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WATER VAPOR MOVEMENT IN FREEZING AGGREGATE BASE MATERIALS

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16. Abstract The objectives of this research were to 1) measure the extent to which water vapor movement results in water accumulation in freezing base materials; 2) evaluate the effect of soil stabilization on water vapor movement in freezing base materials; 3) determine if the corresponding changes in water content are sufficient to cause frost heave during winter; 4) determine if the corresponding changes in water content are sufficient to cause reductions in stiffness during spring; 5) evaluate relationships between selected material properties, freezing conditions, and the occurrence and impact of water vapor movement; and 6) numerically simulate heat and water movement in selected pavement design scenarios. The research involved extensive laboratory and field testing, statistical analyses, and numerical modeling. For a pavement section with a base layer underlain by a capillary barrier, water vapor movement into a frost-susceptible base material can lead to frost heave of the base layer from long-term water vapor movement into a frost-susceptible base material can lead to frost heave of the base layer cannot drain until nearly reaching positive pore pressures, only non-frost-susceptible base material should be assumed to remain in a saturated condition during the entire service life of the pavement.			ovement results in ter vapor movement cient to cause frost to cause reductions in conditions, and the novement in selected trical analyses, and water vapor ring a typical winter mulation of water eave of the base layer ase layer during res, only non-frost- llary barriers, and the e of the pavement. 23. Registrant's Seal	
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UNIT CONVERSION FACTORS

Length:

1 millimeter (mm) = 0.039 inches (in.) 1 centimeter (cm) = 0.394 inches (in.) 1 meter (m) = 3.28 feet (ft) or 1.09 yards (yd)

Area:

1 square meter $(m^2) = 10.764$ square feet (ft^2) or 1.195 square yards (yd^2)

Volume:

1 cubic meter $(m^3) = 35.314$ cubic feet (ft^3) or 1.307 cubic yards (yd^3)

Mass:

1 kilogram (kg) = 2.202 pounds (lb)

Temperature:

degrees Celsius (°C) = $5/9 \cdot ((\text{degrees Farenheit (°F)}) - 32)$ degrees Kelvin (K) = (degrees Celsius (°C)) + 273.15

Pressure:

1 kilopascal (kPa) = 0.145 pound-force per square inch (lbf/in.²)

Energy:

1 Joule (J) = 0.7376 foot-pound-force (ft·lbf)

Power:

1 watt (W) = 0.7376 foot-pound-force per second (ft·lbf/s)

Conductance:

1 microsiemen (μ S) = 1×10⁻⁶ ampere/volt (A/V)

Number:

1 mole (mol) = 6.0221415×10^{23}

LIST OF ACRONYMS

AASHTOAmerican Association of S	State Highway and Transportation Officials
ACPA	American Concrete Pavement Association
ASCE	American Society of Civil Engineers
ASAE	American Society of Agricultural Engineers
ASHRAE American Society of Heating, Ref	frigeration, and Air Conditioning Engineers
ASTM	American Society for Testing and Materials
BYU	Brigham Young University
СТВ	Cement-Treated Base
CRC	Chemical Rubber Company
FHWA	Federal Highway Administration
LVDT	Linear Variable Differential Transformer
LWD	Light Weight Deflectometer
MDD	
NRCCAN	Northeastern Region Certified Crop Adviser
PCA	Portland Cement Association
PFWD	Portable Falling-Weight Deflectometer
PVC	Polyvinyl Chloride
RF	
SHAW	Simultaneous Heat and Water
SSG	Soil Stiffness Gauge
STP	
SWCC	Soil-Water Characteristic Curve

	TDR
Transportation Research Board	TRB
	TRR
	UDOT

EXECUTIVE SUMMARY

The objectives of this research were to 1) measure the extent to which water vapor movement results in water accumulation in freezing base materials; 2) evaluate the effect of soil stabilization on water vapor movement in freezing base materials; 3) determine if the corresponding changes in water content are sufficient to cause frost heave during winter; 4) determine if the corresponding changes in water content are sufficient to cause reductions in stiffness during spring; 5) evaluate relationships between selected material properties, freezing conditions, and the occurrence and impact of water vapor movement; and 6) numerically simulate heat and water movement in selected pavement design scenarios. The research involved extensive laboratory and field testing, statistical analyses, and numerical modeling.

The results of the laboratory testing, which included gradations, Atterberg limits, soil classifications, specific gravity and absorption values, electrical conductivity values, moisture-density relationships, soil-water characteristic curves, moisture-stiffness curves, hydraulic conductivity values, and frost susceptibility assessments, were used to characterize each material and enable subsequent statistical analyses. Testing of both untreated and treated materials enabled investigation of a wide variety of material properties. The results of the field testing, which included temperature, moisture content, water potential, elevation, and stiffness data over time, provided the basis for comparing pavement sections with and without capillary barriers and established the framework for numerical modeling.

In a pavement section with a capillary barrier underlying the base layer, water vapor movement from the subgrade through the capillary barrier may be expected to increase the water content of the base layer by 1 to 3 percent during a typical winter season in northern Utah for base materials similar to those studied in this research. During winter, cold temperatures create an ideal environment for water vapor to travel upward from the warm subgrade soil below the frost line, through the capillary barrier, and into the base material. Soil stabilization can lead to increased or decreased amounts of water vapor movement in freezing base materials depending on the properties of the stabilized soil, which may be affected by gradation, mineralogy, and stabilizer type and concentration. Accumulation of water from long-term water vapor movement into frost-susceptible base materials underlain by a capillary barrier can lead to frost heave of the

base layer as it approaches saturation, as water available in the layer can be redistributed upwards to create ice lenses upon freezing. However, the incremental increase in total water content that may occur exclusively from water vapor movement during a single winter season in northern Utah would not be expected to cause measurable increases in thaw weakening of the base layer during spring. Because water in a base layer overlying a capillary barrier cannot drain until nearly reaching positive pore pressures, the base layer will remain indefinitely saturated or nearly saturated as demonstrated in this research. For materials similar to those studied in this research, potentially important material properties related to the occurrence of water vapor movement during freezing include dry density, percent of material finer than the No. 200 sieve, percent of material finer than 0.02 mm, apparent specific gravity, absorption, initial water content, porosity, degree of saturation, hydraulic conductivity, and electrical conductivity. The rate at which water vapor movement occurs is also dependent on the thermal gradient within the given material, where higher thermal gradients are associated with higher amounts of water vapor movement. The numerical modeling supported the field observations that the capillary barrier effectively trapped moisture in the overlying base material, causing it to remain saturated or nearly saturated throughout the monitoring period.

Only non-frost-susceptible aggregate base materials should be specified for use in cold climates in conjunction with capillary barriers, and the base material in this case should be assumed to remain in a saturated or nearly saturated condition during the entire service life of the pavement. Further study is recommended on water vapor movement in freezing aggregate base materials.

1.0 INTRODUCTION

1.1 Problem Statement

The performance of pavement structures in cold regions can be greatly affected by the ingress and accumulation of moisture in subsurface layers. Beneath the pavement surface layer, which is typically asphalt or concrete, the subsurface layers include the base, subbase, and subgrade with increasing depth. Water in these layers can lead to degradation of the pavement structure through differential frost heave during winter and attendant thaw weakening during spring. Pavement structures comprising frost-susceptible materials often exhibit increased roughness and cracking during winter and experience reductions in bearing capacity during spring, which can lead to further cracking (1, 2).

Two common frost heave mitigation techniques are stabilization of frost-susceptible materials and use of capillary barriers in pavement structures (3, 4, 5). When properly utilized, stabilizers can reduce frost susceptibility by binding soil particles together, decreasing the permeability of the soil, binding free water in chemical bonds, and/or depressing the freezing point of the pore water in the soil (6, 7, 8, 9). A variety of stabilizers are available in the industry for this purpose, and materials engineering protocols have been developed for determining optimum stabilizer contents for specific materials (10, 11).

Capillary barriers are commonly constructed using crushed, open-graded drainage rock to ensure high porosity and hydraulic conductivity. Frequently specified as subbase layers, capillary barriers may be both underlain and overlain by a geotextile to provide filtering action against particularly fine-grained subgrade soils and granular base materials (*12*). Although the use of these "rock cap" designs can provide a drainage path for water seeping downwards through the pavement, the primary purpose of a capillary barrier in this application is to prevent the vertical movement of liquid water from the native subgrade soil upwards into the base layer. While installation of a capillary barrier can effectively eliminate the upward movement of liquid water through capillary rise, the porous nature of the capillary layer may readily facilitate the movement of water vapor through the pavement structure (*13*).

Water vapor movement is a well-known phenomenon and has been studied in fields such as soil science (14, 15, 16, 17), animal science (18), building construction (19, 20), textiles (21), agronomy (22), and food science (23, 24); however, this topic has been largely neglected in the pavement literature. Although providing positive drainage is a general consideration in pavement design, aspects of water vapor movement are not typically considered by pavement engineers.

Within the pavement industry, anecdotal and theoretical evidence suggests that moisture can enter pavement layers in which the ingress of liquid moisture is not expected (25). For example, researchers who conducted a field experiment to demonstrate the effects of water on pavement stiffness during spring thaw attributed the unexpected yet significant accumulation of moisture directly below the pavement surface to condensation, which could only have occurred due to water vapor flow through the free-draining gravelly soil more than 6 ft above the water table (26). In addition, another source states that the accumulation of ice often observed between a granular base and the bottom of an asphalt surface layer is most likely caused by water vapor flowing through permeable granular base material; the thermal gradient created during cold months causes the water vapor to freeze through the process of deposition when it comes in contact with the less permeable asphalt layer (12). Also, in a laboratory experiment conducted to investigate the movement of water vapor through an aggregate base material in response to temperature gradients, frost heave was unexpectedly observed in the specimen, indicating that water vapor movement may play a role in the frost heave mechanism (27). With increasing recognition of the potential importance of water vapor movement in freezing pavement structures, additional research on this topic is warranted. Specifically, pavement engineers need to better understand the degree to which water vapor movement affects the moisture contents of base materials in pavement structures with capillary barriers.

1.2 Research Objectives and Scope

Six objectives were developed for this research:

1. Measure the extent to which water vapor movement results in water accumulation in freezing base materials.

- 2. Evaluate the effect of soil stabilization on water vapor movement in freezing base materials.
- 3. Determine if the corresponding changes in water content are sufficient to cause frost heave during winter.
- 4. Determine if the corresponding changes in water content are sufficient to cause reductions in stiffness during spring.
- 5. Evaluate relationships between selected material properties, freezing conditions, and the occurrence and impact of water vapor movement.
- 6. Numerically simulate heat and water movement in selected pavement design scenarios.

The scope of this research included both laboratory and field investigations. In the laboratory, water vapor movement and frost heave were measured in freezing experiments involving three aggregate base materials and two stabilizers. In the field, two experimental pavement sections differing by the presence of a capillary barrier were constructed in northern Utah for moisture, frost heave, and stiffness monitoring through a fall, winter, and spring. All of the materials tested in the laboratory or in the field were classified and then subjected to absorption, specific gravity, moisture-density, water potential, stiffness, and permeability testing. Statistical analyses and numerical modeling were completed on selected data sets.

1.3 Outline

This report contains five chapters. Chapter 1 gives the problem statement, states the research objectives, and describes the scope of the research. Chapter 2 presents background information on water movement through soils, moisture-related damage mechanisms in pavements, techniques for mitigating frost heave, and the mathematical theory behind numerical modeling. Chapter 3 provides a detailed description of the laboratory and field testing performed in this research. Chapter 4 describes the results of the laboratory and field testing and the associated analyses. Chapter 5 offers conclusions and recommendations based on the research findings.

2.0 BACKGROUND

2.1 Overview

While soils and aggregates must be moistened to be compacted properly during construction, moisture can damage pavements once they are in service. Engineers need to understand mechanisms of moisture transport through soil media so they can design roadways to be less susceptible to the ingress and accumulation of in-situ moisture following construction. Of particular importance to this research, the following sections discuss water movement through soils, damage mechanisms of frost heave and thaw weakening, and techniques for mitigation of frost heave and thaw weakening. This chapter also includes the mathematical theory underpinning a numerical model that can be used to simulate the movement of water in both liquid and vapor phases.

2.2 Water Movement through Soils

Moisture-related damage in pavements can potentially result from the movement of both liquid water and water vapor in response to gradients in moisture, temperature, and/or concentration of dissolved ions, such as salts (28, 29). Specifically, water moves from areas with higher matric and/or osmotic potentials toward areas with lower potentials, and the rate of water movement depends on the magnitudes of the gradients and the hydraulic conductivity and/or vapor diffusivity of the soil. Water potential is the potential energy of water per unit volume in a given state relative to pure water with no external forces acting on it and at a reference atmospheric pressure, temperature, and elevation (30). Because water found in soil pores typically has a lower energy state than pure water, water potential usually has a negative value (31). Because water moves from areas of higher potential to areas of lower potential, gradients in moisture, temperature, and ion concentration influence both the direction and rate of water flow through soil.

