50 yr (98%)	100 yr (99%)
KAYCEE	485055	N4343	W10638	4660	44.8	431	566	856	947	1046	1162	1324	1457	1496	1606	1704
LAGRANGE	485260	N4138	W10410	4582	47.3	254	340	530	591	657	736	847	938	965	1041	1110
LANDER WSO //	485390	N4249	W10844	5563	44.4	507	682	1067	1192	1327	1489	1715	1903	1958	2115	2255
LARAMIE FAA AIRPORT	485415	N4119	W10541	7266	40.7	783	939	1237	1323	1414	1517	1655	1764	1795	1882	1958
LOVELL	485770	N4450	W10824	3837	44.7	582	775	1194	1328	1474	1647	1887	2086	2144	2310	2458
MEDICINE BOW	486120	N4154	W10612	6570	41.6	686	830	1107	1188	1273	1371	1501	1605	1635	1718	1791
MIDWEST 1 SW	486195	N4325	W10617	4840	47.2	284	392	636	717	805	912	1062	1189	1226	1332	1428
MOORCROFT	486395	N4416	W10457	4275	44.1	698	858	1174	1268	1367	1481	1635	1758	1793	1892	1979
MORAN 5 WNW	486440	N4351	W11035	6798	35.6	1574	1766	2103	2195	2289	2394	2529	2634	2663	2744	2814
NEWCASTLE	486660	N4351	W10413	4265	46.5	483	615	884	967	1055	1158	1298	1413	1445	1539	1621
PATHFINDER DAM	487105	N4228	W10651	5930	44.7	540	672	934	1013	1097	1194	1325	1431	1461	1546	1621
PAVILLION	487115	N4315	W10841	5440	44.8	431	591	953	1072	1202	1359	1580	1765	1819	1975	2115
PHILLIPS	487200	N4138	W10429	4982	47.6	156	223	384	439	500	575	682	774	801	879	950
POWELL	487380	N4445	W10846	4378	46.7	365	506	826	932	1049	1189	1388	1556	1605	1746	1874
REDBIRD 1 NW	487555	N4316	W10418	3880	46.4	459	606	919	1019	1126	1253	1430	1575	1617	1738	1845
RIVERTON	487760	N4301	W10823	4954	42.8	784	1018	1511	1665	1831	2025	2294	2514	2578	2758	2919
ROCHELLE 3 E	487810	N4336	W10454	4496	45	516	668	987	1086	1193	1319	1492	1633	1674	1790	1894
ROCK SPRINGS	487840	N4135	W10913	6367	44.1	454	597	902	998	1102	1225	1396	1536	1577	1693	1796
SARATOGA	487990	N4127	W10648	6790	42.6	642	766	1000	1068	1139	1220	1327	1412	1436	1503	1562
SEMINOE DAM	488070	N4208	W10653	6838	43.2	676	818	1091	1171	1255	1351	1480	1582	1611	1693	1765
SHERIDAN WSO //	488155	N4446	W10658	3964	44.6	434	596	960	1080	1212	1369	1592	1779	1834	1990	2132
SHERIDAN FIELD STA	488160	N4451	W10652	3800	44	613	796	1181	1302	1431	1584	1794	1966	2015	2157	2283
SUNDANCE	488705	N4424	W10421	4750	43.7	717	858	1124	1202	1283	1375	1498	1595	1623	1700	1768
THERMOPOLIS	488880	N4339	W10812	4313	46.5	384	526	846	951	1066	1203	1398	1561	1608	1745	1868
TORRINGTON EXP FARM	488995	N4205	W10413	4098	48.3	191	275	477	546	624	718	855	971	1005	1105	1196
UPTON	489205	N4406	W10437	4261	43.3	926	1099	1424	1518	1616	1727	1874	1991	2024	2116	2196
WHALEN DAM	489604	N4215	W10438	4294	47.7	327	421	614	674	738	814	917	1001	1025	1094	1155
WHEATLAND 4 N	489615	N4207	W10457	4638	48.7	125	189	350	407	473	553	672	775	806	896	979
WORLAND	489770	N4401	W10758	4061	44.4	726	932	1360	1493	1635	1801	2030	2216	2270	2422	2558
YELLOWSTONE PARK //	489905	N4458	W11042	6230	40.2	852	1007	1296	1379	1465	1563	1693	1795	1823	1904	1975
YODER	489925	N4156	W10418	4231	48.4	200	278	457	516	582	661	774	869	896	976	1049

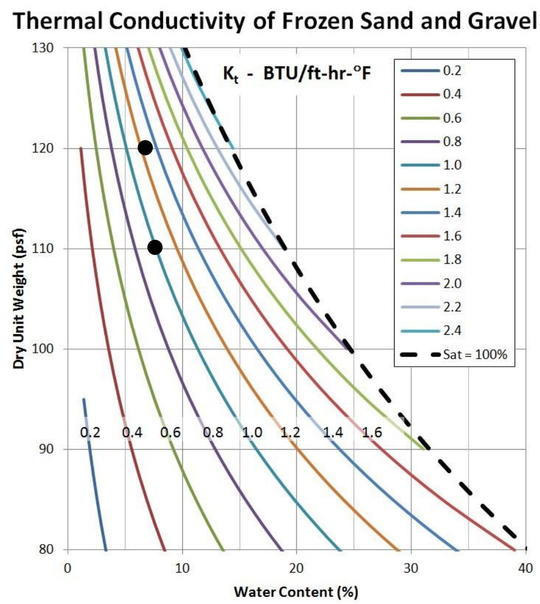


Figure 21. Graph. Thermal Conductivity of Frozen Sands and Gravels (after USACE, 1988).

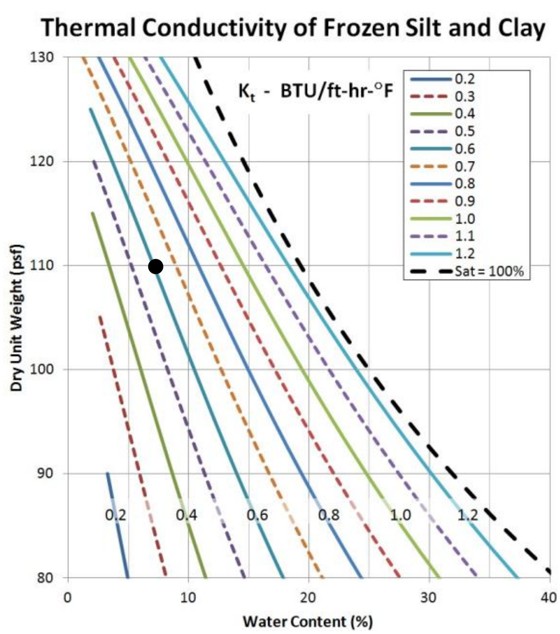


Figure 22. Graph. Thermal Conductivity of Frozen Silt and Clay Soils (after USACE, 1988).

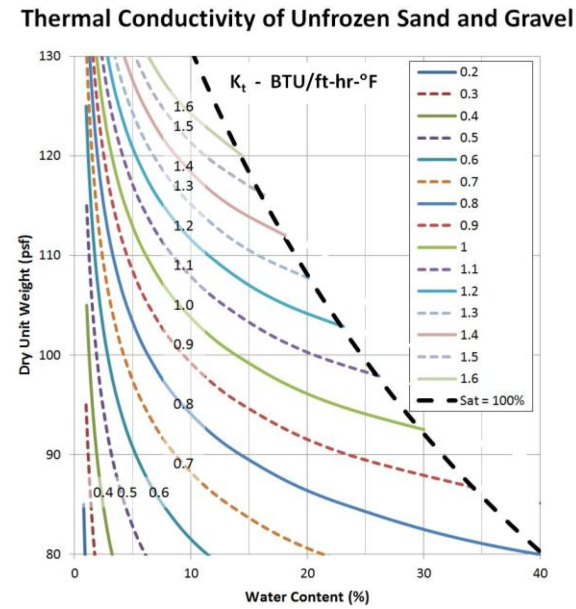


Figure 23. Graph. Thermal Conductivity of Unfrozen Sands and Gravels (after USACE, 1988).

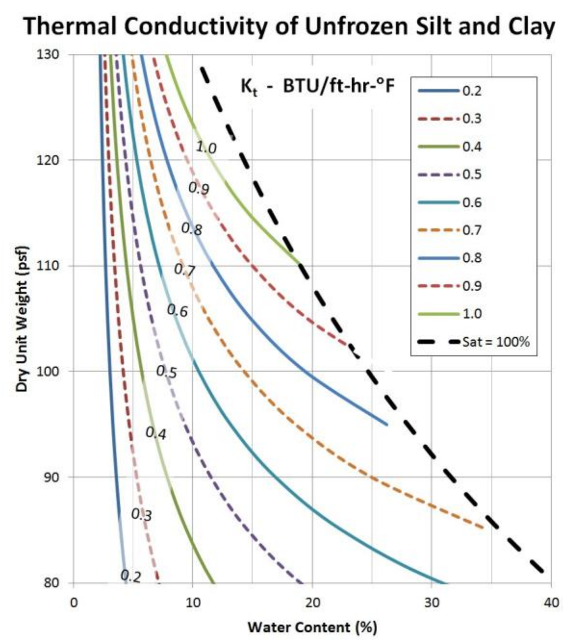


Figure 24. Graph. Thermal Conductivity of Unfrozen Silt and Clay Soils (after USACE, 1988).

CHAPTER 3. SITE DESCRIPTION

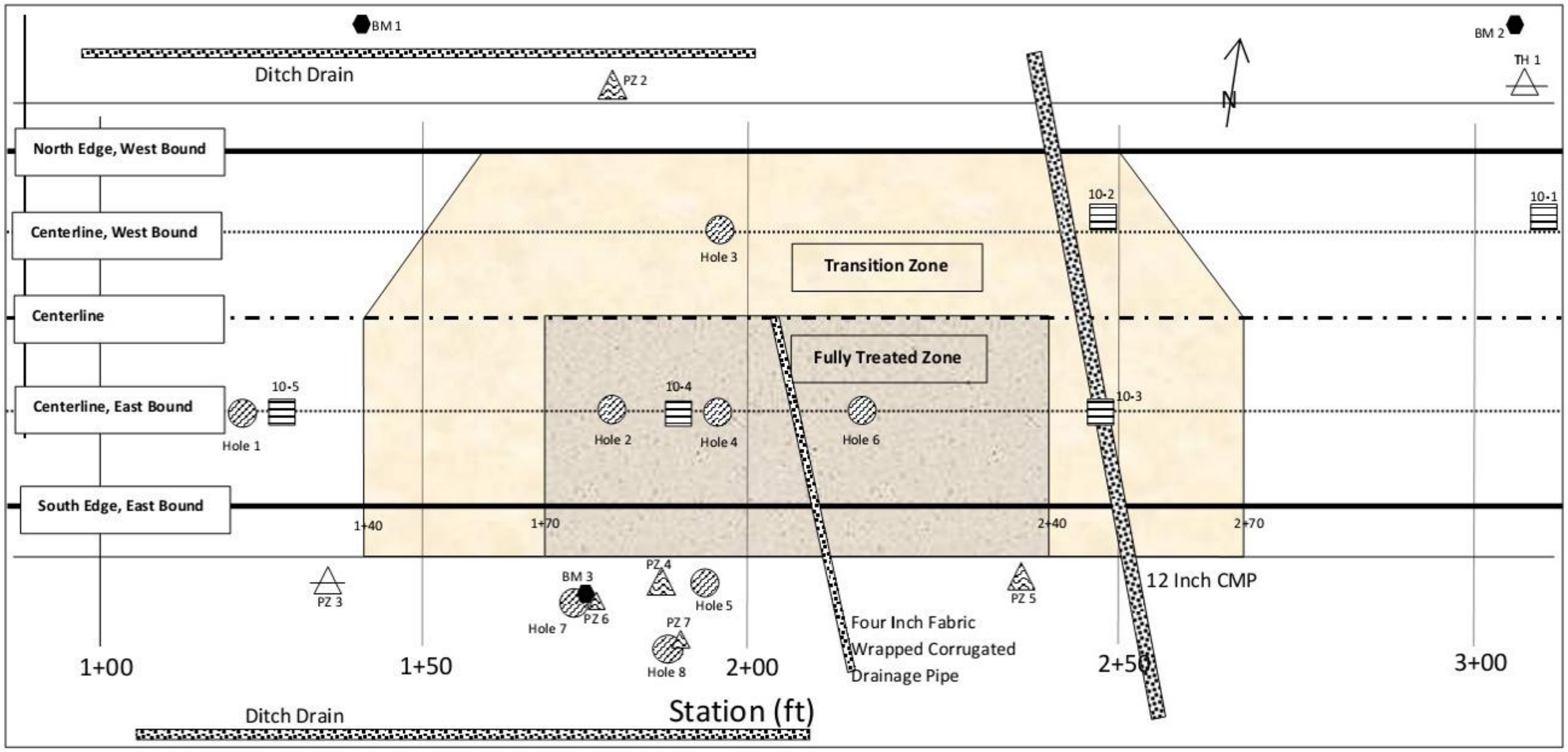
Wyoming Highway WY-70 crosses a geologic ridge line 7.2 km (4.5 mi) west of Encampment at milepost 51.8. This ridge, seen in figure 25 below the black bar, has a strike angle of approximately N80^{o}W. The dip is relatively flat at approximately 10 to 20 degrees. The road crosses the ridge obliquely through a road cut. During construction of the cut section in the late 1980's, the contractor found the material to be hard enough to break teeth on his equipment. While there is no record of the construction, apparently the caprock (which has been reported as both granite and quartzite) is very tough while the underlying layers may be less so. Core samples of the rock were not taken during any investigation, but bedrock depths to the east and west of the cap are deeper than that of the cap.

Figure 25. Photo. Ridge Line Crossing WY70 at MP51.8.

The contractor was permitted by WYDOT to alter the vertical alignment to minimize the rock excavation. He smoothed the surface and buried the caprock in about three feet of silty backfill, changing the vertical alignment to provide smooth transition zones above and below the cut.

SITE INVESTIGATIONS

Three drilling programs have taken place at the site since 1990. Two site investigations were performed to gather details about the site while the third was to install the instrumentation for this research. Each has added information about the site, but none have been conclusive about the nature of the bedrock surface. Figure 26 shows a plan view of the site with all the boreholes and installed drains.



Notes:

4/27/2000 – Piezometers PZ2, PZ4 and PZ5. TH1 was located approximately 125 feet east of PZ2, but was backfilled. PZ3 was located approximately 50 feet west of PZ4, but was not found.

4/22/2010 – Boreholes 10-1 to 10-5. Benchmarks BM1 and BM2 may have been installed.

11/3/2011 – Holes 1 to 8. Piezometers were installed in Hole 7 (PZ6) and Hole 8 (PZ7). Benchmark 3 was installed in Hole 7.

Figure 26. Sketch. Plan View of Site Showing Borings, Piezometers, and Benchmarks.

Drilling on April 17, 2000

The first investigation took place in the spring of 2000 (Miller, 2000). The report is given in appendix A. Five boreholes were drilled and piezometers were installed in four of them. Test Hole TH 1 and Piezometer PZ 2 are located 5.5 m (18 ft) north of the centerline while the other three are about 5.2 m (17 ft) south of the centerline. All are off the shoulder of the highway.

The report summarizes previous remedial work performed at the site. Based on a recommendation from the FHWA, a clean gravel French drain was installed at a depth of 1 m (3 ft) sometime in the early 1990s. This apparently had little effect on the heaving, so in the later 1990s, a fabric wrapped, 102 mm (4.0 in.) perforated PVC underdrain was installed at a depth of 1 m (3 ft) from the centerline of the highway to daylight on the south slope of the embankment. This is located in the middle of the heaving zone, and, although not stated in the report, it is assumed to be on the surface of the caprock. It was backfilled with a stable, non-heaving sand. This drain was also ineffective as the frost heave is at a maximum value on both sides of the drain with a large dip in the center caused by the non-heaving fill.

Miller, who performed the investigation in 2000, recommended that a lateral six-inch diameter PVC drain be installed on each side of the road, 6.7 m (22 ft) from the centerline. He specified an installation depth of 1.8 m (6 ft), but cautioned with the high bedrock on the south side, excavation may be difficult and that special provisions should be made for the excavation, such as jackhammering. In fact, the rock on the south side proved even more difficult to excavate than anticipated, so the trench was only excavated a few feet between STA 1+60 and 2+10 and the line was placed approximately 1 m (3 ft) higher than specified.

Drilling on April 22, 2010

In the early spring of 2010, Rawlins WYDOT field personnel conducted a survey of the road surface along the centerline of the highway and the centerline of the east bound lane. They both indicated a substantial heave on the surface and the presence of the dip in east bound lane.

A second drilling program occurred in spring 2010. Sullivan (2010) reviewed the work that had been done to that time and described the results of drilling five holes in the center of the driving lanes (appendix B). Two borings were drilled to the east of the heave zone in the center of the west bound lane and three were drilled across the heave zone in the center of the east bound lane. The three eastern most holes indicated bedrock depths greater than 2.1 to 2.9 m (7.0 to 9.5 ft). The borehole 10-4 at current station 1+88 had a depth of 1.4 m (4.5 ft). This is located where the heave has been the most significant when driving from the west.

Drilling on November 3, 2011

Eight boreholes were drilled to install the thermal instrumentation, instrumentation post and auxiliary benchmark for this project. Appendix C contains the boring logs for these holes. Two holes, Hole #2 and Hole #4 are in the center of the east bound lane and are on either side of borehole 10-4 from the 2010 investigation. The bedrock depths are 2.0 m (6.5 ft), 1.4 m (4.5 ft) and 2.3 m (7.5 ft) over a 6 m (20-ft) distance. Similarly, Hole #1 from this drilling is 3.0 m (10 ft) from 10-5 from the 2010 investigation. Bedrock was at 0.8 m (2.5 ft) in Hole #1 and 1.7 m (5.5 ft) in 10-5. The bedrock surface, shown in profile in figure 27, appears to be very irregular and rough, as if the contractor stopped trying to make further progress and just filled in the site over the rubble.

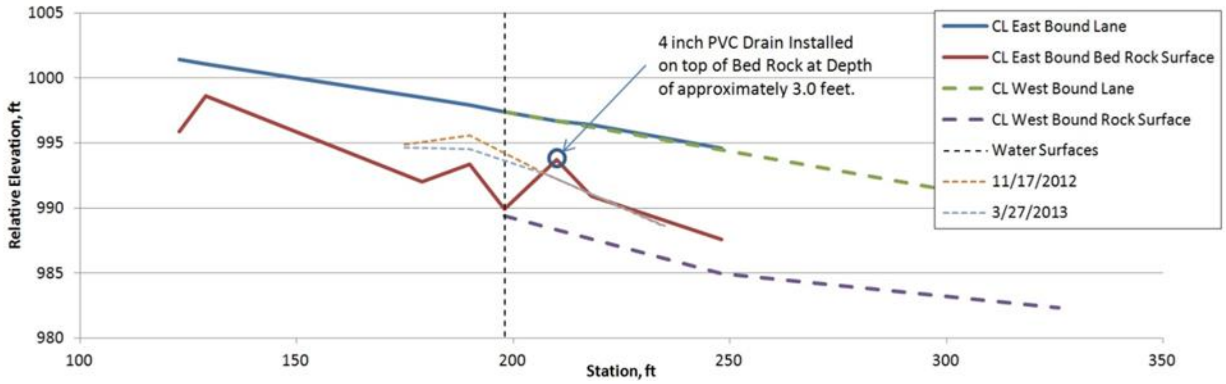


Figure 27. Graph. Profile View of Site. Red line is the bedrock at the center of east bound lane. Heavy dashed line is the bedrock at the center of west bound lane.

SOIL PROFILE

Figure 27 shows a profile view of the site. All of the borings through the road surface indicate 115 to 175 mm (4.5 to 7.0 in.) of plant mix pavement (PMP) over 150 to 230 mm (6.0 to 9.0 in.) of base aggregate. Based on gradation curves, shown in figure 28, the subgrade soil in this area has been classified as “Silty Sand with Gravel”, as SM in the Unified Soil Classification System and as A-4(0) and A-2-4(0) in the AASHTO Soil Classification System. The soils in the two western boreholes are somewhat sandier and lighter in color than the other soils but still fall in the same categories. Based on the Corps of Engineers Frost Susceptibility Classification, all the soils are in the F4b classification group, the highest level of susceptibility. There was evidence of weathered rock in some holes in the bottom 0.3 m (1.0 ft) of drilling.

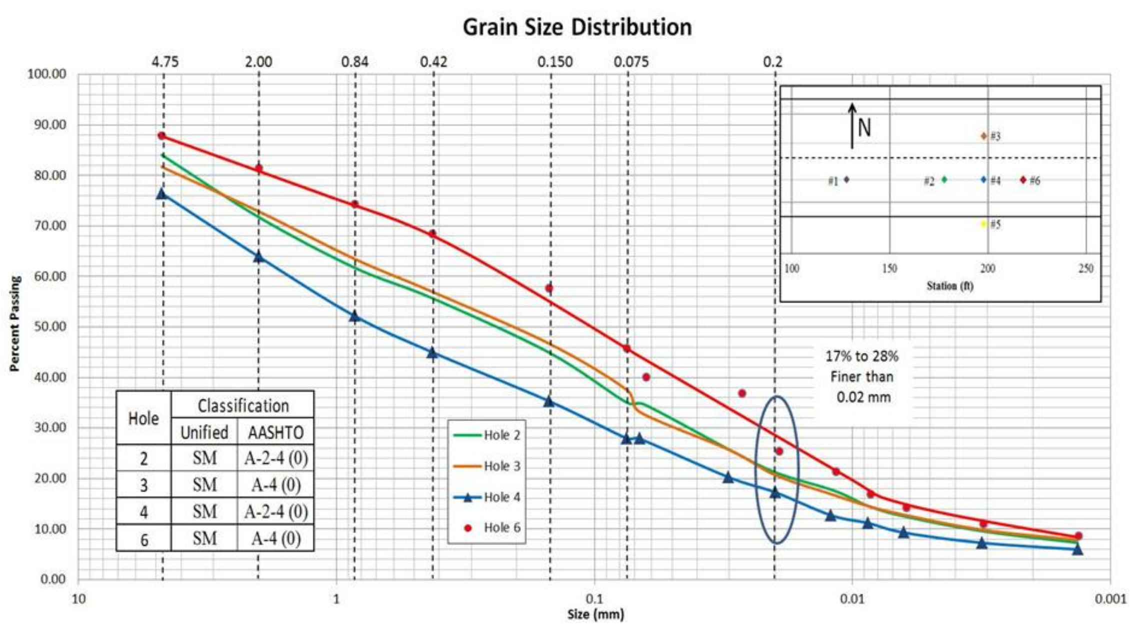


Figure 28. Graph. Grain Sized Distribution Curves for Sub-Grade Soils.

BEDROCK SURFACE

Figure 29 shows the dip in the rock surface from south to north at STA 1+90. Note that the circled bedrock point at +16 feet is from hole PZ2 which is located at STA 1+70, 6 m (20 ft) up grade from STA 1+90. The slope appears to be a continuation of the slope of the south bank of the highway cut and may suggest a dipping fault line that created the notch for the road cut initially.

However, the French drain is assumed to sit on the caprock at STA 2+10. It appears there are large variations in the surface elevation of the caprock. Figure 30 is the plan view of the site with depths to bedrock shown next to the borehole locations. Based on the drilling programs presented above, notes and discussion about the installed drains, and the change in grade of the road surface, the following assumptions may be made:

- In general, the bedrock slopes from south to north.
- There appears to be a trough at STA 1+40 that goes diagonally to the northeast between PZ2 and Hole 3 towards the east end of the north ditch drain.
- The surface is probably not smooth but rocky and/or with holes from rock that was gouged out from the surface. The surface may have been leveled using the broken and crushed rock, gravel and sand. Some of the borings indicate gravel at the bottom.
- There appears to be a more or less level platform extending from STA 1+70 to 2+20 at a depth of 2.5 to 4.5 feet (0.8 to 1.4 m) predominantly on the south side of the highway. This area is shown inside the dashed line in figure 30 and includes the French drain that is presumably on top of the bedrock. All of that area corresponds to the primary area of the heave. This would be the rock that the contractor wanted to avoid, and once passed on the east bound lane, the slope of the road surface increases toward Encampment. The distance is long enough to make the contractor want to work around it, but there does not appear to be any rock comparable on the north side of the highway.

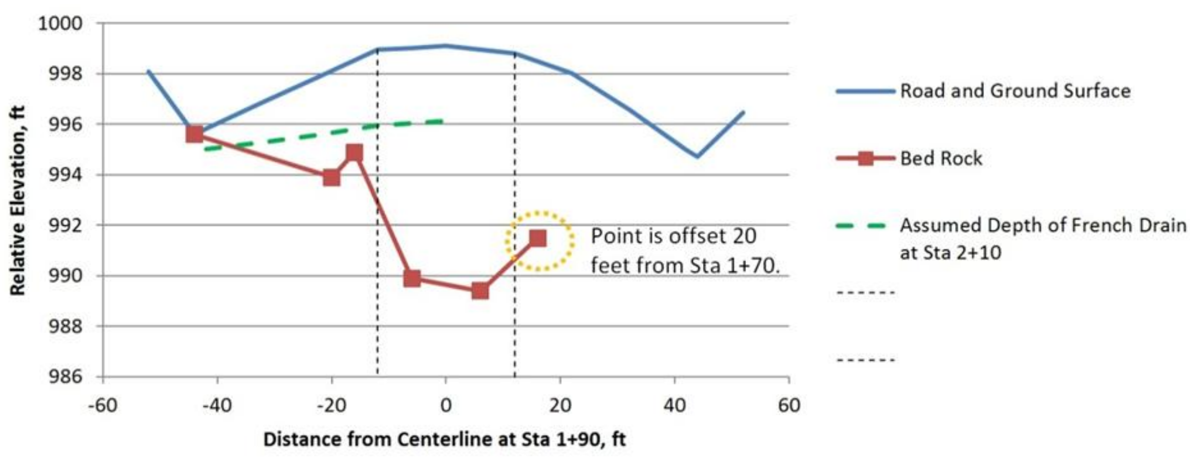


Figure 29. Graph. Bedrock Elevations, French Drain and Road Surface at STA 1+90.

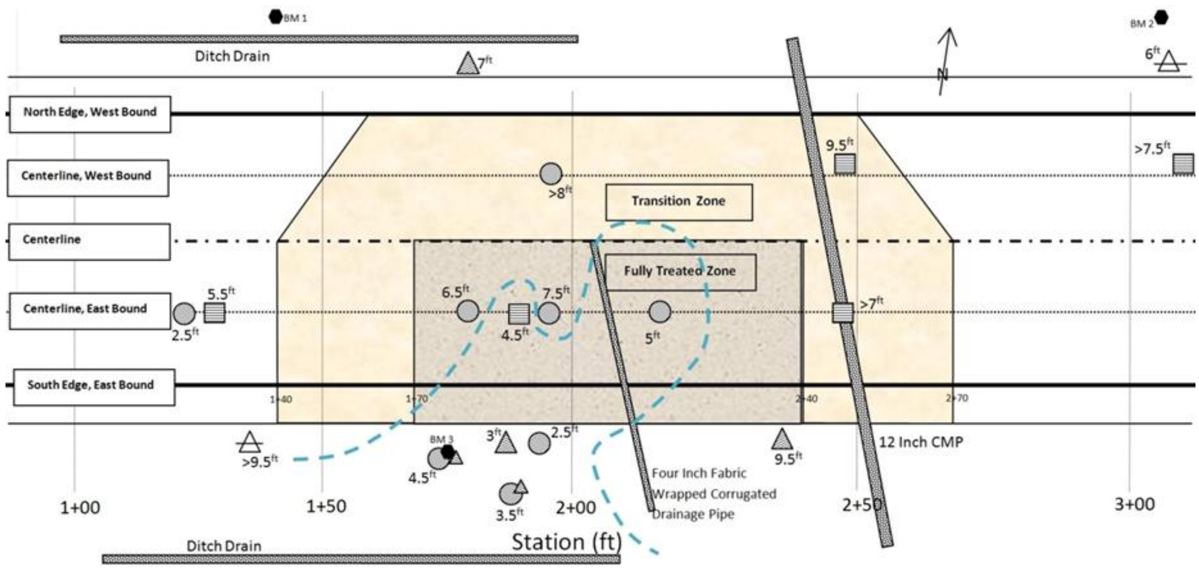


Figure 30. Sketch. Plan View Showing Depth to Bedrock and Proposed Outline of Caprock.

The configuration of the bedrock surface is important because it affects the groundwater flow under the site.

WATER

The role of water in heaving has been an impetus for several remedial measures at the site over time. Two French drains were installed in the 1990s (figure 26) and two lateral drains were installed in 2001. Four piezometers were installed during the drilling in 2000, but readings were not taken over time and Piezometer 3 has been lost. Piezometer readings for the duration of the project are shown in figure 31.

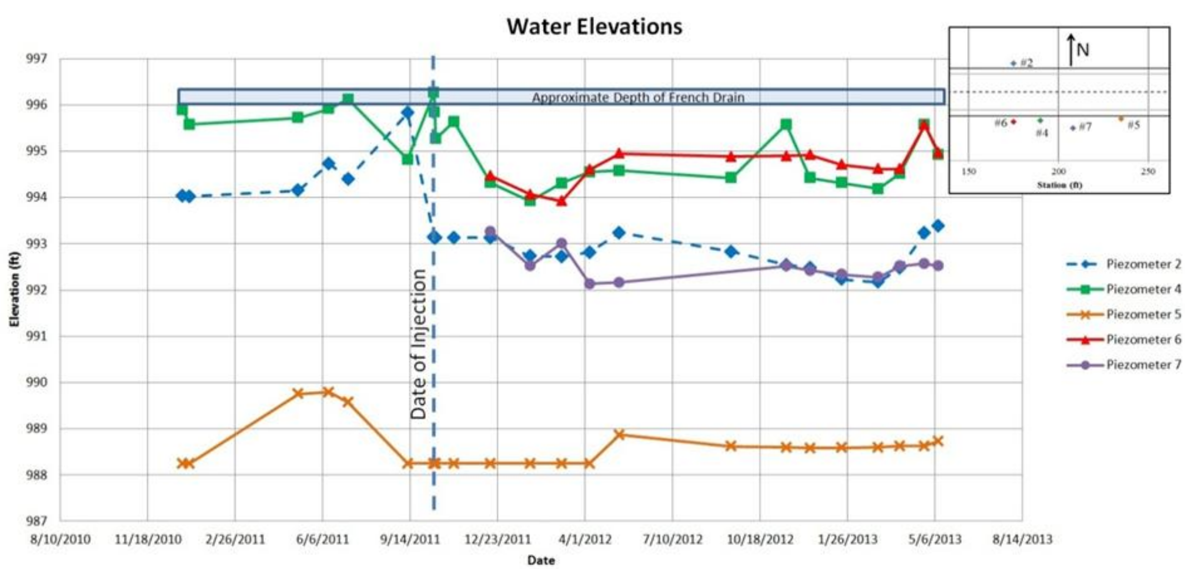


Figure 31. Graph. Water surface elevations.

Readings were taken after the first project site meeting on December 14, 2010. The next readings in May indicated a rise of over 300 mm (1 ft) in PZ5 but less than 152.4 mm (6 in.) in PZ2 and PZ4. In general, the water level differences indicate that the change in head occurs from the south side to the north side under the road, therefore, most of the groundwater flow goes from the south to the north side of the highway. Some water mounds up when it hits the caprock that acts as a small dam. The mounded water can flow over the caprock and then cascades down the slope beyond piezometer PZ7. This is shown in figure 32. The heads are fairly small over the bedrock and the large change in head probably does not indicate a large flow in that direction.

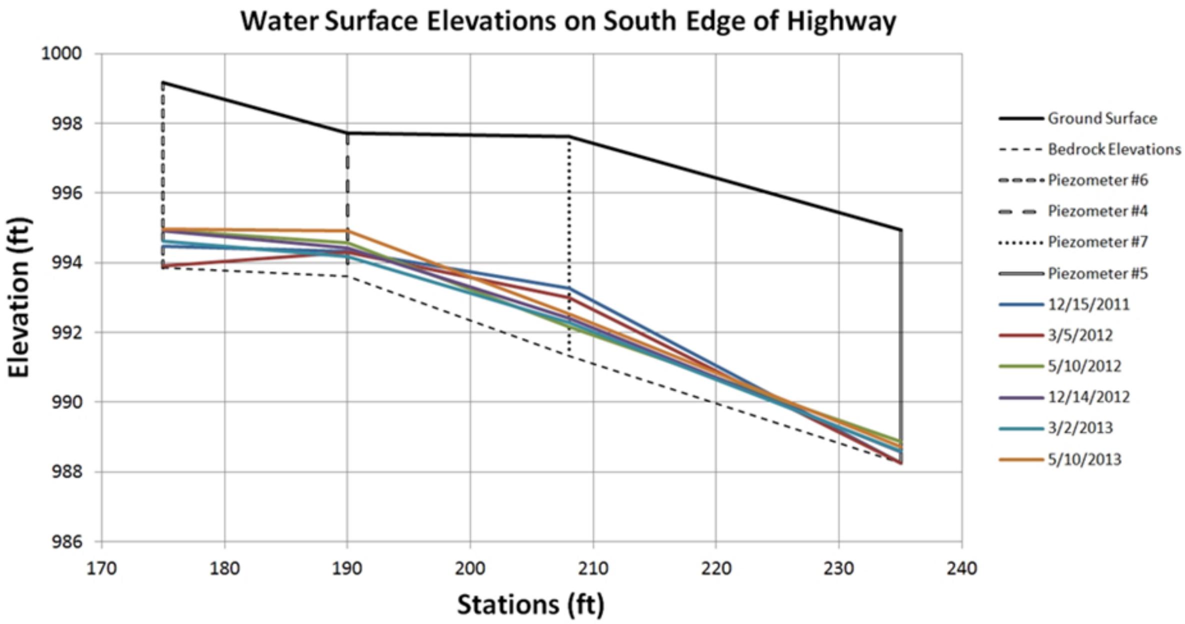


Figure 32. Graph. Water Surface Elevations Along South Edge of Highway.

PZ4 had an interesting reaction during construction from November 3-6, 2011. The water level apparently rose over 0.3 m (1 ft) on the first day of injections, but dropped 0.3 m (1 ft) in the next two days. The weight of the foam would be insufficient to cause this much rise. The injection must have created a pressure wave in the soil water and air that forced the rise and then rapidly dissipated.

Two more piezometers were installed with the other instrumentation. PZ6 is directly across the road from PZ2 and 4.6 m (15 ft) to the west of PZ4. During the project, the water level in PZ6 stayed about 100 mm (4 in.) above the level in PZ4 and supports the idea of flow going north under the road.

More interesting is the difference between PZ4 and PZ7. They are about 3 m (10 ft) apart with the depth to bedrock being about 0.2 m (0.5 ft) lower in PZ7. The water level difference is almost 0.6 m (2 ft) between the two, which is greater than would be anticipated by pure Darcy flow but is probably influenced by the rock surface.

TEMPERATURE

The National Weather Service (NWS) indicates that the average temperature for Encampment is 4.8°C (40.6°F). Figure 33 shows the monthly average high and low temperatures from Encampment and the hourly temperatures recorded in the instrumentation box at the site. The hourly average temperature in the box between May 10, 2012 and May 10, 2013 is 4.6 °C (40.3°F).

The NRCS SNOTEL Webber Springs Station No. 852 is located about four miles from the site and actual temperatures are recorded there. However, there is a 366 m (1200 ft) elevation difference between the two sites, hence the Webber Springs temperatures are somewhat colder than those at the site.