Water potential is often discussed in terms of matric and osmotic potentials. Matric potential is the capillary component of water potential that is driven by the cohesive and adhesive properties of water, and it is manifest by the formation of a meniscus at the soil-water

interface (31, 32). Matric potential is affected by the radius of curvature of the meniscus that forms at the air-water interface. The radius of curvature is in turn affected by the radius of the capillary space in the soil, the surface tension of the water in the soil pores, and the contact angle between the soil water and the soil particles (13). Matric potential becomes more negative as the radius of the capillary space becomes smaller, the surface tension of the pore water becomes greater, or the contact angle becomes lower (33). The radius of the capillary space in the soil is determined by the sizes and relative density of the soil particles. The surface tension of the pore water, which results from cohesion of the water particles, increases as salt concentration increases and as temperature decreases (34, 35, 36). The contact angle between the soil particles and the pore water is determined by how strongly the water molecules interact with one another relative to how strongly they interact with the soil particles, where high contact angles are typical of hydrophobic soils (37). When the soil structure and water properties can be assumed not to change, the matric potential is mainly influenced by changes in the water content, as described in Equation 2.1 (38):

$$\psi = \psi_e \left(\frac{\theta}{n}\right)^{-b}$$
(2.1)
where $\psi = \text{matric potential}, \frac{J}{kg}$
 $\psi_e = \text{air-entry potential}, \frac{J}{kg}$
 $\theta = \text{volumetric water content}, \frac{m^3}{m^3}$
 $n = \text{porosity}, \frac{m^3}{m^3}$
 $b = \text{pore-size distribution index}$

The air-entry potential referred to in Equation 2.1 is the water potential at which the largest pores in the soil just begin to drain, or fill with air (39); the larger the largest pore, the higher the air-entry potential; therefore, soils with fine particles like clays have low air-entry values, and soils with larger particles like gravels have high air-entry values. The pore-size distribution index is a value that describes the size distribution of pores with a radius larger than one micron (40).

The osmotic potential of a solution is defined as the negative pressure that would need to be applied to pure water to prevent it from flowing across a semi-permeable membrane into the given solution (41). While pure water has an osmotic potential of zero, saline solutions have decreasing osmotic potential with increasing salt concentration (13). Temperature also affects osmotic potential because a higher temperature increases the activity of the dissolved ions, thereby decreasing osmotic potential (42). Osmotic potential as affected by both salt concentration and temperature is shown in the van't Hoff formulation given in Equation 2.2 (6, 38):

$$\pi = -v \cdot \phi \cdot m \cdot R \cdot T$$
(2.2)
where $\pi = \text{osmotic potential}, \frac{J}{kg}$
 $v = \text{number of ions per salt molecule}$
 $\phi = \text{osmotic coefficient}$
 $m = \text{molal solute concentration}, \frac{\text{mol}}{\text{kg}}$
 $R = \text{gas constant}, 8.314 \frac{J}{\text{mol K}}$
 $T = \text{temperature}, K$

The osmotic coefficient referred to in Equation 2.2 accounts for departures from ideal behavior of the solvent as the solute concentration increases (43); a value of 1.0 reflects an absence of solutes, while values decreasing toward 0.0 indicate increasing solute concentrations. When the amounts of solutes can be assumed not to change within a given volume of soil, the osmotic potential is influenced by changes in the water content to the extent that such changes cause changes in solute concentration.

Relationships between water potential and water content of soil can be graphically represented and analyzed in the form of soil-water characteristic curves (SWCCs) (44). Figure 2.1 gives an example SWCC for a soil. The equation of the trend line in Figure 2.1 is shown in Equation 2.3:


Figure 2.1 Example of a soil-water characteristic curve.

$$y = \ln(\Psi_{E}) + bx$$
(2.3)
where $y = \text{average potential}, \frac{J}{kg}$
 $\Psi_{E} = \text{air-entry potential}, \frac{J}{kg}$
 $b = \text{pore-size distribution index}$
 $x = \ln(\text{degree of saturation})$

Consistent with the laws of equilibrium, water in both liquid and vapor forms moves from areas of high potential to areas of low potential (29). The rate at which water flows through a porous medium is affected by the magnitude of the potential gradient and the conductivity or diffusivity of the soil matrix, which is in turn affected by the structure and degree of saturation of the soil matrix. Regarding the structure of the soil matrix, higher porosity and lower tortuosity within the soil matrix promote higher rates of both liquid and vapor flow (13); however, concerning the effects of degree of saturation, liquid water flow is increased and water vapor flow is decreased with increasing degree of saturation. Darcy's Law describing the flow of a fluid, such as water, through a porous medium, such as soil, is presented in Equation 2.4 (45):

$$Q = \frac{-k \cdot A}{\mu} \cdot \frac{(\Delta P)}{L}$$
(2.4)
where $Q = \text{total discharge, } \frac{\text{m}^3}{\text{s}}$
 $k = \text{permeability, m}^2$
 $A = \text{cross-sectional area, m}^2$
 $\Delta P = \text{change in pressure, } \frac{\text{kg}}{\text{m} \cdot \text{s}^2}$
 $\mu = \text{viscosity, } \frac{\text{kg}}{\text{s} \cdot \text{m}}$
 $L = \text{length over which the pressure drop takes place, m}$

Viscosity, as described in Equation 2.3, describes the fluidity of water, in the context of this research, moving through pore spaces in soil; the viscosity of liquid water increases with decreasing temperature.

In a pavement structure, changes in temperature, water content, and solute concentration can affect the movement of water in both liquid and vapor forms. Removal of heat during winter in cold regions can lead to freezing of pore water, in particular, which can greatly reduce both the matric and osmotic potentials of the affected layers, causing water to move upwards into the frozen zone.

2.3 Damage Mechanisms of Frost Heave and Thaw Weakening

In cold regions, frost heave and thaw weakening can be particularly detrimental to affected pavement structures. Frost heave is the vertical displacement of the ground surface that occurs during freezing of a frost-susceptible soil in the presence of subsurface water (*12, 46, 47*). Frost heave commences when individual pores in the soil matrix become filled with ice and/or supercooled water and generate heaving pressures exceeding the overburden pressure. Ice formations can be due to in-situ and/or segregational freezing. In-situ freezing is the freezing of

water that already exists in a soil matrix, and it is the main method of ice formation if soil freezing happens quickly (*48*). When water freezes, it increases in volume by approximately 9 percent; however, in-situ freezing of unsaturated soils typically results in heave equivalent to less than 3 percent of the height of the frozen zone (*49, 50*).

On the other hand, segregational freezing is the freezing of incoming water that moves into the freezing zone from unfrozen soil, and it can result in the formation of thick ice lenses, as shown in Figure 2.2 (*51*), and significant vertical displacement (*52*). Segregational freezing is possible when gradients in matric and osmotic potential occur, typically caused by the apparent drying of a soil due to ice formation, and moisture is drawn to the freezing front from the warmer underlying soil strata, as illustrated in Figure 2.3 (*53*). Because factors affecting frost heave can vary spatially across a site, differential frost heave, as shown in Figure 2.4 (*54*) for a pavement structure, frequently occurs.

As ice begins to form in the pores between soil particles, the diameter of the capillaries is effectively reduced. Furthermore, because freezing soil is much the same as drying soil with



Figure 2.2 Ice lenses.



Figure 2.3 Formation of ice lenses in a pavement structure.



Figure 2.4 Pavement damage from frost heave.

regards to the removal of liquid water, the matric potential decreases as water freezes (33). Both of these effects lead to an increased upward movement of water from areas deeper in the soil matrix toward the freezing front (12, 55). However, unlike the removal of water that occurs at a drying front, liquid water freezes, water vapor is deposited, and ice accumulates at a freezing front. Because freezing can typically occur much faster than drying, extreme hydraulic gradients can develop in frozen soils that can cause water to flow toward the freezing front very rapidly (55, 56).

Ice forms in soil generally beginning in the center of a pore space and then extending radially outwards. In the freezing process, ions that may have been present in the pore water are excluded, as ice forms in a pure state. Therefore, as freezing progresses, increases in solute concentration in the remaining, supercooled pore water surrounding the ice lead to lower osmotic potential and an increasingly reduced freezing point of the supercooled water (*57*). As additional heat is removed, water molecules from the supercooled solution within the pores are continually incorporated into the growing pore ice until the freezing point of the solution is depressed below the soil temperature, assuming the overburden pressure is not excessive (*28*). Although an increase in solute concentration leads to an increase in the surface tension of water, which decreases matric potential, it also leads to higher unfrozen water contents in freezing soils at a given temperature, which increase matric potential. Because the latter effect is comparatively more influential than the former, increasing solute concentration is generally expected to increase matric potential in frozen soils and therefore inhibit the occurrence of frost heave (58).

Frost-susceptible soils develop low matric potentials upon freezing, while still maintaining sufficient hydraulic conductivity for water to move upwards from unfrozen underlying strata into the frozen zone. Silts, in particular, meet these criteria, but gravels, sands, and clays typically do not (12, 46); gravel and sand particles are not generally fine enough to support development of low matric potentials, and clays are too impermeable to water even though their capillarity is high due to small pore sizes (46, 59). The United States Army Corps of Engineers has developed a system for classifying the frost susceptibility of soils based in part on the percentage by mass finer than 0.02 mm (46).

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While frost heave resulting from the presence of frost-susceptible soils in pavement structures can lead to extensive cracking and roughness of the pavement surface during winter, springtime thawing can lead to marked reductions in the bearing capacity of affected pavement structures. As pavements thaw from the top down, melt water can become trapped above still-frozen underlying layers, leading to conditions of super-saturation in the road bed (*12*). Although ice lenses that form in the pavement structure during winter are a primary source of water, infiltration of surface water through the pavement surface can intensify the problem. When an asphalt surface layer is porous due to improper compaction, or when joints and/or cracks in the surface of concrete or asphalt pavement exist, additional melt water from snow and ice above the surface can also penetrate into the upper layers of the pavement structure. For this reason, properly compacting asphalt surface layers to 3 to 8 percent voids, sealing of concrete joints, and sealing of all cracks are necessary to ensure an impermeable surface (*60*).

Under conditions of super-saturation, a pavement structure may be unable to support heavy traffic loads, as development of positive pore water pressures under loading can lead to loss of inter-particle friction and therefore to reduced stiffness and strength (*61, 62, 63, 64, 65*). Trafficking of a pavement in this condition can lead to rapid development of fatigue cracking as shown in Figure 2.5 (*53*). While some clays may not experience significant frost heave, thaw weakening may still occur as segregated ice melts during spring (*66*). Indeed, many researchers believe that thaw weakening is the most important frost-action damage mechanism (*12, 62, 64,*



Figure 2.5 Pavement damage from thaw weakening.

67, 68). Studies have shown that the bearing capacity of susceptible pavement structures can be reduced during spring by 15 to 60 percent of fall values, depending on soil type and other factors (62, 64). In particular, because of their high susceptibility to frost heave, silty soils generally experience more pronounced thaw weakening than other soil types (69).

2.4 Mitigation of Frost Heave and Thaw Weakening

To mitigate frost heave, at least one of three essential factors, including sustained freezing temperatures, frost-susceptible soils, and available water, must be eliminated. According to the United States Army Corps of Engineers, the most common method used to mitigate frost heave in pavements is placement of non-frost-susceptible materials within the freezing zone (46, 70, 71, 72), which is often coupled with installation of a capillary barrier, as shown in Figures 2.6 (73) and 2.7 (74). The materials used to construct these layers are most commonly coarse-grained, free-draining materials, such as gravels, with the thickness of the capillary barriers typically ranging between 1.5 and 5.0 ft (75). Non-woven geotextiles are commonly placed above and below the capillary barriers to prevent fines from migrating into the layers while still allowing water drainage to occur (12). As specifically shown in Figure 2.7, specialized geosynthetics, which are much thinner, can also be used as capillary barriers.

When non-frost-susceptible materials are not available for a project, other materials may be improved using chemical stabilization. Chemicals commonly used for soil stabilization



Figure 2.6 Schematic of a pavement section with a gravel capillary barrier.



Figure 2.7 Schematic of a pavement section with a geosynthetic capillary barrier.

include cementitious products such as portland cement, lime, and fly ash, as well as chloridebased salts (10, 76). Although the results of one study indicate that cementitiously-stabilized base layers may be too rigid for placement over frost-susceptible subbase and/or subgrade materials in pavements that experience deep frost penetration (62), improvements in material strength and durability achieved from the proper use of cementitious stabilizers can greatly reduce the frost susceptibility of the treated layer itself, increasing its resistance to both frost heave and spring thaw damage (75, 77, 78, 79). The use of cementitious stabilizers may also allow for base layers of decreased thickness when compared to untreated base materials because the increase in rigidity allows for loads to be distributed more evenly over a larger area (79). Application ratios for cement stabilization vary widely but frequently range from 3 to 14 percent (79, 80).

Various forms of chlorides, including calcium, magnesium, and sodium, can also be used for soil stabilization (81). Unpaved roads stabilized with calcium chloride, for example, have been shown to resist washboarding, erosion, and raveling; significantly reduce aggregate loss; and improve safety by reducing dust (82, 83). In semiarid and arid climates, unpaved roads treated with magnesium chloride exhibited satisfactory rideability for more than 2 years with no maintenance (84). Additional benefits from chloride-based salts included an increase in dry density, derived from enhanced particle lubrication during compaction, and a depression in the freezing point of the pore water, which can virtually eliminate frost heave as long as the salt concentration remains sufficiently high (6, 8, 9, 85). Because salt has been shown to leach out of treated materials over time, so that it may not be present in appreciable quantities several years after stabilization (86, 87), the effects of stabilization may be temporary. Therefore, their use in frost heave and thaw weakening mitigation may not be effective over a long time period (86). Application ratios for chloride-based salts frequently range from 0.1 to 1 percent (82, 85, 88, 89).

Frost heave and thaw weakening are both caused by the ingress and accumulation of moisture; therefore, the methods of mitigation for each are typically the same (90). However, even if the amount of moisture in a pavement structure is not significant enough to cause frost heave, thaw weakening could still occur as a result of moisture accumulation immediately beneath a pavement surface. Therefore, measures taken to mitigate frost heave, while helpful, may not be sufficient to completely mitigate thaw weakening, and thick pavement layers and/or spring load restrictions may be required to prevent premature failure (53, 91).

2.5 Mathematical Theory behind Numerical Modeling

The movement of water in both liquid and vapor phases within a soil medium can be simulated with the Simultaneous Heat and Water (SHAW) model. The SHAW model is a one-dimensional finite-difference model that simulates coupled heat, water, and solute transfer in atmosphere-plant-snow-residue-soil systems (92) and has been widely applied to soil freezing and thawing events (93, 94, 95). A one-dimensional water balance equation including liquid water flow and vapor flow is expressed in the SHAW model as shown in Equation 2.5 (92):

$$\frac{\partial \theta_l}{\partial t} + \frac{\rho_i}{\rho_l} \frac{\partial \theta_i}{\partial t} = \frac{\partial}{\partial z} \left[K \left(\frac{\partial \Psi}{\partial z} + 1 \right) \right] + \frac{1}{\rho_l} \frac{\partial q_v}{\partial z}$$
(2.5)

where t = time, s

 θ_l = volumetric liquid water content, $\frac{m^3}{m^3}$ t = time, s ρ_i =density of ice, $\frac{kg}{m^3}$ ρ_l = density of liquid water, $\frac{kg}{m^3}$

$$\theta_i$$
 = volumetric ice content, $\frac{\text{m}^3}{\text{m}^3}$
 z = depth, m
 K = hydraulic conductivity, $\frac{\text{m}}{\text{s}}$
 Ψ = matric potential, m H₂O
 q_v = water vapor flux, $\frac{\text{kg}}{\text{m}^2\text{s}}$

The relationship between matric potential and volumetric water content is expressed with the following Equation 2.6 (96):

$$\Psi = \Psi_E \left(\frac{\theta_1}{\theta_s}\right)^{-b}$$
(2.6)

where $\Psi = \text{matric potential, m H}_2\text{O}$ $\Psi_E = \text{air-entry potential, m H}_2\text{O}$ $\theta_l = \text{volumetric liquid water content, } \frac{\text{m}^3}{\text{m}^3}$ $\theta_s = \text{saturated soil water content, } \frac{\text{m}^3}{\text{m}^3}$ b = pore-size distribution index

Soil hydraulic conductivity as a function of matric potential or water content is defined in Equation 2.7 (96):

$$K = K_s \left(\frac{\Psi_E}{\Psi}\right)^{\left(2+\frac{3}{b}\right)}$$
(2.7)

where K = hydraulic conductivity, $\frac{m}{s}$

 K_s = saturated hydraulic conductivity, $\frac{m}{s}$ Ψ_E = air-entry potential, m H₂O Ψ = matric potential, m H₂O b = pore-size distribution index When ice is present in soil, the matric potential is expressed as a function of temperature as shown in Equation 2.8 (97):

$$\Psi = \frac{L_f}{g} \left(\frac{T}{T + 273.16} \right)$$
(2.8)

$$\Psi = \text{matric potential, m H_2O}$$

$$L_f = \text{latent heat of fusion, } \frac{J}{\text{kg}}$$

$$g = \text{gravitational acceleration, } 9.81 \frac{\text{m}}{\text{s}^2}$$

$$T = \text{temperature, }^{\circ}C$$

where

When ice is present, the maximum liquid water content is defined from temperature in Equations 2.6 and 2.8; ice content is therefore the water content exceeding the maximum liquid water content.

The one-dimensional energy conservation equation including soil freezing and thawing, heat conduction, and convective heat transfer, with both liquid water and water vapor, follows in Equation 2.9 (92):

$$C_{s} \frac{\partial T}{\partial t} - \rho_{i} L_{f} \frac{\partial \theta_{i}}{\partial t} = \frac{\partial}{\partial z} \left[\lambda_{s} \frac{\partial T}{\partial z} \right] - \rho_{l} c_{l} \frac{\partial (q_{l}T)}{\partial z} - L_{v} \left(\frac{\partial q_{v}}{\partial z} + \frac{\partial \rho_{v}}{\partial t} \right)$$
(2.9)
where C_{s} = soil volumetric heat capacity, $\frac{J}{m^{3} \circ C}$
 T = temperature, °C
 t = time, s
 ρ_{i} = density of ice, $\frac{\text{kg}}{\text{m}^{3}}$
 L_{f} = latent heat of fusion, $\frac{J}{\text{kg}}$
 θ_{i} = volumetric ice content, $\frac{\text{m}^{3}}{\text{m}^{3}}$
 z = depth, m

$$\lambda_{s} = \text{soil thermal conductivity, } \frac{W}{m \circ C}$$

$$\rho_{l} = \text{density of liquid water, } \frac{\text{kg}}{\text{m}^{3}}$$

$$c_{l} = \text{specific heat of liquid water, } \frac{\text{J}}{\text{kg} \circ C}$$

$$q_{l} = \text{liquid water flux, } \frac{\text{m}}{\text{s}}$$

$$L_{v} = \text{latent heat of vaporization, } \frac{\text{J}}{\text{kg}}$$

$$q_{v} = \text{water vapor flux, } \frac{\text{m}}{\text{s}}$$

$$\rho_{v} = \text{density of water vapor, } \frac{\text{kg}}{\text{m}^{3}}$$

Thermal conductivity and volumetric heat capacity of soils are calculated as functions of soil particle-size distribution, bulk density, soil water content, and ice content with the theory initially developed by de Vries (*98*).

The surface boundary condition of the system is determined by the surface energy balance equation given as Equation 2.10:

$$R_{n} = H + L_{v}E + G$$
(2.10)
where $R_{n} = \text{net radiation}, \frac{W}{m^{2}}$
 $H = \text{net radiation}, \frac{W}{m^{2}}$
 $L_{v} = \text{latent heat of vaporization}, \frac{J}{\text{kg}}$
 $E = \text{evaporation rate at the surface}, \frac{\text{kg}}{\text{m}^{2}\text{s}}$
 $G = \text{ground heat flux}, \frac{W}{\text{m}^{2}}$

 R_n is defined as shown in Equation 2.11 (99):

$$R_{n} = (1 - \alpha)R_{s} + \sigma\varepsilon_{s}(\varepsilon_{a}T_{a}^{4} - T_{s}^{4})$$
(2.11)
where $R_{n} = \text{net radiation}, \frac{W}{m^{2}}$
 $\alpha = \text{surface albedo}$
 $R_{s} = \text{solar radiation}, \frac{W}{m^{2}}$
 $\sigma = \text{Stefan-Boltzmann constant}, 5.67 \times 10^{-8} \frac{W}{m^{2}K^{4}}$
 $\varepsilon_{s} = \text{emissivity of surface}$
 $\varepsilon_{a} = \text{atmospheric emissivity}$
 $T_{a} = \text{air temperature}, ^{C}$
 $T_{s} = \text{surface temperature}, ^{C}$

Sensible heat flux and evaporation rate at the surface are determined using Equations 2.12 and 2.13, respectively (99):

$$H_{s} = \rho_{a}c_{a} \frac{(T_{s} - T_{a})}{r_{H}}$$

$$E_{s} = \frac{\rho_{vs} - \rho_{va}}{r_{v}}$$
(2.12)
$$H_{s} = \text{heat flux at the surface, } \frac{W}{m^{2}}$$

$$\rho_{a} = \text{density of air, } \frac{\text{kg}}{\text{m}^{3}}$$

$$c_{a} = \text{specific heat of air, } \frac{\text{J}}{\text{kg}^{\circ}\text{C}}$$

$$T_{s} = \text{surface temperature, } ^{\circ}\text{C}$$

$$T_{a} = \text{air temperature, } ^{\circ}\text{C}$$

$$r_{H} = \text{resistance to surface heat transfer, } \frac{\text{s}}{\text{m}}$$

$$E_{s} = \text{evaporation rate at the surface, } \frac{\text{kg}}{\text{m}^{2}\text{s}}$$

$$\rho_{vs} = \text{surface vapor density, } \frac{\text{kg}}{\text{m}^3}$$

 $\rho_{va} = \text{atmospheric vapor density, } \frac{\text{kg}}{\text{m}^3}$
 $r_v = \text{resistance for vapor transfer, } \frac{\text{s}}{\text{m}}$

The variable r_v is taken to be equal to r_H . Through these equations, the SHAW model allows analysis of the movement of water in both liquid and vapor phases within a soil medium.

2.6 Summary

Engineers need to understand mechanisms of moisture transport through soil media so they can design roadways to be less susceptible to the ingress and accumulation of in-situ moisture following construction. Liquid water and water vapor move through soil in response to gradients in water potential, which is often discussed in terms of matric and osmotic potentials. Matric potential is the capillary component of water potential that is driven by the cohesive and adhesive properties of water, and it is manifest by the formation of a meniscus at the soil-water interface. The osmotic potential of a solution is defined as the negative pressure that would need to be applied to pure water to prevent it from flowing across a semi-permeable membrane into the given solution. Consistent with the laws of equilibrium, water in both liquid and vapor form moves from areas of high potential to areas of low potential. Relationships between water potential and water content of soil can be graphically represented and analyzed in the form of SWCCs.

In cold regions, frost heave and thaw weakening can be particularly detrimental to affected pavement structures. In-situ freezing is the freezing and expansion of the water that already exists in a soil matrix, and it is the main method of ice formation if soil freezing happens quickly. Segregational freezing is the freezing of incoming water that moves into the freezing zone from unfrozen soil, and it can result in the formation of thick ice lenses and significant vertical displacement. While frost heave resulting from the presence of frost-susceptible soils in pavement structures can lead to extensive cracking and roughness of the pavement surface during winter, springtime thawing can lead to marked reductions in the bearing capacity of affected pavement structures.

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To mitigate frost heave, at least one of three essential factors, including sustained freezing temperatures, frost-susceptible soils, and available water, must be eliminated. The most common method used to mitigate frost heave in pavements is placement of non-frost-susceptible materials within the freezing zone. When non-frost-susceptible materials are not available for a project, other materials may be improved using chemical stabilizers such as cementitious products and chloride-based salts. Improvements in material strength and durability achieved from the proper use of cementitious stabilizers can greatly reduce the frost susceptibility of the treated layer, increasing its resistance to both frost heave and spring thaw damage. Benefits from treating materials with chloride-based salts include an increase in dry density, derived from enhanced particle lubrication during compaction, and a depression in the freezing point of the pore water, which can virtually eliminate frost heave as long as the salt concentration remains sufficiently high. The movement of water in both liquid and vapor phases within a soil medium can be simulated with the SHAW model.

3.0 PROCEDURES

3.1 Overview

This research involved both laboratory and field testing of several materials as described in the following sections. Two aggregate base materials were specifically selected for treatment with portland cement and calcium chloride. These included a natural aggregate obtained from a pavement rehabilitation project along Redwood Road near Bluffdale, Utah, and a crushed aggregate base material obtained from the McGuire Pit in Willard, Utah, both of which are representative of base materials used by the Utah Department of Transportation (UDOT) in northern Utah. Four additional materials were selected because of their use in construction of an experimental field site on 400 North at approximately 1400 West in Orem, Utah. These included a crushed aggregate base material, a dense-graded subbase material, an open-graded subbase material, and the native subgrade. All of these material samples were shoveled into 5-gallon buckets from their respective field sites for transport to the Brigham Young University (BYU) Highway Materials Laboratory. The following sections provide a detailed description of the laboratory and field testing.

3.2 Laboratory Testing

The following sections describe the laboratory procedures applied to each of the materials to determine the gradation, Atterberg limits, soil classification, specific gravity and absorption, electrical conductivity, moisture-density relationship, SWCC, moisture-stiffness curve, and hydraulic conductivity. The frost susceptibility of each base material in an untreated state was investigated, as well as that of the Redwood and McGuire materials with Type I/II cement and calcium chloride dihydrate treatments. Statistical analyses were also performed to investigate possible relationships between frost susceptibility data and other soil characteristics that were determined through laboratory testing.

3.2.1 Gradation

Upon arrival at the laboratory, the buckets of material were individually emptied into metal pans for drying to constant weight in an oven at 140°F. Afterwards, the entire bulk sample of each material was processed through a sieve machine to separate it by particle size. The sieve sizes used in this process included 0.75 in., 0.50 in., 0.375 in., No. 4, No. 8, No. 16, No. 30, No. 50, No. 100, and No. 200. The particles retained on each sieve, as well as those that passed the No. 200 sieve, were stored in separate 5-gallon buckets. Once the processing was complete, the total weight of material in each size category was determined, and a master gradation was determined from these weights.

The master gradation was followed in preparation of all samples of that material, including a sample prepared specifically for washed sieve analysis, which was performed in general accordance with American Society for Testing and Materials (ASTM) D422 (Standard Test Method for Particle-Size Analysis of Soils). In this test, a target total sample weight of 3.3 lb was reached by weighing out calculated amounts of individual particle sizes and separately washing each one over the given sieve and all smaller sieves in order from large to small. The washed samples were then dried to constant weight in an oven at 230°F, and the dry weights were used to compute the actual gradation for the given material. In addition, following completion of the washed sieve analysis, 0.11 lb of the material passing the No. 200 sieve was subjected to hydrometer testing, again in general accordance with ASTM D422, to determine the percentage of material finer than 0.02 mm as needed for soil classification purposes.

3.2.2 Atterberg Limits

After the sieve analyses were complete, Atterberg limits were determined on the fraction finer than the No. 40 sieve in general accordance with ASTM D4318 (Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils). For each material, the proper amounts of each sieve size were calculated to achieve a 2.2-lb sample. The proper amounts of each sieve size for the No. 30 sieve size and smaller were then weighed out and screened on the No. 40 sieve, and only the material passing the No. 40 sieve was used in the testing. This information was used, along with the material gradations, to determine soil classifications.