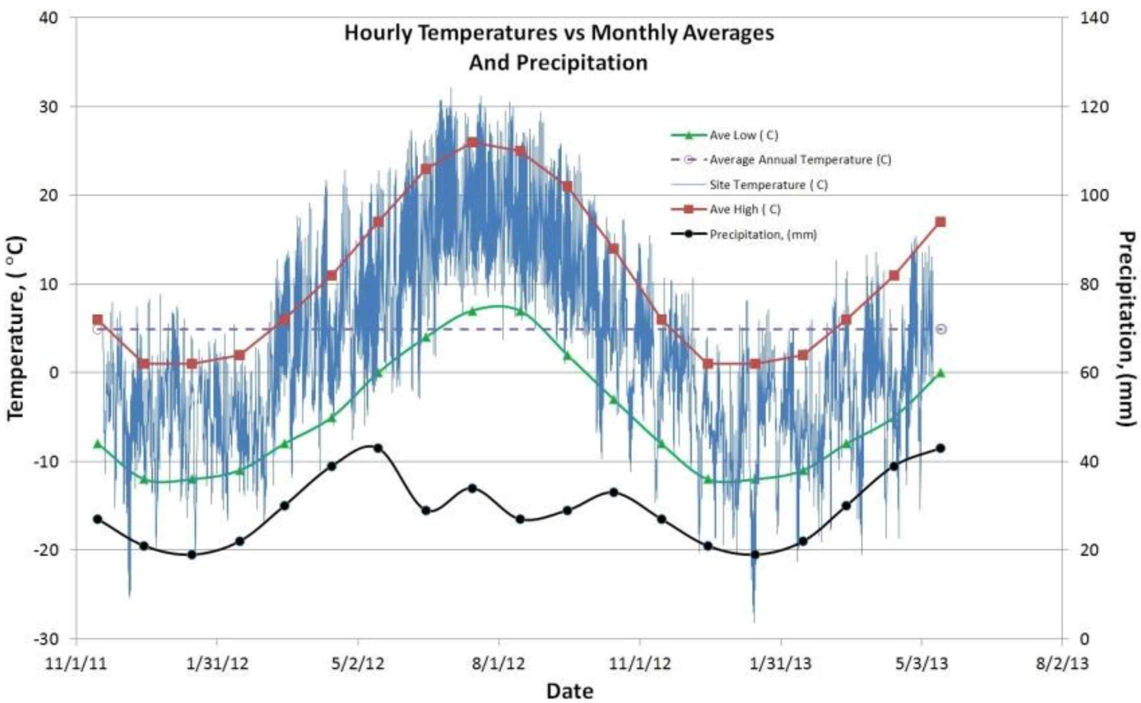


Figure 33. Graph. Monthly Average High and Low Temperatures and Precipitation for Encampment and Hourly Temperatures from the Instrumentation Box.

CHAPTER 4. CONSTRUCTION

Three phases of construction occurred over a period of about five weeks:

1. Injecting the Uretek expanding structural polymer foam to create a layer with a thickness of 75 mm (3 in.).
2. Milling the road surface to make the final surface smoother.
3. Installing and initializing the monitoring equipment.

Prior to the start of construction, three road surface profiles had been measured over the summer and early fall to establish a preconstruction elevation baseline against which the post injection and post milling road surface elevations could be compared.

INJECTION

October 10, 2011

Concrete Stabilization Technologies (CST), WYDOT and the University of Wyoming (UW) personnel met at the site Monday morning, October 10, to finalize the scope of work and outline the construction procedures. Two WYDOT flaggers maintained traffic control for the duration of the construction. After the traffic was diverted to the west bound lane for a distance of about 91 m (300 ft), CST took a level survey to establish a pre-injection road profile. Figure 34 shows the working setup at the site. They then laid out a 1.8 m (6 ft) square grid pattern on the road surface from STA 1+70 to 2+40 to locate the injection points over the heave area. Holes having 19 mm (0.75 in.) diameters were drilled through the pavement to a depth of 0.5 m (18 in.) on the grid. Injection tubes 0.6 m (24 in.) long were placed in the holes that would be connected to the nozzle of the injection gun. Upon testing their equipment, CST determined that one of their material pumps for the two part Uretek 486 STAR #3 polymer was not working and needed to be replaced. The injection tubes were pulled and the semi-truck was taken back to the Saratoga maintenance shop to complete the repair. After CST left the site, UW made a final preconstruction survey to establish the pre-injection road profile.

Figure 34. Photo. Site Layout showing the CST Urethane Truck, a Service Truck and Trailer, the Injection Hoses, the Leveling System and the Traffic Control

October 11, 2011

CST returned to the site and reinserted the injection rods. In addition to using a laser level, string lines were set along the highway centerline and the center and south edge of the east bound lane. Injection was started at station 2+40. Figure 35 shows the injection process. The injections were performed in pairs across the road to try to produce even lifting. Lifting was also started at station 1+70. Work proceeded from both ends towards the middle. Both ends were lifted the necessary 75 mm (3 in.) so that the string line could be set on the road surface at both ends to get proper elevation of the string. Spacer blocks lifted the string an additional 75 mm (3 in.), so irregularities in the surface would not touch the string and alter the elevation. A 3.7 m (12 foot) 2X4 was also used as a straight edge. Small tapers were added to each side of the treated areas to improve the transition onto the lifted areas for vehicles. The middle section was not treated that day due to potential difficulties caused by the poor condition of the asphalt in that area which had significant cracking. Warning signs were placed to warn drivers of the hazardous condition of the roadway.

Figure 35. Photo. Injecting the Uretek at STA 2+10, the location of the dip. Also shown are some of the Injection Tubes.

October 12, 2011

CST started working on injecting the middle section of the eastbound lane between the two humps. However, difficulties arose with the injection guns plugging. For most of the day, only one injection gun was functioning and was used singularly while the other gun was being serviced. The asphalt in this area had many cracks and thus was difficult to inject without severely damaging the road surface. Care was taken to inject slowly and with closer spacing to prevent uneven lifting and breaking apart of the road surface. In addition to the deep injection, some shallow surface injections were used to help lift small areas of pavement up to the correct elevation. The shallow injections were needed to prevent extra lifting of surrounding areas as caused by the deep injection. Injection and leveling continued throughout the day.

October 13, 2011

CST added 9 m (30 ft) long tapers to the east and west ends of the east bound lane, from STA 1+40 to 1+70 and from STA 2+40 to 2+70. They also did a few final touch ups of the surface elevation to the eastbound lane. Traffic control was switched to the north side of the road and CST proceeded to construct a smooth taper across the road from the centerline (which was lifted 76.2 mm (3 in.)) to the white line on the north edge with zero lift. Both deep (0.8 m (30 inches)) and shallow injections were used to keep the road surface smooth and intact.

Occasionally, the material may spread out beyond the areas wanted. During the injection process, there was no noticeable sign of unwanted lateral spread of the material at the treatment depth. Some material leaked out right under the surface of the asphalt and through the holes for the probes, but no visible material spread out into the road ditches or out of the outlets of the drains. The injection process was completed pending approval by WYDOT. CST did a post-injection survey of the site.

October 15, 2011

CST met with Scott Kinniburgh from WYDOT to drive the site. Mr. Kinniburgh drove the site four times in each direction at 105 kph (65 mph) to evaluate how smooth the ride was in a vehicle. It was agreed that the road did not cause an immediate safety risk even though it was not smooth. The road needed to be milled down to provide a smoother ride, but there was not a lot more that could be done about the rough ride using injections so it the injection process was deemed complete. A post injection survey of the site before milling was completed by UW on October 16 so that estimates could be made of the thickness of the insulation layer.

MILLING

On October 17, 2011, CST and Pavement Solutions came to the site to mill the road surface to provide a smooth ride for vehicles. Pavement Solutions filled up their water tanker in Encampment and arrived at the site ready to start work. The work was delayed several hours by falling snow that reduced visibility and obscured the road surface. When the snow stopped, WYDOT used a snowplow to clear the work area. The surface of the road was rougher than Pavement Solutions had expected so they had to start in smooth areas and mill off the high spots in order to provide a smoother ride for the milling machine. This was necessary so that the milling machine would be able to mill the whole road surface without gouging. Milling was then done over the whole work area to make a continuously smooth surface. After milling, a ride test was performed at posted speed limits and it was deemed satisfactory. Figure 36 shows the changes to the original baseline elevation after injection and then after grinding for the centerline. The same graphs for the remaining profiles can be found in the appendix D. The milling did cause some additional damage to the road surface.

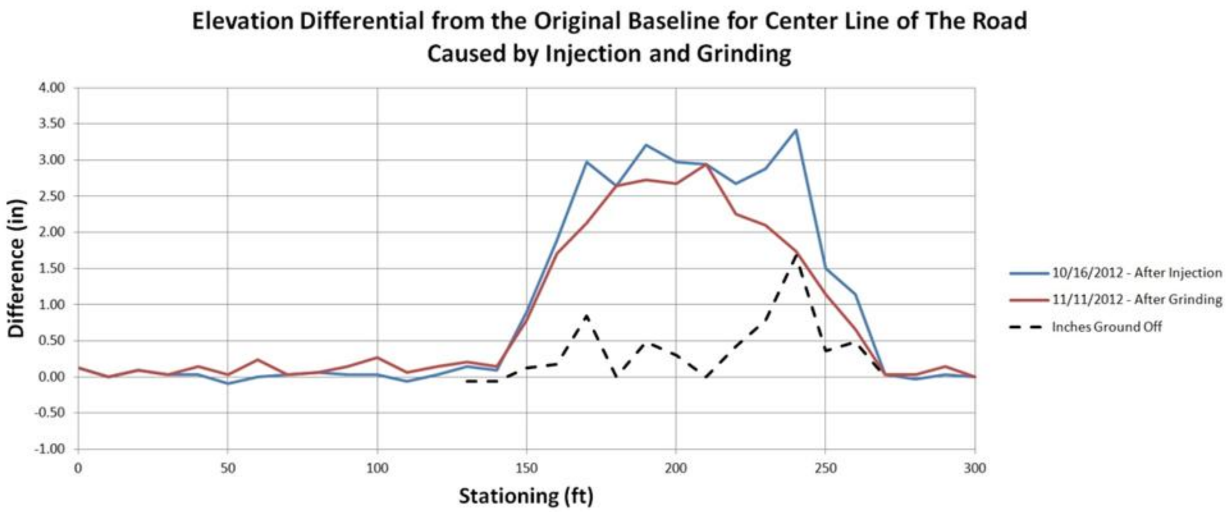


Figure 36. Graph. Injection Thickness and Final Surface after Milling at the Centerline.

TEMPERATURE MONITORING USING THERMISTORS

The in-situ soil temperatures were measured using thermistors and data was collected in a Campbell-Scientific Data Acquisition System. Thermistors are typically made of a ceramic or polymer with an electrical resistance that varies linearly with temperature. They are connected to a device that runs a current through the thermistor and the resistance is determined. Thermistors are physically more robust and more accurate than most other temperature gauges such as thermocouples but function over a smaller temperature range of -90°C to 130°C (-130°F to 266°F).

INSTALLATION OF INSTRUMENTATION

The thermal instrumentation was installed on November 3 when the drill rig became available. Six holes were drilled to place the thermal instrumentation, one to hold the post used to attach the instrumentation box and one more to create a new surveying benchmark. The new benchmark was necessary because the two existing benchmarks were buried under the snow on the north side of the north ditch, where they had been inaccessible during the previous winter.

Prior to the installation, the top three thermistors for each of the six holes were attached to a piece of foam household corner trim molding to hold the thermistors in place at specified locations, shown in figure 37. The depths were selected to be just below the road base and above the injection, just below the injection, and 254 mm (10 in.) below the previous thermistor. The thermistors were glued into holes drilled in the molding with their heads extending 12 mm (1/2 in.) beyond the face of the molding and taped so that they would not move when the molding was placed in the ground and the borehole backfilled. The heads were extended to insure they would make positive contact with the sidewall of the boring. The cables were also taped in place to reduce the potential of the cables being damaged during the placement and backfill compaction.

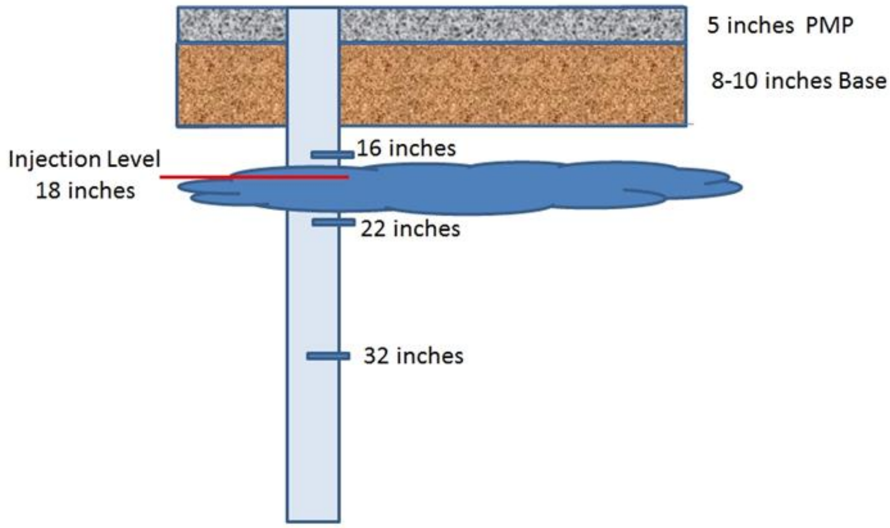


Figure 37. Sketch. Upper Thermistor Probe Depth relative to the Road Profile and the Injection Depth. (1 in. = 25.4 mm)

Figure 38 shows the first probe was located 100 mm (4 in.) from the top of the molding, which was buried at a depth of 305 mm (12 in.). The final one or two thermistors were attached in the field because their location was a function of the depth to bedrock. After the hole was drilled, the depth was measured and the prepared molding was cut one foot shorter than the depth of the hole. One or two holes were drilled in the molding with the bottom thermistor located about four inches from the bottom of the hole at the top of the bedrock. The molding was placed in the hole and the cuttings were backfilled into the hole using a very effective tamper built by Scott Kinniburgh shown in figure 39. The cuttings were compacted to the top of the molding.

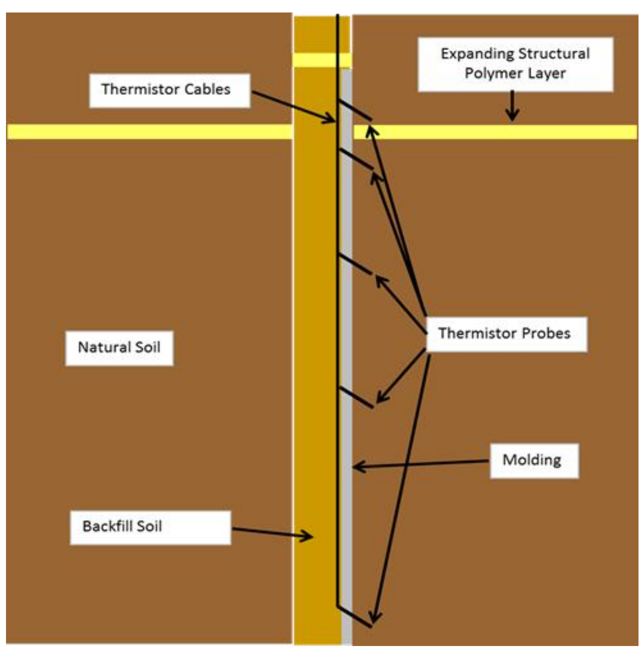


Figure 38. Sketch. Diagram of Instrumentation Installation.

During the injection phase, a 200 mm (8 in.) diameter Sonotube concrete form was filled with the polymer foam to create a 1.2 m (4 ft) length of foam the diameter of the hollow stem auger hole. A 75 mm (3 in.) segment of the form was cut and placed on the backfill at the top of the molding. The foam plug was then sealed in the hole using a can of Great Stuff expanding insulation foam to fill the annulus between the plug and the borehole. This was covered by more compacted soil up to the base of the asphalt. Finally, the hole was completed with a layer of cold patch compacted to the existing road surface. The thermistor cables were routed from the borehole to the side of the pavement through grooves cut by the Saratoga shop crew using a concrete saw. The cables were then placed in PVC conduit pipe that routed the cables to the instrumentation post where the data collection box was mounted.

Figure 39. Photo. Kinniburgh Tamper Compacting Cold Mix to the Surface

November 3, 2011

The UW group met with Scott Kinniburgh and his crew from the Saratoga WYDOT maintenance shop at the site. Locations were marked on the road for the drill holes. The WYDOT crew sawed 12.5 mm (1/2 in.) wide and 50 mm (2 in.) deep grooves into the road using a concrete saw to extend the thermistor cables from the hole locations to the side of the road. The crew had also dug a cable trench approximately 0.3 to 0.6 m (1 to 2 ft) deep and 1.5 to 3 m (5 to 10 feet) from the south edge of the road between STA 1+30 and 2+20 in which to run the PVC cable conduit.

Drilling began when the drill rig and crew arrived at noon. After traffic was moved to the east bound lane, the first hole drilled was Hole 3 located at Station 1+98 in the middle to the west bound travel lane. This hole was drilled first so that it could be finished and allow the west bound lane to be kept open for the duration of drilling. This hole was drilled to a depth of 2.4 m (8 ft) to bedrock. Figure 40 shows the borehole locations and the zones treated with expanding structural polymer. The boring logs are located in appendix C. The holes were not numbered in order of drilling but according to a convenient arrangement for later analysis. It is less confusing to refer to the holes by their final designation rather than the order drilled.

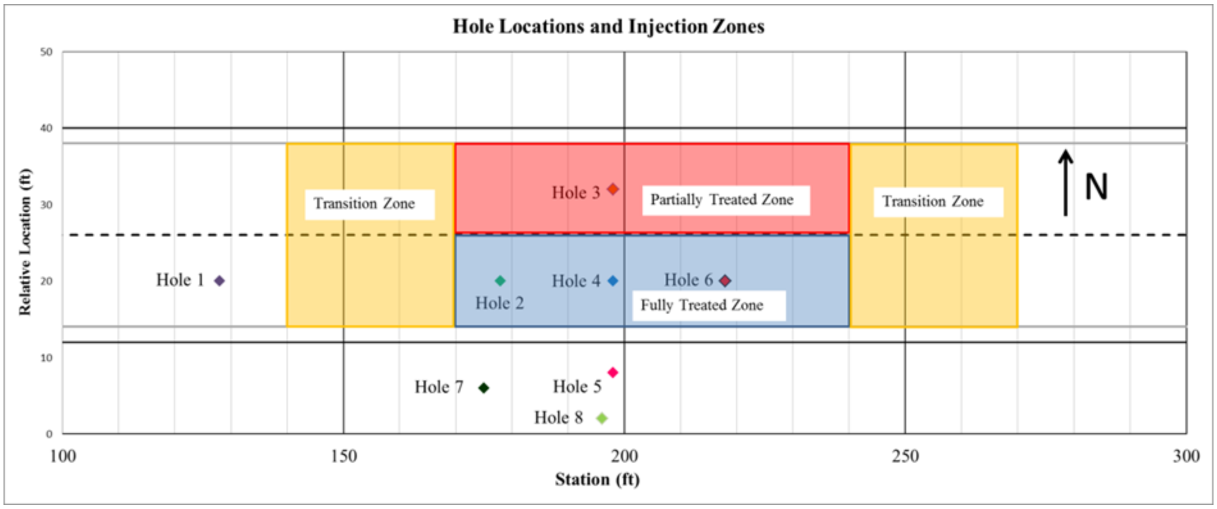


Figure 40. Sketch. Borehole Layout and Injection Zones.

While drilling Hole 3, the hole was dry until the last foot above bedrock, but upon completion, water began to rise up in the hole. The water level was not allowed to stabilize, but the piezometers on the north and south sides of the road indicated that the static water level would be about three feet below the surface. The instrumented molding was placed after the last two thermistors were drilled, glued and taped. The molding had been cut so that its top was one foot below the pavement.

As the hole was back filled, the water flowing in created a soft soil slurry. It was not possible to compact the soil properly with the available equipment. The hole was filled with the drill cuttings that were mixed with the borehole slurry to form a thicker mud using a long piece of rebar and it was left to drain overnight. The soil thickened somewhat because the water level lowered, and the next morning, more soil was compacted on top to provide a bridge of soil on which to compact the cold mix asphalt.

This hole was located in the tapered zone, so the foam thickness was approximately 38 mm (1-1/2 in.) thick. A heat flux sensor was glued under a 50 mm (2 in.) thick piece of the Sonotube foam plug and was placed at a depth of about 0.6 m (24 in.) to correspond roughly to the depth of the Uretek. More soil was compacted back into the borehole and then finished with asphalt to the level of the road surface.

A large amount of make-up soil was required for this hole, about five or six shovel fulls. When drilling, the cuttings will either rise up in the auger or, if the soil is of low density, it will compact into the side of the boring. It was obvious that the amount of silty sand coming out was a smaller volume than the hole itself and the additional soil required confirmed that. This indicated the soil was of low density as if the depth of freezing caused the void volume to be larger than would be expected in a compacted subgrade.

The next hole drilled was Hole 5 located at Station 1+98, about 1.5 m (5 ft) off the south side of the road into the ditch. It was drilled to bedrock at a depth of 1.1 m (3.75 ft). There was no water in the hole at the time of drilling or back filling. The bottom thermistor was added to the molding to be right above the bedrock and the molding was cut to fit in the hole. The top of the molding was placed in the hole at a depth of 305 mm (1 ft). No foam plug was placed in this hole since it was outside the treatment zone.

Hole 7 was drilled next at Station 1+75 at a distance of 2.4 m (8 ft) from the south edge of the roadway. The hole was drilled to bedrock at a depth of 1.4 m (4.5 ft). This hole was used for placing a piezometer designated as Piezometer 6. The piezometer was made of 38 mm (1.5 in.) diameter PVC pipe. The pipe was cut to the depth needed and a cap was glued on the bottom of the pipe. Holes having a diameter of 3.2 mm (1/8 in.) were drilled in the pipe near the bottom to allow water to enter the piezometer. A threaded adaptor was glued onto the top of the pipe to hold a threaded cap.

A new rebar benchmark was added to this borehole as well, designated as Benchmark 3 (BM3). Benchmark 3 was added closer the roadway to provide easier access to a benchmark during the winter months when Benchmarks 1 and 2 were buried under several feet of snow. To make the benchmark, the bottom of the hole was filled with concrete to hold the benchmark in place. A piece of #4 rebar was cut to an appropriate length to stick out 102 mm (4 in.) above the top of the ground surface. The rebar was pushed into the concrete and down to the bottom of the hole. A PVC sheath was place around the rebar. The hole was then backfilled to hold the piezometer and the benchmark in place.

Hole 8 was drilled at station 1+93 at a distance 3.7 m (12 ft) from the south edge of the road into the ditch. This hole was drilled to place the post for the equipment box and the solar panel as well as a new piezometer designated as Piezometer 7. The hole was drilled to a depth of 1.22 m (4 ft). The 102 mm (4 in.) post and the 38 mm (1.5 in.) diameter piezometer were placed back in the hole and the hole was backfilled. The post had to be cut notched on one corner at the bottom to allow the piezometer to be placed in the same hole.

Holes 6, 4, and 2 were drilled at Stations 2+18, 1+98, and 1+78 and to depths of 1.67 m, 2.29 m and 2.00 m (5.5, 7.5, and 6.5 ft) respectively. These holes were all similar in nature and all drilled along the centerline of the east bound lane. All the holes were dry. The instrumented moldings were prepared as in Hole #3, placed, compacted and covered with a 75 mm (3 in.) thick foam plug, backfilled and covered with an asphalt patch. A heat flux sensor was glued to the bottom of the foam plug in Hole 4.

The final hole drilled was Hole 1 at Station 1+28 along the centerline of the east bound lane The hole could only be drilled to a depth of about 0.8 m (2.5 ft) before refusal. Since the hole was so shallow, the last two thermistors for the hole were not added to the molding. The molding was cut to the needed length and placed so the top was at a depth of 305 mm (12 in.). The hole was backfilled up to the bottom of the asphalt. It is interesting to note that Hole 10-5, drilled during 2010, is located about 3 m (10 ft) away and has a depth of 1.7 m (5.5 ft). On further consideration, it is possible that Hole 1 hit a rock suspended in the fill and does not go down to the actual bedrock surface.

For all of the holes drilled in the roadway, the cables for the thermistors were run through the cuts sawed into the road surface earlier out to the edge of the roadway. The cables were covered with a piece of 12.5 mm (1/2 in.) diameter flexible insulation foam and then were sealed with a special polymer caulking material. WYDOT also repaired several pieces of the road surface that had been damaged by the injection and grinding processes using hot patch mix. The equipment box and the solar panel were attached to the post facing south away from the roadway. The thermistor cables were set in the ditch away from the road and the project concluded for the day.

Table 3 summarizes the depths of the thermistors in each of the boreholes. The top three thermistors are at the same depth so their temperatures can be compared directly. The lower probes were placed as a function of the depth of bedrock and were useful to determine the temperature distributions below the freezing depths.

Table 3. Thermistor Depths for Each Hole (1 foot = 0.305 m)

	Depths to Thermistor Probes (ft)					
	Hole #					
Probe	1	2	3	4	5	6
1	1.33	1.33	1.83	1.33	1.33	1.33
2	1.87	1.87	2.33	1.87	1.87	1.87
3	2.67	2.67	3.17	2.67	2.67	2.67
4	NP	4.42	4.67	4.00	3.58	4.08
5	NP	6.17	8.67	5.33	N	5.50

November 4, 2011

Hole 3 had dried enough overnight to be backfilled and compacted although compaction was still difficult. It was completed as described above. The primary task for the day was to run the thermistor cables from the edge of the road through PVC pipes to the instrumentation box at Station 1+93. The PVC conduit pipe was to protect the cables from damage and was buried in the trench cut by WYDOT.

The cable trenches from the boreholes extended to the south edge of the pavement shoulder. Each set of cables were then run through a 25.4 mm (1 in.) diameter PVC pipe from the shoulder to a main conduit which was finally buried at a depth of 305 mm (1 ft). The conduit size was increased as each cable bundle was added to it. The conduits from both the east and west sides ended at the instrumentation post.

The layout of the boreholes changed from the time that the thermistors were ordered to when the final locations were determined as the nature of the site and the injection became better understood. Significantly, the borehole locations came closer together so that the cables on the thermistors were longer than finally required. In some cases, the cables were 15 m (50 ft) too long. All of the excess cables were bundled, wrapped, and placed in a 3 m (10 foot) long piece of 100 mm (4 in.) diameter PVC pipe. The cables were fed through two pipe saddles to two 38 mm (1.5 in.) diameter PVC pipes that fed the cables into the instrumentation box. Figure 41 shows the final cable layout.

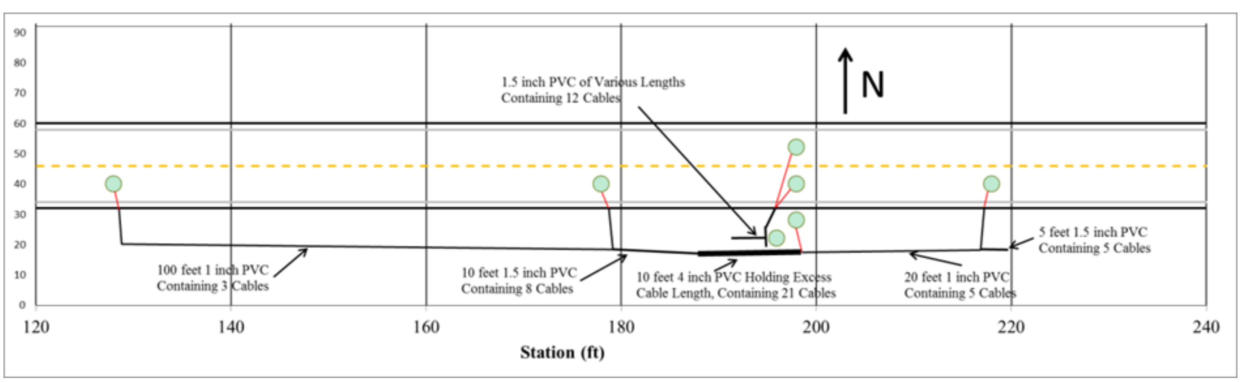


Figure 41. Sketch. Wiring Layout and Conduit Location

Repairs, Temporary Baseline Survey, and Wiring

The final instrumentation wiring was not completed during the installation trip. A trip was taken on November 11 to measure a second baseline road profile and finish the work not completed on the last trip. The ends of all the pipes were filled with expanding foam insulation and the junction of the pipes was surrounded with the expanding foam insulation. After all the pipes and conduit were sealed and protected, WYDOT came to the site on a later date and buried all of the pipe to protect it from damage.

Since the roadway elevations were changed by injection and milling of the road surface, a survey of the site was conducted to establish a baseline against which to compare the winter surveys. During surveying, the elevation of BM 3 was determined so that it could be used in place of Benchmarks 1 and 2 if they were buried by snow. The elevation of BM3 was determined to be 304.395 m (998.67 feet) relative to the assigned elevation of 304.800 m (1000.00 ft) at Benchmark 1. Most of the thermistors were wired into the data logger and the multiplexer. Time restrictions prevented the completion of wiring. The wiring was completed on November 18, 2012. The power was turned on to the system and the data acquisition program was sent to the data logger. This was the conclusion to the construction portion of the project.

SUMMARY

The construction process began with the injection of the polymer. The road surface was milled and then all of the instrumentation was placed and wired to the data logger. The collection of temperature data began on November 18, 2011 and the site was monitored over the next two winters. Chapter 5 is a discussion of the results of the data collection and the project

CHAPTER 5. RESULTS

Chapter 3 described the site characteristics and why frost heaving is so significant in this small area. Chapter 4 presents the instrumentation and measures that were used to determine the state of the system. This chapter will present the results of data collection over the winters of 2011-2012 and 2012-2013.

The objective questions to be evaluated are:

- Can injection of the foam nondestructively level the road surface and make the road safer to drive without full reconstruction of the site?
- Will the foam provide a sufficient thermal barrier to reduce or eliminate the frost heave caused by penetration at the site?
- Will the continuous blanket of foam create a barrier to moisture during the spring thaw?

Maintaining the same sequence, the surface movements will be presented first, the thermal data will be presented next and the water data will be considered last.

ROAD SURFACE MOVEMENT

Surveying – Longitudinal Elevation Differences

Figure 42 shows the frost heave occurring on the Centerline of the east bound lane. The thinner solid black line is the heave determined on January 26, 2011 before the treatment was applied. The maximum heave measured is 66 mm (2.6 in.) at STA 1+90, but the significant feature is the 33 mm (1.3-in.) drop at STA 2+10 followed by the 25.4 mm (1.0-in.) rise in the next 3 m (10 ft). The dip, right in the middle of the travel lane, was created by the non-heaving backfill over the French drain and it has created significant problems and complaints.

The heavy black line at the bottom of the figure represents the thickness of the structural polymer foam along the centerline of the lane. The secondary vertical axis indicates thicknesses of 0 to 83 mm (0 to 3.25 in.) with the average thickness being a little less than 76.2 mm (3 in.).

The bolded colored lines are the differences between the measured elevations and the baseline elevations after the foam treatment. The dashed lines are from the winter season of Year 1 (2011-2012) while the solid lines are from the winter season of Year 2 (2012-2013). The maximum heave under the highest bump is less than 12.7 mm (0.5 in.). One set of readings taken on March 5, 2012 reaches 17.8 mm (0.7 in.) between STAS 1+50 and 2+60 while the rest of the readings are less than 12.7 mm (0.5 in.). The resulting movement is essentially unnoticeable to a driver.

The heave along the centerline of the highway is shown in figure 43. The heave prior to treatment measured on January 26, 2011, is about 73.7 mm (2.9 in.) at STA 1+90. There is no dip along the centerline because the French drain on top of the buried caprock extended from the centerline to the south. The foam thickness is uniform at three inches over the center of the treated zone.

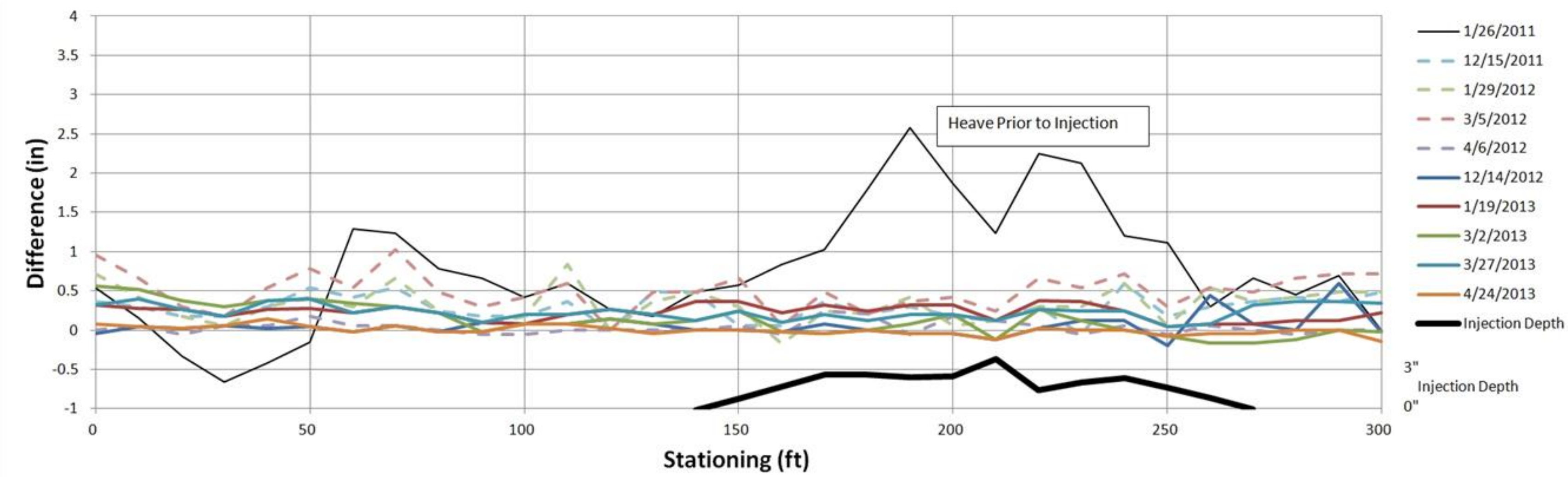


Figure 42. Graph. Frost heave along centerline of east bound lane (CLEB) – Year 1 (Dashed) and Year 2 (Solid)

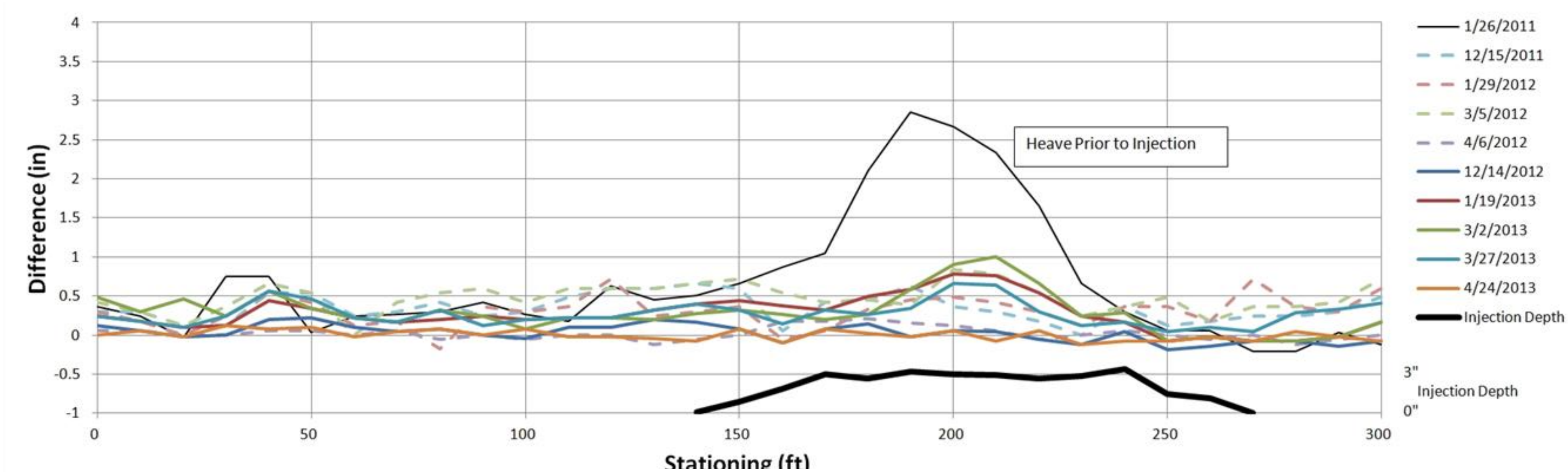


Figure 43. Graph. Frost heave along centerline of the highway (CL) – Year 1 (Dashed) and Year 2 (Solid)

A small amount of heave occurs at STA 2+10 where the caprock is closest to the surface. Two reasons may be the likely cause. First, the caprock may alter the thermal regime locally that could provide more water adjacent to the freezing front and create more ice lenses. As no temperature gages were installed on the centerline, it is not possible to verify that hypothesis. A second reason may be that the taper in foam thickness to the north occurs at the centerline, so it is possible that some heat loss is occurring here causing the temperature to drop lower at that point. Nonetheless, the heave is significantly less than that before the treatment.