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3.2.3 Soil Classification

The collected gradation and Atterberg limits data were used to classify all materials in general accordance with the American Association of State Highway and Transportation Officials (AASHTO) M145 (Standard Specification for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes), ASTM D2487 (Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)), and the United States Army Corps of Engineers Frost Design Soil Classification System.

3.2.4 Specific Gravity and Absorption

Specific gravity and absorption tests were performed in general accordance with ASTM C127 (Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate) and ASTM C128 (Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate), respectively. Testing was performed on 5.5-lb samples of the coarse fractions, or fractions larger than the No. 4 sieve, and 1.1-lb samples of the fine fractions, or fractions finer than the No. 4 sieve. Samples were weighed out separately from the buckets of material following the master gradation prepared for the given material. The harmonic mean of the specific gravities for the coarse and fine fractions was then computed for each material. This information was used, along with dry density, to compute the porosity of compacted specimens prepared in subsequent testing.

3.2.5 Electrical Conductivity

Electrical conductivity was determined using the material passing the No. 40 sieve, consistent with procedures developed at the Texas Transportation Institute (*57, 100*). For a given test, 0.011 lb of material was mixed with 0.220 lb of deionized water in a polyethylene container. The solution was stirred with a glass rod to encourage dissolution of any salts that were present in the sample. The bottles were then closed to prevent evaporation of water from the solution, which was allowed to equilibrate at room temperature for 14 days.

After equilibration, electrical conductivity testing began. To measure the electrical conductivity of the samples, a testing probe equipped with a dual platinum-plate electrode was

used. The probe was calibrated daily using a standard conductivity solution. The specific standard solution used was determined based on the expected conductivity of the solutions to be tested. The apparatus was standardized using a conductivity solution of 450 μ S/cm for all untreated samples and a conductivity solution of 1500 μ S/cm for all cement- and salt-treated samples. When the probe was inserted in each solution to obtain a reading, as illustrated in Figure 3.1, it was placed to a uniform depth, ensuring the probe was completely immersed in the solution, and the samples were gently agitated to cause the solution to completely surround the probe. Once the reading on the meter stabilized, the electrical conductivity reading was recorded. The probe was then removed from the solution and cleaned with deionized water before being inserted in the next solution.



Figure 3.1 Electrical conductivity testing.

3.2.6 Moisture-Density Relationship

Separate moisture-density curves were prepared for the materials in untreated and treated conditions. While all of the materials were evaluated in the untreated condition, only the Redwood and McGuire materials were evaluated in treated conditions.

For the untreated specimens, materials were weighed out in bowls according to the corresponding master gradations for a minimum of five approximately 4-lb samples, as shown in

Figure 3.2. Water was then added to the samples to meet specific target moisture contents above and below an estimate of the optimum moisture content for the material, with successive water contents usually being spaced by at least 1 percentage point. Following mixing, each sample concentration was sealed in a plastic bag, as shown in Figure 3.3, and allowed to equilibrate for a minimum of 24 hours before compaction.

For the treated specimens, either portland cement or calcium chloride was mixed into the material before compaction. When cement was used, it was consistently applied at a concentration of 3.0 percent by weight of dry aggregate for both materials, while calcium chloride was consistently applied at a concentration of 1.0 percent by weight of dry aggregate for both materials. In general accordance with ASTM D558 (Standard Test Methods for Moisture-Density (Unit Weight) Relations of Soil-Cement Mixtures), the samples to be treated with cement were weighed out in two fractions, coarse and fine, with the coarse fraction containing material retained on the No. 4 sieve and the fine fraction containing material passing the No. 4 sieve. All the water required for a given sample was added to the coarse fraction and allowed to equilibrate in a sealed bag for a minimum of 24 hours. Immediately before compaction, the specified amount of cement was then uniformly blended with the dry fine fraction, and this mixture was then incrementally mixed into the moistened coarse fraction, as shown in Figure 3.4.

The calcium chloride was pre-mixed in a portion of the water that was added to the specimens at the time of mixing, as shown in Figure 3.5, so that it would already be dissolved upon contact with the aggregate. After a 24-hour equilibration period in a sealed bag, the calcium chloride-treated specimens were then ready for compaction.

All of the samples were compacted into 4-in.-diameter steel molds to a target height of 4.6 in. The molds were pre-weighed to allow computation of the wet weight of the specimen following compaction. The compaction effort applied to each material was chosen based on the familiarity of the researchers with the use of the materials in the field. The material collected from the McGuire Pit was compacted in general accordance with ASTM D1557 (Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft³ (2,700 kN-m/m³))), which is the typical specification for base materials used on UDOT

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Figure 3.2 Weigh-out of sieved aggregate base material.



Figure 3.3 Moisture equilibration of uncompacted samples.



Figure 3.4 Preparation of cement-treated specimen.



Figure 3.5 Preparation of calcium-chloride-treated specimen.

projects. The material collected from the Redwood Road project, as well as all materials collected from 400 North in Orem, were compacted in general accordance with ASTM D698 (Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft³ (600 kN-m/m³))). The same compaction effort was applied uniformly to a material, regardless of treatment type.

Specimens were compacted in either five or three lifts with 25 blows per lift of either a 10- or 5.5-lb Proctor hammer, and the surface of each lift was lightly scarified before compaction of the next lift to ensure adequate mechanical coupling between the layers. To prevent evaporation of water from the loose sample during compaction, the bowl was covered with a plastic bag. Immediately following compaction, three blows were applied to each specimen using a finishing tool, shown in Figure 3.6, specifically designed to flatten the specimen surface. The weight and height of each specimen were then measured, the latter being determined using a



Figure 3.6 Application of finishing tool.

micrometer at four equally spaced locations around the perimeter of the specimen. Each specimen was then extruded from the compaction mold, placed into a pre-weighed bowl, and weighed before and after drying to constant weight in a 230°F oven for a period of approximately 48 hours. The collected data were used to determine the OMC and corresponding maximum dry density (MDD) of each material.

3.2.7 Soil-Water Characteristic

For measurement of SWCCs, specimens were prepared in such a way that free-free resonance testing and hydraulic conductivity testing could be performed afterwards, as described in subsequent sections. For this testing, the four 400 North materials were all evaluated in the untreated condition, while the Redwood and McGuire materials were evaluated in both untreated and treated conditions, where the treatments were the same as the two used in moisture-density testing. For each combination of material and treatment type, three replicate specimens were prepared, for a total of 30 specimens. Each specimen was compacted to a target height of 4.6 in. inside a standard plastic mold having a 4-in. diameter and an 8-in. height; the plastic molds were inserted into a metal mold to provide support during compaction as shown in Figure 3.7. Before the specimens were compacted, the interior walls of the molds were coated with a thin layer of submersible caulking, as shown in Figure 3.8, to minimize preferential water flow down the insides of the molds during hydraulic conductivity testing.

For the 400 North materials, specimens were compacted at water contents corresponding to field-measured dry densities, as explained in a later section, while the Redwood and McGuire materials were both compacted at the respective OMC values determined in the laboratory. The same compaction levels used in moisture-density testing were applied to these specimens. After compaction, all the specimens were sealed in plastic bags, as shown in Figure 3.9, and allowed to equilibrate at 70°F in a computer-controlled environmental chamber. The equilibration period was 10 days for untreated samples and 28 days for treated samples, which allowed the cement-treated specimens to substantially cure before testing.

After the conditioning period, the specimens were tested in general accordance with ASTM D5298 (Standard Test Method for Measurement of Soil Potential (Suction) Using Filter



Figure 3.7 Compaction of a specimen.



Figure 3.8 Mold prepared with caulking.



Figure 3.9 Moisture equilibration of compacted specimens.

Paper). This testing involved equilibration of the specimens with Whatman No. 42 filter papers, which were dried in a desiccator and then placed in hydraulic contact with the specimen surfaces and suspended above the specimen surfaces for measurement of matric potential and total potential, respectively. To achieve these conditions, two 1.67-in.-diameter filter papers were placed between two 2.76-in.-diameter filter papers, and the set of four was placed on the smoothest region of the specimen surface but away from the edges so that none of the filter papers were in contact with the mold wall, as shown in Figure 3.10. A circle of thin plastic sheeting with a diameter of about 6 in. was then placed over the filter papers to keep them clean, and a cylindrical steel weight having a diameter of 3 in. and a weight of approximately 2 lb was placed on top of the plastic circle to press the filter papers down tightly against the specimen surface. As a stand, a very short length of polyvinyl chloride (PVC) pipe having a diameter of approximately 1.9 in. and a length of 0.36 in. was then placed on top of the steel weight, as shown in Figure 3.11, and two additional 2.76-in.-diameter filter papers were placed on top of the PVC pipe, one resting on the other, as shown in Figure 3.12. These were approximately centered on top of the specimen away from the mold wall, and one edge of each filter paper was



Figure 3.10 Filter papers for measuring matric potential.



Figure 3.11 Configuration for placement of filter papers.



Figure 3.12 Filter papers for measuring total potential.

folded upwards to facilitate easy handling with tweezers as shown in Figure 3.13. In no case were bare hands used to place the filter papers, as contamination from skin oils can provide inaccurate test results.

After the filter papers were in place, a plastic lid was snapped onto each mold, and electrical tape was then applied around the edges of the lid to seal it closed, as shown in Figure 3.14. The specimens were then allowed to equilibrate for at least 14 days in an environmental chamber maintained at 70°F and 50 percent relative humidity. Following the equilibration period, the specimen molds were carefully opened one at a time in the environmental chamber. Each of the two filter papers on top of the steel weight was removed as fast as possible and sealed in a small metal container for weighing, as shown in Figure 3.15. The steel weight was then removed from the specimen surface, and the middle two filter papers from the set of four were also quickly removed and each sealed in a small metal container for weighing. The containers were then opened and placed in an oven maintained at 230°F for a minimum of 2 hours for drying of the filter papers to constant weight. From the weights measured before and

after drying, the moisture contents of the filter papers were determined for direct mapping to either the matric or total water potential of the specimen. In each case, the average of the two replicate filter papers was reported. From ASTM D5298, the equations used to determine water potential for filter papers with water contents less than and greater than 45.3 percent are presented in Equations 3.1 and 3.2, respectively:

$$h = 5.327 - 0.0779w_f \tag{3.1}$$

$$h = 2.412 - 0.0135w_f \tag{3.2}$$

where h = water potential, kPa

 w_f = gravimetric water content of the filter paper, percent

After all the filter papers were removed, the specimens were subjected to free-free resonance testing, as described in the next section, and were then dried in an oven maintained at either 104 or 140°F to a predetermined weight corresponding to a predetermined target moisture content.



Figure 3.13 Handling of filter papers.



Figure 3.14 Moisture equilibration of specimens for measurement of soil-water characteristic curve.



Figure 3.15 Tools for removal of filter papers.

At this point, new filter papers were placed on top of the specimens, and the same steps previously described were repeated for another round of testing. This testing was performed at five different moisture contents for each of the specimens, after which SWCCs were plotted for each material.

3.2.8 Moisture-Stiffness Relationship

As previously described, free-free resonance testing was performed in conjunction with SWCC testing, which enabled a measurement of stiffness at each of the five moisture contents evaluated for development of the SWCCs. For this testing, the technique outlined in ASTM Special Technical Publications (STP) 1437 (Resilient Modulus Testing for Pavement Components) was followed to determine the resonant frequency and corresponding Young's modulus value of each of the specimens. To complete the testing, each specimen was placed on a stand surfaced with closed-cell foam to provide acoustic isolation for the specimen, as shown in Figure 3.16. The top of the specimen was instrumented with an accelerometer, which was mounted in a disk of closed-cell foam, as shown in Figure 3.17, on which an annular steel weight was placed to ensure good mechanical coupling between the accelerometer and the specimen surface. The bottom of the specimen was struck with a light hammer equipped with a load cell. Both the accelerometer and load cell were connected to a computer, which was programmed to record accelerometer output upon receiving a signal from the load cell. From a computer display of the induced stress wave amplitudes across the range of excited vibration frequencies, the resonant frequency was selected. In this manner, the resonant frequency was measured several times, and the values were averaged for calculation of the modulus of each specimen at specific moisture contents. The results were plotted to show the relationship between stiffness and moisture content for each combination of material and treatment type. At the conclusion of all SWCC and stiffness testing, the specimens were dried to a constant weight in an oven maintained at either 104 or 140°F. Final dry weights were used to backcalculate actual moisture contents for which SWCC and stiffness testing were performed.



Figure 3.16 Accelerometer used for free-free resonance testing.



Figure 3.17 Free-free resonance testing for stiffness measurement.

3.2.9 Hydraulic Conductivity

After completion of all SWCC and free-free resonance tests, hydraulic conductivity testing was performed on the same set of specimens in general accordance with ASTM D5084 (Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter) using the apparatus and method developed in previous research (*101*). To allow water to flow through the specimens, the bottom of each plastic mold was carefully removed using a rotary cutting tool, or, in the case of the open-graded subbase material, numerous small holes were drilled into the bottom of each specimen mold. Filter fabric was then glued over the bottom of each specimen to minimize migration of fines out of the specimen during testing. With the valve on the lower portion of the permeameter closed, the lower portion of the permeameter was filled with water. Each specimen was then placed inside flexible couplers that allowed the specimen to be secured by hose clamps to the lower portion of the permeameter and to an open standpipe above the specimen, as shown in Figure 3.18. After the specimen was secured, the open standpipe attached to the top of the specimen was filled with water to allow the specimen to soak for a minimum of 4 hours.



Figure 3.18 Configuration for saturation of specimens.

After this conditioning period, the specimen was removed and visually inspected as shown in Figure 3.19. If the bottom of the specimen was not uniformly soaked, the specimen was returned to the permeameter for additional conditioning for a minimum of 24 hours. After soaking, the specimen was placed in the permeameter, and a 60-in. standpipe was secured to the specimen and filled with water. The valve in the lower portion of the permeameter was then opened to start the flow of water through the specimen. The rate of flow was allowed to stabilize before readings were taken. To measure the water flow rate through a specimen, a graduated cylinder was placed at the discharge location of the permeameter, and a stopwatch was used to record the amount of time required for a specified volume of water to collect in the graduated cylinder. Multiple readings were taken and used to calculate the average hydraulic conductivity for each specimen.

The testing of specimens with lower permeability, particularly those including stabilization agents, was facilitated by pressurizing the water flowing through the specimens. The pressurization was accomplished by adding an air-tight lid with an air hose fitting, as shown in Figure 3.20, to the top of the water-filled standpipe and supplying air at a regulated pressure ranging from 0 to 10 psi to maintain a constant pressure head on the specimen. For specimens that had extremely low permeability, duct tape was wrapped around the top of the specimen molds, as shown in Figure 3.21, to ensure a proper seal under the elevated pressure that was applied during those tests. During this testing, the upper and lower portions of the permeameter were held together using rope, as shown in Figure 3.22, to ensure stability.



Figure 3.19 A saturated specimen.



Figure 3.20 Lid for standpipe.



Figure 3.21 Specimen prepared for hydraulic conductivity testing.



Figure 3.22 Configuration for hydraulic conductivity testing.
3.2.10 Frost Susceptibility

Two sets of frost heave tests were performed. Both focused exclusively on the Redwood, McGuire, and 400 North base materials collected for evaluation in this research. The first set included only untreated specimens, while the second set included both untreated and treated specimens. The following sections explain the testing procedures.

3.2.10.1 Frost Susceptibility of Untreated Materials

The first frost heave test was designed to evaluate the effect of moisture availability on the frost heave characteristics of untreated specimens, including the ingress and/or redistribution of moisture within the materials. For this testing, a total of three specimens of each of the Redwood, McGuire, and 400 North base materials were prepared, for a total of nine specimens. Each specimen was compacted to a target height of 9 in. inside a 6-in.-diameter mold that had been greased with light oil to minimize adfreezing of the specimen to the mold walls during testing. Three types of molds were utilized for each material type.

The first mold type was a standard plastic cylinder mold that had been trimmed to a height of 9.5 in. Four slits, equally spaced around the circumference, were cut into the mold to prevent the mold itself from impeding the movement of the overburden weight should heaving of the specimen occur. The specimens compacted in these molds were designed to be placed directly into the water bath of the frost heave testing apparatus, as shown in Figure 3.23, but not experience any water ingress. This configuration was included in the experimentation to investigate how water already in a specimen is redistributed when a thermal gradient typical of top-down freezing in the field is imposed on it.

The second mold type consisted of the same standard plastic cylinder mold as the first type, but, in addition to the other modifications made, several 0.125-in.-diameter holes were drilled into the base of the mold; about 24 holes were drilled approximately 1 in. apart in a line about 0.5 in. above the bottom, and seven additional holes were drilled in the bottom of the mold to allow capillary rise from the bath water into the specimens during testing. This configuration, illustrated in Figure 3.24, was included to investigate the amount of water ingress that occurs in freezing specimens exposed to shallow water tables.



Figure 3.23 Schematic of frost heave test configuration for untreated specimen closed to water ingress.

The third mold type was a 10-in. length of PVC pipe with an inside diameter of 6 in. Each specimen compacted in a pipe mold was supported at its base during testing by a matching PVC coupler and a 6.5-in. length of PVC pipe designed to elevate the bottom of the specimen above the water table in the frost heave testing apparatus as shown in Figure 3.25. This configuration was included to investigate the amount of water ingress that occurs in freezing specimens in situations where water movement is limited to that which occurs in the form of water vapor. To permit the flow of liquid water from the water bath under the specimen during testing, several holes were drilled into the shorter length of PVC pipe where it would be submerged in water, and a stiff plastic disk pre-drilled with numerous 0.5-in.-diameter holes was placed on the upper lip inside the coupler, as shown in Figure 3.26, to provide structural support to the bottom of the specimen placed inside it. In addition, a No. 200 stainless steel mesh was placed over the plastic disk to minimize the migration of fines out of the specimen during testing. A thick grease was applied to the insides of both ends of the PVC coupler to ensure water-tight seals.



Figure 3.24 Schematic of frost heave test configuration for untreated specimen open to capillary rise.

Materials for all specimens were weighed out and compacted using the methods described in previous sections, except that each of the frost heave specimens had twice as many lifts as the previously compacted specimens and each lift was compacted with 56 blows instead of 25; the total number of blows per unit volume was consistent with the requirements stated in ASTM D1557 and D698 for modified and standard compaction, respectively. In addition, a special base unit was fabricated to hold each of these mold types during compaction. After compaction, all specimens were sealed in plastic bags to maintain the desired moisture content for 3 days, until testing could begin.

In preparation for frost heave testing, the water bath, illustrated in Figure 3.27, was filled with deionized water to a depth of approximately 1.5 in., and the chamber temperature was set to 35.6°F for 24 hours prior to the start of the test. All of the testing equipment, as well as the specimen assemblies, which were still sealed in plastic bags, were also placed in the chamber to cool before testing.



Figure 3.25 Schematic of frost heave test configuration for untreated specimen open to water vapor movement.

Following the cooling period, the weight and height of each specimen were recorded. The height was measured in four locations equally spaced around the circumference of each specimen using a micrometer. Then, 10-lb steel overburden weights were oiled and placed into the molds on the tops of the specimens to simulate an approximately 4-in.-thick pavement surface layer, and the weight of each specimen was also measured in this configuration. A circle of thin plastic sheeting with a diameter of approximately 10 in. was then placed over the top of each specimen and securely taped around the full perimeter of the mold to seal it. The specimens compacted into the third mold type were then pressed into the PVC bases. Based on the number of available data logger channels, all specimens, except the McGuire material compacted in the first mold type and the 400 North material compacted in the second mold type, were then



Figure 3.26 Mold prepared for untreated specimen open to water vapor movement.



Figure 3.27 Configuration of water bath for untreated specimens.

instrumented at their top and bottom surfaces with thermocouples to accommodate monitoring of the temperature profiles within the specimens during testing. The sides of the specimens, as well as the PVC bases of the specimens compacted in the third mold type, were then wrapped in foam insulation to minimize lateral heat loss, and all the specimens were placed into the water bath. The water bath itself was covered by an insulated acrylic sheet positioned just above the water surface and fabricated with nine holes through which the bases of the specimens were inserted. The foam insulation around the specimens extended down to the acrylic sheet insulation, which consisted of a 2-in.-thick piece of Styrofoam, so that heat flow out of the bath was substantially limited to that which occurred through the individual specimens. A linear variable differential transformer (LVDT) was then positioned over the center of the overburden weight placed on each specimen to monitor changes in specimen heights during testing; as shown in Figure 3.28, the LVDTs were mounted to an aluminum frame designed for this purpose. All of the thermocouples and LVDTs were attached to a data logger programmed to record readings on 15minute intervals.

After all preparations were complete, the chamber was closed, and the air temperature in the chamber was set to 29.3°F to begin the testing. The bath water was maintained at temperatures between 34.7 and 36.5°F by the use of heat tape triggered by temperature sensors in



Figure 3.28 Frost heave testing of untreated specimens.

the water bath. The chamber and water bath temperatures were held constant at these values for a period of 8 days, at which point the temperature in the chamber was dropped to 28.4°F to further cool the specimens. The chamber and water bath temperatures were held constant at these new values for a period of 22 days.

After a total of 30 days, the frost front had penetrated to nearly the bottoms of the specimens suspended above the water table in the PVC bases but had not yet begun to penetrate the specimens that were placed directly in the water bath. Therefore, the specimens suspended above the water table were removed from the bath, the thermocouples were detached, the circles of plastic sheeting and tape were removed, the PVC bases were detached, and the specimens were immediately weighed with and without the overburden weights, as was done prior to testing, to facilitate analysis of the ingress of moisture that occurred within the materials during testing. After removal of the overburden weights, a micrometer was again used to measure the heights of the specimens at the same four locations at which measurements were previously taken so that the amounts of frost heave could be calculated.

For continued testing of the specimens placed directly in the water bath, the three specimens that were removed were immediately replaced with three dummy specimens to prevent accelerated heat loss from the water bath. Each dummy specimen consisted of coarse sand placed in a standard plastic cylinder mold matching the first mold type previously described; thus, water uptake by the dummy specimens was not possible. To initiate freezing of the specimens that were placed directly in the water bath, the temperature in the chamber was then dropped to 26.6°F for 5 days, 24.8°F for 6 days, 23.0°F for 5 days, 21.2°F for 9 days, and finally to 19.4°F for the remaining 5 days of the testing period. After a total of 30 days, the frost front had penetrated to near or below the middles of the specimens.

Upon completion of the testing, the remaining six test specimens were removed from the bath, the thermocouples were detached, the circles of plastic sheeting and tape were removed, and the specimens were immediately weighed with and without the overburden weights as was done prior to testing. The bottom of each specimen was carefully dried before it was weighed. After removal of the overburden weights, a micrometer was again used to measure the heights of the specimens at the same four locations at which measurements were previously taken.

Immediately following weight and height measurements, moisture profiles for the specimens were determined to facilitate analysis of the redistribution of moisture that occurred within the materials during testing. For the analysis, each specimen was removed from its mold in five lifts of approximately equal volume, as shown in Figure 3.29. Because the top half of each specimen was frozen from testing, a pick was used to manually break apart the specimens. As shown in Figure 3.30, the collected materials were placed in pre-weighed bowls and then weighed before and after drying to constant weight in an oven maintained at 230°F so that gravimetric moisture contents could be computed. The moisture profiles of the specimens were then plotted for visual inspection.



Figure 3.29 Separation of a specimen into lifts for moisture profile determination.



Figure 3.30 Drying of specimens for moisture profile determination.

3.2.10.2 Frost Susceptibility of Untreated and Treated Materials

The second frost heave test was designed to evaluate the effect of water vapor movement on the frost heave characteristics of both untreated and treated specimens, including the ingress of moisture within the materials. For this testing, untreated specimens were prepared using the Redwood, McGuire, and 400 North materials, and treated specimens were prepared using the Redwood and McGuire materials, consistent with previous testing. For the treated specimens, either portland cement or calcium chloride was mixed into the material before compaction. When cement was used, it was consistently applied at a concentration of 3.0 percent by weight of dry aggregate for both materials, while calcium chloride was consistently applied at a concentration of 1.0 percent by weight of dry aggregate for both materials.

For each combination of material and treatment type, three replicate specimens were prepared, for a total of 21 specimens. Each specimen was compacted to a target height of nearly 8 in. inside a standard plastic mold having a 4-in. diameter and an 8-in. height. Before the specimens were compacted, the interior walls of the molds were oiled to minimize adfreezing of the specimen to the mold walls during testing. In addition, 0.375-in.-diameter holes were drilled in the bottom of the molds to allow water vapor movement upwards into the specimens, and a fine screen was placed inside the molds to minimize the migration of fines out of the specimens during testing.

Materials for all specimens were weighed out and compacted using the methods described in previous sections, except that specimens requiring modified compaction were created using 9 lifts with 25 blows per lift and specimens requiring standard compaction were created using 5 lifts with 25 blows per lift; the total number of blows per unit volume was consistent with the requirements stated in ASTM D1557 and D698 for modified and standard compaction, respectively. As previously described, the plastic molds were inserted into a metal mold to provide support during compaction. After compaction, all the specimens were sealed in plastic bags and allowed to equilibrate at approximately 70°F on the laboratory bench for a minimum of 28 days, which allowed the cement-treated specimens to substantially cure before testing. After the curing period, all specimens were placed in an oven at 140°F for a period of 2 days to reduce their water contents; drying in this manner decreased their water potentials and provided for increased water vapor movement during the testing and therefore better comparisons among the treatments.

As in the first frost heave test, the water bath was filled with deionized water to a depth of approximately 1.5 in., and the chamber temperature was set to 35.6°F for 24 hours prior to the start of the test. All of the testing equipment, as well as the specimens, which were still sealed in plastic bags, were also placed in the chamber to cool before testing.

Following the cooling period, the weight and height of each specimen were recorded. The height was measured in four locations equally spaced around the circumference of each specimen using a micrometer. A circle of thin plastic sheeting with a diameter of approximately 6 in. was then placed over the top of each specimen and securely taped around the full perimeter of the mold to seal it.

For this testing, the water bath that was used in the previous testing was covered by a newly designed acrylic sheet, shown in Figure 3.31, which was positioned just above the surface of the 2-in.-deep water bath. The sheet was fabricated with 25 holes for holding up to 25 specimens, as shown in Figure 3.32. An 8-in.-thick block of Styrofoam prepared with matching



Figure 3.31 Configuration for placement of untreated specimens above water bath.



Figure 3.32 Configuration of water bath for untreated and treated specimens.

holes was placed over the acrylic sheet, as shown in Figure 3.33, both to insulate the water bath and to provide lateral insulation around the specimens; this configuration ensured that heat flow out of the bath was substantially limited to that which occurred through the individual specimens. To suspend the specimens above the water bath, a base having a height of 2.5 in. and a diameter of 4 in. was cut from PVC pipe and positioned in the water at the bottom of each hole. To permit the flow of liquid water from the water bath under the specimens during testing, several holes were drilled into the base where it would be submerged in water. This configuration, illustrated in Figure 3.34, was specifically designed to investigate the amount of water ingress that occurs in freezing specimens in situations where water movement is limited to that which occurs in the form of water vapor.