It is significant to note that the readings taken on January 26, 2011 were unlikely to the be the maximum heave that would occur during the winter season. Data analysis over the next two years indicates that the maximum heave normally occurs during the early portion of March. Scaling the results would increase the January measurements by a factor of at least 25%.

During the initial negotiations for the project, the frost heave on the north side of the highway was not considered to be a concern. The bump was not considered severe and there was not as much driver discomfort on the north side. Surveying was not performed on the north side during the first two site visits, but was started during the research phase. Figure 44 shows that over 76.2 mm (3.0 in.) of heave occurred on the north edge during the Year 1 (dashed) while 44.5 mm (1.75 in.) occurred during Year 2. The most likely reason for this difference in heave is the road surface condition. An overlay was performed on the road in the mid 1990s. A tack coat was not applied between the initial surface and the overlay as an experiment to reduce maintenance costs. The thickness of the overlay after milling the surface after the injection was less than 25.4 mm (1 in.) over much of the area. During the spring and summer of 2012, plates of the asphalt began to slide on the cold joint due to the traffic, to the point of pinching the instrumentation cables crossing the road. A decision was quickly made to place another overlay on the stabilized surface. The overlay made the surface black as opposed to the lighter grey of the weathered asphalt. The different albedo most likely caused the change in heave between the first and second year heave. Longitudinal frost heave diagrams for all five lines are presented in appendix E.

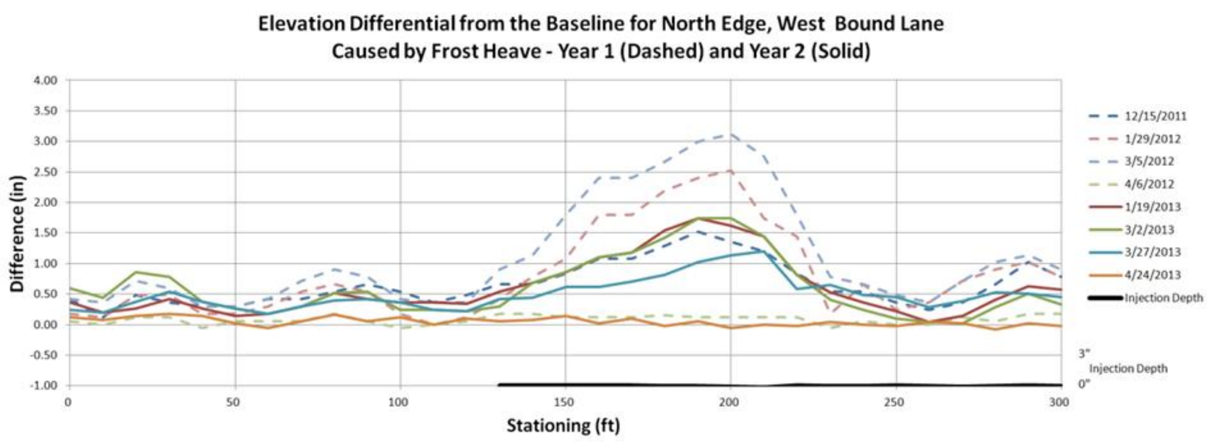


Figure 44. Graph. Elevation differences along the north edge of the west bound lane (NEWB)

Surveying – Transverse Elevation Differences

The significance of the heave reduction can be seen in the transverse direction across the highway. Figure 45 shows the heave at STA 2+00. The heavy black line on the bottom shows the full thickness of the injection on the east bound lanes (South/Left) and the taper in thickness to zero on the north edge. The thin line between -6 and 0 feet is the heave measured on January 26, 2011 before the treatment. The dashed lines are the readings during Year 1 after the injections and the solid lines are the readings during Year 2. The heaves at -6 ft, the centerline of the east bound lane are all below 12.7 mm (0.5 in.). A portion of the heave on the south edge of the lane may be attributed to irregular injection adjacent to the embankment slope. Injection would stop when the foam would begin to flow out the embankment face. One modification to the injection plan for future work would be to create a vertical barrier along the outside shoulder. This barrier would prevent lateral loss of foam and reduce the heat loss to the side.

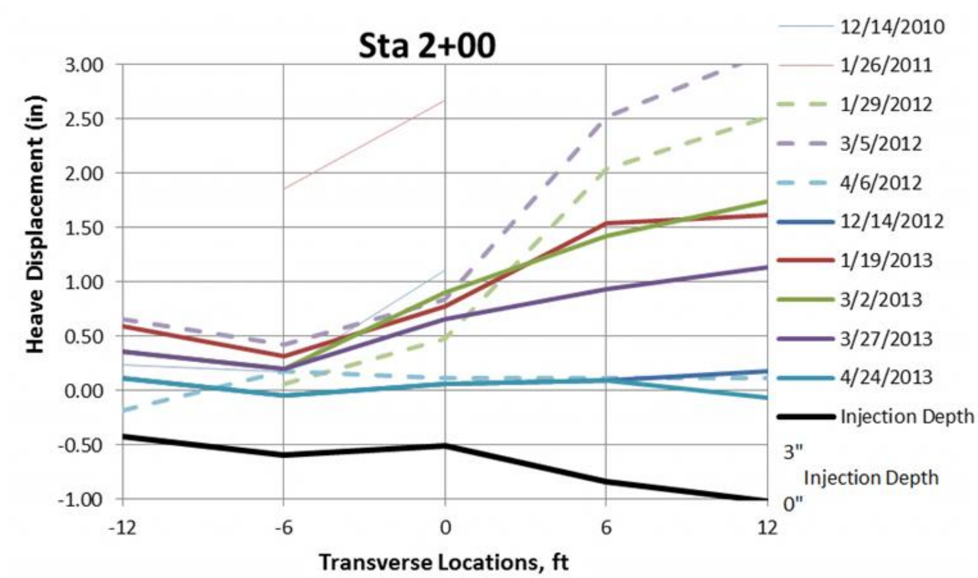


Figure 45. Graph. Elevation differences across the highway at STA 2+00. The east bound lane is on the left.

A layer of snow covers the ground surface during the winter that acts as a thermal blanket. The ground temperature under the snow stays right at 0°C (32°F). The warmth on this side compared to the colder surface temperature under the cleared road surface could also affect the depth and rate of freezing and allow somewhat more heave to occur along that edge. This temperature difference could also explain in part why shoulders tend to show more distress than the pavement next to them.

Surrounding stations show similar patterns, with the heave on the right decreasing in both directions while the heave in the CLEB remains at a minimum. Transverse sections at STA 1+60 and STA 2+30 are shown in figure 46 and figure 47, respectively. The heave on the treated side remains small and the heave on the west bound, partially untreated side is decreasing away from the maximum heaves at STA 2+00. Appendix F presents ten diagrams between STA 1+00 to STA 2+70.

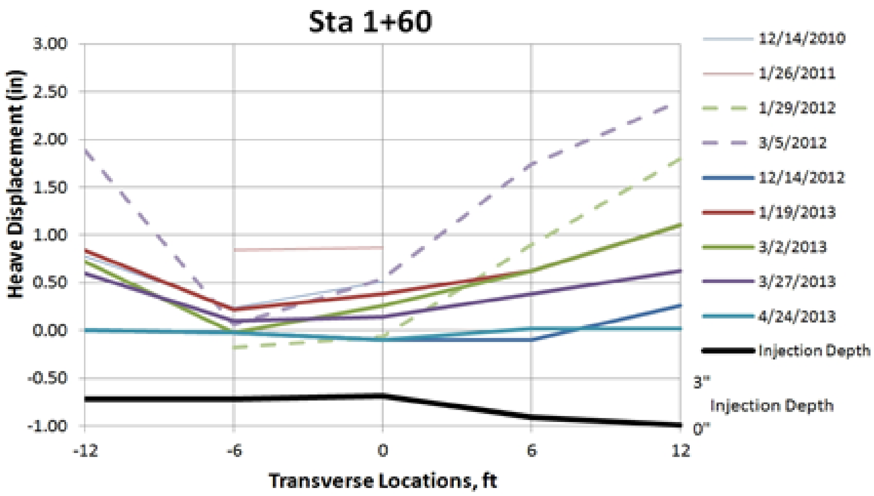


Figure 46. Graph. Elevation differences at STA 1+60

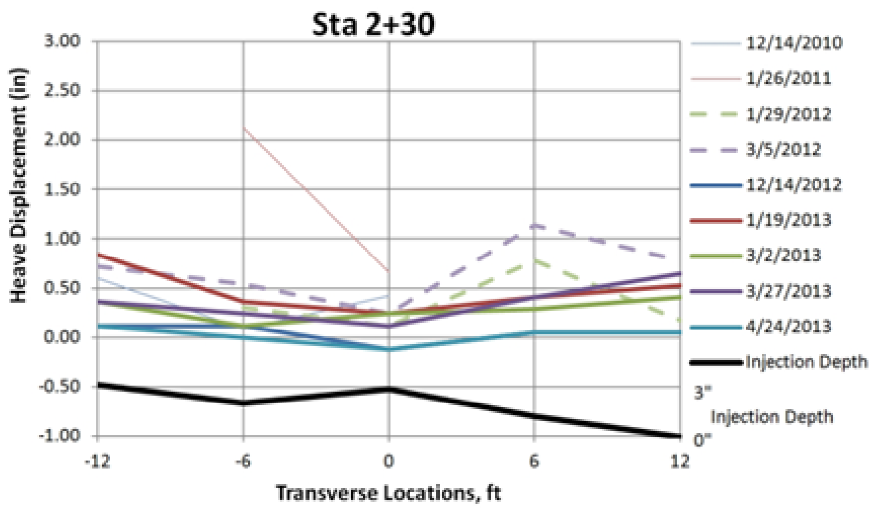


Figure 47. Graph. Elevation differences at STA 2+30

TEMPERATURES

The temperature below the road surface was determined at boreholes one through six, shown in figure 26. The top three thermistors in all the holes were located at fixed distances 410, 560 and 820 mm (1.33, 1.87 and 2.67 ft) below the surface, shown in figure 48 except for Hole #3, which is located 150 mm (6 in.) deeper. The top thermistor was located just above the foam while the second thermistor was just below the foam. The fourth and fifth thermistors were spaced depending on the depth of the hole. Temperature readings at all thermistors were collected every hour from November 18, 2011 to May 10, 2013. Hole #5 was off the road surface and covered with snow most of the season and its temperature patterns are very different. It will be evaluated separately.

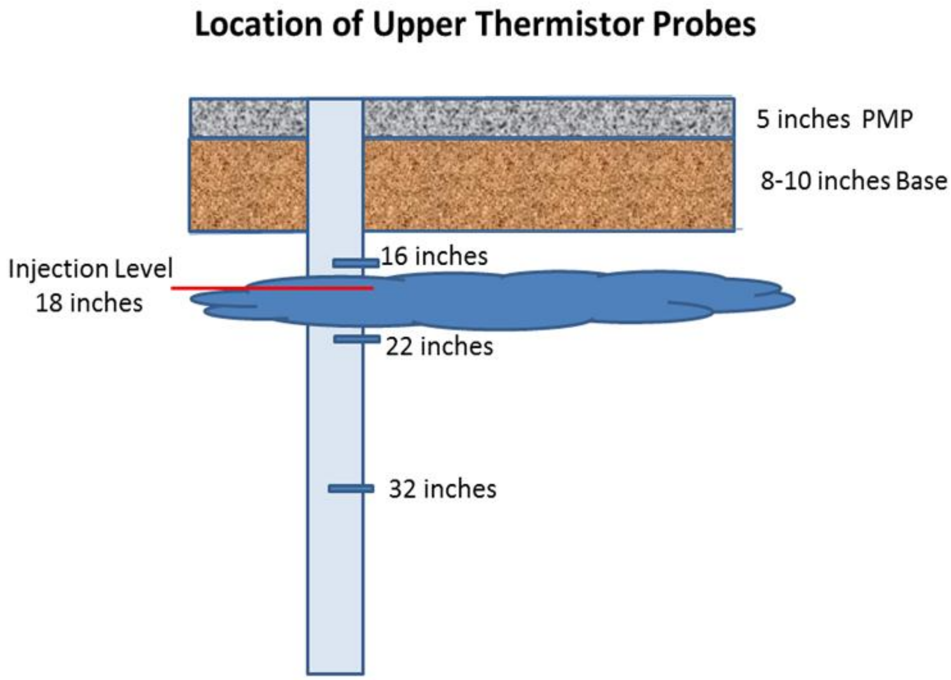


Figure 48. Sketch. Location of top three thermistors relative to the pavement and the foam. (1 in. = 25.4 mm)

Temperature Readings at Depth

Temperature readings over the two winter seasons for the five thermistors in Hole #4 are shown in figure 49 and for the three thermistors in Hole #1 are shown in figure 50. (The wiring for the bottom thermistor in Hole #4 was damaged by the moving asphalt during the summer and gave erratic readings during the rest of the time.) Both of these holes are located on the centerline of the east bound lane, with Hole #4 having 76.2 mm (3 in.) of foam and Hole #1 having none.

The average temperature of the top thermistor in Hole #4 during the first winter is about -2^{o}C (28.4°F) with a minimum temperature about -3^{o}C (26.6°F). The average temperature in Hole #1 during the same time is about -3^{o}C (26.6°F) while the minimum is at -6^{o}C (21.2°F). Similar results hold for the second year. The key temperature readings are at the second and third depths, 560 and 820 mm (1.87 and 2.67 ft), immediately above and below the polymer injection. The average temperature during Year 1 at the 560 mm (1.87 ft) depth in Hole #4 is at 0^{o}C (32°F) with a small dip to -1^{o}C (30.2°F) while the average temperature in Hole #1 is approximately -2^{o}C (28.4°F) with a minimum temperature of -4^{o}C (24.8°F). The green line representing the data at 820 mm (2.67 ft) depth is well above zero in Hole #4 and well below zero in Hole #1. Little freezing is taking place below the foam layer in Hole #4 so the heave there is minimal. Conversely, the frozen depth is well below 820 mm (2.67 ft) in Hole #1. Minimal heaving occurs in the area of Hole #1 because the soil is coarser and is not as frost susceptible.

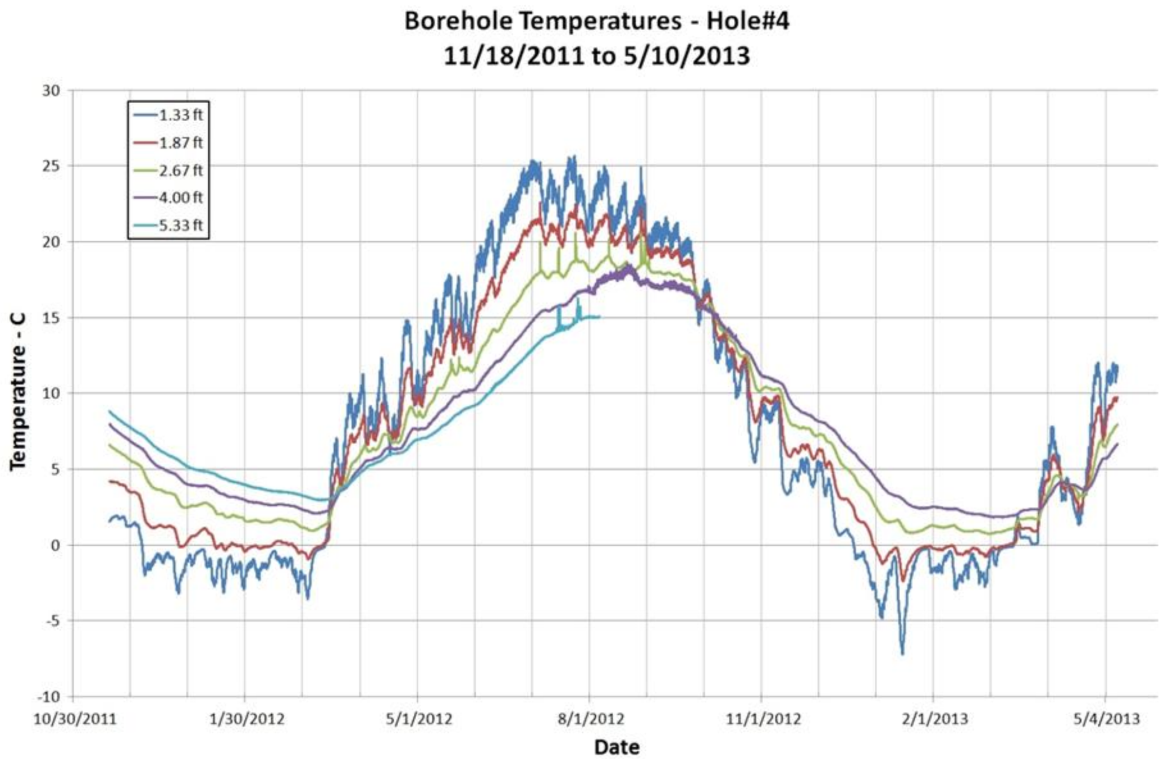


Figure 49. Graph. Temperature readings for the five thermistors in Borehole #4.

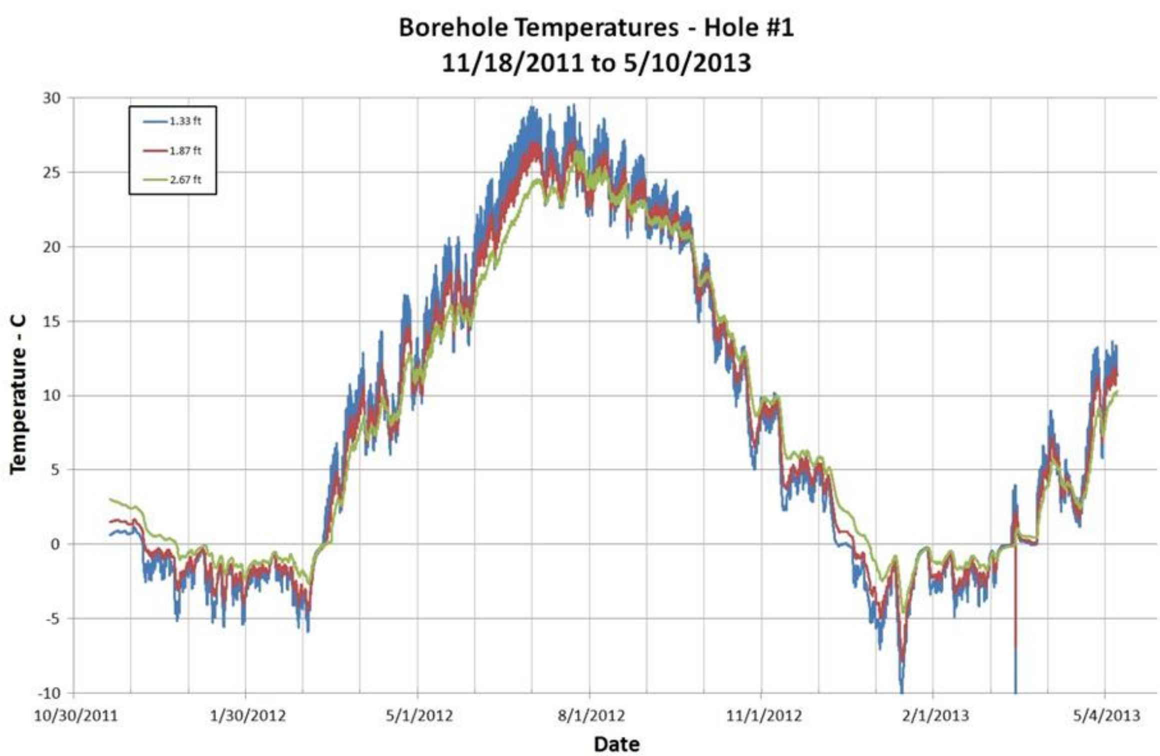


Figure 50. Graph. Temperature readings for the five thermistors in Borehole #4.

Figure 51 combines all these results so they can be compared. The horizontal bars represent the hourly temperatures for each thermistor averaged over the freezing season. For example, the bottom bar in Hole #1 at -3 °C (26.6°F) is the average temperature from the hourly readings in the first thermistor at a depth of 405 mm (1.33 ft) determined between December 6, 2011 and March 11, 2012. These are the first and last days that the thermistor registered a temperature below 0 °C (32°F). This range was used for all the other first year averages. The range of days for the second year is from December 17, 2012 to March 14, 2013.

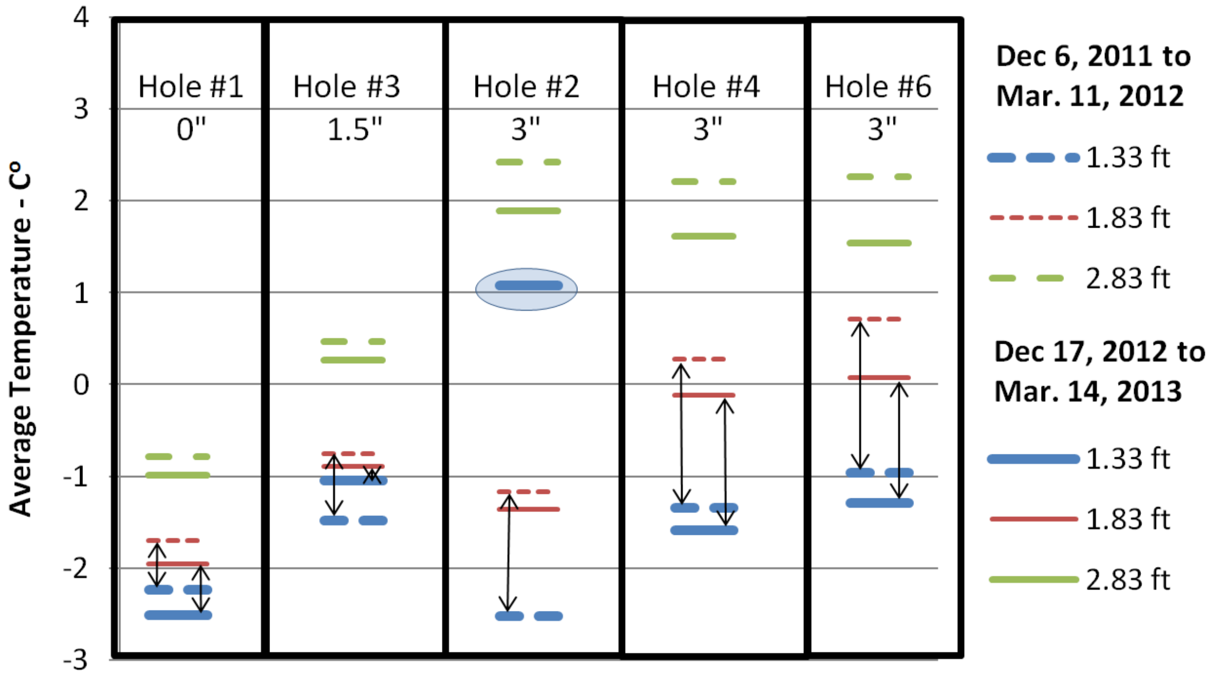


Figure 51. Graph. Thermistor readings averaged over the freezing season.

There are two anomalous features in this figure. The circled blue line is a consequence of the erratic readings obtained after the wiring was damaged. The second feature is the very small temperature range between the first two thermistors in Hole #3 during Year 2. It may also be attributed to wiring damage, but the two thermistors tracked each other very closely throughout the winter.

Four significant features stand out in this diagram.

- The difference in average temperatures in the first two thermistors in Hole #1 with no foam is small compared to the differences in Hole #3 with 38 mm (1-1/2 in.) of foam and to Holes #2, #4, and #6 with 76.2 mm (3 in.) of foam, which are more than twice as great as in Hole #1. The layer of foam is between these two thermistors, so the difference should be significant.
- The temperature difference between the second and third thermistors is much greater in the holes with three inches of foam. The foam is providing an active barrier to heat loss so the average temperature is greater under the foam than without it.
- The average temperature in the third thermistor in Holes #2, #4, and #6 is uniformly 1.5^{o}C (34.7°F) above freezing with the foam, while the thermistor in Hole #1 is one degree below freezing (30.2°F). This may create a heat reservoir, which raises the

temperature in the upper levels. However, this is not showing up in Hole #2, so the high water levels around the caprock may influence these values also.

- The third thermistor in Hole #2 is at the same temperature as are those in Holes #4 and #6, whereas the top two thermistors in Hole #2 are behaving more like those in Hole #1. It is possible that the foam was injected deeper in #2 than the other holes. That would have the effect of "raising" the thermistors and keeping both in the freezing zone, while the third thermistor would still be below the foam and act about the same as the other holes.

Graphs of temperature readings for all the holes are presented in appendix G.

Effect of Freezing on Heave

Figure 52 compares the temperature data during Year 1 in Hole #4 to the frost heave over that time at STA 2+00 along the five survey lines. Specifically, the heavy black dashed line is the heave measured along the centerline over Hole #4. Very little heave occurs until the temperature at 570 mm (1.87 ft) briefly drops below 0°C (32°F). Even then, the heave is small because the depth of frost penetration is very small.

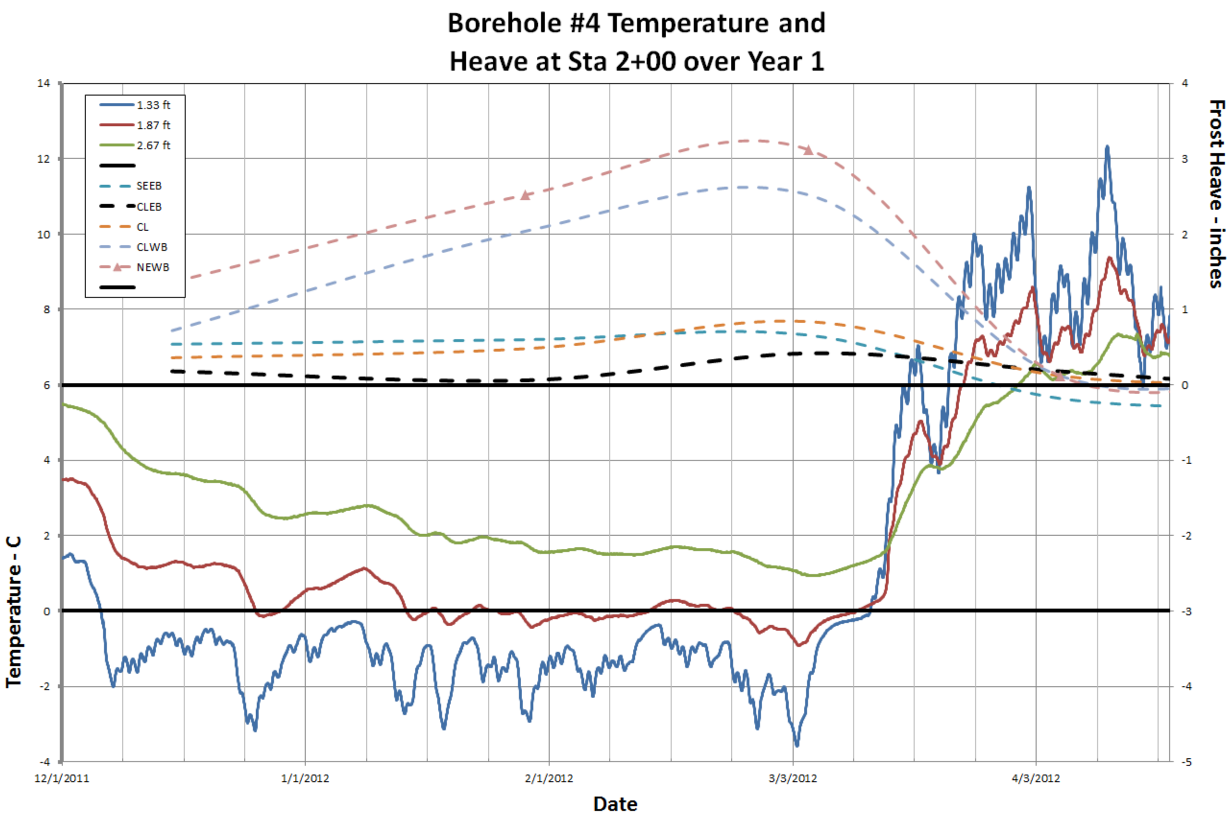


Figure 52. Graph. Hole #4 Temperature and Heave at STA 2+00 – Year 1.

Figure 53 shows the same information as figure 52 except it is for Year 2. A cold spell occurred on January 11-16, 2013, when the low reached -28.0°C (-18.4°F) on the 14th. This drove the soil temperatures down at all three levels, but especially the thermistor at 570 mm (1.87 ft), which dipped to -2.4°C (27.7°F). This allowed early freezing to occur, but as the soil temperature decreased through the rest of the season, the ice appears to have melted and the surface dropped a small amount and stabilized.

The average temperature during Year 2 (2012-2013) was a little colder than during Year 1 (2011-2012) with an air temperature freezing index, FI, of 1355 Degree-days compared to 1206 Degree-Days. The heave patterns are lower in Year 2, which is likely due to the black surface of the pavement after the overlay was placed. Secondly, the movement of the thinned pavement in plates during the spring and summer of 2011 crimped some of the thermistor cables and caused some erratic temperature readings in Holes #2, #3 and #6.

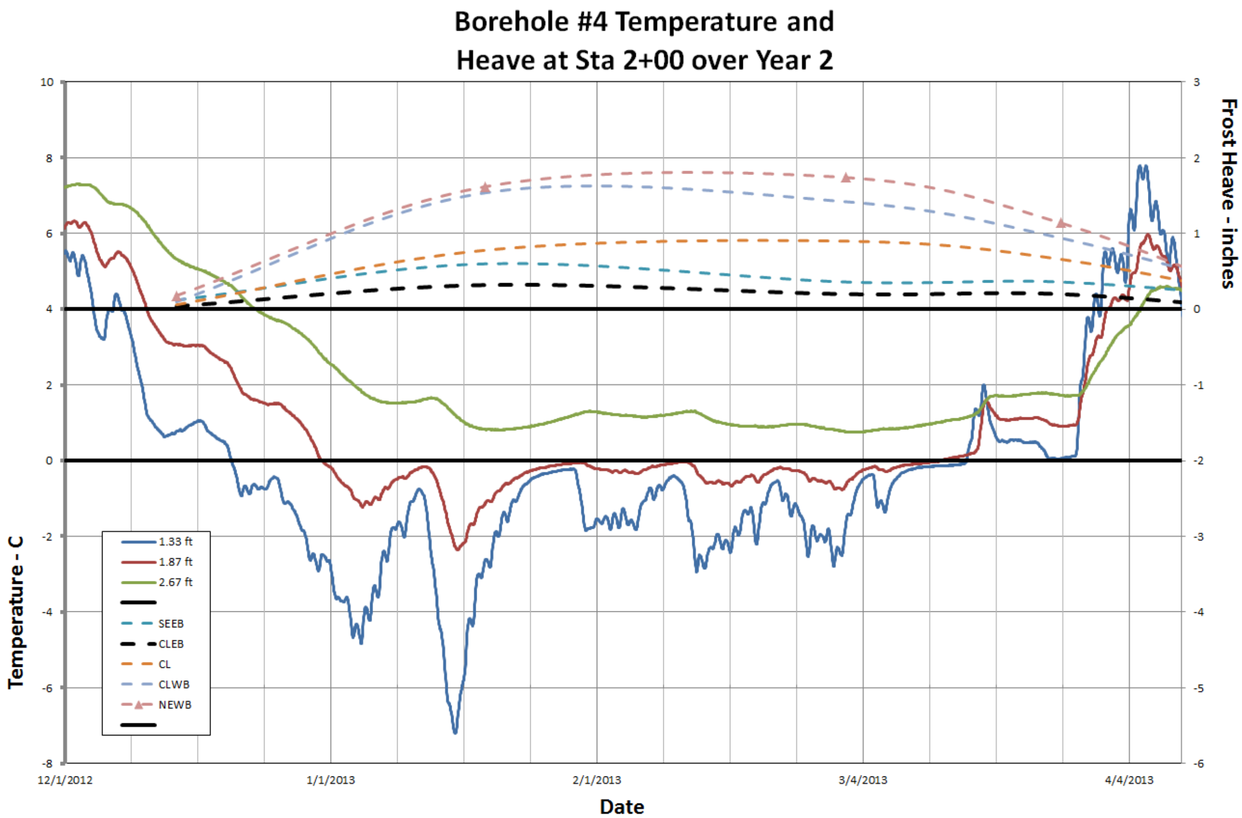


Figure 53. Graph. Hole #4 Temperature and Heave at STA 2+00 - Year 2.

Figure 54 compares the temperature data to heave at Hole #3. Hole #3, located on the centerline of the west bound lane, is in the taper zone and only has about 37 mm (1-1/2 in.) of foam for insulation. The red line at a depth of 570 mm (1.87 ft) is consistently below 0°C (32°F) and the green line at depth 813 mm (32 in.), decreases to 0°C (32°F) on February 1, 2012 and stayed on 0°C (32°F) through the rest of the season. The heavy dashed line is the centerline heave over Hole #3 and indicates that the heave increases progressively throughout the season. The slower that the freezing front advances into the soil, the greater the opportunity for water to flow upward and to freeze into an expanding ice lens. Finally, as the soil temperatures increase above 0°C (32°F) around March 15, 2012, the ice melts and the pavement quickly drops back to the normal elevation during the summer.

Holes #2 and #6 show similar patterns as Hole #4. Figures for all of the holes over both years are presented in appendix H.

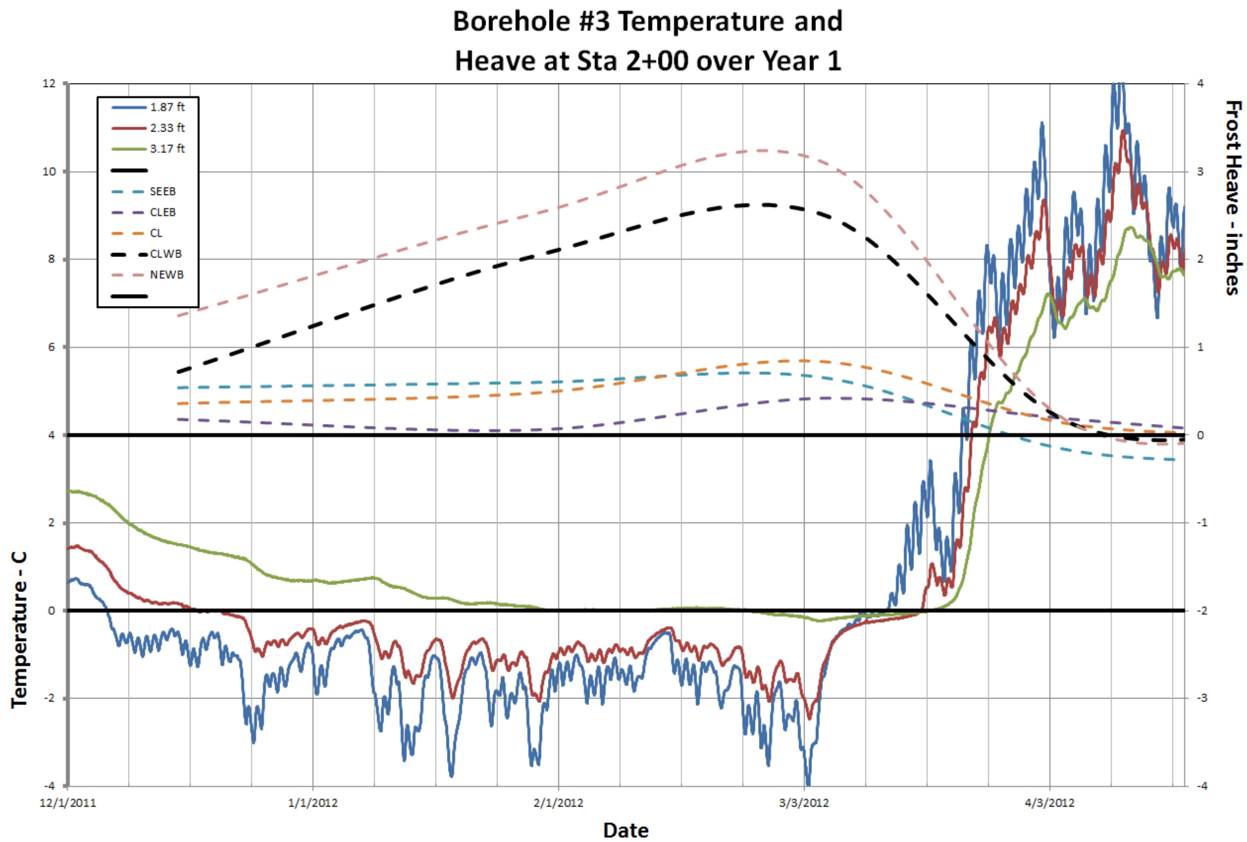


Figure 54. Graph. Temperature and heave comparison in Borehole #3, Year 1.