When the specimens were ready for placement in the test apparatus, a thermocouple was attached to the bottom of most of the specimens before the specimens were pressed downwards into the holes in the Styrofoam block onto the prepared bases. A 4-in.-diameter acrylic disk was then placed on top of each specimen, over the plastic sheeting, and another thermocouple was then positioned to monitor the temperature of the top of the specimen through a hole drilled in the disk for that purpose, as shown in Figure 3.35. An LVDT was then positioned over the



Figure 3.33 Untreated and treated specimens prepared for frost heave testing.



Figure 3.34 Schematic of frost heave test configuration for untreated and treated specimens open to water vapor movement.



Figure 3.35 Untreated and treated specimens positioned above water bath.

center of the acrylic disk to monitor changes in specimen heights during testing; as shown in Figure 3.36, the LVDTs were again mounted to an aluminum frame designed for this purpose. All of the thermocouples and LVDTs were attached to a data logger programmed to record readings on 15-minute intervals. After all 21 specimens were installed, four dummy specimens were placed in the remaining holes of the bath.

After all preparations were complete, the chamber was closed, and the air temperature in the chamber was set to 26.6°F to begin the testing. The bath water was maintained at 35.6°F by the use of heat tape triggered by temperature sensors in the water bath. The chamber and water bath temperatures were held constant at these values for a period of 14 days, at which point the temperature in the chamber was dropped to 24.8°F for another 14 days and then to 23.0°F for the final 14 days of the test.

After a total of 6 weeks, the frost front had penetrated to nearly the bottoms of the specimens. Therefore, the specimens were removed one by one from the bath, the



Figure 3.36 Frost heave testing of untreated and treated materials.

thermocouples were detached, the circles of plastic sheeting and tape were removed, and the specimens were immediately weighed to facilitate analysis of the ingress of moisture that occurred within the materials during testing. Immediately before weighing, the bottom of each specimen was blotted with a paper towel to remove all water droplets, but care was taken not to disturb or remove any soil material during the process. A micrometer was again used to measure the heights of the specimens at the same four locations at which measurements were previously taken so that the amounts of frost heave could also be calculated.

3.2.11 Statistical Analysis

Statistical analyses were performed to investigate possible relationships between frost susceptibility data and other soil characteristics that were determined through laboratory testing. Specifically, the dependent variables of frost heave and water ingress, as applicable, were of interest in the analyses that were performed. The independent variables included material type, treatment type as applicable, percent of material passing the No. 200 sieve, percent of material finer than 0.02 mm, specific gravity, absorption, electrical conductivity, dry density, porosity, air-entry potential, pore-size distribution index, hydraulic conductivity, initial water content, and initial degree of saturation. In the analyses, a *p*-value, or level of significance, less than or equal to 0.15 indicated a statistically significant correlation. For significant correlations, plots displaying the relationships between the given variables were prepared for examination.

3.3 Field Testing

Field experimentation was conducted to measure the extent to which water vapor movement results in water accumulation in freezing base materials and to determine if the corresponding changes in water content are sufficient to cause frost heave during winter or reductions in stiffness during spring. The field testing was performed in 2009 and 2010 on a site located on 400 North at approximately 1400 West in Orem, Utah. At this location, the mean annual precipitation is 14 to 18 in., the mean annual air temperature is 42 to 50°F, the water table is 36 to 60 in. deep, and the frost-free period is 130 to 170 days in length according to the National Cooperative Soil Survey (*102*). The highest air temperature experienced at the site

during the testing period was 83.7°F, and the lowest temperature was 1.4°F. For Orem, Utah, the air freezing index is 1000°F-days, and the design frost depth is 30 in. (*103*).

Two experimental pavement sections were constructed during September 2009 in the shoulder of a minor arterial in Orem, Utah. They comprised an asphalt surface layer and a dense-graded base layer underlain by either an open-graded subbase or a dense-graded subbase, as shown in Figure 3.37. All the materials used in the construction of the pavement section with the capillary barrier were identical to the materials used in the pavement section without the capillary barrier, as well as the rest of the newly constructed road, with the exception of the open-graded subbase, which was brought in specifically for this experimentation. The sections were each 10 ft by 10 ft in horizontal extent. The section with the open-graded subbase, which functioned as a capillary barrier, was specifically designed to facilitate the movement of only water vapor into the base layer; therefore, during construction of that section, specific efforts



Figure 3.37 Schematic of experimental pavement sections.

were made to block all sources of liquid water from the base layer, including capillary rise, lateral seepage, and downward percolation.

The section involving construction of a capillary barrier required excavation down to the native subgrade soil. After the site was excavated as shown in Figure 3.38, the subgrade density and modulus were measured at four locations using a nuclear density gauge and at five locations using a soil stiffness gauge (SSG), respectively, as shown in Figure 3.39. Nuclear density tests were performed in general accordance with ASTM D6938 (Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth)), and SSG testing was performed in general accordance with ASTM D6758 (Standard Test Method for Measuring Stiffness and Apparent Modulus of Soil and Soil-Aggregate In-Place by Electro-Mechanical Method). Two subsurface probes, one that measured temperature and volumetric water content and another that measured water potential, were embedded 2 in. into the subgrade, as shown in Figures 3.40 and 3.41, at a total depth of 26 in. from the finished pavement surface; the sensors were placed approximately 1 ft apart. The geomembrane was then installed, reaching from the subgrade to the top of the base layer as shown in Figure 3.42, and the single vertical seam was sealed with industrial-strength tape to prevent lateral movement of water from the surrounding pavement into the experimental section. The lower geotextile was



Figure 3.38 Excavation of pavement section with capillary barrier down to subgrade.



Figure 3.39 Stiffness and density measurements of subgrade.



Figure 3.40 Placement of water potential sensors in subgrade.



Figure 3.41 Careful compaction of subgrade around subsurface sensors.



Figure 3.42 Placement of geomembrane around excavated area.

then placed directly onto the subgrade, and the capillary barrier, which was a layer of crushed, open-graded drainage rock as shown in Figure 3.43, was placed on top of the geotextile. After the open-graded drainage rock was manually leveled, which was done using hand tools as shown in Figure 3.44, the upper geotextile was installed, and the dense-graded base material was placed, as shown in Figures 3.45 and 3.46. The dense-graded base material was then compacted into place as depicted in Figures 3.47 and 3.48, and its density was also measured, as shown in Figure 3.49, at four locations. The upper and lower geotextiles provided filtering action against the dense-graded base material and the subgrade to prevent the migration of fines into the capillary barrier. Placement of the geomembrane and the layer of drainage rock was intended to isolate the base layer from all sources of liquid water so that any accumulation of moisture in the base layer could be attributable to water vapor movement.

Six additional subsurface probes, three that measure temperature and volumetric water content and three that measure water potential, were placed in the dense-graded base layer. The first set was placed 2 in. above the bottom of the base layer, for a total depth of 12 in. from the finished pavement surface, the second set was centered vertically in the middle of the base layer at a total depth of 10 in. from the pavement surface, and the final set of sensors was placed 2 in. below the top of the base layer at a total depth of 8 in. from the pavement surface. All of the sensors were approximately centered in the 10 ft by 10 ft section.



Figure 3.43 Placement of open-graded subbase layer on top of geotextile.



Figure 3.44 Leveling of open-graded drainage layer.



Figure 3.45 Placement of geotextile over open-graded drainage layer.



Figure 3.46 Placement of base material over geotextile.



Figure 3.47 Initial compaction of base material.



Figure 3.48 Final compaction of base material.



Figure 3.49 Stiffness and density measurements of base material.

The section without a capillary barrier did not require full excavation, as the pavement layers were already constructed using the desired materials, but trenches were necessarily dug to enable installation of the same sensors at the same locations as those that were installed in the section with a capillary barrier. Figures 3.50 and 3.51 illustrate the placement of the subsurface sensors in this pavement section. No geomembranes or geotextiles were utilized in construction of this section. All of the sensors were connected to a data logger mounted in a sealed metal box buried several feet behind the curb, as shown in Figure 3.52. The data logger was programmed to record readings on 15-minute intervals.

After all the trenches were backfilled, the site was cordoned off to protect the shallow subsurface sensors before paving, as shown in Figure 3.53, and the protruding upper edges of the geomembrane around the site with the capillary barrier were carefully tacked down to the surface of the compacted base, as shown in Figure 3.54, to ensure a good seal at the bottom of the asphalt. The density and modulus of the base layer were then measured at four locations using a nuclear density gauge and at five locations using an SSG, respectively. The road was then paved with 6 in. of hot mix asphalt, as shown in Figure 3.55, and opened to traffic on September 29, 2009. The asphalt mix design was obtained from the foreman in charge of the paving. As soon



Figure 3.50 Careful compaction of subgrade around subsurface sensors in pavement section without capillary barrier.



Figure 3.51 Placement of subsurface sensors in base material.



Figure 3.52 Placement of data logger in ground.



Figure 3.53 Marking of experimental pavement sections prior to paving.



Figure 3.54 Edge of geomembrane tacked down to base layer.



Figure 3.55 Placement of asphalt on experimental section.



Figure 3.56 Placement of joint sealant along edge of experimental section.

as the asphalt cooled sufficiently, self-leveling joint sealant was placed between the asphalt and the concrete curb, as shown in Figure 3.56, to prevent moisture from draining into the base material through that joint. The assumption at that point was that the base material was

effectively isolated from all liquid water sources and could only take on water through the upwards movement of water vapor from the subgrade through the open-graded drainage layer. On October 16, 2009, the surface and the middle of the asphalt layer were instrumented with thermocouples to enable temperature monitoring. For both thermocouple wires, a shallow groove extending approximately 1 ft into the road was sawn into the asphalt surface, as shown in Figure 3.57. To install the subsurface temperature sensor, a 0.125-in.-diameter hole was drilled to a depth of 3 in. at the end of the groove in the asphalt, and the tip of one thermocouple wire was positioned at the bottom of the hole. To install the surface temperature sensor, the tip of another thermocouple wire was situated above the hole at the surface of the asphalt. Both thermocouple wires were then routed through the groove, over the curb, and through a trench to the data logger, as shown in Figure 3.58. The wires were then permanently mounted to the pavement and curb using epoxy as shown in Figure 3.59. A temperature sensor was also placed



Figure 3.57 Preparation of site for instrumentation of asphalt layer with thermocouple wires.



Figure 3.58 Placement of thermocouple wires at site.



Figure 3.59 Application of adhesive over thermocouple wires.

in the air approximately 20 in. above the ground beyond the edge of the roadway, as shown in Figure 3.60.

After all the sensors were in place, the monitoring period began. This period lasted approximately 6 months and consisted of sensor data acquisition, elevation surveys that were performed every two weeks using a rod and level, and portable falling-weight deflectometer (PFWD) testing as described in the following sections specifically addressing temperature, liquid water content, matric potential, frost heave, and stiffness. Numerical modeling was also performed to simulate heat and water movement in the pavement sections.



Figure 3.60 Placement of thermocouple wire for measuring air temperature.

3.3.1 Temperature

The type of sensor selected for measuring temperatures in the subgrade and base layers of the two pavement sections utilizes thermistors to perform its function. The thermistor is embedded in the body of a multi-function probe with a three-prong design (*104*). As previously explained, thermocouples were selected for measuring temperatures in the asphalt layer and air. In this research, temperature measurements obtained from various locations were plotted against time and analyzed to determine frost penetration depth, for example, and also to aid in the interpretation of moisture content and matric potential data. The temperature histories at selected depths were also used as inputs in and for validating numerical models developed for each of the experimental pavement sections.

3.3.2 Liquid Water Content

The same multi-function probe selected for measuring subsurface temperatures was also used to measure the volumetric liquid water content of the subgrade and base layers. The probe incorporates a dielectric-type sensor with an interrogation zone coinciding with two of the three prongs on the probe (104). Because the dielectric value of ice is more like soil than liquid water (105), the sensor cannot detect water in solid form. Plotting the data and directly comparing the liquid water contents of the base layers immediately before the onset of winter and immediately after thawing was of primary interest in this research for determining how the presence of a capillary barrier influences the movement of water in a freezing pavement system.

3.3.3 Matric Potential

The sensor selected for measuring matric potential of the subgrade and base layers incorporates a ceramic disk to which the sensor is calibrated. When liquid water in the ceramic disk is in equilibrium with liquid water in the surrounding soil, the matric potential of the liquid water in the ceramic disk is equal to that of the liquid water in the soil (*104*). Because evaporation and freezing events, for example, both have the same effect on liquid water content, temperature data were also collected and consulted in this research to aid in interpretation of the matric potential data. In this research, the matric potential data were plotted and evaluated with respect to water movement in the experimental pavement sections.

3.3.4 Frost Heave

To measure the occurrence of any frost heave of the experimental pavement sections, elevation surveys were performed using a rod and level, as shown in Figure 3.61, every 2 weeks during the monitoring period. Changes in elevation of a selected point in the middle of each section relative to two fixed reference points established as benchmarks were the basis for computing frost heave of the pavement sections. The elevation data were plotted against time for evaluation in conjunction with the sensor data.



Figure 3.61 Surveying of experimental pavement sections.

3.3.5 Stiffness

In conjunction with and at the same locations as the elevation surveys, the stiffness of the individual pavement layers was measured using a PFWD as shown in Figure 3.62. Data were collected in general accordance with ASTM E2583 (Standard Test Method for Measuring Deflections with a Light Weight Deflectometer (LWD)). The PFWD consists of a 7.87-in.-diameter load plate, onto which a 44.1-lb weight is dropped from a height of 30 in., and three sensors for measuring pavement surface deflection at radial distances of 0, 12, and 24 in. from the center of the load plate. A seating load was applied before actual measurements were taken to ensure that the load plate was properly situated on the ETB surface. Three PFWD tests were



Figure 3.62 Portable falling-weight deflectometer testing at experimental pavement sections.

then immediately performed at the given location, and the plate load and deflections were all recorded for analysis.

Modulus values for the pavement layers were backcalculated from the PFWD measurements using BAKFAA, a linear elastic analysis program commonly used in pavement analysis (*106*). As inputs for all layers, a Poisson's ratio of 0.35 and full interface bonding were assumed for all backcalculations. The pavement layer thicknesses specified in the program were equal to the actual layer thicknesses in the field.

Backcalculations of layer modulus values were performed in two steps. In the first step, the subbase layer and subgrade were treated as a single layer, and the asphalt and base layers were treated as a single layer. The thickness of the subbase/subgrade layer thickness was semiinfinite by default, while the thickness of the asphalt/base layer was specified to be 14 in. Seed modulus values of 150 and 50 ksi were specified for the layers, and these values were both designated as changeable in the software. The modulus value of the subbase/subgrade layer resulting from this backcalculation was then held constant for use in the second step.

In the second step, with the subbase layer and subgrade still combined, the asphalt/base layer was redefined as two separate layers. The thicknesses of the individual asphalt and base layers were specified to be 6 and 8 in., respectively, and the corresponding seed modulus values were set at 500 and 50 ksi. The modulus values of both the asphalt and base layers were designated as changeable during the backcalculation, but the modulus of the subbase/subgrade layer was held constant at the previously determined value. The results of this second backcalculation were recorded for the individual layers.

For analyzing the stiffness of the two pavement sections through time, the modulus value for a given layer at a given time was divided by the initial modulus value of that layer corresponding to the beginning of the monitoring period. The modulus ratios were then plotted against time for inspection.

3.3.6 Numerical Modeling

The SHAW model was used to numerically simulate heat and water movement in the experimental pavement sections. The time period from October 17, 2009, to April 12, 2010, which was the full extent of the monitoring period, was simulated using data from both local and regional sources. Specifically, air temperature was measured at the site, while wind speed and humidity data were obtained from hourly observations recorded at a UDOT weather station located in Provo; because an asphalt layer was present at the top of the pavement section, minor variations in wind speed and humidity between the location of the weather station and the location of the field site were not expected to be practically important. Daily solar radiation was estimated based on cloud cover observations from the Salt Lake City Airport using the relation given by Flerchinger et al. (*107*). Precipitation was based on daily measurements from the Salt Lake City Airport and hourly measurements from UDOT weather stations in Provo and Orem.

Simulations were initialized with moisture contents and temperature profiles measured in the field on October 17. A unit gradient in soil water potential, i.e., gravity flow, was assumed at the depth of 24 in.; the lower temperature boundary was defined by observed temperature in the subgrade material at the 24-in. depth. Initial values of the soil hydraulic parameters K_s , θ_s , ψ_e , and b were based on laboratory measurements. Because the asphalt layer was newly placed and compacted, it was initially assumed to be impermeable, but field observations of water content in the base layer indicated an increase in water content in response to rainfall events. Calculations using the field-measured asphalt density of 128.7 pcf and the theoretical maximum specific gravity of 2.448 provided by the asphalt supplier revealed that, rather than having voids in the acceptable range of 3 to 8 percent (60), the asphalt actually had 16.2 percent voids, meaning it was improperly compacted during construction. Therefore, for the purposes of modeling, the asphalt layer was assigned a saturated hydraulic conductivity of 1.0 cm/hr to allow infiltration through the asphalt. Other assumed input values for the asphalt were 0.20 J/kg and 5 for the airentry value and pore-size index value (b), respectively, to ensure rapid drainage with low residual water content of the asphalt. Percentages of sand and silt, used primarily for soil thermal conductivity calculations, were set to 99 and 1 percent, respectively. Precipitation events measured at the Salt Lake City Airport and the UDOT weather stations in Provo and Orem were screened for rainfall events based on observed air temperature at the field site; precipitation was assumed to be rain, as opposed to snow, if the air temperature was above 0°C. Because snow was plowed off the road surface, snow events were not input into the model.

Figure 3.63 displays a schematic of the site characteristics that were used for each simulation. Air temperature was used as an upper boundary condition, and subgrade temperature was used as a lower boundary condition. Simulations for the pavement section with and without the capillary barrier contained information in layers 8 to 12 for the dense-graded and open-graded subbase material, respectively.





3.4 Summary

This research involved both laboratory and field testing of several materials. A natural aggregate obtained from a pavement rehabilitation project along Redwood Road near Bluffdale, Utah, and a crushed aggregate base material obtained from the McGuire Pit in Willard, Utah were specifically selected for treatment with Type I/II portland cement and calcium chloride dihydrate. Four additional materials were selected because of their use in construction of an experimental field site on 400 North at approximately 1400 West in Orem, Utah, including a crushed aggregate base material, a dense-graded subbase material, an open-graded subbase material, and the native subgrade.

In the laboratory, each of the materials was subjected to testing to determine gradation, Atterberg limits, soil classification, specific gravity and absorption, electrical conductivity, moisture-density relationship, SWCC, moisture-stiffness curve, and hydraulic conductivity. Frost susceptibility testing was performed on untreated Redwood, McGuire, and 400 North base materials in each of three mold types to investigate the amount of water ingress that occurs in
freezing specimens in the absence and presence of shallow water tables and in situations where water movement is limited to that which occurs in the form of water vapor. Additional frost susceptibility testing was performed to evaluate the effect of water vapor movement on the frost heave characteristics of both untreated and treated specimens, including the ingress of moisture within the materials. For this testing, untreated Redwood, McGuire, and 400 North base materials and treated Redwood and McGuire materials were evaluated; for the treated specimens, either portland cement or calcium chloride specimens were prepared at 3.0 or 1.0 percent concentrations by weight of dry aggregate. Statistical analyses were also performed to investigate possible relationships between frost susceptibility data and other soil characteristics that were determined through laboratory testing.

Field experimentation was conducted to measure the extent to which water vapor movement results in water accumulation in freezing base materials and to determine if the corresponding changes in water content are sufficient to cause frost heave during winter or reductions in stiffness during spring. The field testing included the construction of two experimental pavement sections during September 2009 in the shoulder of a minor arterial in Orem, Utah. They comprised an asphalt surface layer and a dense-graded base layer underlain by either an open-graded subbase or a dense-graded subbase. Density and modulus for the subgrade and dense-graded base layer were measured in the field. Subsurface probes for measuring temperature, volumetric water content, and water potential were embedded in the subgrade and at three depths in the base layer of each section. Geotextiles were placed directly under and directly above the open-graded subbase layer, providing filtering action against the subgrade and the dense-graded base material, respectively. A geomembrane was placed around the dense-graded base and open-graded subbase layers in the section with the capillary barrier to isolate the base layer from all sources of liquid water. During the monitoring period, subsurface measurements of temperature, liquid water content, and matric potential were taken, along with surface measurements of frost heave and stiffness. The SHAW model was then used to numerically simulate heat and water movement in the experimental pavement sections. The discovery that the asphalt was leaking made use of the SHAW model even more invaluable, as it became the only means of quantifying water vapor flux in the pavement system.

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4.0 RESULTS

4.1 Overview

The results of the laboratory and field testing are presented in the following sections for the Redwood, McGuire, and 400 North materials. The findings are specific to the materials tested and may therefore not be applicable to other material types. A hyphen in a table indicates that the given value was not measured or was not applicable.

4.2 Laboratory Testing

The following sections present the results of the laboratory procedures applied to each of the materials, including the determination of gradation, Atterberg limits, soil classification, specific gravity and absorption, electrical conductivity, moisture-density relationship, SWCC, moisture-stiffness curve, hydraulic conductivity, and frost susceptibility. Statistical analyses were also performed to investigate correlations between frost susceptibility data and other laboratory test results.

4.2.1 Gradation

The results of particle-size analyses for the Redwood and McGuire materials are shown in Figures 4.1 and 4.2, respectively, while the results of particle-size analyses for the 400 North dense-graded base, dense-graded subbase, open-graded subbase, and subgrade materials are shown in Figures 4.3 through 4.6, respectively. The results of both sieve and hydrometer analyses are presented in each figure. While the particle-size distributions for the 400 North open-graded subbase and subgrade were each noticeably different from the others, the particlesize distributions for the Redwood, McGuire, and 400 North dense-graded base and subbase were very similar.

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Figure 4.1 Particle-size distribution for Redwood material.



Figure 4.2 Particle-size distribution for McGuire material.



Figure 4.3 Particle-size distribution for 400 North dense-graded base.



Figure 4.4 Particle-size distribution for 400 North dense-graded subbase.



Figure 4.5 Particle-size distribution for 400 North open-graded subbase.



Figure 4.6 Particle-size distribution for 400 North subgrade.

4.2.2 Atterberg Limits

In the determination of Atterberg limits, plastic limit testing showed that all of the materials were non-plastic; therefore, no liquid limit testing was performed.

4.2.3 Soil Classification

The AASHTO, Unified, and frost classifications for each of the tested materials are shown in Table 4.1. In the AASHTO classification system, all of the materials except the 400 North subgrade were classified the same, while the Unified and frost classification systems exhibited greater discrimination. In particular, the frost classification ranged from not frost susceptible to frost and thaw-weakening susceptible.

Material		AASHTO Classification	Unified Classification	Frost Classification
Redwood		A-1-a (0)	SP-SM (poorly graded sand with silt)	S2 (low to medium frost susceptibility)
McGuire		A-1-a (0)	GP (poorly graded gravel)	PFS (possibly frost susceptible)
	Dense-Graded Base	A-1-a (0)	SW-SM (well-graded sand with silt and gravel)	F2 (b) (frost and thaw- weakening susceptible)
400 North	Dense-Graded Subbase	A-1-a (0)	SW-SM (well-graded sand with silt and gravel)	F1 (frost susceptible)
	Open-Graded Subbase	A-1-a (0)	GP (poorly graded gravel)	NFS (not frost susceptible)
	Subgrade	A-3 (0)	SP-SM (poorly graded sand with silt)	S2 (low to medium frost susceptibility)

Table 4.1 Soil Classifications

4.2.4 Specific Gravity and Absorption

Table 4.2 shows the apparent specific gravity and absorption computed for each material in the untreated and treated condition, as applicable. Absorption was calculated only for materials in the untreated condition, as treatment with portland cement or calcium chloride was assumed not to affect the absorption.

	Material	Apparent Specific Gravity	Absorption (%)
	Untreated	2.58	
Redwood	Cement-Treated	2.59	3.3
	Salt-Treated	2.58	
	Untreated	2.69	
McGuire	Cement-Treated	2.70	1.2
	Salt-Treated	2.69	
	Dense-Graded Base	2.68	1.9
400 North	Dense-Graded Subbase	2.66	1.5
	Open-Graded Subbase	2.67	1.1
	Subgrade	2.63	1.1

Table 4.2 Apparent Specific Gravity and Absorption

4.2.5 Electrical Conductivity

The average electrical conductivity of each aggregate base material and treatment combination is shown in Table 4.3. The untreated samples had the lowest electrical conductivity, while the salt-treated samples had the highest electrical conductivity. Increases in the electrical conductivity of the cement- and salt-treated samples compared to the untreated samples result from the introduction of ions to the aggregate base material through cement hydration and salt dissolution, respectively.

	Electrical Conductivity (µS/cm)	
	Untreated	295
Redwood	Cement-Treated	1187
	Salt-Treated	2153
	Untreated	62
McGuire	Cement-Treated	1199
	Salt-Treated	2093
400 North	Dense-Graded Base	94

Table 4.3 Electrical Conductivity

4.2.6 Moisture-Density Relationship

Moisture-density curves for the Redwood and McGuire materials in both the untreated and treated condition are shown in Figures 4.7 and 4.8, respectively. In both cases, the cement treatment increased the OMC value while the salt treatment decreased the OMC value relative to the OMC for the untreated material. Adding either cement or salt to the Redwood material resulted in a higher MDD than the untreated material, but such a trend was not apparent with the McGuire material.



Figure 4.7 Moisture-density curve for Redwood material.



Figure 4.8 Moisture-density curve for McGuire material.

Moisture-density curves for the 400 North dense-graded base, dense-graded subbase, and subgrade are shown in Figures 4.9 through 4.11, respectively. The dense-graded base and subbase materials had similar OMC and MDD values, which were in both cases higher than the OMC and MDD values for the subgrade material. A moisture-density curve was not prepared for the 400 North open-graded subbase material; instead, the OMC value was assigned to be the absorption value for that material. A summary of the OMC and MDD values for all the materials is given in Table 4.4.



Figure 4.9 Moisture-density curve for 400 North dense-graded base.



Figure 4.10 Moisture-density curve for 400 North dense-graded subbase.



Figure 4.11 Moisture-density curve for 400 North subgrade.

Ma	aterial	Optimum Moisture Content (%)	Maximum Dry Density (pcf)
	Untreated	9.5	126.0
Redwood	Cement-Treated	9.8	128.5
	Salt-Treated	9.1	128.1
	Untreated	5.8	142.2
McGuire	Cement-Treated	6.5	141.5
	Salt-Treated	5.5	142.1
	Dense-Graded Base	8.5	136.0
400 N 4	Dense-Graded Subbase	8.0	135.5
400 North	Open-Graded Subbase	1.1	96.6
	Subgrade	5.8	105.3

 Table 4.4 Optimum Moisture Content and Maximum Dry Density

4.2.7 Soil-Water Characteristic

The SWCCs resulting from water potential testing of the Redwood, McGuire, and 400 North materials are shown in Figures 4.12 through 4.17, and the pore-size distribution index *b* and the air-entry potential value Ψ_E computed for each of the materials are displayed in Table 4.5. For each material, the difference between the total potential and the matric potential can be attributed to the osmotic potential.