Average and Maximum Depth of Frost Penetration

The upper thermistors were placed to determine the temperatures around the foam layers. When the freezing temperatures in a borehole cross the thermistors, that determines the depth of freezing at that time. However, it does not give an absolute depth when the temperature is not at freezing around a thermistor.

Plotting the thermistor values as a function of depth produces graphs that show the depth at which the temperature crosses freezing. Figure 55 and figure 56 plot the average temperatures shown in figure 51 plotted against depth for Years 1 and 2, respectively. Reading the values at 0°C (32°F) gives the values shown in table 4 for Average Depth.

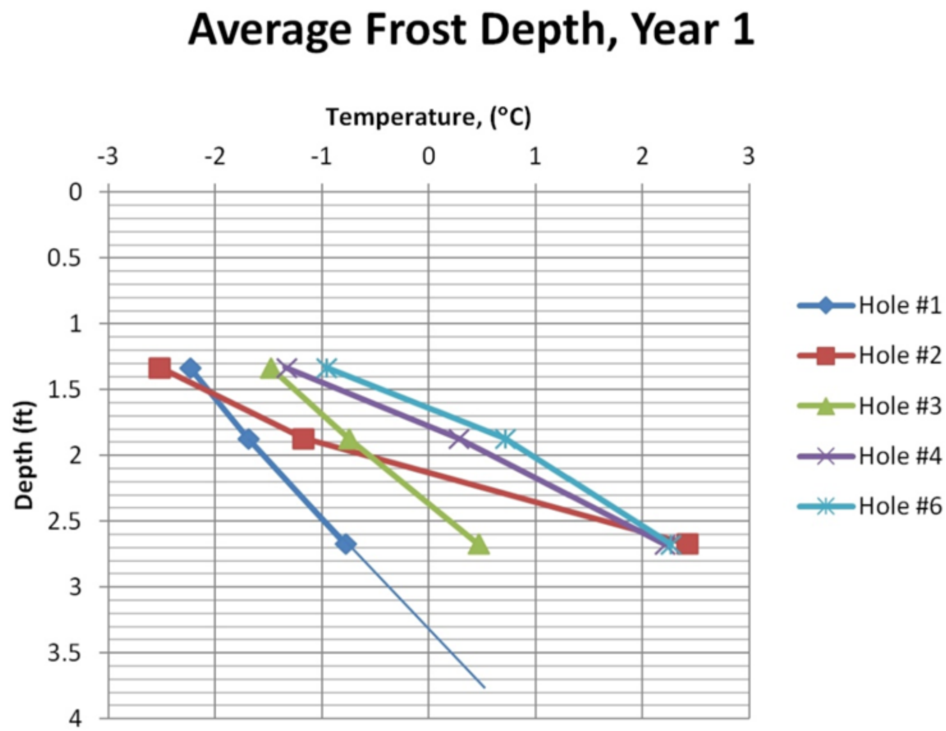


Figure 55. Graph. Average Borehole Temperatures with Depth, 2011-2012.

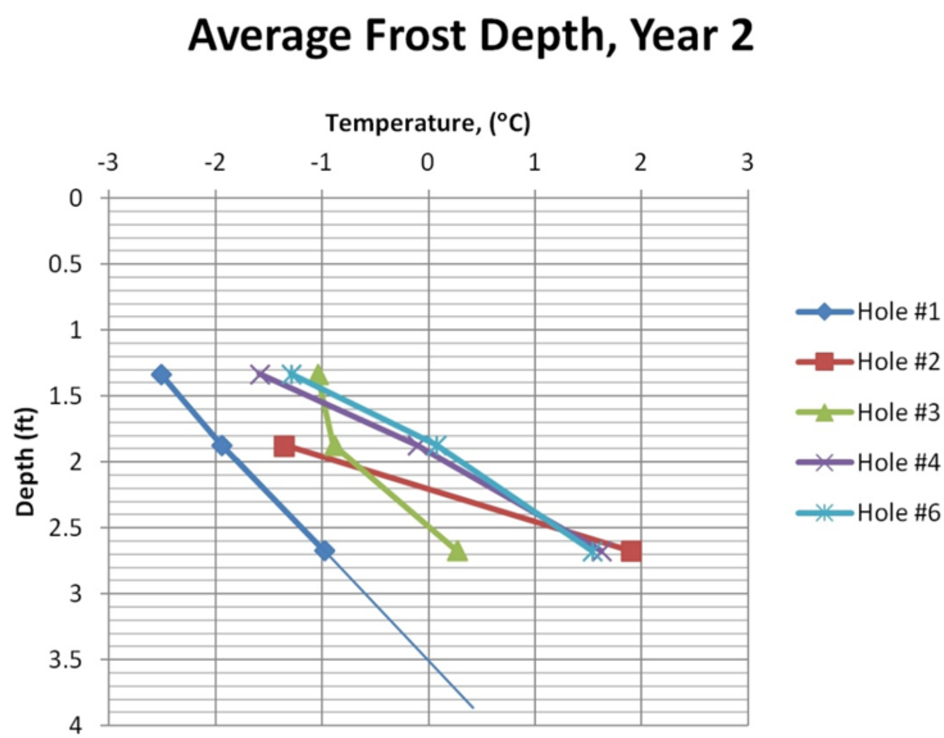


Figure 56. Graph. Average Borehole Temperatures with Depth, 2012-2013.

Table 4. Average and Maximum Depth of Freezing in the Boreholes (1.00 ft = 0.3048 m)

	2011-2012		2012-2013	
	Average Depth (ft)	Maximum Depth (ft)	Average Depth (ft)	Maximum Depth (ft)
Hole #1	3.30	3.80	3.50	3.80
Hole #2	2.13	2.13	2.21	2.60
Hole #3	2.36	2.85	2.50	3.15
Hole #4	1.78	2.25	1.93	2.45
Hole #6	1.63	2.20	1.84	2.40

Figure 57 through 61 plots the minimum temperatures during both years in each borehole. For example, figure 49 plots the temperatures for the duration of the project for Hole #4. During the first year, the minimum temperature for the top three thermistors occurs on March 3, 2012. Those values are plotted in figure 60. During the second year, two low temperatures are evident.

A cold spell occurred on January 15, 2013 that produced the lowest values in the top two thermistors. However, it occurred early enough in the season that the third thermistor was not able to respond, so it had a lower minimum temperature on February 28, 2013, just before the temperatures started to increase. The thermistor values for both of these dates were plotted to determine the maximum frost depth that was recorded in table 4.

Averaging the average frost depths in table 4 for Holes #2, #4 and #6 (having the full 76.2 mm (3 in.) of foam) gives 560 mm (1.84 ft). The average depth in Hole #1 is 1.04 m (3.40 feet). In a general sense, the effect of the 76 mm (3 in.) of foam is equivalent to 475 mm (1.56 ft) of soil, or 25.4 mm (1 in.) of foam equals about 152 mm (6 in.) of soil.

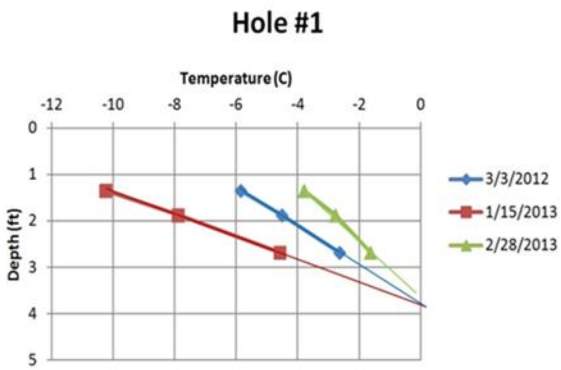


Figure 57. Graph. Hole #1. Minimum Temperatures versus Depth indicating Maximum Depth of Freezing.

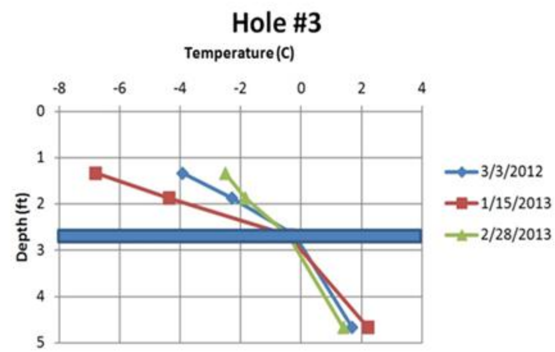


Figure 59. Graph. Hole #3. Minimum Temperatures versus Depth indicating Maximum Depth of Freezing.

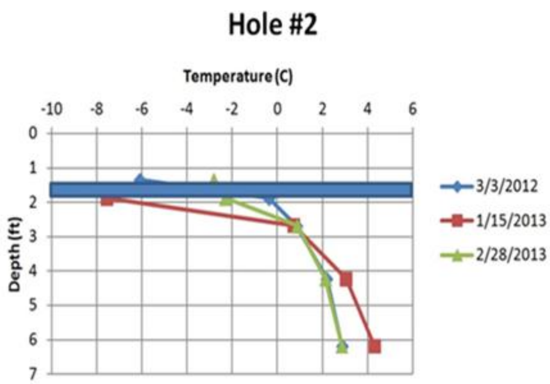


Figure 58. Graph. Hole #2. Minimum Temperatures versus Depth indicating Maximum Depth of Freezing.

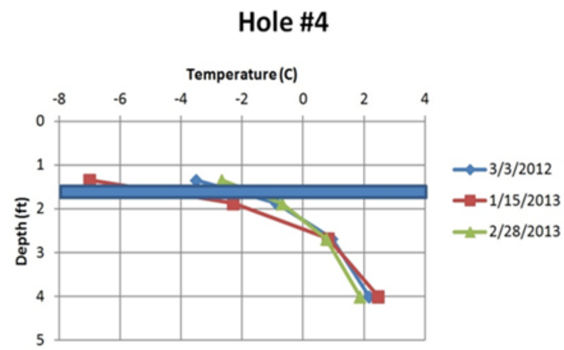


Figure 60. Graph. Hole #4. Minimum Temperatures versus Depth indicating Maximum Depth of Freezing.

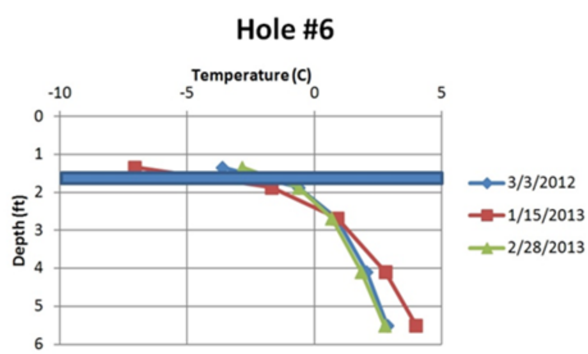


Figure 61. Graph. Hole #6. Minimum Temperatures versus Depth indicating Maximum Depth of Freezing.

Borehole #5

Hole #5 is located off of the road surface and adjacent to the data acquisition post. During the winter months, the location was covered with snow. The snow acts as an insulation layer and protects the soil surface from direct contact with freezing air. This is clearly shown in figure 62 in which the blue line, representing the depth of 405 mm (1.33 ft), stays within one-half a degree of 0°C the entire time it is covered with snow. An example of the insulation properties of the snow is that on January 14, 2013, the air temperature was -28°C (-18.4°F) and the top thermistor in Hole #5 reached a minimum temperature of -0.3°C (31.5°F) on January 22, 2013 eight days later while the top thermistor in Hole #1 reached -10.3°C (13.5°F), 27 hours after the minimum air temperature occurred.

Because the temperatures remain high in the snow-covered area adjacent to the road, the frost heave under the road shoulder will usually be greater because the soil water is able to flow laterally and provide water for freezing. This would be a likely cause for the shoulders of the road breaking up more than the road surface itself.

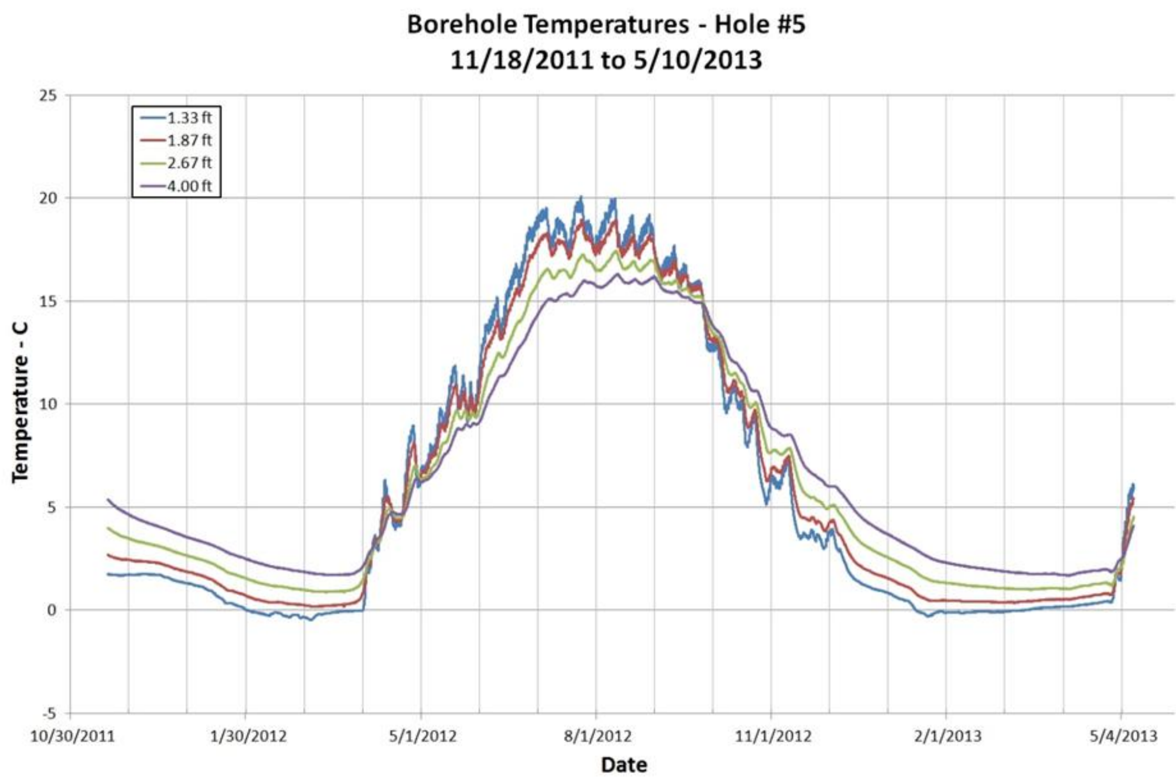


Figure 62. Graph. Temperature Reading for the four thermistors in Hole #5

AIR FREEZING INDEX

The air freezing index is the sum of the average daily degrees below freezing throughout the freezing season. It gives an indication of the thermal driving potential to cause the soil to freeze. The temperatures are measured at a height of 1.4 m (4.5 ft) above the ground surface. This was approximately the height of the instrumentation box when viewed from the roadway. The freezing index is determined by adding the temperatures together cumulatively from a day prior to the freezing season in the fall and then stopping when the value increases past the minimum value in the spring. The air freezing curves from the site are shown in figure 63. The upper red line, representing Year 2, is fully formed with a well defined maximum and a well defined minimum. The difference between those two values is the Air Freezing Index. In this case, the value is 169 minus -584 or 753 °C-Days acting over 167 days.

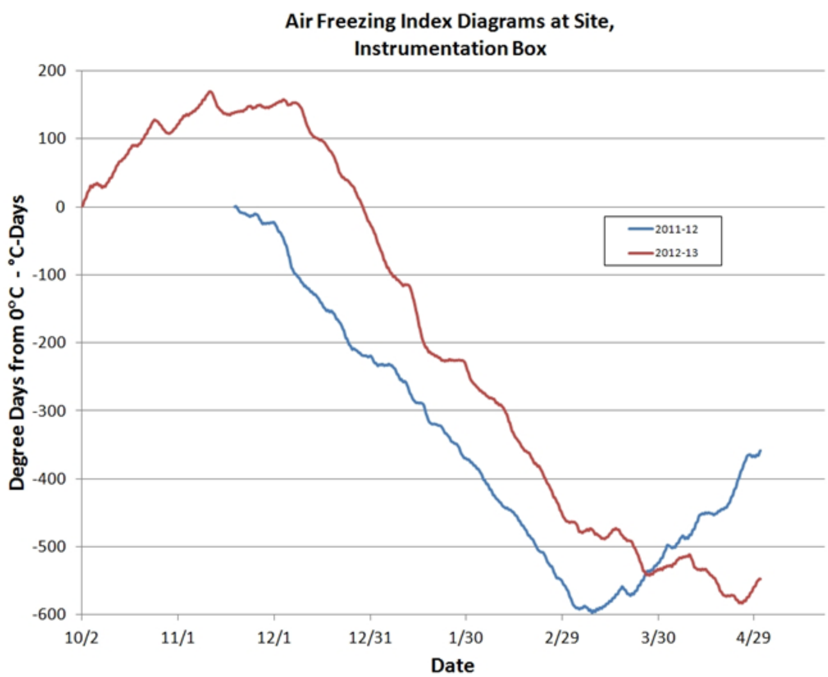


Figure 63. Graph. Air Freezing Index curves at the Site.

Unfortunately, the curve for Year 1 is truncated at the earliest days because the instrumentation was not installed until mid November. In order to get a representative value for the upper portion of the curve, a several step process was required. The first step was to compare other freezing index curves from other weather sites to the Year 2, 2012-2013 data.

There are three NWS weather stations in the Encampment area and the Webber Springs SNOTEL station between the site and Battle Mountain Pass shown in figure 64. Two NWS stations are in town, about 1.6 km (1 mi) apart. Formally, both stations, 483048 and 483050, are closed and do not measure temperatures, just snow data. However, Tony Bergantino of the Wyoming Water Resources Data System (WRDS) was able to locate temperature data for both stations. The data was identical between the two stations, except 483050 had a longer period of record than 483048. It is important to note that both data sets were indicated to be estimates, not actually recorded values.

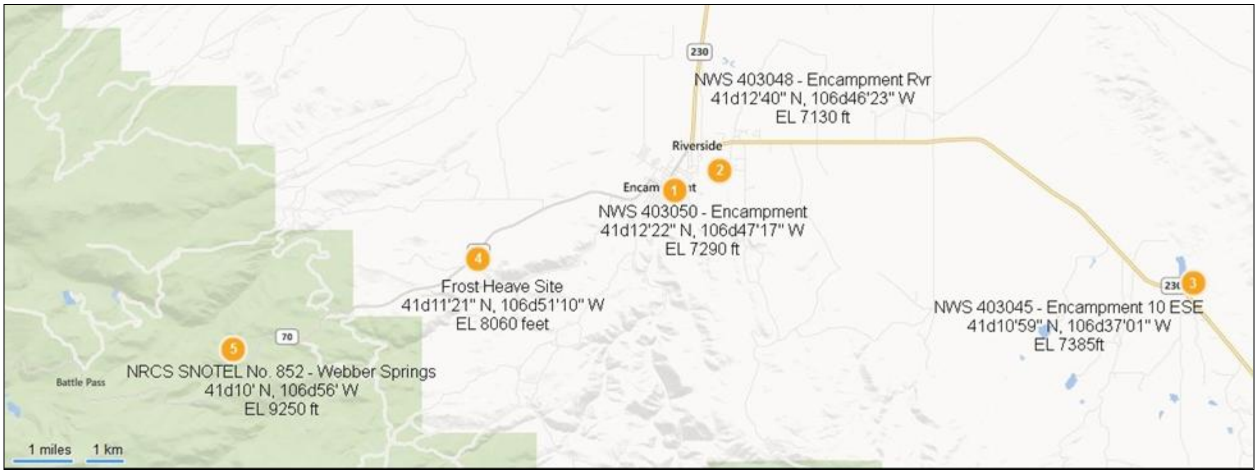


Figure 64. Map. Locations of National Weather Stations and SNOTEL Station Relative to Site.

Figure 65 shows the freezing index diagram for the site, NWS 483050 and Webber Springs. The data record for NWS 483050 ended on January 17, 2013, so is not useful. However, the shapes of the site and Webber Springs curves are very similar, with the Webber Springs data being somewhat expanded from the site curve because of its location at a higher elevation. The freezing index for the site is 753 °C-days while that of Webber Springs is 817 °C-days. A scaling ratio is determined by dividing the site's FI value by the Webber Springs FI, to get 0.92.

Figure 66 shows the air freezing index curve for Year 1 in which the curve for Webber Springs has been shifted down so that the minimum values match. The general shape of the Webber Springs curve agrees with the site curve and figure 67 shows that a scaling ratio of 0.94 matches site curve exactly. Therefore, a scaling ratio between 0.92 and 0.94 would be appropriate between these two sites. The Freezing Index for the site in Year 1 is equal to 73 minus -597 or 670 °C-Days (1206 °F-Days) acting over 137 days.

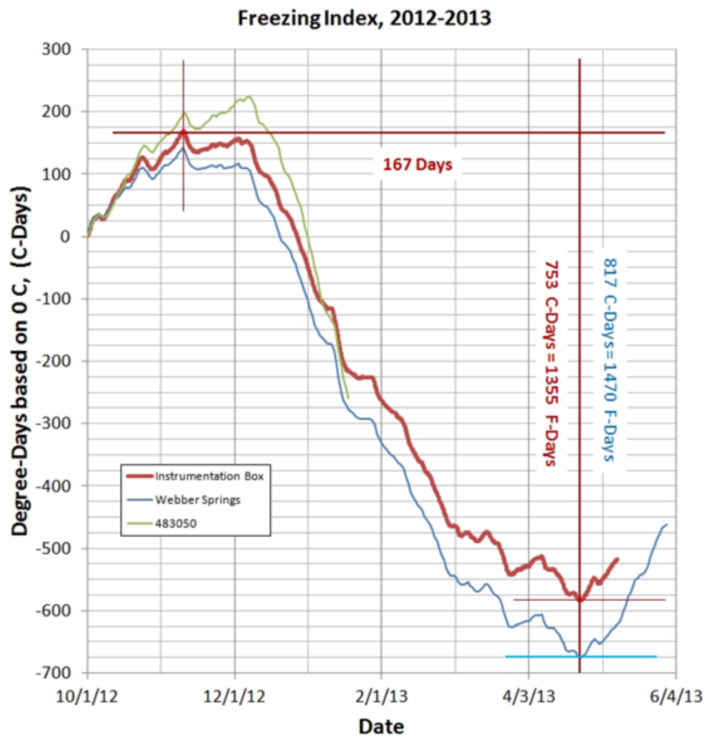


Figure 65. Graph. Air Freezing Index at the Site, NWS 483050 and at Webber Springs, 2012-2013.

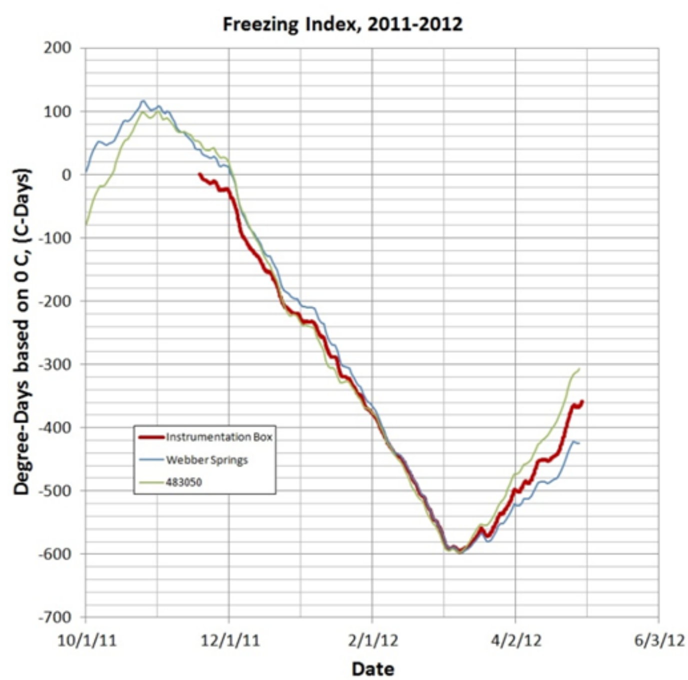


Figure 66. Graph. Air Freezing Index for 2011-2012.

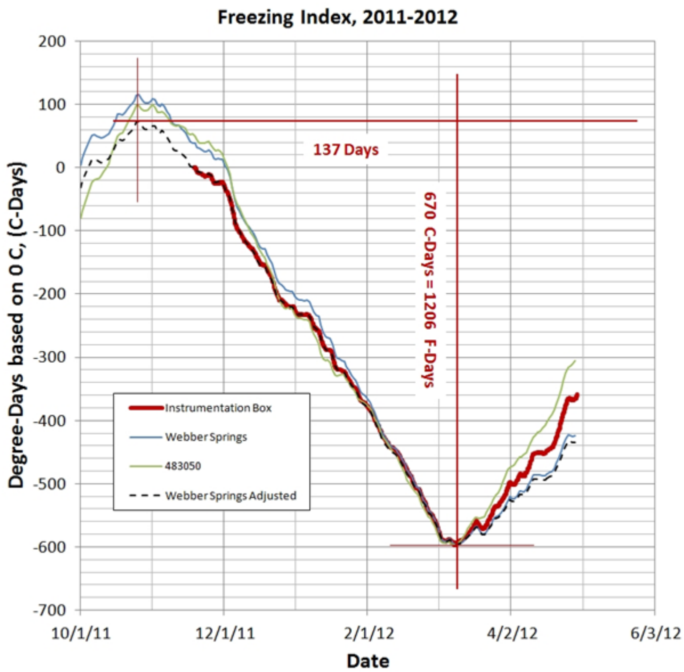


Figure 67. Graph. Adjusted Webber Springs Freezing Index Curve Matches Site Curve.

GROUNDWATER UNDER THE ROAD SURFACE

The state of the groundwater was discussed in chapter 3. In general, it appears that the water is flowing from the west, hits the caprock around STA 2+00 where it mounds up and then separates, with most water flowing to the north and some flowing over the caprock where it can supply water for the freezing front. Regarding figure 30, there appears to be a slight lowering of the water surface that could be due to the lack of vertical flow from the road surface, but lacking longer-term records of historic water levels, it would not be possible to say that the layer of impermeable foam above would be the sole reason.

However, the foam prevents water from flowing upward into the freezing soil above the foam layer. The foam creates a consistent impervious barrier to prevent water from reaching the freezing silty sand above the foam. Since the water is cutoff, no segregational heaving can occur, especially if the covered pavement surface is intact. This may be just as significant effect as the insulation effect. Note that this is even more effective than the insulation barriers made from foam panels. A large amount of water can easily come up through the gaps between the panels, therefore not affecting an impervious cutoff.

A final consequence of the layer is that the water content at the spring thaw should be much lower because the excess water was not allowed to freeze around the lenses. This should provide better, uniform support for the road surface, less likelihood of the soil weakening due to an over-saturated condition and minimal differential movement of the soil.

CHAPTER 6. PREDICTION OF FROST DEPTH AND SELECTION OF THICKNESS OF POLYMER FOAM INJECTION

Several models exist to estimate the depth of frost penetration, i.e., Stefan, 1887, Neumann, 1860 (reported in Lunardini, 1980), Soliman etal, 2008. The most common appears to be the Modified Berggren Equation (MBE) initially developed by Aldrich and Paynter in 1953. A simplified procedure is presented in the Minnesota Department of Transportation's Pavement Design Manual (MNDOT, 2007). This model is useful to predict frost depths in multiple pavement or soil/aggregate layers.

EQUIVALENT VOLUMETRIC HEAT FOR THE POLYMER FOAM

The Volumetric Heat of Latent Fusion, L, is the amount of heat required to convert the liquid water to ice in a body of unit volume. Once all the water has frozen, heat will flow through a layer by conduction. (The dry unit weight times the water content in the equation for L calculates the weight of water per unit volume.) The latent heat is a capacitance term because it represents the amount of heat that must be added to the soil to allow the water inside it to freeze.

Because the Uretek repels water and has none in its structure, it does not have a Heat of Latent Fusion. However, it behaves in a similar manner because the carbon dioxide contained in the closed cells has to be cooled by conduction through the thin, low conductivity cell walls. It absorbs an amount of heat before it allows the temperatures to change across it. There are no published values for Volumetric Heat of Latent Fusion for polymer foams, but its effects have to be accounted for before the heat flux can be adequately described.

Having established the depth of freezing in soil profiles with and without the polymer injection layer in this project, this information can be used to determine an appropriate value for an Equivalent Latent Heat for the Polymer Foam, L_F. Using that information, the depth of freezing can be predicted without and with a foam insulation layer to determine the depth of injection required to prevent freezing in the underlying layers during seasons with different freezing indices.

MNDOT FROST PENETRATION MODEL

The MNDOT model was described in Chapter 2. To make a prediction requires following these basic steps:

1. Define the Road Profile:
    a. Determine the Layer Thicknesses, h_i (inches), for the pavement, the base courses and the subgrade soils.
    b. Determine the Dry Unit Weight and the Water Content of each layer.
        i. MNDOT recommends:
            1. Portland Cement Concrete:
                a. $\gamma_d = 140$ pcf
                b. w = 2%

2. Bituminous Pavement:
    a. γ_d = 138 pcf
    b. w = 0%.

2. Determine the Freezing Index for the site:
    a. Use known values. For this project, the actual Freezing Indices were equal to 670 °C-Days (1206 °F-Days) in 2011-2012 and 753°C-Days (1355 °F-Days) in 2012-2013.
    b. Select a value of Freezing Index from a table, such as table 2, Air Freezing Index for Selected Wyoming Communities (NOAA, 2012).
        i. MNDOT recommends using the 30 year return period for design.
        ii. The NOAA table does not give values for 30 year return periods, so select either the 25 year or 50 year values (1445°F-Days or 1506 °F-Days), or estimate the 30 year value by interpolation (1460 °F-Days).
3. Determine the thermal properties of the layers
    a. k_t = Thermal Conductivity (BTU/ft hr °F)
        i. Use figures 21 through 24.
        ii. Kirsten (1949) Equations

$$k_{t-Frozen\ Sand} = \frac{\left[0.076(10)^{0.013\gamma_d} + 0.032(10)^{0.0146\gamma_d} w\right]}{12}$$

$$k_{t-Unfrozen\ Sand} = \frac{\left[\left(0.7\log(w) + 0.4\right)10^{0.01\gamma_d}\right]}{12}$$

$$k_{t-Frozen\ Silt-Clay} = \frac{\left[0.01(10)^{0.022\gamma_d} + 0.085(10)^{0.008\gamma_d} w\right]}{12}$$

$$k_{t-Unfrozen\ Silt-Clay} = \frac{\left[\left(0.9\log(w) - 0.2\right)10^{0.01\gamma_d}\right]}{12}$$

        iii. MNDOT recommends:
            1. k_t of Portland Cement Concrete = 6 BTU/ft^2 hr °F/in.
               = 0.5 BTU/ft hr °F
            2. k_t of Bituminous Pavement = 10 BTU/ft^2 hr °F/in.
               = 0.83 BTU/ft hr °F
        iv. Use the frozen conductivity for freezing, use the unfrozen conductivity for thawing.
    b. Volumetric Heat of Latent Fusion for granular materials:
        i. $L = 1.43\ (\gamma_d\ w)$ (BTU/ft^3)
            1. γ_d = Dry Unit Weight (pcf)
            2. w = Water Content (%)
4. Determine the Freezing Index Required (*FIR*) for each layer using Stefan's equation (figure 18):
    a. $$FIR_i = \frac{{z_i}^2 L_i}{48 K_{t-i}}$$
    b. If $FIR_i < FI$,
        i. Calculate the remaining *FI* by subtracting FIR_i from *FI*.
        ii. Repeat step 4)a. for the next layer.

c. If $FIR_i \geq FI$

   i. Calculate the depth of frost penetration into that layer using Stefan's equation (figure 18):

   $$z_i = \sqrt{\frac{48k_{t-i}FI}{L_i}}$$

   where FI is the remaining FI from the previous layer.

   ii. z equals the sum of the previous layer thicknesses plus z_i.

COMPARISON OF PREDICTED TO ACTUAL FREEZING DEPTHS

The validity of this procedure can be evaluated by comparing the predicted frost depths to those measured at the site. The Freezing Index, FI , is an average value determined over the winter season. It makes no assumption about the distribution of the colder days so it is most appropriate to use the average frost depths given in table 4 rather than the maximum depths, which are influenced by the coldest periods of the winter.

In addition, because the polymer foam in Holes #2, #3, #4 and #6 alters the frost depth, the predicted depths can be varied by adjusting the value of the Equivalent Volumetric Latent Heat of the polymer. All eight predicted depths were determined and compared to the ten actual depths (Holes #4 and #6 had identical physical properties). The errors were squared and summed. Using a least-squares procedure, the best-fit value of Equivalent Volumetric Latent Heat was determined to be 1290 BTU/ft^2 and that value was used in the following descriptions.

Hole #1 – Year 1

Hole #1 is about 3 m (10 ft) from Hole #10-5 from the April, 2010 drilling which occurred just after the winter season. From the #10-5 boring, the asphalt thickness was 6.5 inches (165 mm) and the base course was 200 mm (7.9 in.) with a water content of 3.7 percent. The subgrade silty sand had a water content of 1.9 percent. The recommended conductivity was used for the asphalt and the water content was assumed zero. The subgrade unit weight was not measured, but it was soft and easy to drill. Its unit weight was assumed to be 1.7 g/cm^3 (105 pcf). The Freezing Index for the first year was 1206 °F-Days. The analytic procedure in table 5. The actual measured depth of frost penetration was 3.30 feet (1005.4 mm) and the predicted depth was 3.38 feet (1030.2 mm). The correlation using the Stefan's equation is excellent.

Table 5. Frost Depth Approximation for Hole #1 - Year 1.
Estimated Depth = 3.38 ft, Actual Depth = 3.30 feet.

Hole #1		#10-5				FI =1206				
Material	γ_d (pcf)	w (%)	L (BTU/ ft^3)	h_i (inches)	k_t (BTU/ft^2 hr °F/in)	FIR_i (°F-Days)	FI_i (°F-Days)	z_i (inches)	Σz_i (inches)	Σz_i (ft)
Asphalt	138	0	0	6.5	10	0	1206	6.5	6.5	0.54
Base Course	120	3.7	634.9	7.9	18	45.9	1160.1	7.9	14.4	1.20
Subgrade Silty Sand	105	1.9	285		3.5	1160.1	0	26.1	40.5	3.38

Holes #4 and #6 – Year 1

Holes #4 and #6 are adjacent to Hole #10-4. The pavement thickness was 178 mm (7.0 in.) and the base course thickness was 7.4 inches (188 mm). The water content from 2010 for the base course is 4.4 percent and for the silty sand is 7.0 percent. Holes #4 and #6 have 76.2 mm (3 in.) of polymer at a depth of 457 mm (18 in.). Based on tests reported in chapter 2, the polymer has a conductivity of 0.192 BTU/ft^2 hr °F/in. The model predicted a frost depth of 530 mm (1.74 ft) while the actual measured depth was 542 mm (1.78 ft) in Hole #4 and 496.8 mm (1.63 ft) in Hole #6, an error of 48 mm (1.9 in.). See table 6.

Table 6. Frost Depth Approximation for Holes #4 and #6 - Year 1.
Estimated Depth = 1.74 ft, Actual Depth at Hole #4 = 1.78 feet.
Actual Depth in Hole #6 = 1.63 feet.

Holes #4 and #6		#10-4				FI =1206				
Material	γ_d (pcf)	w (%)	L (BTU/ft^3)	h_i (inches)	k_f (BTU/ft^2 hr °F/in)	FIR_i (°F-Days)	FI_i (°F-Days)	z_i (inches)	Σz_i (inches)	Σz_i (ft)
Asphalt	138	0	0	7	10	0	1206	7.0	7.0	0.58
Base Course	120	4.4	755	7.4	18	47.9	1158	7.4	14.4	1.20
Subgrade Silty Sand	105	7	1051	3.6	7.8	36.5	1122	3.6	18.0	1.50
Polymer Foam	7		1290		0.192	1122	0	2.8	20.8	1.74

Hole #2 – Year 1

The actual measured depth of frost penetration in Hole #2 was 649 mm (2.13 ft). Using the same approximation for Hole #2 as for Holes #4 and #6 above results in an error of 119.4 mm (4.7 in.). However, as was seen in figure 51, Hole #2 has behaved differently from Holes #4 and #6 (the other two holes with 76.2 mm (3 in.) of polymer), appearing as if the polymer foam were injected somewhat deeper than the others. Table 7 was recomputed with a depth of 167.6 mm (6.6 in.) for the subgrade layer and the calculated depth of penetration was 602 mm (1.98 ft), yielding an error of 43.2 mm (1.7 in.).

Table 7. Frost Depth Approximation for Hole #2 - Year 1.
Estimated Depth = 1.99 ft, Actual Depth = 2.13 feet.

Holes #4 and #6		#10-4				FI =1206				
Material	γ_d (pcf)	w (%)	L (BTU/ft³)	h_i (inches)	k_t (BTU/ft² hr °F/in)	FIR_i (°F-Days)	FI_i (°F-Days)	z_i (inches)	Σz_i (inches)	Σz_i (ft)
Asphalt	138	0	0	7	10	0	1206	7.0	7.0	0.58
Base Course	120	4.4	755	7.4	18	47.9	1158	7.4	14.4	1.20
Subgrade Silty Sand	105	7	1051	3.6	7.8	36.5	1122	3.6	18.0	1.50
Polymer Foam	7		1290		0.192	1122	0	2.8	20.8	1.74

Hole #3 – Year 1

Hole #3 is also adjacent to Hole #10-4. Hole #3 has 38 mm (1.5 in.) of foam at a depth of 762 mm (30 in.). Table 8 shows an estimated frost depth of 807.7 mm (2.65 ft) as opposed to the actual 719 mm (2.36 ft) or a difference of 92 mm (3.63 in.).

Table 8. Frost Depth Approximation for Hole #3 - Year 1.
Estimated Depth = 2.65 ft, Actual Depth = 2.36 feet.

Hole #3		#10-4				FI =	1206			
Material	γ_d (pcf)	w (%)	L (BTU/ft³)	h_i (inches)	k_t (BTU/ft² hr °F/in)	FIR_i (°F-Days)	FI_i (°F-Days)	z_i (inches)	Σz_i (inches)	Σz_i (ft)
Asphalt	138	0	0	7	10	0	1206	7.0	7.0	0.58
Base Course	120	4.4	755	7.4	18	47.9	1158	7.4	14.4	1.20
Subgrade Silty Sand	105	7	1051	15.6	7.8	684.8	473	15.6	30.0	2.50
Polymer Foam	7		1290		0.192	473	0	1.8	31.8	2.65

Holes #4 and #6 – Year 2

The Freezing Index during Year 2 was 1355 °F-Days, or 12.4 percent colder than Year 1. Consequently, the freezing depths were deeper during the second year. The predicted frost depth for Holes #4 and #6 was 582 mm (1.91 ft), while the actual depth for Hole #4 was 588 mm (1.93 ft) and for Hole #6 was 561mm (1.84 ft). See table 9.

Table 9. Frost Depth Approximation for Holes #4 and #6 - Year 2.
Estimated Depth = 1.91 ft, Actual Depth at Hole #4 = 1.91 feet.
Actual Depth at Hole #6 = 1.84 feet.

Hole #6		#10-4				FI =	1355			
Material	γ_d (pcf)	w (%)	L (BTU/ft^3)	h_i (inches)	k_f (BTU/ft^2 hr °F/in)	FIR_i (°F-Days)	FI_i (°F-Days)	z_i (inches)	Σz_i (inches)	Σz_i (ft)
Asphalt	138	0	0	7	10	0	1355	7.0	7.0	0.58
Base Course	120	4.4	755	7.4	18	47.9	1307	7.4	14.4	1.20
Subgrade Silty Sand	105	7	1051	3.6	7.8	36.5	1271	3.6	18.0	1.50
Polymer Foam	7		1290	30	0.192	1260	11	3.0	21.0	1.75
Subgrade Silty Sand	105	7	1051	0.0	7.8	11	0	2.0	23.0	1.91

Summary of Frost Depth Predictions

Table 10 presents a summary of the calculated depths of frost penetration to the measured average depths given in table 4. The largest error was a 97.5 mm (0.32-ft) over-prediction in the first year while five of the ten estimates had errors less than 30.5 mm (0.1 ft) over a range of actual depths from 497 mm (1.63 ft) to 1067mm (3.50 ft). This includes Hole #1 with no foam, Hole #3 with 38.1 mm (1.5 in.) of polymer foam and the other three Holes with 76.2 mm (3.0 in.) of polymer foam. The quality of the results was produced by varying the Equivalent Volumetric Latent Heat of the foam to minimize the least square error of the prediction. The standard deviation of the sum of the errors was 48.8 mm (0.16 feet). This produced a value of Equivalent Volumetric Latent Heat for the foam of 1290 BTU/ft^3. The technique appears to successfully predict the frost depths well enough to use as a predictor in design. These results are based solely on an analysis using Uretek 486 STAR #3 structural polymer at this site and time period.

Table 10. Summary of Measured vs. Calculated Frost Depths for Years 1 and 2.

	2011-2012 - *FI* = 1206 °F-Day			2012-2013 - *FI* = 1355 °F-Day		
Hole #	Measured Depth	Calculated Depth	Difference	Measured Depth	Calculated Depth	Difference
	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
1	3.30	3.38	0.08	3.50	3.51	0.01
2	2.13	1.98	-0.14	2.21	1.99	-0.22
3	2.36	2.65	0.32	2.50	2.68	0.18
4	1.78	1.74	-0.03	1.93	1.91	-0.02
6	1.63	1.74	0.12	1.84	1.91	0.07

EXAMPLES OF FROST HEAVE PREDICTIONS

As an example, using the 30 year Frost Index value recommended by MNDOT, the Design Frost Index for Encampment is FI = 1460 °F-Days. The model predicts a freezing depth of 1100 mm (3.6 ft) with no foam or 536 mm (1.76 ft) with 81.3 mm (3.2 in.) of polymer foam to contain the freezing front in the foam. Given the variability of the injection depth, probably 90 to 105 mm (3.5 to 4.0 in.) of foam should be specified.

The area around Bondurant, WY has had difficulties with frost heave as well. It probably has the same type of silty sand or sandy silt as at the Encampment site. The 30 year Frost Index for Bondurant is evaluated from table 2 as FI = 3300 °F-Days. Assuming the same ground conditions as Encampment, the depth of frost penetration at Bondurant is 1480 mm (4.85 ft) without the foam and 122 mm (4.8 in.) of foam would limit the frost depth to within the foam layer. Design would likely require 140 mm (5.5 in.) to 152.4 mm (6.0 in.) for the injected depth.

SUMMARY

The MNDOT Model has been shown to effectively predict the depth of frost penetration at five boreholes over two seasons. It provided very accurate estimates for profiles with no foam and very reasonable estimates in the profiles with 38 mm (1.5 in.) and 76.2 mm (3.0 in.) of injected Uretek 486 STAR #3 structural polymer. Examples to determine the foam thickness required at two locations in Wyoming produced appropriate depths at those sites.

CHAPTER 7. CONCLUSIONS AND IMPLEMENTATION RECOMMENDATIONS

STUDY OBJECTIVES

The primary objective of this research was to determine whether the controlled injection of structural polymer foam into the subgrade soil would stabilize the frost heave occurring at milepost 51.8 on WY-70. This can be broken into three questions:

- Can injection of the foam nondestructively level the road surface and make the road safer to drive without full reconstruction of the site?
- Will the foam provide a sufficient thermal barrier to reduce or eliminate the frost heave caused by frost penetration at the site?
- Will the continuous blanket of foam create a barrier to moisture during the spring thaw?

CONCLUSIONS

The process of injecting a polymeric foam into the subgrade to reduce frost heave is novel. The research project has been a success in eliminating heave in a dangerous situation. Several important conclusions can be reached concerning this process.

Objective 1 - Can injection of the foam nondestructively level the road surface and make the road safer to drive without full reconstruction of the site?

1. The three-inch layer of foam injected below the base course of the highway has significantly reduced or eliminated the frost heave under the road.
2. Depending on the initial quality of the road surface, the injection can be performed without additional surface treatment. The prior condition of the road surface and the effects of milling eventually required repaving of the surface.
3. The foam may have provided some structural support to spread out the heaving and reduce local effects; however, this was not tested in this project.

Objective 2 - Will the foam provide a sufficient thermal barrier to reduce or eliminate the frost heave caused by frost penetration at the site?

1. It has been shown that the thermal regime under the foam has altered the pattern of freezing associated with frost heave. At this location, the 76 mm (3.0 in.) layer of foam was sufficient to prevent the soil beneath the layer from freezing. The soil and aggregate above the layer will continue to freeze, but the aggregate is not frost susceptible and the layer prevented upward flow of the water into the subgrade to prevent segregational freezing.

Objective 3 - Will the continuous blanket of foam create a barrier to moisture during the spring thaw?

1. There has been a slight reduction in water level after the foam was injected which could have decreased the unsaturated flow to the freezing front.

2. The foam layer has prevented segregational heaving from developing both above and below the layer. This has prevented the extra amount of water in the profile that would be associated with the ice lenses from being available during the spring thaw. Therefore, the soil profile should not become over-saturated and experience the strength reduction generally attributed to the spring thaw.

OTHER OBSERVATIONS

While outside the study of the objectives, several other observations were made during the construction and research that have significant impact on its usefulness as a remediation procedure.

1. The process can be accomplished in a short time frame. In this case, the project was functionally completed in five working days. To reconstruct a highway with foam insulation panels could easily take weeks or months. Special site preparation and control is required when placing the panels and placing the sub-base material above it.
2. The operation can be performed in a single lane, so that a traffic lane can remain open all the time. Simple routing with a pair of flaggers, Stop/Slow signs and cones is sufficient for traffic control, especially in the area with low traffic counts.
3. There was no need for a detour. Hence, safety is an important factor for both the workers and the driving public. At night, no equipment was left on the site and the pavement was still in sufficient repair that the traffic could reuse the lane after appropriate signage for Slow/Construction was provided.
4. As the polymer foam is hydrophobic, it does not mix with the water and is non-toxic after its reaction occurs. There does not appear to be any negative effects on the environment.
5. There is no outward sign of the remedial action taking place nor anything that should distract a driver passing over that section.

IMPLEMENTATION RECOMMENDATIONS

While this remediation procedure is still in an initial and experimental phase, after two years, it appears to be successfully controlling the frost heaving that existed at the site. It is recommended that this procedure be tested at another site to verify its successful application. A procedure is given in this report to predict the thickness of the required at a site that may have different Freezing Indices.

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APPENDIX A

SITE INVESTIGATION REPORT
NOVEMBER 8, 2000

RECEIVED NOV - 9 2000

THE STATE OF WYOMING

Jim Geringer, *Governor*
Gene Roccabruna, P.E., *Director*

Department of Transportation

5300 BISHOP BOULEVARD (82009) P.O. BOX 1708 CHEYENNE, WYOMING 82003-1708

November 8, 2000

M E M O R A N D U M

PROFESSIONAL GEOLOGIST
GREGORY E. MILLER
PG-2547
WYOMING
11-8-00

TO: J. Gould, P.E., District Engineer, Laramie

FROM: Greg Miller, P.G., Engineering Geologist

SUBJECT: Frost Heave at Milepost 51.8±

PROJECT: CB09B, Battle Mountain Highway, Carbon County

At the request of Tim McGary, District Maintenance Engineer in Laramie, an investigation was conducted at approximate milepost 51.8 on the Battle Mountain Highway to determine the cause of frost heaving at this location. On April 17, 2000, WYDOT auger drilling rig H-821 was utilized to drill five shallow test holes at the site, four of which were cased with 2-inch, perforated PVC to monitor groundwater levels. A map indicating the approximate test hole locations and recommended underdrain improvements is included with this memorandum (see figure 1).

DISCUSSION

The problem area is located within a through cut where very dense granitic bedrock outcrops on both sides of the highway. Saratoga Maintenance Foreman Elvis Herring indicated that when this section of highway was originally constructed, very difficult excavation to grade was experienced in this cut and very dense granite bedrock is present within 3.0 feet of existing finished grade. Auger drilling at the site confirmed the presence of dense bedrock at a depth of 3.0 feet in GW-4. Additionally, there is an unlined irrigation ditch which runs parallel to and above the highway, and is located outside of the right-of-way north of the roadway. This ditch runs water for most, if not all, of the spring/summer irrigation season. Exacerbating the problem is another french drain located approximately half a mile upgrade from milepost 51.8 which adds water to the irrigation ditch. This drain was installed to lower groundwater in the vicinity of a naturally occurring spring located south of the highway at this location.

To remediate the frost heaving problem at milepost 51.8, a recommendation was given by the FHWA to install a french drain (placement of clean drain gravel) to lower groundwater, however this solution was found to be ineffective, as the heaving continued. In a second attempt to eliminate the problem, WYDOT Maintenance personnel installed a fabric wrapped, 4" perforated PVC underdrain system running diagonally from centerline to its outlet in the south ditch section. The approximate flowline depth of this drain is 3.0' below existing grade. This remedy was also ineffective as evidenced by the ongoing frost heaving present on both sides of the in-place PVC underdrain.

The cause of heaving at the site is the shallow groundwater beneath the surfacing section caused by the presence of a shallow, granite bedrock hump that is present within 3.0 feet of existing grade. Water migrating downhill from the irrigation ditch north of the highway and/or water flowing downgrade from the upper french drain is trapped in the vicinity of the impermeable bedrock and then backs up or ponds around this isolated bedrock hump.

RECOMMENDATIONS

The following recommendations are offered for implementation by Saratoga Maintenance personnel. To eliminate the frost heaving, it is imperative to lower groundwater within the affected area. Two perforated PVC underdrains, installed approximately 22 feet from existing centerline, running parallel to the roadway for a distance of 50 feet on both sides of the existing drain pipe (100' total length each side) are recommended to lower groundwater. The underdrains are to be installed utilizing guidelines outlined in Specification #605 of the Standard Specifications for Road and Bridge Construction (figure 2). To ensure that groundwater will be lowered sufficiently to eliminate additional heaving, flowline elevations of 6.0 feet below the existing finished grade are recommended. Because of the dense bedrock present as shallow as 3.0 feet, very difficult excavation is anticipated and some type of hard rock excavation method such as jackhammering will be required.

Six-inch perforated PVC is recommended for the two 100' lengths of proposed underdrain surrounding the existing drain pipe, and 6" non-perforated PVC will be required for outletting the water downgrade. The outlet drains may be daylighted in the ditch section downhill from the perforated section or into any available culverts located downgrade.

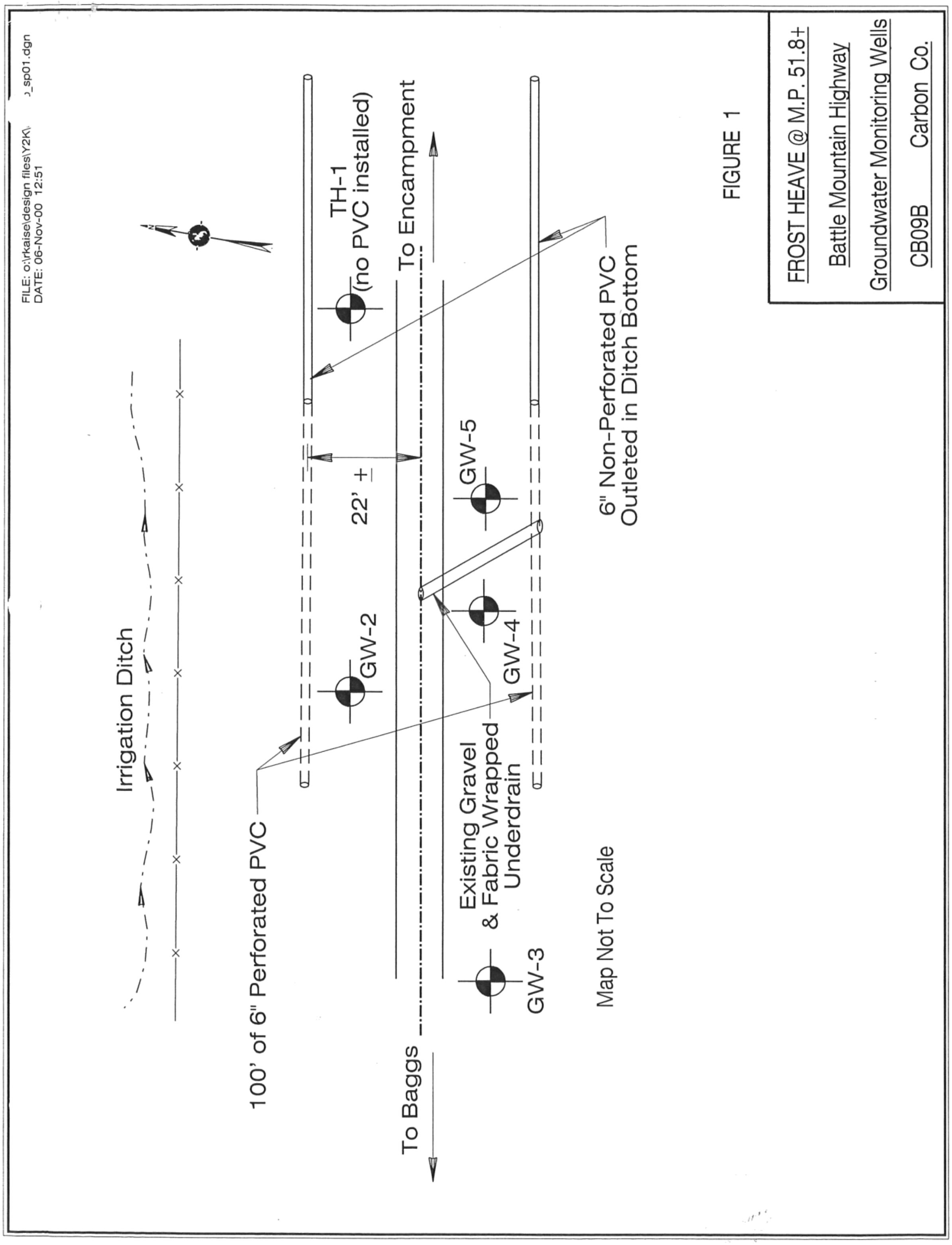
Estimated quantities for the underdrain installation are:

Excavation:	150 c.y.
Gravel for Drains:	75 c.y.
Drainage & Filtration Fabric:	400 sq. yds.
6" Perforated PVC:	320 feet
6" Non-perforated PVC:	240 feet

Reviewed By: Michael P. Schuster
Project Geologist

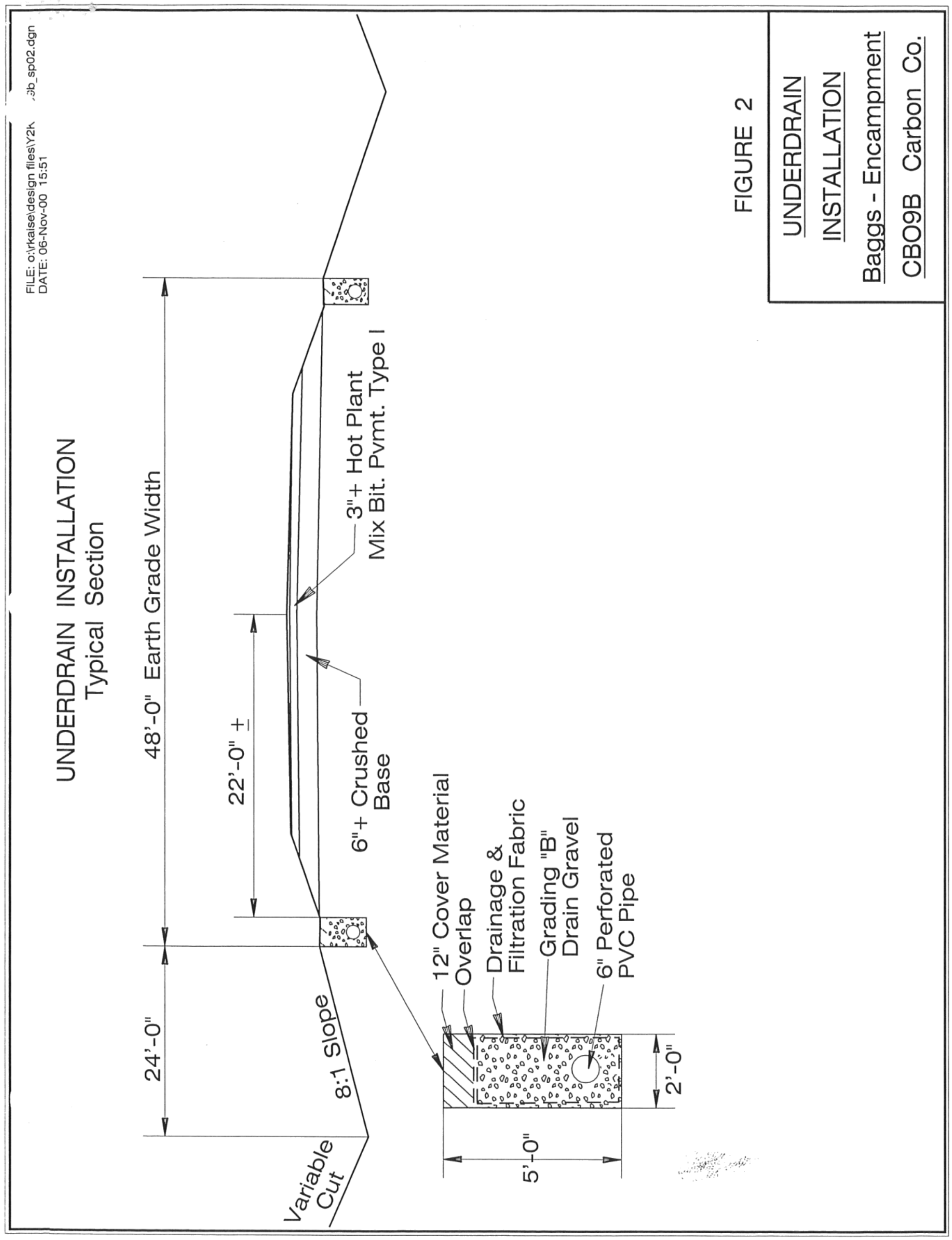
Approved By: S. Michael Hagen
Chief Engineering Geologist

cc: R. Tarango, P.E., Resident Engineer, Rawlins
T. McGary, P.E., District Maintenance Engineer, Laramie
Geology (3)



FILE: o:\rkaise\design files\Y2K\ _sp01.dgn
DATE: 06-Nov-00 12:51
Irrigation Ditch
TH-1
(no PVC installed)
To Encampment
22' ±
GW-5
GW-2
GW-4
GW-3
100' of 6" Perforated PVC
To Baggs
Existing Gravel
& Fabric Wrapped
Underdrain
Map Not To Scale
6" Non-Perforated PVC
Outleted in Ditch Bottom
FROST HEAVE @ M.P. 51.8±
Battle Mountain Highway
Groundwater Monitoring Wells
CB09B Carbon Co.

FIGURE 1


UNDERDRAIN INSTALLATION
Typical Section
48'-0" Earth Grade Width
24'-0"
22'-0" ±
8:1 Slope
Variable Cut
3"+ Hot Plant Mix Bit. Pvmt. Type I
6"+ Crushed Base
12" Cover Material
Overlap
Drainage & Filtration Fabric
Grading "B" Drain Gravel
6" Perforated PVC Pipe
5'-0"
2'-0"
FILE: o:\rkaise\design files\Y2k 3b_sp02.dgn
DATE: 06-Nov-00 15:51
FIGURE 2
UNDERDRAIN INSTALLATION
Baggs - Encampment
CBO9B Carbon Co.

GENERAL LOG FORM

PROJECT: CB09B
LOCATION: Hwy 70 MP 51.8

PR WL SP
SS SN SL Encampent Drainage

DRILLER: Crabtree
GEOLOGIST: GEM

Date: 4-17-00 Sta. ¢ Elev. T.H. Rig 821 Circ. Med. 4" GWS Date

T.H. No.	Depth	Sample Blows/300 mm or Core	Description: consistency, color, soil type, degree, saturation
1	0-6'		Fine-med, dry-sl moist, sand w/ tr. rock
	6'-7.5'	steady grind @ 6'	V. dense steady grind - granite bedrock?
			no PVC - totally dry to 7.5'
2		18' N. of ¢ + 25'	west of french drain
	0'-7'		Fine-med, dry, gravelly sand
	7'-9.5'	V. dense grind	Weath. bedrock? - gravelly sand/silt cuttings
			installed 10' 2" PVC w/ bottom 5.5' perfed

GENERAL LOG FORM

PROJECT: CB09 B
LOCATION: Hwy 70 MP 51.8

PR WL SP
SS SN SL Encamp. Drainage

DRILLER: Crabtree
GEOLOGIST: GEM

Date: 4-17-00 Sta. 100' w of drain 18' So of ¢ ¢ Elev. T.H. Rig 821 Circ. Med. GWS Date

T.H. No.	Depth	Sample Blows/300 mm or Core	Description: consistency, color, soil type, degree, saturation
3	0-9.5'		Compact, dry, fine-med, silty sand - dry to T.D.
			Installed 9' of 2" PVC
4			TH-4 is ~ 12' W. of French Drain + ~16.5' So. of ¢
	0'-3'		Compact-dense, gravelly sand
	3'-5.25		Bedrock - grey v. dense, mafic rock v. tough drilling
			4'10" 2" PVC installed
5			TH-5 is 25' east of drain + 17' So. of ¢
	0'-5		Compact-dense, sl moist, gravelly sand
	5-9.5'		Dry-sl moist, fine-med sand
			9'3" 2" PVC installed

APPENDIX B

SITE INVESTIGATION REPORT
JULY 7, 2010

File

Misc. Proj w/out #5

Department of Transportation

State of Wyoming

Dave Freudenthal
Governor

John F. Cox
Director

July 7, 2010

MEMORANDUM

PROFESSIONAL GEOLOGIST
TODD A. SULLIVAN
PG-2555
7/7/10
WYOMING

TO: Jay Gould, P.E., District Engineer, Laramie

FROM: Todd Sullivan, P.G., Engineering Geologist

SUBJECT: Heave Area, State Highway 70, M.P. 51.8

PROJECT: Agile Assets, Encampment South, Carbon County
Geology/Shared/Agile Assets/Encampment South

At the request of Tim McGary P.E., District Maintenance Engineer in Laramie, a drilling investigation was conducted at the above-referenced road section on April 22, 2010, using WYDOT auger drill H-823. The objective was to measure pavement and base thicknesses in the vicinity of a heave area, and characterize the subgrade and bedrock within the heave area and outside its limits. A total of five test holes were drilled, from which seven soil samples were collected and submitted to the Geology Laboratory for classifications and moisture contents. The investigation was conducted during a snow storm when several feet of snow was on the ground. While drilling at test hole 10-3, a 12" diameter CMP was breached at a 5 foot depth, through which a high flow of water was observed. Saratoga maintenance personnel were present for the breaching, and soon after began a corrective action plan to repair the pipe.

DISCUSSION AND RESULTS

The heave area has been problematic since the original construction of the roadway in the early 1980's. Maintenance has worked on this section several times over the years, and even installed a cross drain pipe in the mid 1990's as per Geology's recommendations. Rawlins field personnel recently ran a short profile for the heaved roadway section and provided arbitrary stationing to Geology for referencing of test holes. The investigation included test holes outside the limits, so stationing was adjusted to reflect positive numbers. The cross drain pipe was installed at approximate Station 0+88, which is presently beneath a dip in the roadway between two adjacent heaves. Test holes were drilled in the heave areas and outside of the heave areas for comparison.

All sampling and drilling results are summarized in the attached table 'Summary of Test Hole and Sample Data'. The surfacing consisted of a PMP mat ranging in thickness from 4.5 to 7.0 inches. The crushed base consisted of dry to slightly moist, silty sand and gravel ranging in thickness from 5.0 to 9.4 inches. Subgrade materials beneath and downgradient from the heave in test holes 10-1, 10-2, and 10-3, consisted of dry to slightly moist, silty sand. Subgrade materials upgradient (west) of the heave consisted of 1.4 to 4.3 feet of gravelly, fractured bedrock overlying very hard, unweathered quartzite bedrock. Ice or frost was not observed in cuttings from beneath the pavement.

Reviewed By: ______________________
Project Geologist

Approved By: ______________________
Chief Engineering Geologist

Attachments: Summary Table (1)
Test Hole Location Map (1)

cc: T. McGary, P.E., District Maintenance Engineer, Laramie
B. Jaure, Maintenance Area Supervisor, Rawlins
Geology (3)

SUMMARY OF TEST HOLE AND SAMPLE DATA

Encampment South
Highway 70 (M.P. 51.8)

TH	STATION/OFFSET	PMP (IN)	BASE (IN)	SAMPLE NO.	SAMPLE DEPTH (FT)	#40	-#200	LL	PI	w%	CLASSIFICATION	NOTES
10-1	2+03, 7.5' Lt.	4.5										Few transverse cracks
			5.0	1	0.4-0.8	28	14.6	NV	NP	3.8	SM, A-1-a	Crushed Base
				2	0.8-3.5	52	23.4	NV	NP	4.8	SM, A-2-4(0)	Dry Silty Sand
					3.5-7.5							Sand and Gravel
10-2	1+25, 7.5 Lt.	5.0										Few transverse & longitudinal cracks
			9.4	3	See notes	27	13.5	NV	NP	3.7	GM, A-1-a	Composite crushed base sample from TH10-2 at 0.4 to 1.2' and TH10-5 at 0.5-1.2'
				4	1.2-9.5	66	41.9	NV	NP	12.7	SM, A-4(0)	Silty Sand, more moisture with depth
					9.5-10.5							Quartzite bedrock, very hard, auger refusal
10-3	1+25, 6.0' Rt.	6.5										High spot milled down, longitudinal crack in EBL
			7.9	5	See notes	27	15.6	NV	NP	4.4	GM, A-1-b	Composite crushed base sample from TH10-3 at 0.5' to 1.2' and TH10-4 at 0.6' to 1.2'
				6	See notes	44	27.7	NV	NP	7.0	SM, A-2-4(0)	Composite subgrade sample from TH10-3 at 1.2 to 7.0' and TH10-4 at 1.2 to 3.1'
10-4	0+67, 6.0' Rt.	7.0										Transverse cracks every 5', heaved area
			7.4	5	See notes	27	15.6	NV	NP	4.4	GM, A-1-b	Composite crushed base sample from TH10-3 at 0.5' to 1.2' and TH10-4 at 0.6' to 1.2'
				6	See notes	44	27.7	NV	NP	7.0	SM, A-2-4(0)	Composite subgrade sample from TH10-3 at 1.2 to 7.0' and TH10-4 at 1.2 to 3.1'
					3.1-4.5							Gravelly fractured bedrock
					4.5-5.0+							Quartzite bedrock, very hard, auger refusal
10-5	0+00, 6.0' Rt.	6.5										Mat in good condition
			7.9	3	See notes	27	13.5	NV	NP	3.7	GM, A-1-a	Composite crushed base sample from TH10-2 at 0.4 to 1.2' and TH10-5 at 0.5-1.2'
				7	1.2-5.5	47	30.0	NV	NP	1.9	SM, A-2-4(0)	Gravelly fractured bedrock
					5.5-6.0+							Quartzite bedrock, very hard, auger refusal

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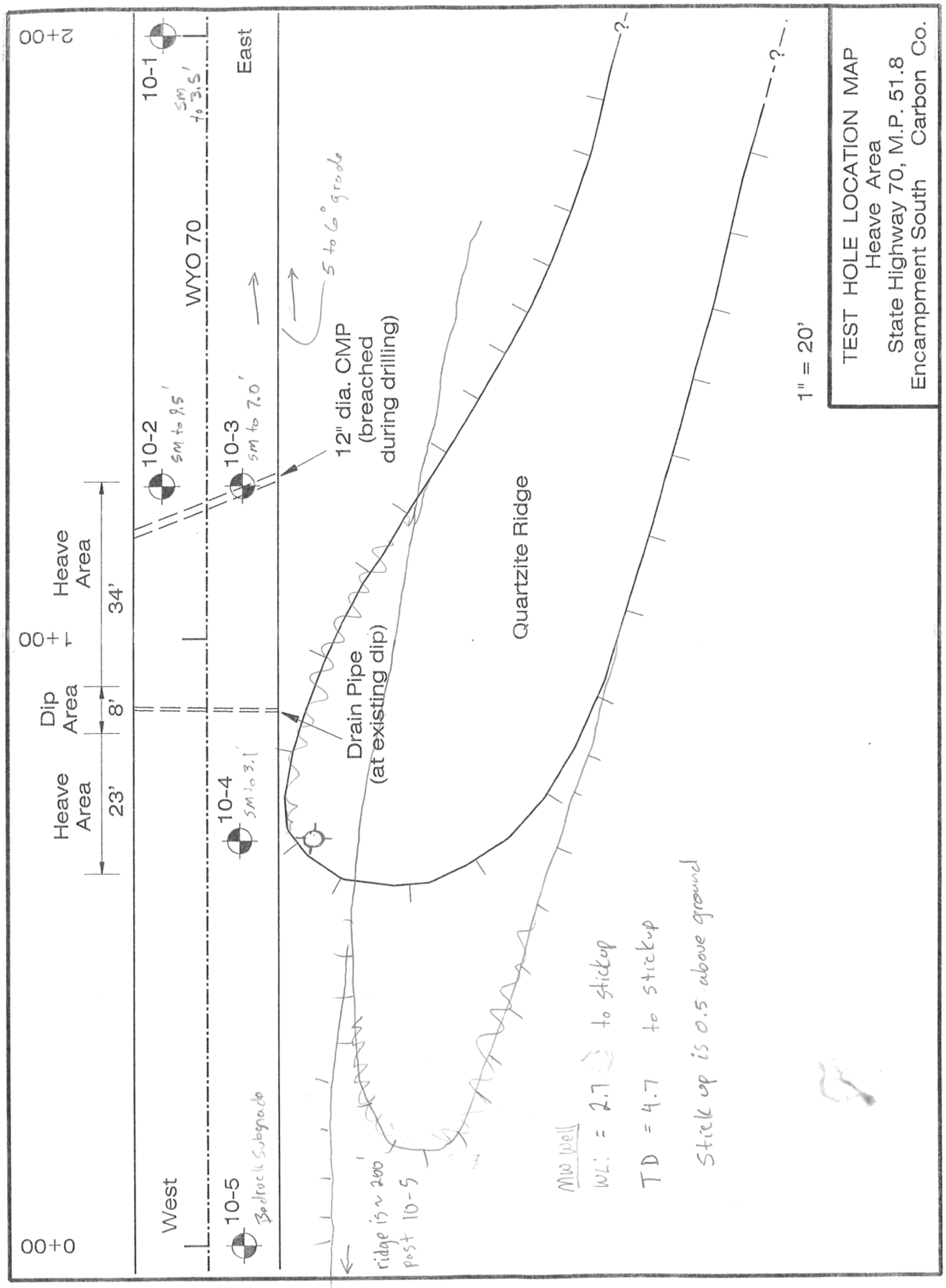

GENERAL LOG FORM

PROJECT: | PR WL SP | DRILLER: Vanderbloom
LOCATION: Encampment South MP. 51.8 | SS SN SL | GEOLOGIST: Sullivan

Date: 4/22/10 Sta. ~~1+80~~ 2+03 7.5' Lt. ℄ Elev. T.H. Rig 823 Circ. Med. GWS Date

T.H. No.	Depth	Sample Blows/Ft. or Core	Control Hole Description: consistency, color, soil type, degree, saturation
10-1	0.0-4.5"		PMP - few transv. cracks - good condit. - alligator cracks WBL Shoulder
	4.5"-9.5"	SM A-1-a Silty Sand w/ Gravel Sample # 1	Crushed Base, sl. Clayey Sand & gravel (~~no~~ crushed) dry
	9.5"-3.5'	0.8-3.5 Sample # 2	Sand, med. grained, clean, dry SM A-2-4(0) Silty Sand
	3.5'-7.5'	1+25	Sand & Gravel, dry, looks like colluvium
		~~1+02~~ 7.5' Lt. ℄	
10-2	0.0-5.0"		PMP - widely-spaced longitudinal & transv. cracks - not bad in WBL
	5.0"-1.2'	Comp # 3 GM A-1-a Silty Gravel w/ Sand	Crushed Base - sl. clayey sand and gravel - coarse gravel sub round
			Sl. moist
	1.2-9.5'	Sample #4 SM A-4(0) Silty Sand	Clayey sand & gravel - sl. moist, more moisture w/ depth
	9.5-10.5		Quartzite - v. hard, insitu? auger refusal 10.5'

10-3 35' down sta. from where french drain is

GENERAL LOG FORM

PROJECT: | PR WL SP | DRILLER: Vanderbloom
LOCATION: Encampment South MP. 51.8 | SS SN SL | GEOLOGIST: Sullivan

Date: 4/22/10 Sta. ~~1+02~~ 1+25 6' Rt. ℄ Elev. T.H. Rig 823 Circ. Med. GWS Date

T.H. No.	Depth	Sample Blows/Ft. or Core	Description: consistency, color, soil type, degree, saturation
10-3	0.0-6.5"		PMP. High spot, milled down - longitud. crak down middle EBL
			& some short arcuate cracks
	6.5"-1.2'	Comp 5 GM A-1-b Silty Gravel w/ Sand	Crushed Base
	1.2'-7.0	Comp 6 SM A-2-4(0) Silty Sand w/ Gravel	Clayey Sand & gravel, sl. moist, larger gravels, sl. moist
			At. 7.0' mangled CMP Material came up in cuttings - Scott
			Kinniburg remembered the cross drains as plastic - aborted hole
			- after pulling augers there is good flow 5' down in hole,
			dug out snow bank in North ROW ditch & did
			confirm we went through a 12" dia. CMP.

10-4 is 25' ~~up sta~~ from french drain

GENERAL LOG FORM

PROJECT: PR WL SP DRILLER: Vandenbloom
LOCATION: Encampment South SS SN SL GEOLOGIST: Sullivan

Date: 4/22/10 Sta. ~~0+44~~ 0 + 67 6' Rt 35' ℄ Elev. T.H. Rig 823 Circ. Med. GWS Date

T.H. No.	Depth	Sample Blows/Ft. or Core	Description: consistency, color, soil type, degree, saturation
10-4	0.0-7"		PMP - transv. cracks ~ 5' spacing (heave area)
	7" - 1.2'	comp 5 GM A-1-b	Base Crushed - sl. moist - sl. Clayey Sand & Gravel
	1.2 - 3.1'	Comp 6 SM A-2-4(0)	Clayey Sand w/ Gravel
	3.1 - 4.5		V. gravelly - fractured bedrock
	4.5+	0 + 00	auger refusal in quartzite bedrock
		Sta. ~~0-23~~ 6' Rt	
10-5	0.0 - 6.5"	/	PMP - v. good shape, no cracks
	6.5" - 1.2'	comp 3 GM A-1-a	Base Crushed,
	1.2 - 5.5	S# 7 SM A-2-4(0) silty sand	V. gravelly - fractured bedrock
	5.5+		Quartzite bedrock, v. hard

Form T-195

Rev. - January 2009

WYDOT
GEOLOGY FOUNDATIONS LAB
TRANSMITTAL FORM

G:/GEOLOGY/Shared/Projects/Agile Assets

Project (ERP) Number [agile assets] sub folder Encampment S, Location Encampment South

Function (i.e PE2, OTH etc.)______ Type (i.e. ST, PR, SL etc.) SP Is this a Bridge that needs to be tested using LRFD procedures? No

Bedrock Information - Age: Formation: Type: Quartzite

Lab Number	Sample Info. Geol. No.	Sample Info. Type	T.H. and Station	Depth of Sample	U.C.	Direct* Shear quick or residual	Triaxial* U.U. C.U. C.D.	Swell* ASTM A,B or C, AASHTO	Consol	Remarks (Please indicate soil / rock type for each sample.)
100364	1	Bag	10-1, 1+80	0.4 - 0.8'						Crushed Base
100365	2		10-1, 1+80	0.8 - 1.2	Bag	Depth =	0.8'-3.5'			Sand .5-1.2
100366	3		10-2, 1+02	0.4 - 1.2						Crushed Base - Composite w/ 10-5
100367	4		10-2, 1+02	1.2 - 8.5'						.6-1.2'
100368	5		10-3, 1+02	0.6 - 1.2						Crushed Base - Composite w/ 10-4
100369	6		10-3, 1+02	1.2 - 7.0'						Clayey S&G - Composite w/ 10-4 1.2-3.5
100370	7		10-5, 0+23	1.2 - 5.5'						
			23 downstation from 0+00							10-4 is @ Sta 0+44

*Please indicate which test procedure needs to be used

Date Received in Lab: 4/22/10
Submitted by: T. Sullivan
Reviewed by: MS

Comments: Need Moisture Contents & Classifications on all Samples,

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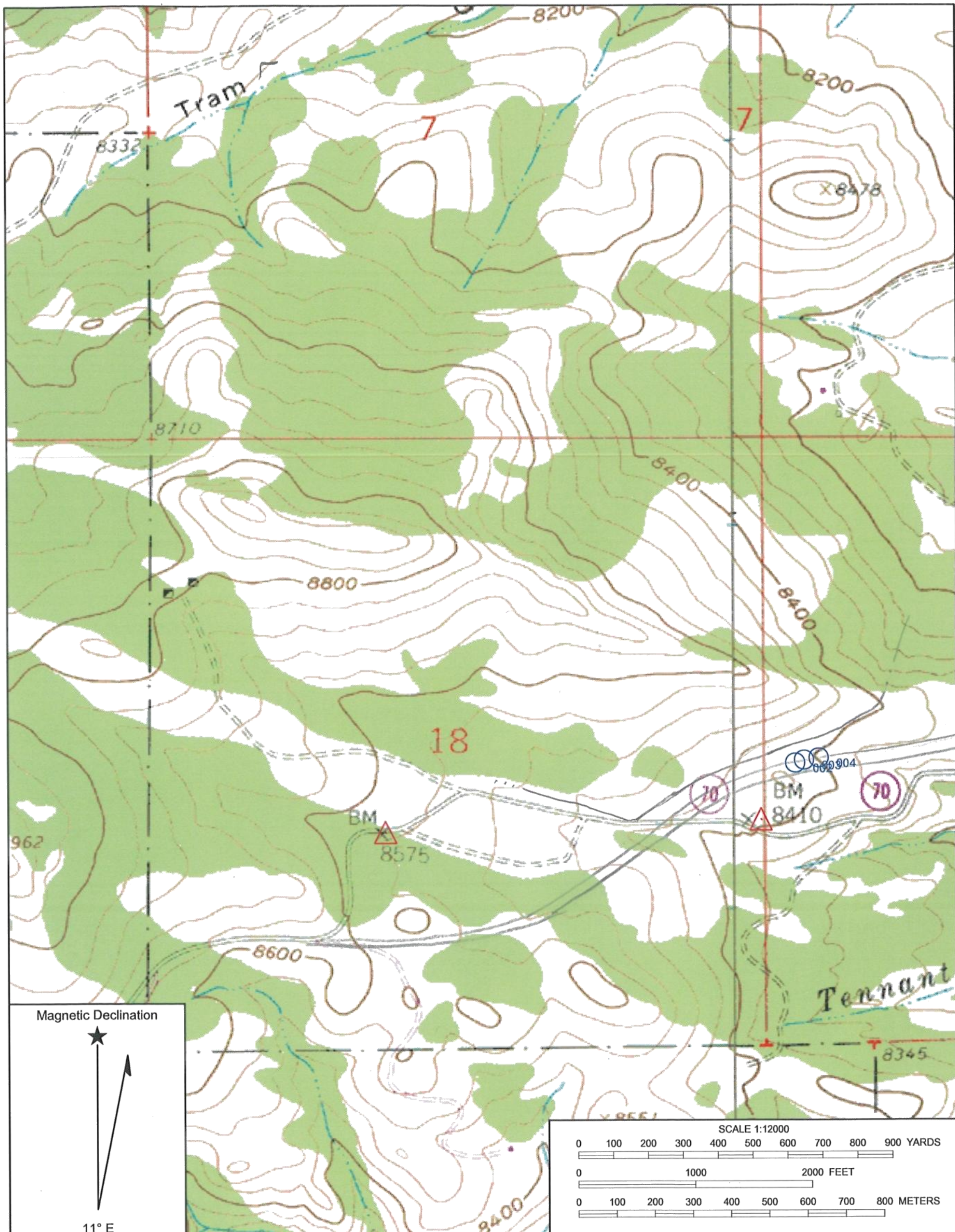
Geology Foundations Laboratory Test Data Summary

PROJECT: Encampment South *Project number:* Encampment South, Agile Assets

Lab No.	TH No.	Geo. No.	Depth	USCS group name	USCS	AASHTO	%<1"	%<3/4"	%<3/8"	%<#4	%<#10	%<#40	%<#200	L.L.	P.L.	P.I.	M%	L.I.	Testing Remarks
	TH 10-1,1+80																		
100364		1	0.4-0.8	silty sand with gravel	SM	A-1-a	100.0	92.3	75.5	61.6	47.9	28.0	14.6	NV	NP	NP	3.78		
100365		2	0.8-1.2	silty sand	SM	A-2-4(0)	100.0	100.0	97.7	94.0	79.6	51.6	23.4	NV	NP	NP	4.76		
	TH 10-2, 1+02																		
100366		3	0.4-1.2	silty gravel with sand	GM	A-1-a	100.0	88.6	65.3	54.2	43.6	26.5	13.5	NV	NP	NP	3.68		
100367		4	1.2-9.5	silty sand	SM	A-4(0)	100.0	100.0	93.8	86.9	78.8	65.9	41.9	NV	NP	NP	12.70		
	TH 10-3, 1+02																		
100368		5	0.6-1.2	silty gravel with sand	GM	A-1-b	100.0	92.3	67.7	55.5	43.5	27.3	15.6	NV	NP	NP	4.44		
100369		6	1.2-7.0	silty sand with gravel	SM	A-2-4(0)	100.0	91.0	74.3	65.2	56.2	43.8	27.7	NV	NP	NP	7.03		
	TH 10-5, 0-23																		
100370		7	1.2-5.5	silty sand	SM	A-2-4(0)	100.0	100.0	95.6	87.7	71.1	47.4	30.0	NV	NP	NP	1.93		

Report Date: 5/6/2010



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Tennant
8345
8400
Magnetic Declination
11° E
SCALE 1:12000
0 100 200 300 400 500 600 700 800 900 YARDS
0 1000 2000 FEET
0 100 200 300 400 500 600 700 800 METERS

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APPENDIX C

DRILLING LOGS
NOVEMBER 11, 2011

GENERAL LOG FORM

PROJECT: WY70 FROST HEAVE THERM, | PR WL SP SS SN SL | DRILLER:
LOCATION: | | GEOLOGIST:

Date: 11/3/11 Sta. WBL +200 ℄ Elev. T.H. Rig 821 Circ. Med. GWS Date

T.H. No.	Depth	Sample Blows/Ft. or Core	Description: consistency, color, soil type, degree, saturation
11-1	0-5.5		SILTY SAND W/GRAVEL, OLIVE BROWN, SL. MOIST
	5.5-8.0	SL. M 7	" , SOME ANGULAR ROCK CHUNKS
11-2	0-2.5	S. SHOULDER MOST.	SILTY SAND W/GRAVEL, BROWN, SL. MOIST → WET
	2.5-4.0	+200	GRAY GRANITE BR
11-3	0-3.5	S. SHOULDER POST	SILTY SAND W/ GRAVEL, BROWN, SL. MOIST → WET
	3.5+	+200	GRAY GRANITE BR.
11-4	0-4.5	S. SHOULDER B/M	SILTY SAND W/ GRAVEL, BROWN, SL MOIST → WET
	4.5+	+175	GRAY GRANITE BR

GENERAL LOG FORM

PROJECT: WY 70 FROST HEAVE THERM. | PR WL SP SS SN SL | DRILLER:
LOCATION: | | GEOLOGIST:

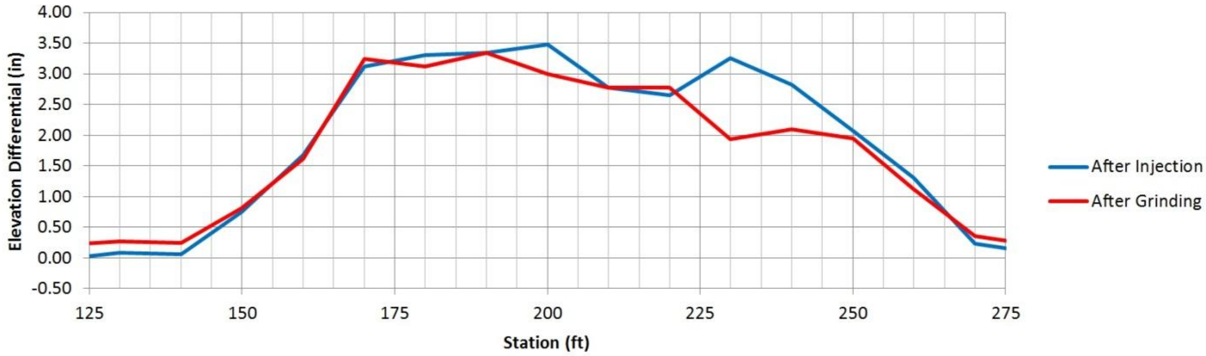
Date: 11/3/11 Sta. EBL +218 ℄ Elev. T.H. Rig Circ. Med. GWS Date

T.H. No.	Depth	Sample Blows/Ft. or Core	Description: consistency, color, soil type, degree, saturation
11-5	0-5.5		SILTY SAND W/GRAVEL, BROWN, SL. MOIST
	5.5+		GRAY GRANITE BR
11-6	0-7.5	EBL +198	SILTY SAND W/ GRAVEL, BROWN, SL. MOIST
	7.5+		GRAY GRANITE BR
11-7	0-6.5	EBL +179	SILTY SAND W/GRAVEL, BROWN, SL. MOIST, FOAM 0-1.0
	6.5+		GRAY GRANITE BR
11-8	0-2.5	EBL + 129	SILT, GRAY, DRY, WEATHERED BR?
	2.5+		GRAY GRANITE BR

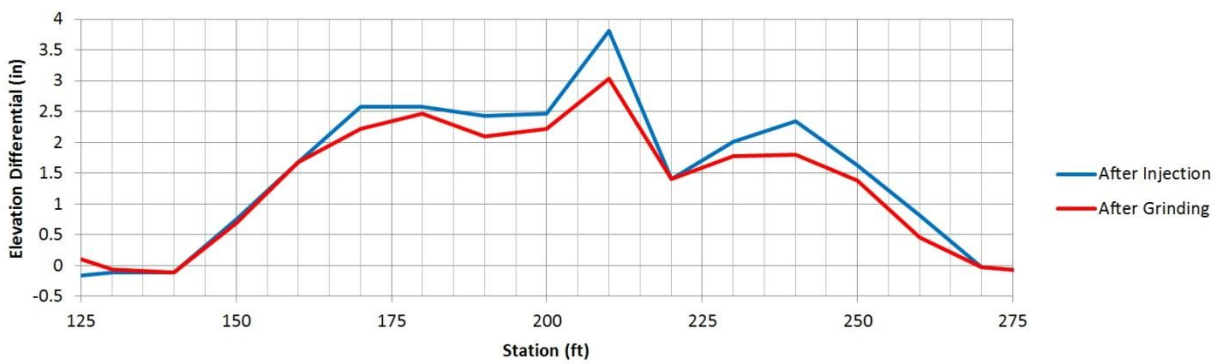
APPENDIX D

INJECTION DEPTH AND MILLED SURFACE AT END OF CONSTRUCTION

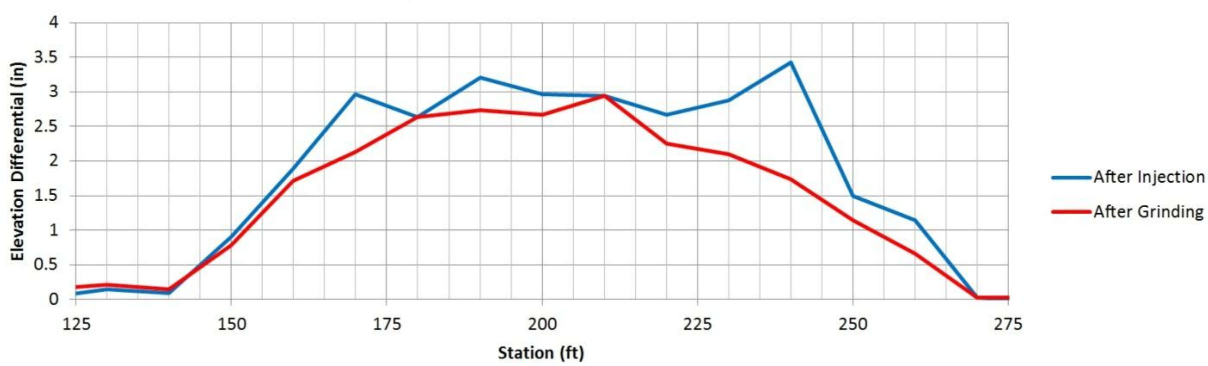
Elevation Differential from the Original Baseline for South Edge of East Bound Lane Caused by Injection and Grinding

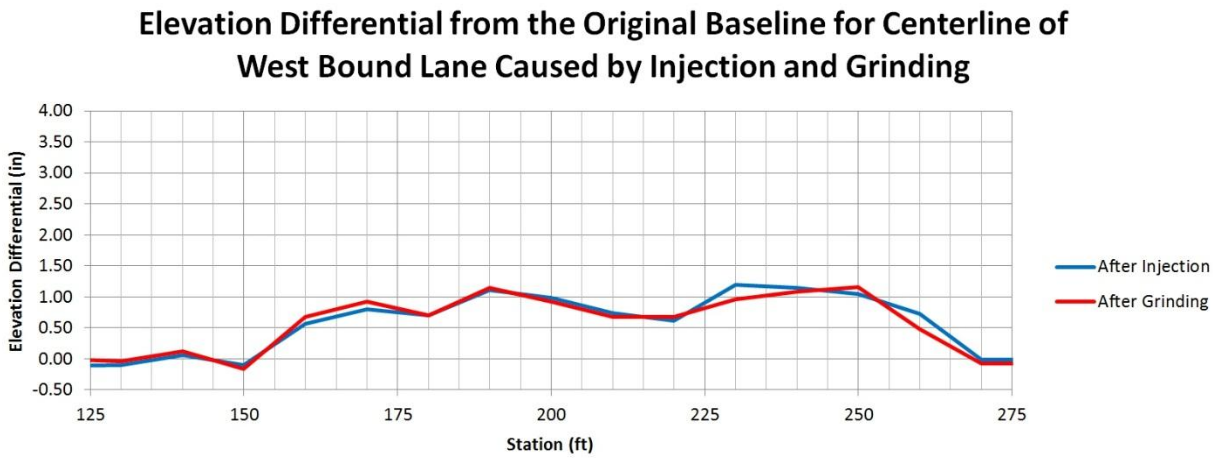


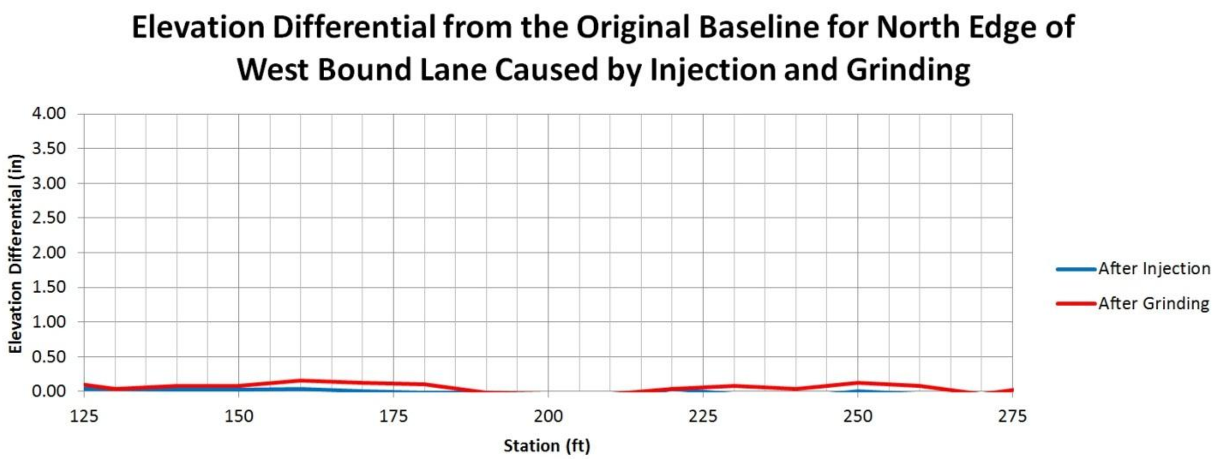
Elevation Differential from the Original Baseline for Centerline of East Bound Lane Caused by Injection and Grinding



Elevation Differential from the Original Baseline Centerline of Highway Caused by Injection and Grinding




Elevation Differential from the Original Baseline for Centerline of West Bound Lane Caused by Injection and Grinding
Elevation Differential (in)
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
125
150
175
200
225
250
275
Station (ft)
After Injection
After Grinding


Elevation Differential from the Original Baseline for North Edge of West Bound Lane Caused by Injection and Grinding
Elevation Differential (in)
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
125
150
175
200
225
250
275
Station (ft)
After Injection
After Grinding

APPENDIX E

ELEVATION DIFFERENCES ACROSS ROAD PROFILES

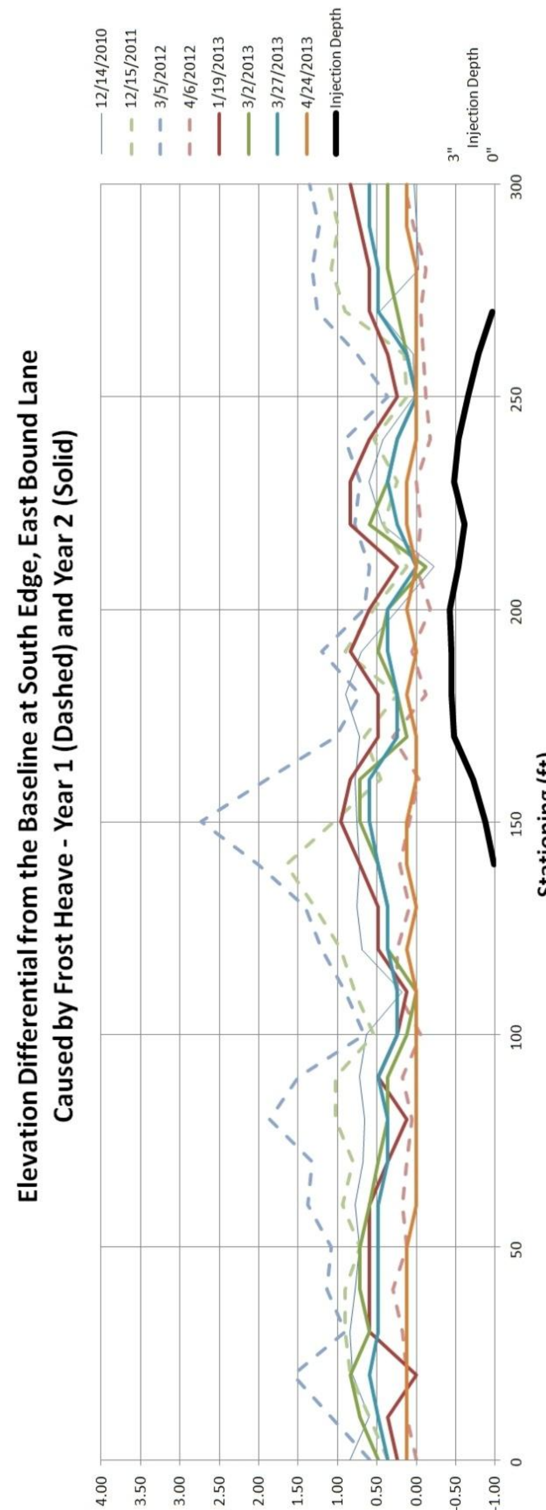


Figure A. 1 South Edge of East Bound Lane

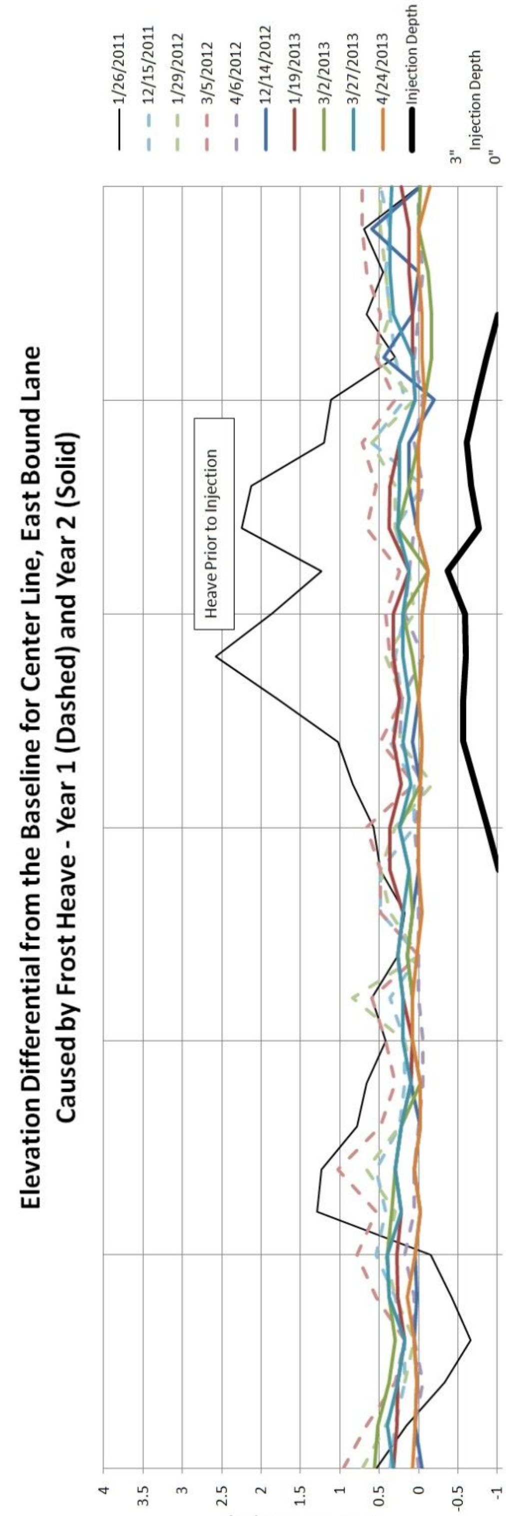


Figure A. 2 Centerline of East Bound Lane

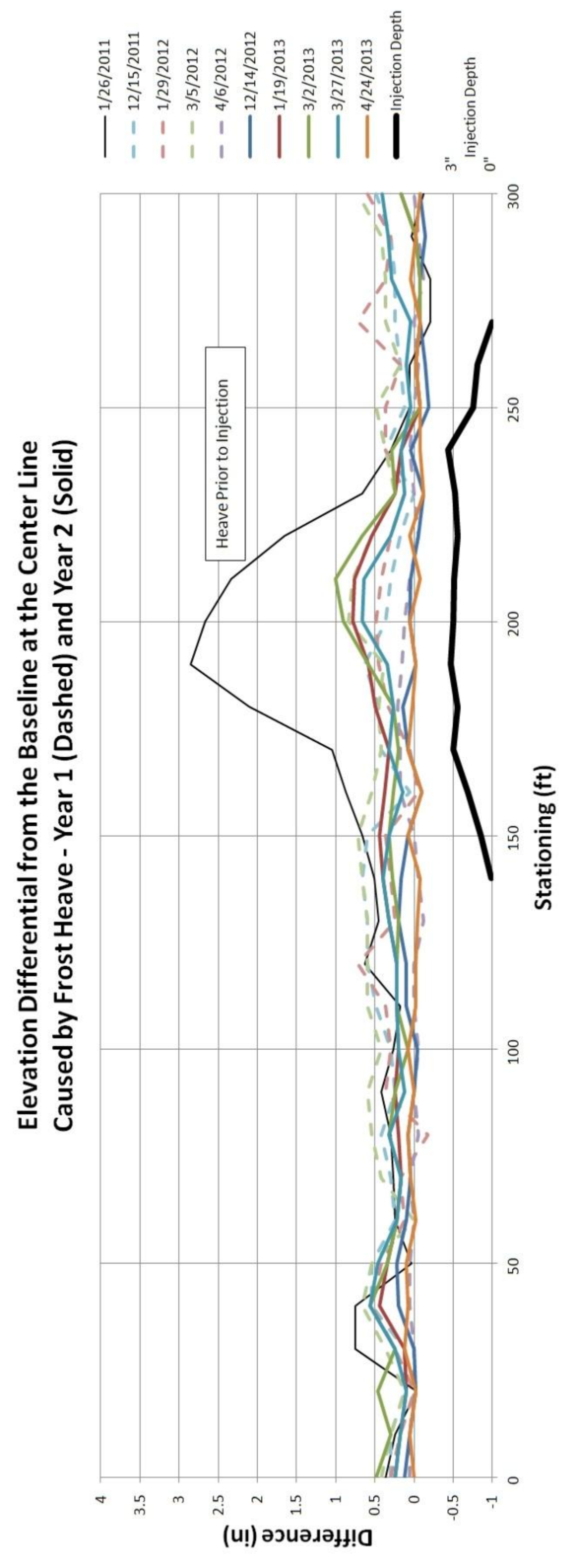


Figure A. 3 Centerline of Highway

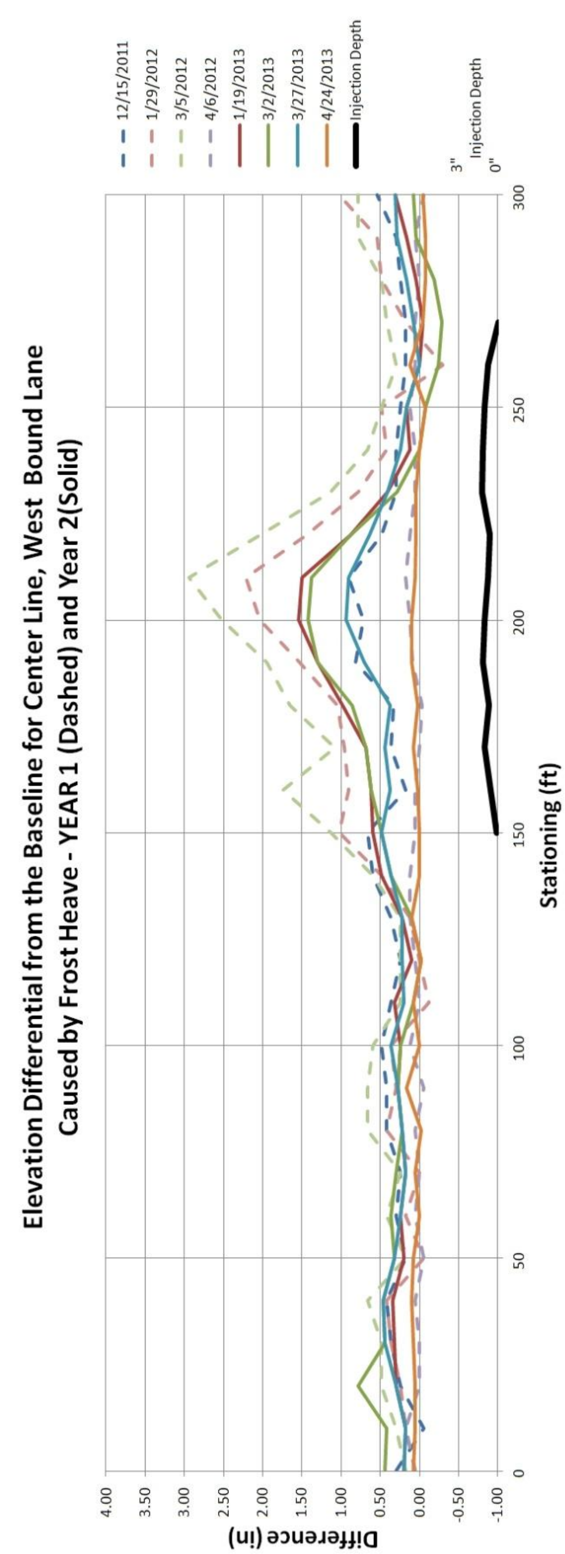


Figure A. 4 Centerline of West Bound Lane

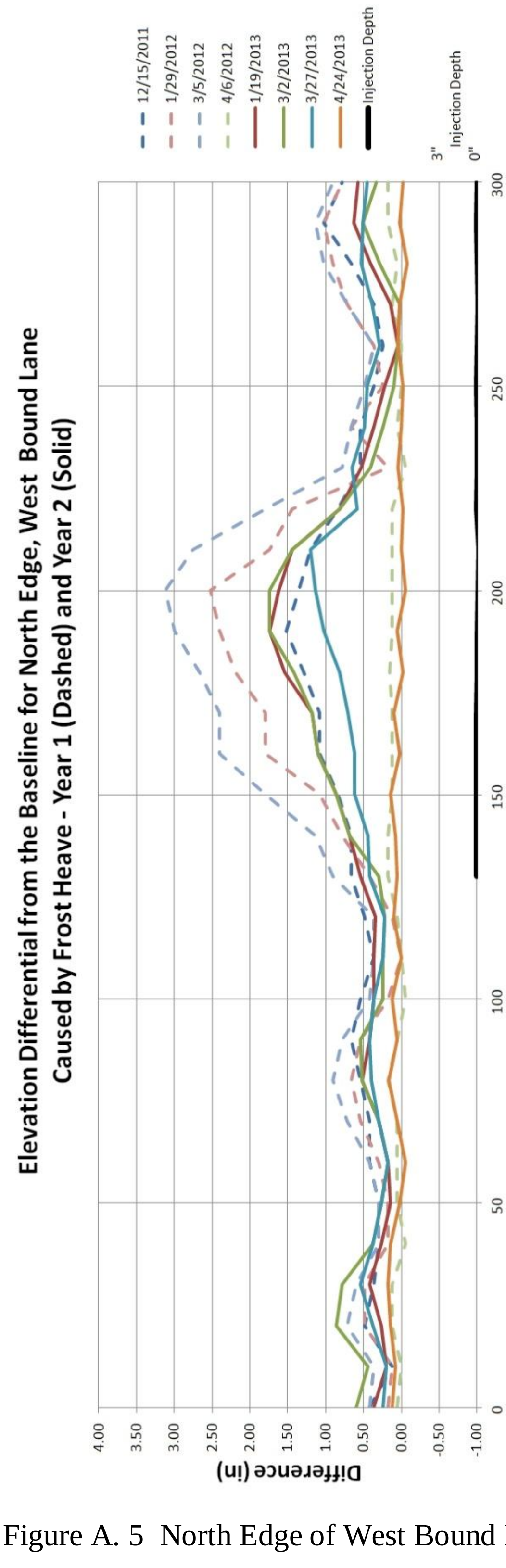
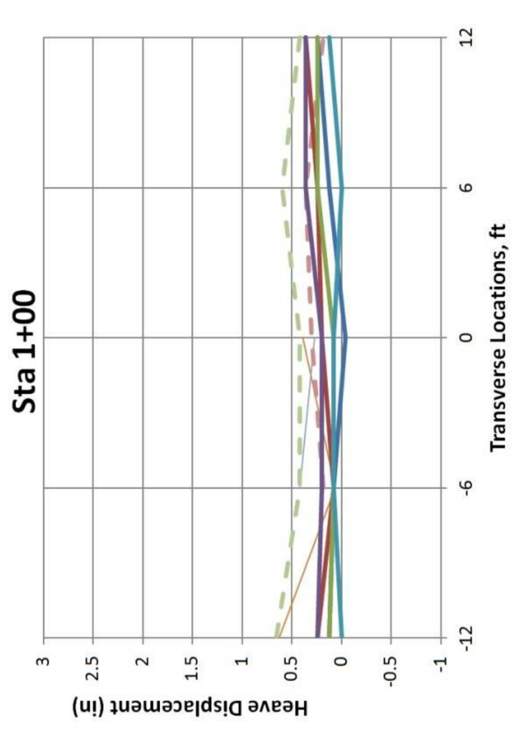
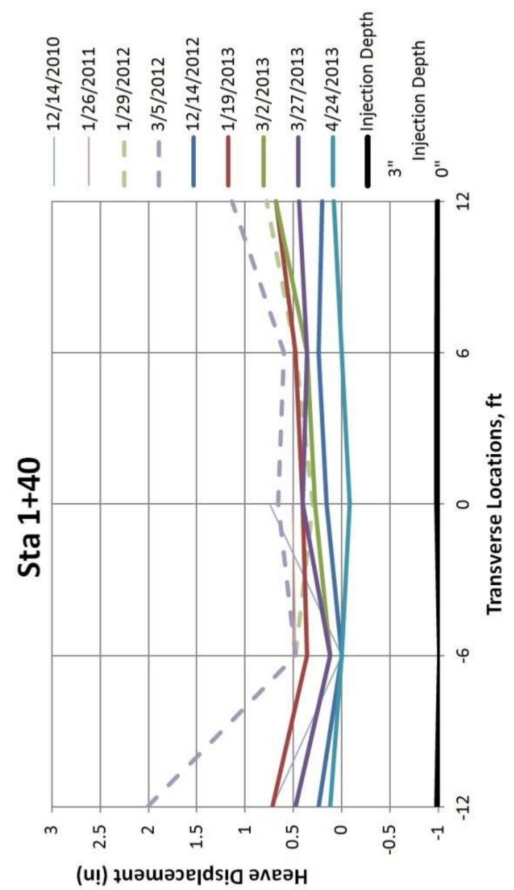


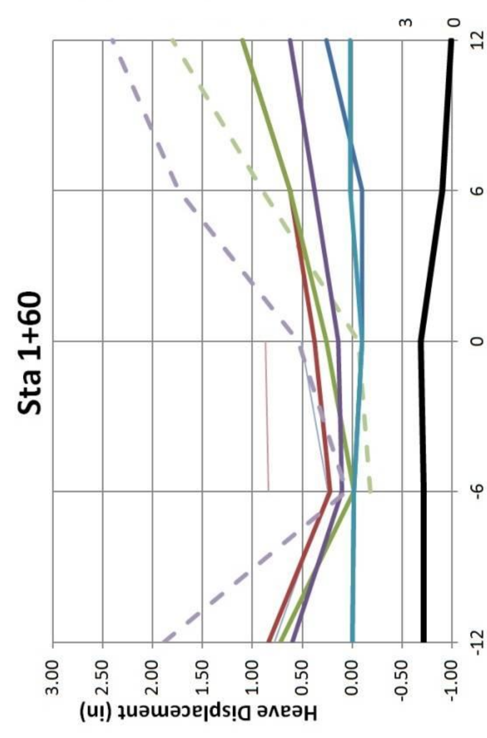
Figure A. 5 North Edge of West Bound Lane

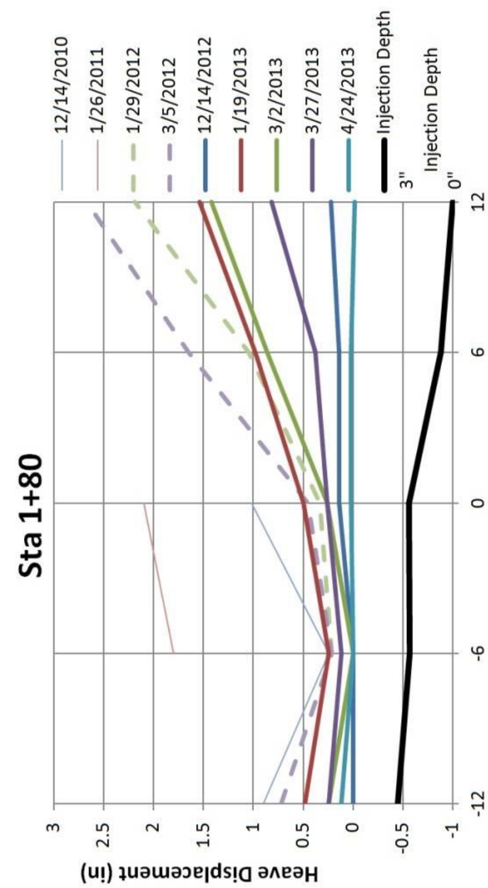
APPENDIX F

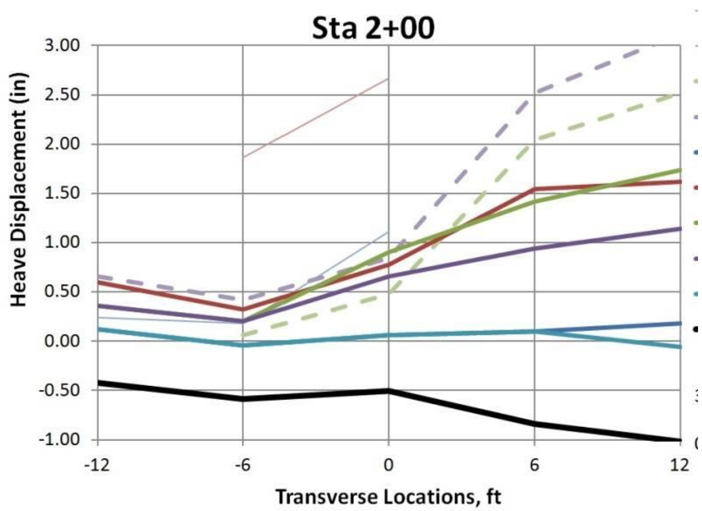
ELEVATION DIFFERENCES IN TRANSVERSE DIRECTIONS

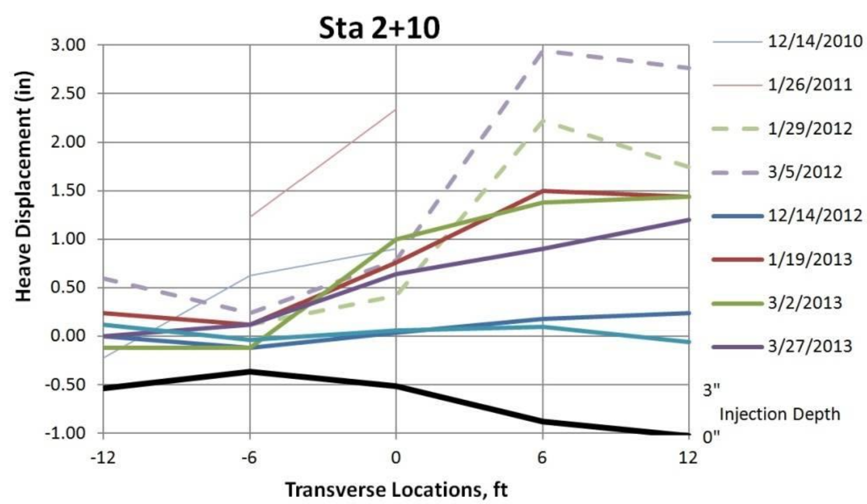

Sta 1+00
Heave Displacement (in)
Transverse Locations, ft

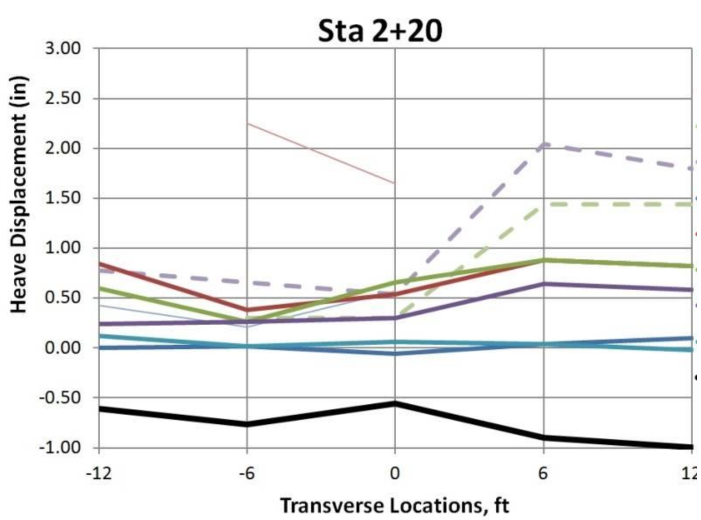

Sta 1+40
Heave Displacement (in)
Transverse Locations, ft
12/14/2010
1/26/2011
1/29/2012
3/5/2012
12/14/2012
1/19/2013
3/2/2013
3/27/2013
4/24/2013
Injection Depth 3"
Injection Depth 0"

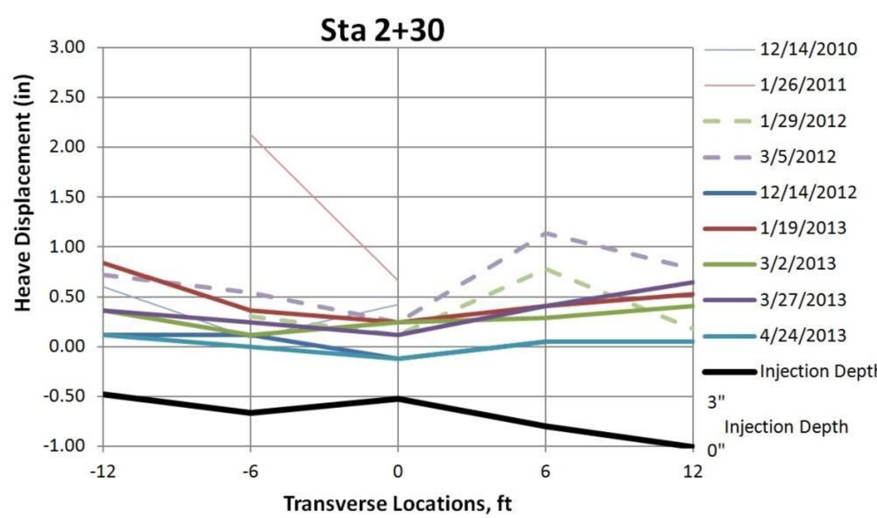

Sta 1+60
Heave Displacement (in)
Transverse Locations, ft

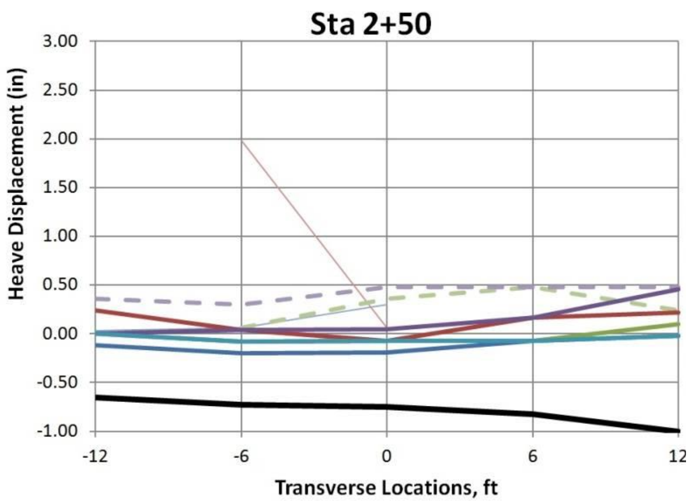

Sta 1+80
Heave Displacement (in)
Transverse Locations, ft
12/14/2010
1/26/2011
1/29/2012
3/5/2012
12/14/2012
1/19/2013
3/2/2013
3/27/2013
4/24/2013
Injection Depth 3"
Injection Depth 0"

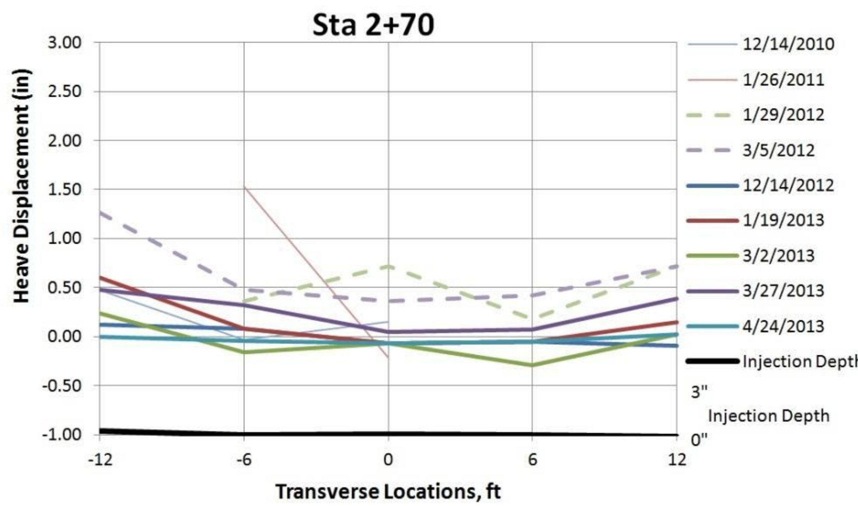

Sta 2+00
Heave Displacement (in)
3.00
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
-1.00
-12
-6
0
6
12
Transverse Locations, ft


Sta 2+10
Heave Displacement (in)
3.00
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
-1.00
-12
-6
0
6
12
Transverse Locations, ft
12/14/2010
1/26/2011
1/29/2012
3/5/2012
12/14/2012
1/19/2013
3/2/2013
3/27/2013
3"
Injection Depth
0"


Sta 2+20
Heave Displacement (in)
3.00
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
-1.00
-12
-6
0
6
Transverse Locations, ft

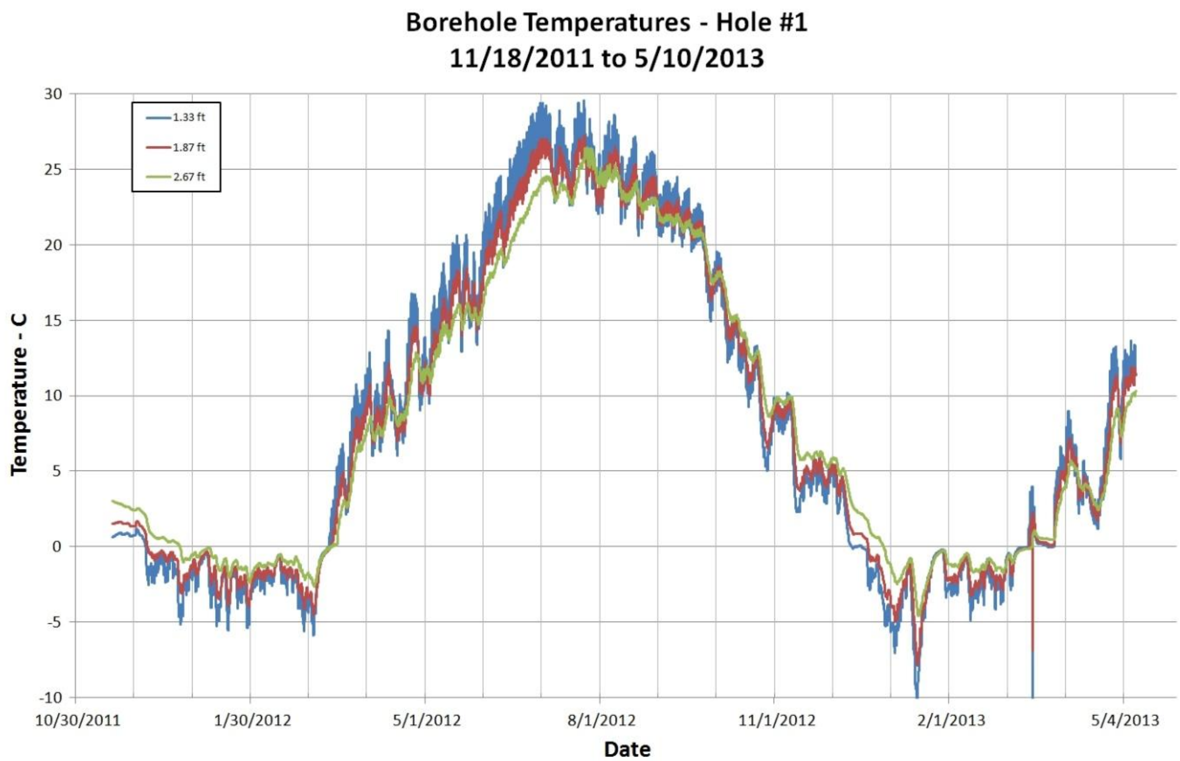

Sta 2+30
Heave Displacement (in)
3.00
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
-1.00
-12
-6
0
6
12
Transverse Locations, ft
12/14/2010
1/26/2011
1/29/2012
3/5/2012
12/14/2012
1/19/2013
3/2/2013
3/27/2013
4/24/2013
Injection Depth
3"
Injection Depth
0"

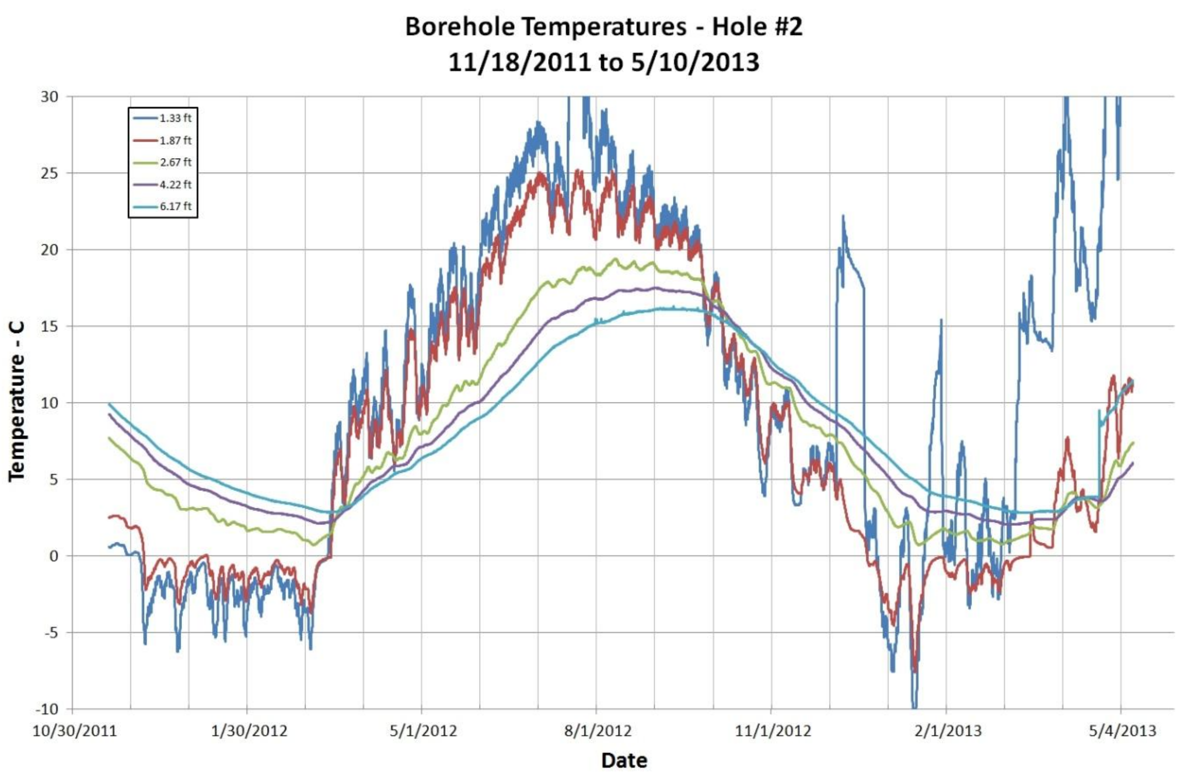

Sta 2+50
Heave Displacement (in)
3.00
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
-1.00
-12
-6
0
6
12
Transverse Locations, ft

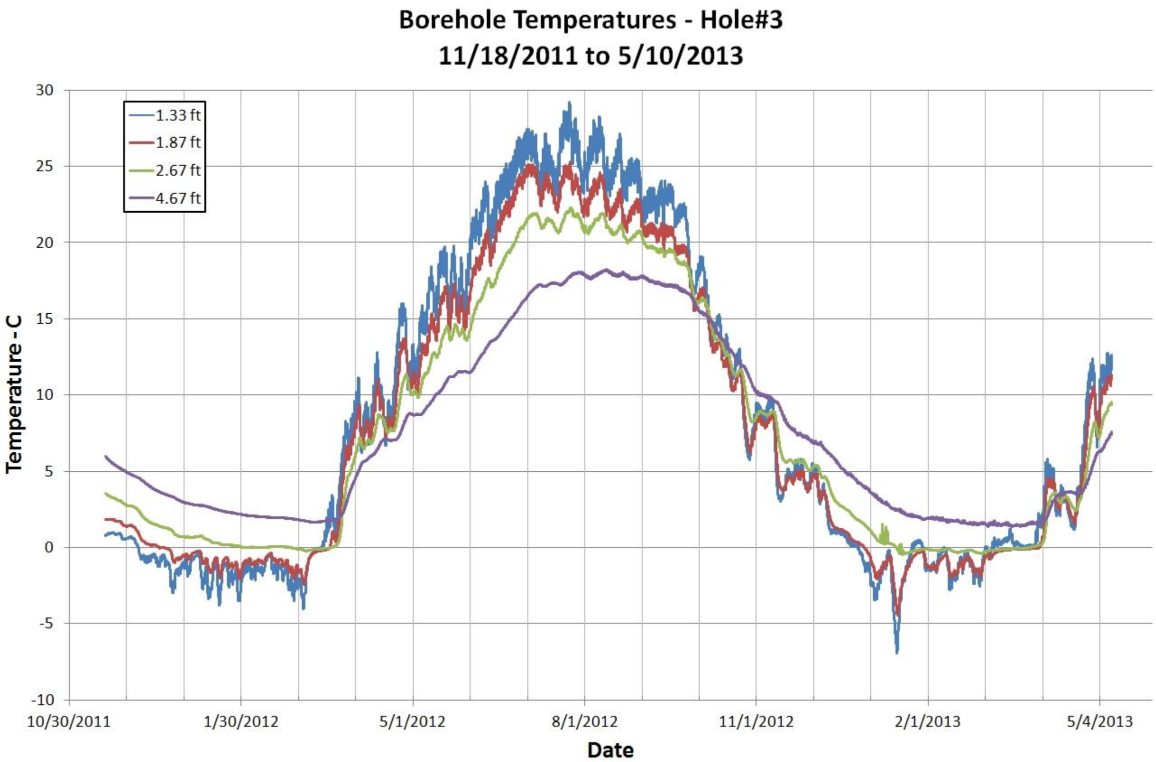

Sta 2+70
Heave Displacement (in)
3.00
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
-1.00
-12
-6
0
6
12
Transverse Locations, ft
12/14/2010
1/26/2011
1/29/2012
3/5/2012
12/14/2012
1/19/2013
3/2/2013
3/27/2013
4/24/2013
Injection Depth
3"
Injection Depth
0"

APPENDIX G

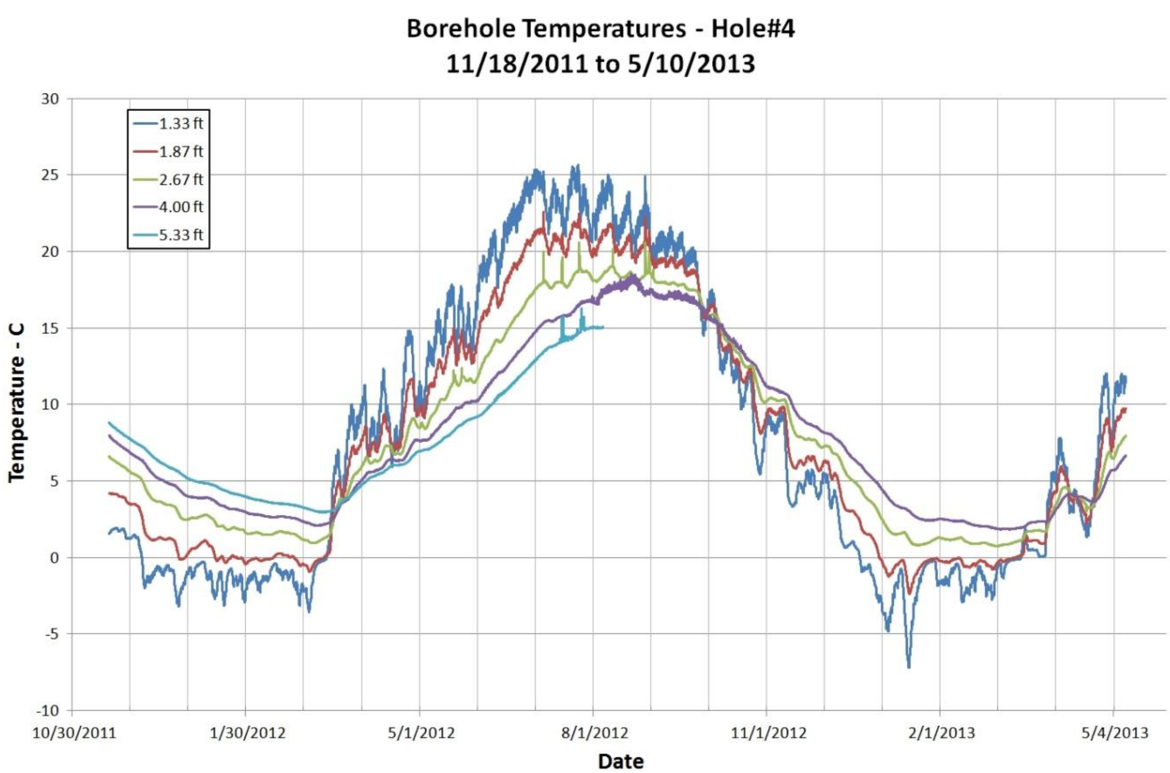
TEMPERATURE VARIATIONS AT HOLES #1 - #6
OVER THE PROJECT DURATION


Borehole Temperatures - Hole #1
11/18/2011 to 5/10/2013
1.33 ft
1.87 ft
2.67 ft
Temperature - C
Date
30
25
20
15
10
5
0
-5
-10
10/30/2011
1/30/2012
5/1/2012
8/1/2012
11/1/2012
2/1/2013
5/4/2013

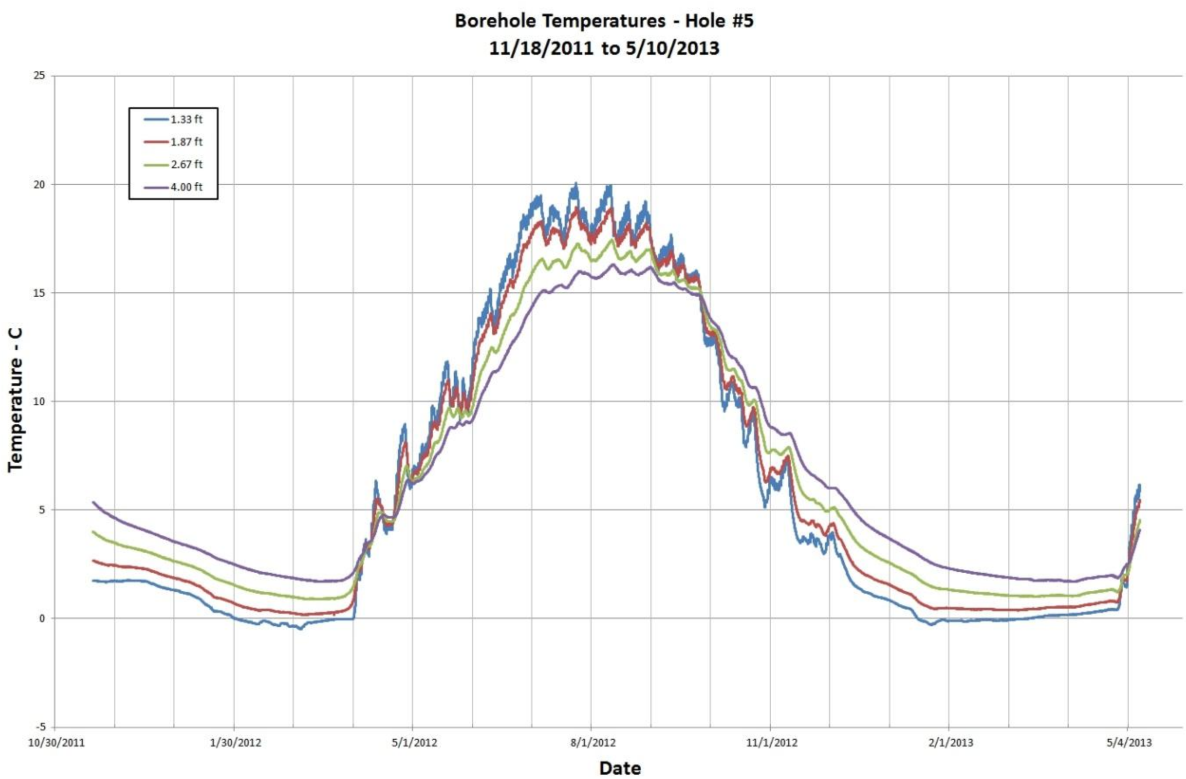

Borehole Temperatures - Hole #2
11/18/2011 to 5/10/2013
1.33 ft
1.87 ft
2.67 ft
4.22 ft
6.17 ft
Temperature - C
Date
30
25
20
15
10
5
0
-5
-10
10/30/2011
1/30/2012
5/1/2012
8/1/2012
11/1/2012
2/1/2013
5/4/2013

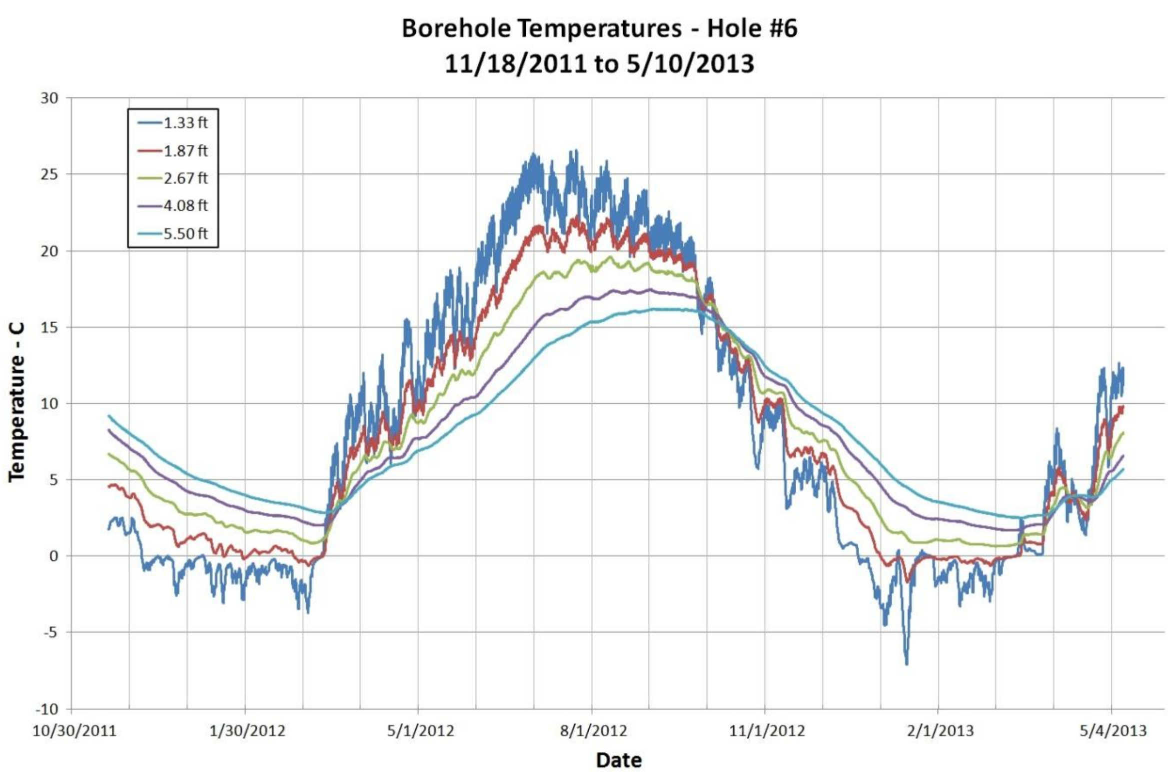


Borehole Temperatures - Hole#3
11/18/2011 to 5/10/2013
1.33 ft
1.87 ft
2.67 ft
4.67 ft
Temperature - C
30
25
20
15
10
5
0
-5
-10
10/30/2011
1/30/2012
5/1/2012
8/1/2012
11/1/2012
2/1/2013
5/4/2013
Date



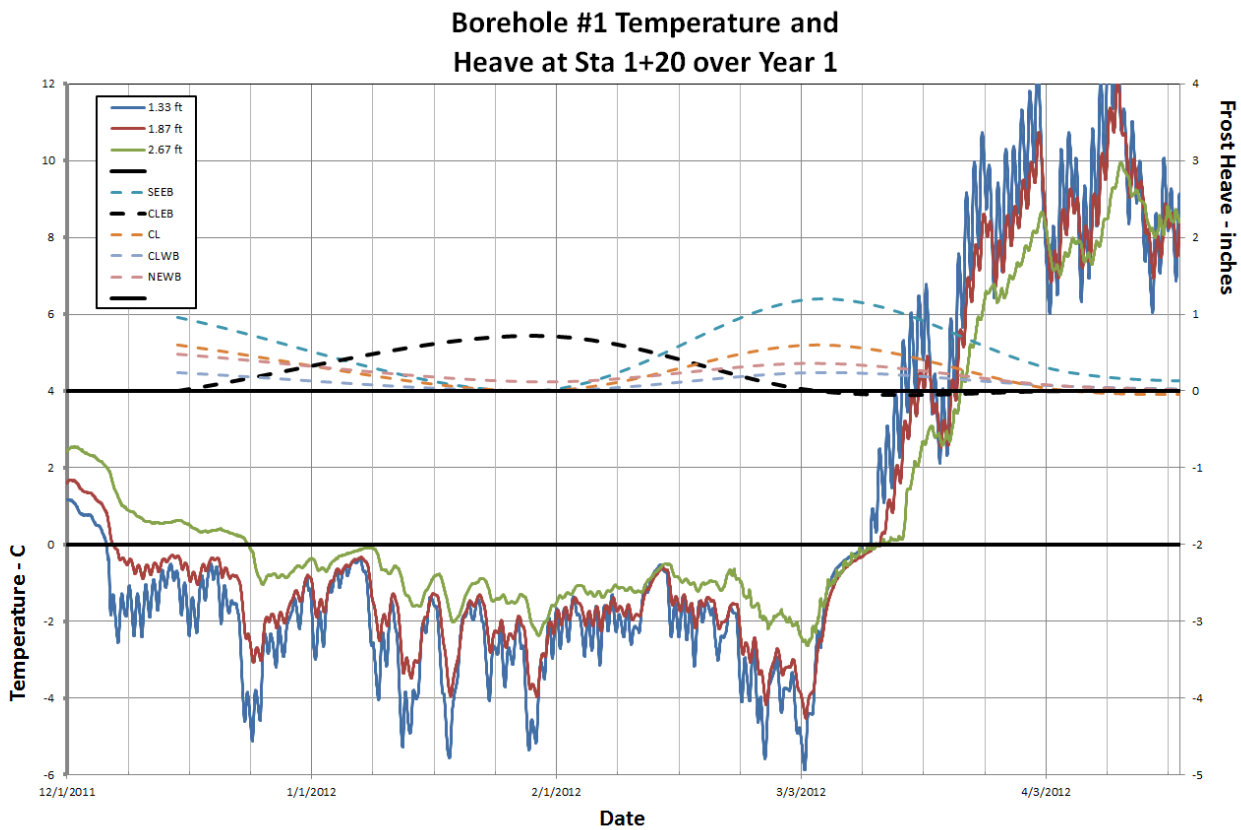
Borehole Temperatures - Hole#4
11/18/2011 to 5/10/2013
1.33 ft
1.87 ft
2.67 ft
4.00 ft
5.33 ft
Temperature - C
30
25
20
15
10
5
0
-5
-10
10/30/2011
1/30/2012
5/1/2012
8/1/2012
11/1/2012
2/1/2013
5/4/2013
Date

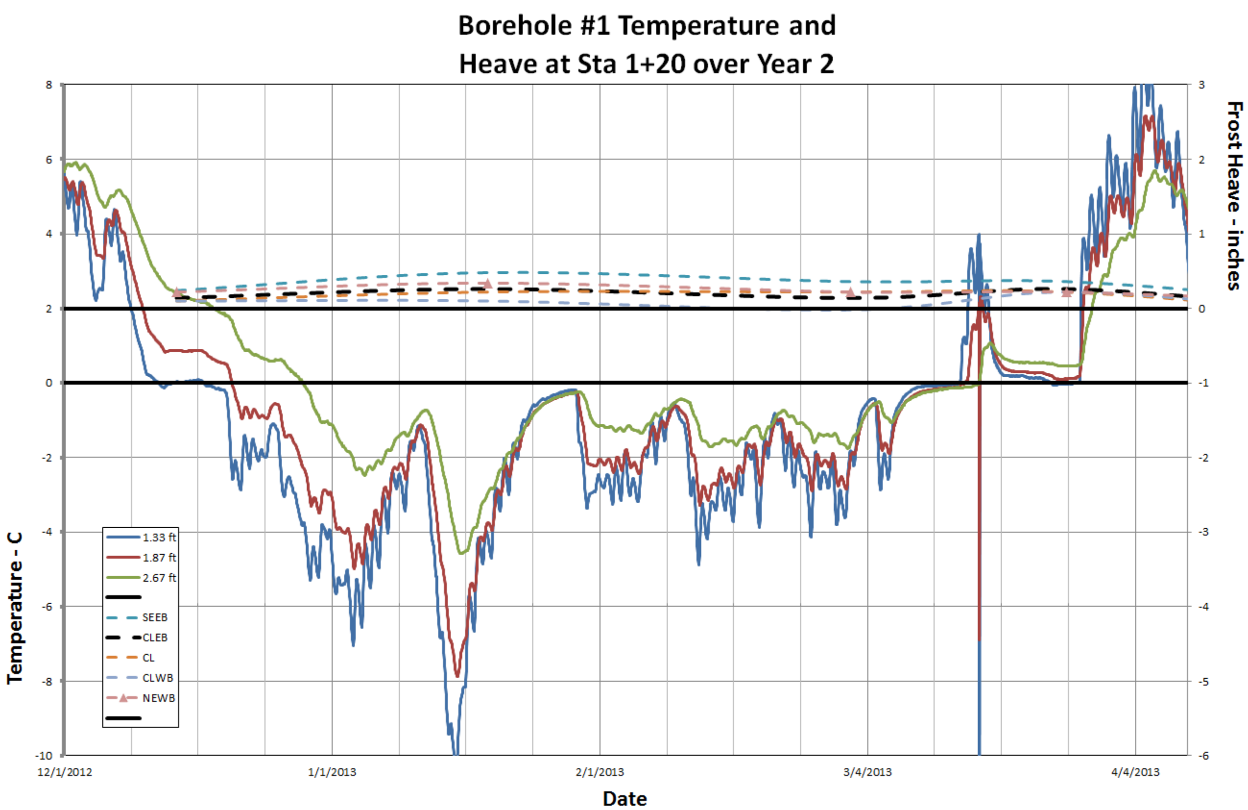

Borehole Temperatures - Hole #5
11/18/2011 to 5/10/2013
1.33 ft
1.87 ft
2.67 ft
4.00 ft
Temperature - C
Date
25
20
15
10
5
0
-5
10/30/2011
1/30/2012
5/1/2012
8/1/2012
11/1/2012
2/1/2013
5/4/2013

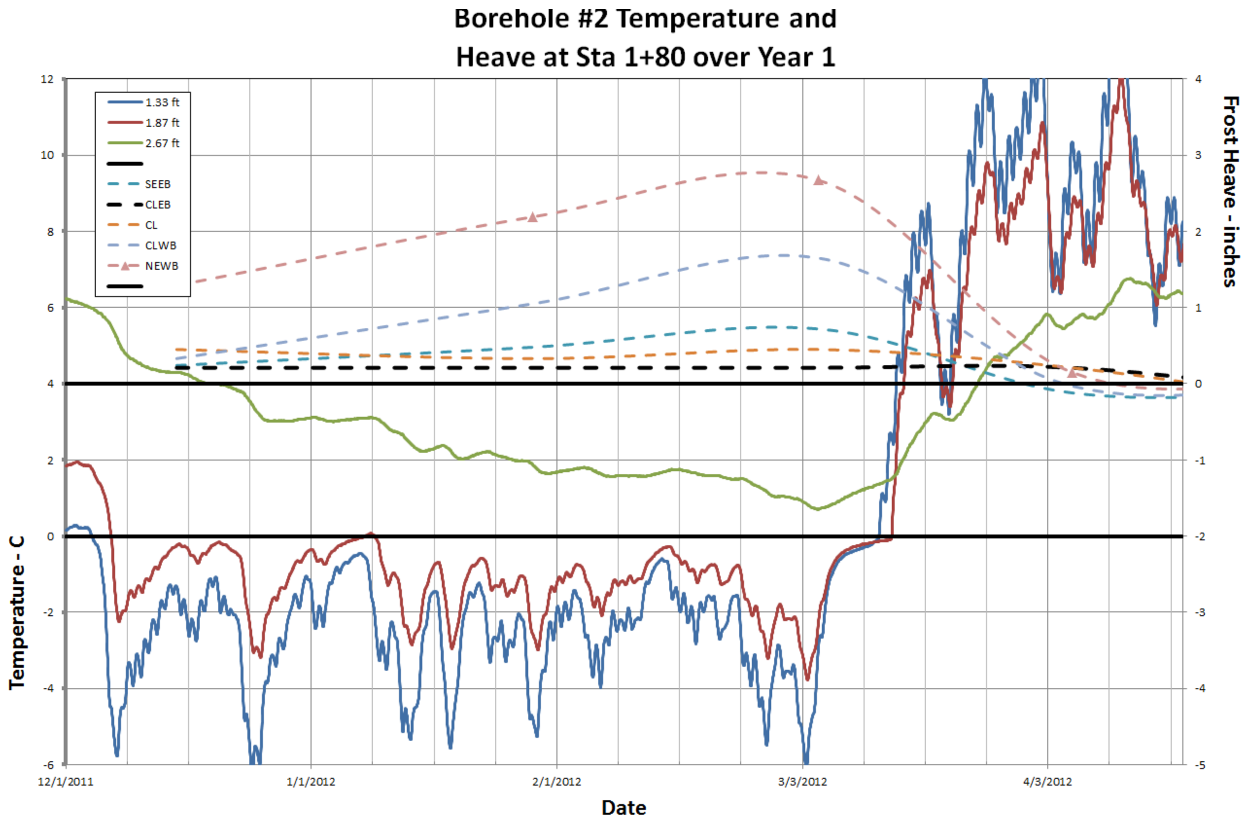

Borehole Temperatures - Hole #6
11/18/2011 to 5/10/2013
1.33 ft
1.87 ft
2.67 ft
4.08 ft
5.50 ft
Temperature - C
Date
30
25
20
15
10
5
0
-5
-10
10/30/2011
1/30/2012
5/1/2012
8/1/2012
11/1/2012
2/1/2013
5/4/2013

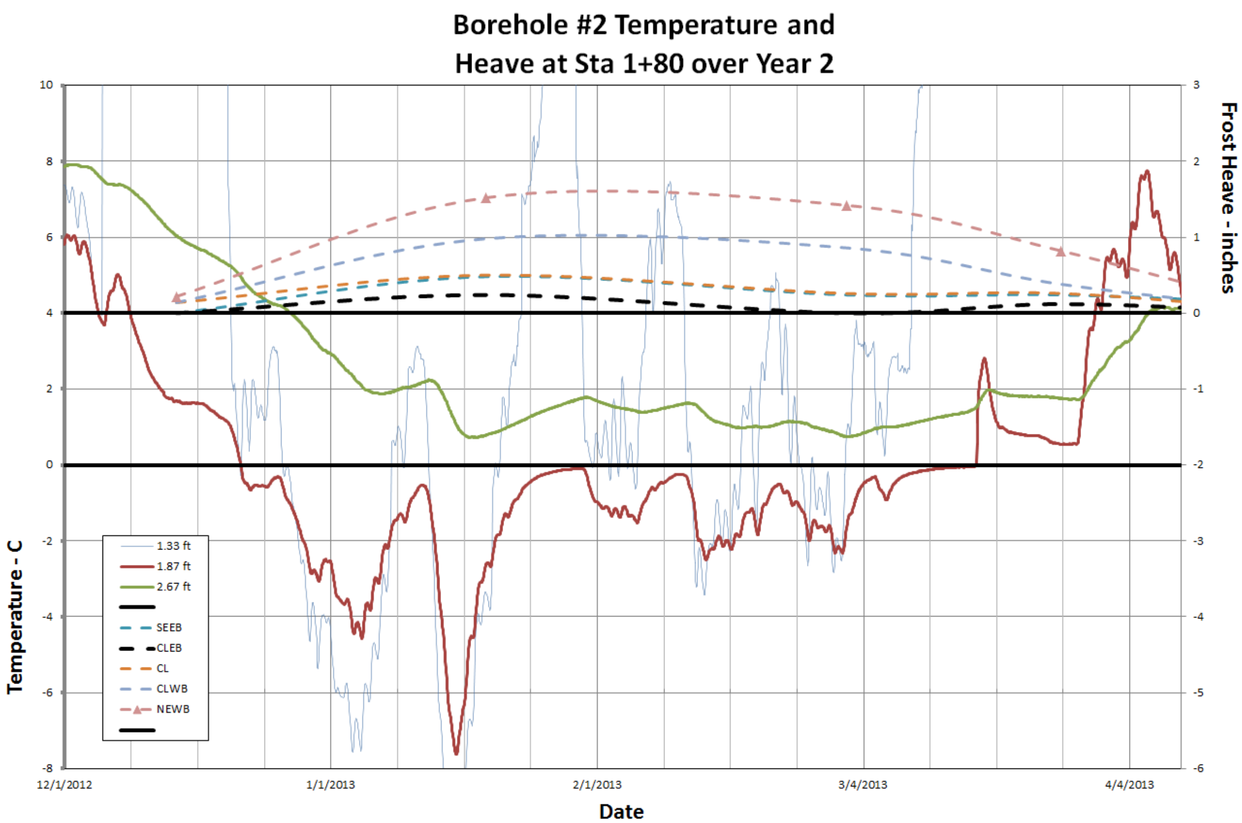
APPENDIX H

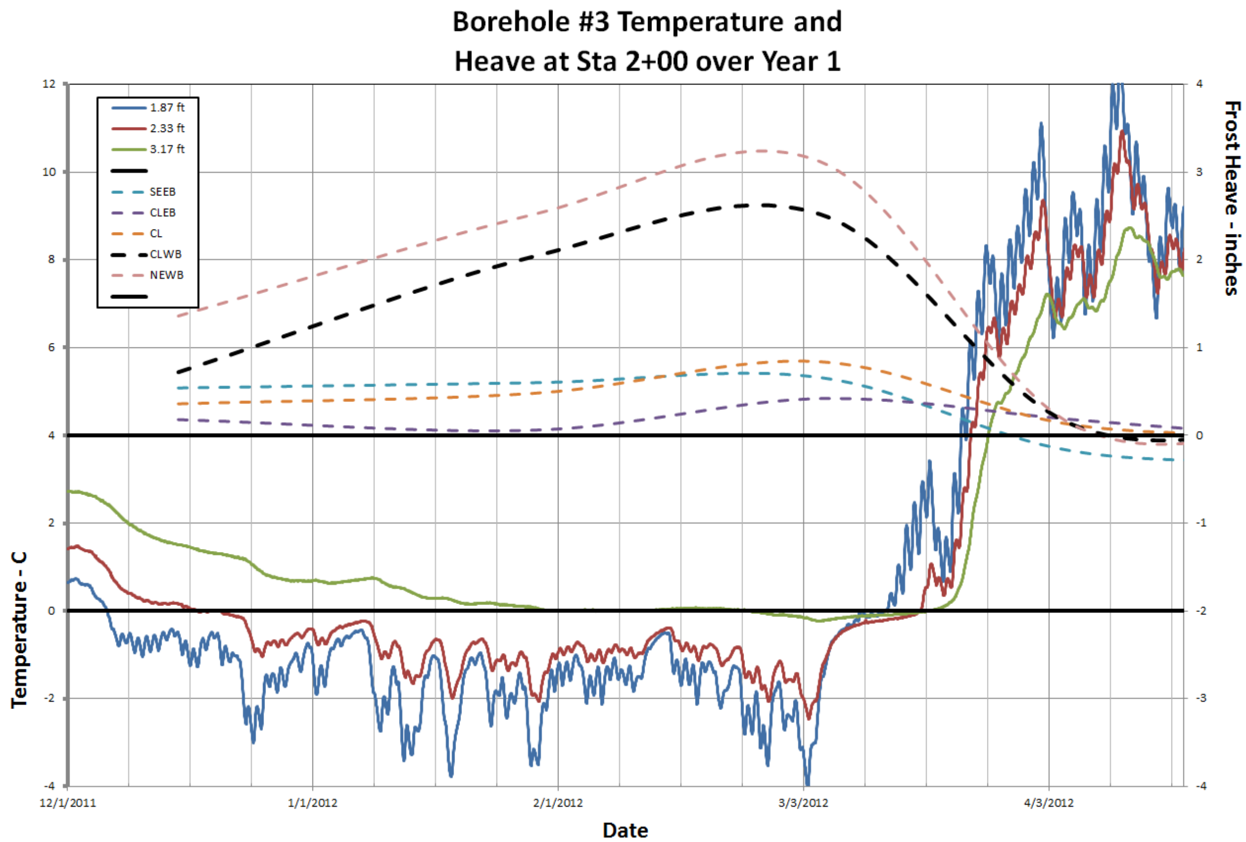
TEMPERATURE AND HEAVE COMPARISON AT BOREHOLE LOCATIONS

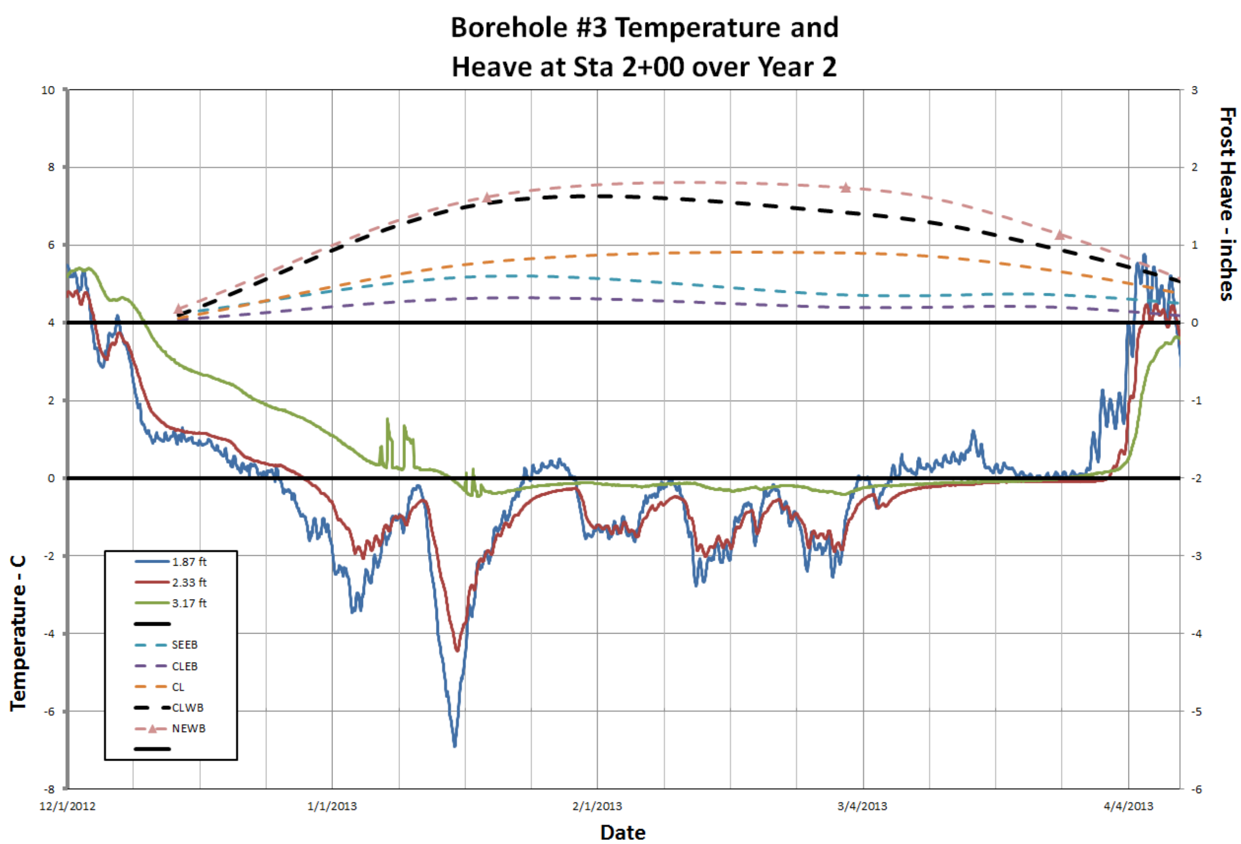

Borehole #1 Temperature and
Heave at Sta 1+20 over Year 1
1.33 ft
1.87 ft
2.67 ft
SEEB
CLEB
CL
CLWB
NEWB
Temperature - C
Frost Heave - inches
Date
12/1/2011
1/1/2012
2/1/2012
3/3/2012
4/3/2012

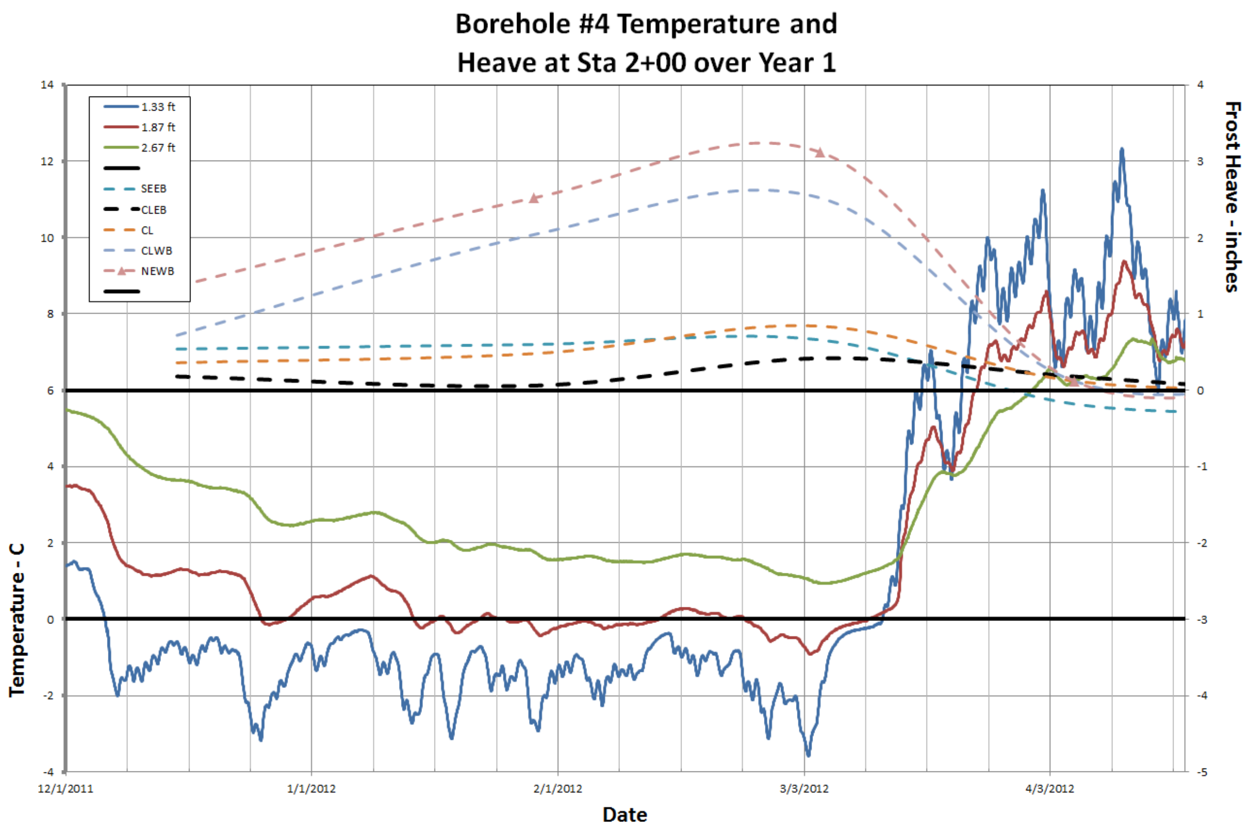

Borehole #1 Temperature and
Heave at Sta 1+20 over Year 2
1.33 ft
1.87 ft
2.67 ft
SEEB
CLEB
CL
CLWB
NEWB
Temperature - C
Frost Heave - inches
Date
12/1/2012
1/1/2013
2/1/2013
3/4/2013
4/4/2013


Borehole #2 Temperature and
Heave at Sta 1+80 over Year 1
1.33 ft
1.87 ft
2.67 ft
SEEB
CLEB
CL
CLWB
NEWB
Temperature - C
Frost Heave - inches
Date
12/1/2011
1/1/2012
2/1/2012
3/3/2012
4/3/2012

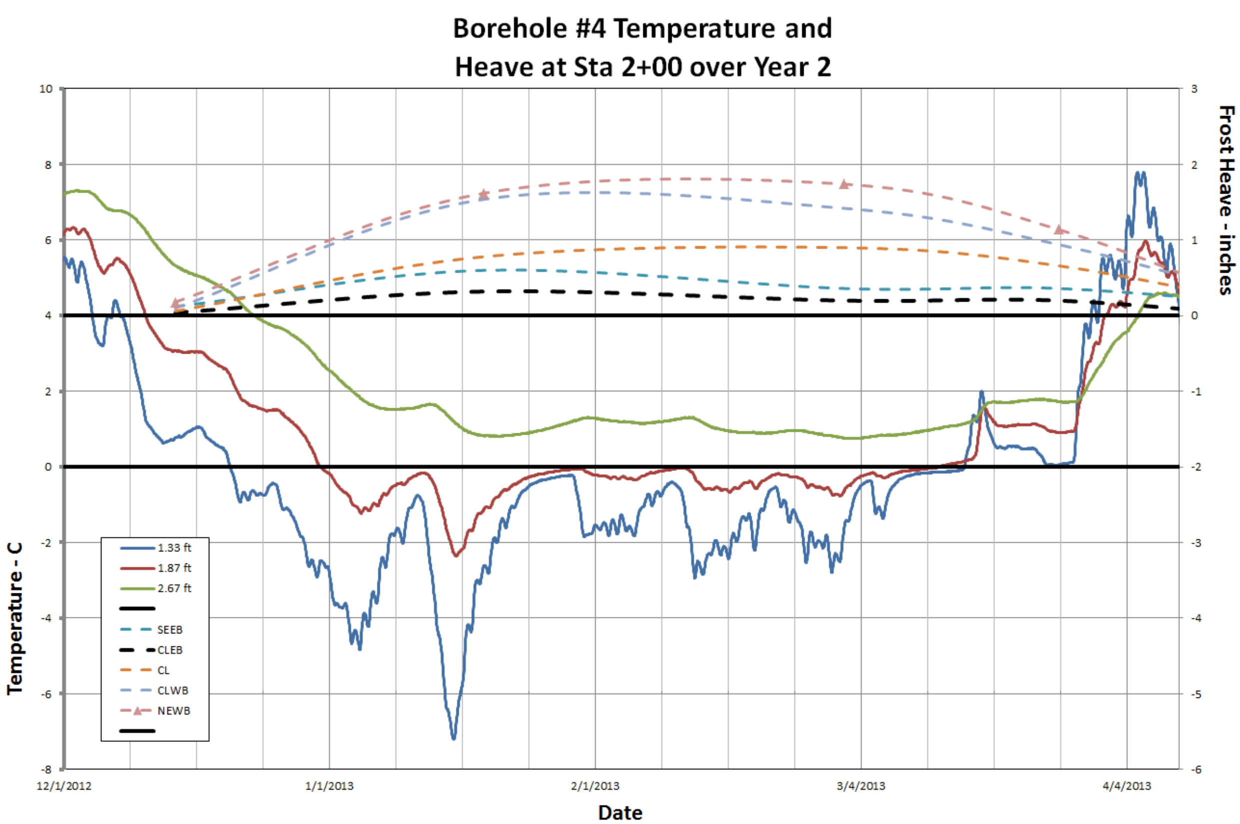

Borehole #2 Temperature and
Heave at Sta 1+80 over Year 2
1.33 ft
1.87 ft
2.67 ft
SEEB
CLEB
CL
CLWB
NEWB
Temperature - C
Frost Heave - inches
Date
12/1/2012
1/1/2013
2/1/2013
3/4/2013
4/4/2013


Borehole #3 Temperature and
Heave at Sta 2+00 over Year 1
1.87 ft
2.33 ft
3.17 ft
SEEB
CLEB
CL
CLWB
NEWB
Temperature - C
Frost Heave - inches
Date
12/1/2011
1/1/2012
2/1/2012
3/3/2012
4/3/2012

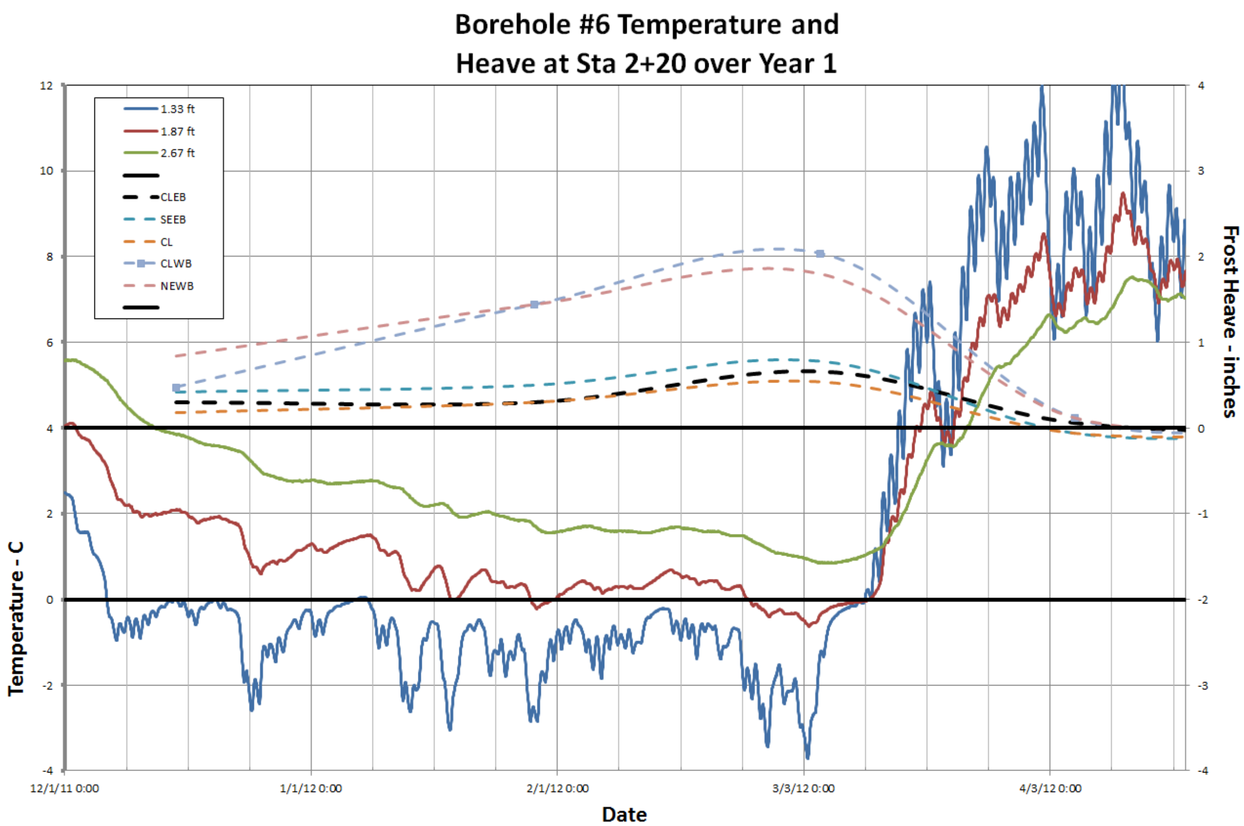

Borehole #3 Temperature and
Heave at Sta 2+00 over Year 2
1.87 ft
2.33 ft
3.17 ft
SEEB
CLEB
CL
CLWB
NEWB
Temperature - C
Frost Heave - inches
Date
12/1/2012
1/1/2013
2/1/2013
3/4/2013
4/4/2013

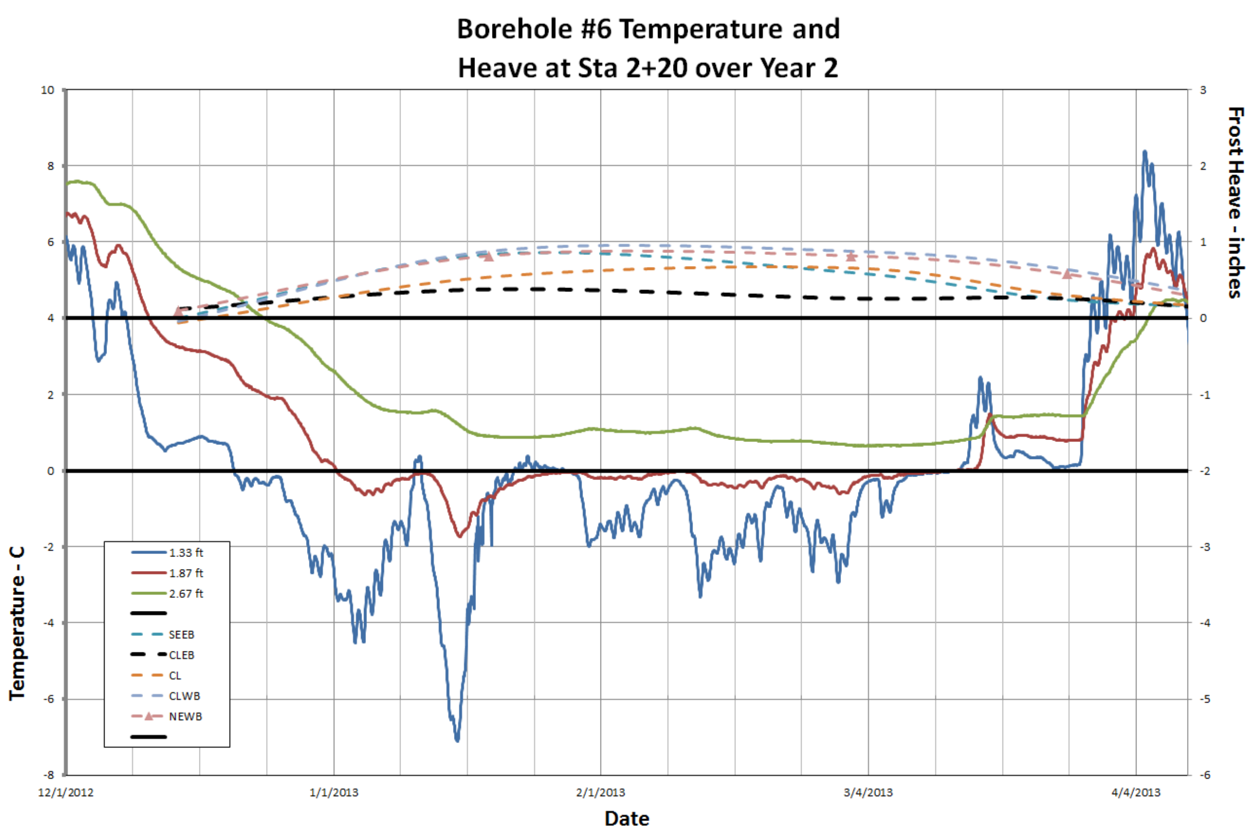


Borehole #4 Temperature and
Heave at Sta 2+00 over Year 1
1.33 ft
1.87 ft
2.67 ft
SEEB
CLEB
CL
CLWB
NEWB
Temperature - C
Frost Heave - inches
Date
12/1/2011
1/1/2012
2/1/2012
3/3/2012
4/3/2012



Borehole #4 Temperature and
Heave at Sta 2+00 over Year 2
1.33 ft
1.87 ft
2.67 ft
SEEB
CLEB
CL
CLWB
NEWB
Temperature - C
Frost Heave - inches
Date
12/1/2012
1/1/2013
2/1/2013
3/4/2013
4/4/2013


Borehole #6 Temperature and
Heave at Sta 2+20 over Year 1
1.33 ft
1.87 ft
2.67 ft
CLEB
SEEB
CL
CLWB
NEWB
Temperature - C
Frost Heave - inches
12/1/11 0:00
1/1/12 0:00
2/1/12 0:00
3/3/12 0:00
4/3/12 0:00
Date


Borehole #6 Temperature and
Heave at Sta 2+20 over Year 2
1.33 ft
1.87 ft
2.67 ft
SEEB
CLEB
CL
CLWB
NEWB
Temperature - C
Frost Heave - inches
12/1/2012
1/1/2013
2/1/2013
3/4/2013
4/4/2013
Date

ACKNOWLEDGEMENTS

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John Christopher Potter was funded for a part of his graduate work and thesis during the first year of the project. Two undergraduate students, Jordan Roberts and Logan Williamson, provided surveying and data collection support during the second year.

The author has a great appreciation for Scott Kinniburgh, Maintenance Foreman and his crew at the Saratoga, WY District 1 Maintenance Shop. Their enthusiasm and dedication to maintaining safe highways in their area of Wyoming needs to be recognized.