The figures showing SWCCs for the Redwood and McGuire materials demonstrate the effects of cement and salt treatments on water potential. While the cement-treated specimens



Figure 4.12 Soil-water characteristic curves for total potential of Redwood material.



Figure 4.13 Soil-water characteristic curves for matric potential of Redwood material.



Figure 4.14 Soil-water characteristic curves for total potential of McGuire material.



Figure 4.15 Soil-water characteristic curves for matric potential of McGuire material.



Figure 4.16 Soil-water characteristic curves for total potential of 400 North materials.



Figure 4.17 Soil-water characteristic curves for matric potential of 400 North materials.

had different SWCCs than the untreated specimens for both total and matric potentials, the salttreated specimens were different than the untreated specimens mainly with respect to total potential; the matric potentials for the salt-treated specimens were very similar to those for the untreated specimens. Regarding the 400 North materials, the dense-graded base and subbase materials were also very similar.

		Total Potential		Matric Potential	
Material		Pore-Size Distribution Index (ln(kPa))/ (-ln(S))	Air-Entry Potential (kPa)	Pore-Size Distribution Index (ln(kPa))/ (-ln(S))	Air-Entry Potential (kPa)
	Untreated	2.57	-1540.9113	6.75	-7.0106
Redwood	Cement-Treated	9.61	-126.1421	11.46	-44.7030
	Salt-Treated	1.97	-16752.1194	8.48	-1.6197
McGuire	Untreated	1.26	-986.0222	5.29	-0.4185
	Cement-Treated	2.08	-1399.9047	5.75	-5.4028
	Salt-Treated	2.02	-17185.2888	4.22	-2.0719
	Dense-Graded Base	1.48	-250.6383	5.12	-0.2043
400 North	Dense-Graded Subbase	1.21	-421.5261	4.48	-0.6873
	Open-Graded Subbase	4.32	-0.0103	4.36	-0.0003
	Subgrade	1.65	-84.1958	6.07	-0.0011

Table 4.5 Soil-Water Characteristic Curve Parameters

4.2.8 Moisture-Stiffness Relationship

Moisture-stiffness curves for the Redwood, McGuire, and 400 North materials are shown in Figures 4.18 through 4.20, respectively, in which the highest moisture content evaluated generally corresponded to the respective OMC value. Tabular results for this testing are available in Appendix A. In most cases, the materials exhibited a peak modulus value within the range of moisture contents tested. The occurrence of a peak suggests that drying below the OMC value may initially yield an increase in stiffness, while further drying may lead to a subsequent decrease. Among the Redwood and McGuire materials, the peaks were most



Figure 4.18 Moisture-stiffness curves for Redwood material.



Figure 4.19 Moisture-stiffness curves for McGuire material.



Figure 4.20 Moisture-stiffness curves for 400 North materials.

pronounced in the cement-treated specimens, while among the 400 North materials the peaks were most pronounced in the dense-graded base and subbase materials. In all cases, the stiffness associated with the lowest moisture content evaluated in the testing was higher than the stiffness associated with the highest moisture content.

4.2.9 Hydraulic Conductivity

The results of the hydraulic conductivity testing are displayed in Table 4.6. For the Redwood material, treatment with either cement or salt increased the hydraulic conductivity, while for the McGuire material treatment with cement decreased the hydraulic conductivity compared to the values measured for the untreated and salt-treated specimens, which were very similar. Among the 400 North materials, the dense-graded base and subbase materials had much

Material		Specimen	Dry Density (lb/ft ³)	Hydraulic Conductivity (ft/day)
		1	124.5	0.003
	Untreated	2	124.2	0.001
		3	124.9	0.007
	Comont	1	122.9	0.020
Redwood	Treated	2	124.1	0.009
	ITeated	3	124.4	0.007
		1	125.7	0.135
	Salt-Treated	2	123.4	0.152
		3	124.3	0.439
		1	138.4	0.031
	Untreated	2	138.9	0.053
		3	137.9	0.110
	Cement- Treated	1	141.4	0.002
McGuire		2	142.1	0.005
		3	141.0	0.001
	Salt-Treated	1	139.4	0.062
		2	139.1	0.063
		3	139.9	0.069
	Dansa Cradad	1	136.0	0.433
	Dense-Graded	2	135.1	0.718
	Dase	3	135.6	0.288
	Dansa Cradad	1	131.6	0.085
	Subbase	2	132.1	0.177
100 North	Subbase	3	132.1	0.141
400 Notui	Open Greded	1	95.1	5714
	Subbase	2	95.1	5271
	Subbase	3	96.6	5266
		1	105.1	13.34
	Subgrade	2	104.8	12.38
		3	104.6	11.55

Table 4.6 Hydraulic Conductivity

lower hydraulic conductivity values than the subgrade and open-graded base materials. The open-graded base material was especially permeable due to its lack of fines.

4.2.10 Frost Susceptibility

The following sections give the results of the two frost susceptibility tests performed in the laboratory, one including only untreated specimens and the other including both untreated and treated specimens.

4.2.10.1 Frost Susceptibility of Untreated Materials

The results of frost susceptibility testing on the untreated materials, which focused on evaluating the effects of moisture availability on the frost heave characteristics of untreated specimens, including the ingress and/or redistribution of moisture within the materials, are presented in Figures 4.21 through 4.23 and Tables 4.7 and 4.8. Temperature profiles and thermal gradients for each of the instrumented specimens through time are shown in Appendix A. During this testing, the heat tape in the water bath did not turn on for the first 30 days of testing because the heat generated by the water pumps in the water bath prohibited cooling of the water bath to the trigger level under the initial freezing conditions.

Figures 4.21 through 4.23 show the amounts of frost heave experienced by the specimens placed directly in the water bath and closed to water ingress, placed directly in the water bath and open to capillary rise, and suspended above the water bath and open to water vapor movement, respectively. Shrinkage of the McGuire and 400 North aggregate base materials was observed in all of the mold configurations. This shrinkage can be attributed to thermal contraction upon cooling associated with the coefficient of thermal expansion and also to decreases in matric potential arising from higher surface tension at the air-water interfaces. The specimens suspended above the water bath also exhibited net shrinkage, suggesting that the amounts of incoming water vapor were not sufficient in this case to cause heaving of the materials. The flat line segments in Figure 4.23 correspond to a temporary failure of the data acquisition equipment.

Concerning the specimens that were placed directly in the water bath, which were exposed to lower chamber air temperatures than those to which the specimens suspended above the water bath were exposed, measurable frost heave was exhibited by the Redwood material in both cases. In the specimen without the possibility of water ingress, the frost heave was due exclusively to the redistribution of water in the specimen. The redistribution of water was most



Figure 4.21 Heave of specimens closed to water ingress.



Figure 4.22 Heave of specimens open to capillary rise.



Figure 4.23 Heave of specimens open to water vapor movement.

Material		Apparent Heave (in.)	Thermal Contraction (in.)	Actual Heave (in.)
	Closed	0.119	0.012	0.131
Redwood	Open	0.235	0.009	0.244
	Vapor	-0.014	0.016	0.002
	Closed	-0.003	0.006	0.003
McGuire	Open	-0.009	0.008	-0.001
	Vapor	-0.184	0.017	-0.167
	Closed	-0.007	0.005	-0.002
400 North	Open	-0.005	0.007	0.002
	Vapor	-0.013	0.016	0.003

Table 4.7 Heave of Untreated Specimens

Ma	Average Thermal Gradient (°F/in.)	
	Closed	0.52
Redwood	Open	0.56
	Vapor	0.20
	Closed	-
McGuire	Open	0.42
	Vapor	0.17
	Closed	0.39
400 North	Open	-
	Vapor	0.18

 Table 4.8 Thermal Gradient in Untreated Specimens

pronounced at the highest thermal gradient imposed on the specimen, which occurred at the end of the testing after the specimens suspended above the water table had already been removed. The Redwood specimen suspended above the water bath may have yielded similar results if a higher thermal gradient could have been applied.

As an analysis of the data in Figures 4.21 through 4.23, Table 4.7 shows computed values for the apparent heave, thermal contraction, and actual heave of each specimen. The apparent heave was calculated as the total change in height relative to the initial height, measured immediately before freezing, while the thermal contraction was calculated as the decrease in height relative to the initial height associated with thermal contraction upon initial cooling. The actual heave was calculated as the subsequent increase in height that occurred during testing relative to the thermally contracted height, which was the shortest height recorded during testing. Table 4.8 gives the average thermal gradients over the full duration of testing for those specimens that were instrumented with thermocouples.

Figures 4.24 through 4.26 show the moisture profiles measured after frost susceptibility testing of the Redwood, McGuire, and 400 North materials, respectively. For the Redwood



Figure 4.24 Moisture profiles for Redwood specimens.



Figure 4.25 Moisture profiles for McGuire specimens.



Figure 4.26 Moisture profiles for 400 North specimens.

material, the moisture profiles of the specimens placed directly into the water bath were very similar, showing an obvious upwards redistribution of water. Specifically, these specimens had higher moisture contents near the top where moisture was accumulating in the vicinity of the freezing front and lower water contents in the middle where moisture was leaving faster than it was being replenished. The specimen that was open to capillary rise exhibited a slightly higher moisture content overall than the specimen that was closed. The moisture profile of the specimen that was suspended above the water bath exhibited only a slight increase in water content towards the top of the specimen.

The moisture profiles for the McGuire specimens did not indicate any significant water movement, although the specimen that was open to capillary rise had a slightly higher moisture content through the middle than the other two specimens, and the specimen that was suspended over the water bath exhibited a slightly higher moisture content than the specimen that was closed to water ingress.

For the 400 North base material, the moisture profiles of the specimen that was closed to water ingress and the specimen that was suspended above the water bath were very similar,

although the latter specimen exhibited a slight increase in water content compared to the former. The specimen that was open to capillary rise had a higher moisture content than the other two specimens, however, suggesting that, for a longer test time, it may have experienced frost heave in this condition.

4.2.10.2 Frost Susceptibility of Untreated and Treated Materials

The results of frost susceptibility testing on the untreated and treated materials, which focused on evaluating the effects of cement and salt treatments on water vapor movement during freezing, are presented in Figures 4.27 through 4.33 and Tables 4.9 and 4.10. Temperature profiles and thermal gradients for each of the instrumented specimens through time are shown in Appendix A. During this testing, the heat tape in the water bath did not turn on until approximately 30 days into the test because the heat generated by the water pumps in the water bath prohibited cooling of the water bath to the trigger level. Figures 4.27 through 4.33 and Table 4.9 show the amounts of frost heave experienced by the specimens. In every case, the specimens exhibited net shrinkage, and, as manifest in the figures by frequent abrupt deviations in the data, the small changes in height approached the limit of resolution of the LVDTs that were used for data collection. As explained previously, the shrinkage can be attributed to thermal contraction upon cooling and also to decreases in matric potential, although the latter



Figure 4.27 Heave of untreated Redwood specimens.



Figure 4.28 Heave of cement-treated Redwood specimens.



Figure 4.29 Heave of salt-treated Redwood specimens.



Figure 4.30 Heave of untreated McGuire specimens.



Figure 4.31 Heave of cement-treated McGuire specimens.



Figure 4.32 Heave of salt-treated McGuire specimens.



Figure 4.33 Heave of 400 North dense-graded base specimens.

effect may have been less pronounced in these specimens as a result of the much lower water contents at which the specimens were tested. While all of the specimens experienced reductions in height during testing, the data show that selected untreated and salt-treated Redwood specimens and salt-treated McGuire specimens were experiencing slow rates of frost heaving by the end of the testing. Over a longer testing time, these specimens may have eventually heaved above their original heights; however, in this testing, the heaving did not exceed the thermal contraction experienced by the specimens.

Combined with the binding effect of especially the cement in selected specimens, the smaller size of these specimens compared to those in the first frost susceptibility test made creation of moisture profiles more difficult; instead, the total change in water content that occurred during freezing is displayed in Table 4.11. The moisture contents of all the specimens increased during testing. Because the specimens were suspended over the water bath, the increase in moisture content can be attributed solely to water vapor movement.

The treatments had different effects on each material. For the Redwood material, specimens treated with calcium chloride experienced the largest increase in moisture content, while those treated with cement experienced the smallest increase in moisture content. For the McGuire material, specimens treated with cement experienced the largest increase in moisture content, while the untreated specimens experienced the smallest increase in moisture content. One likely cause for these inconsistencies is the specific interaction occurring between the treatments and the materials themselves, in which gradation and mineralogy most likely play a role, and another possible cause pertains to the amount of cement or calcium chloride that was added. For example, the McGuire material may require a higher percentage of cement than the Redwood material to slow or stop the movement of water vapor.

While the measured increase in moisture contents may not have been sufficient to markedly decrease the stiffness of the specimens, continuing accumulation of moisture over a longer period would be expected to have detrimental effects. For example, if the laboratory testing period had been extended to 3 months, which is the typical length of a winter season in Utah, the water content would have been expected to increase from 1.5 to 3.1 percent, depending on the material, at the same thermal gradient of approximately 0.75°F/in., and those changes in water content could be sufficient to cause frost heave and reductions in stiffness.

Material		Specimen	Apparent Heave	Thermal Contraction	Actual Heave
		1	-0.008	0.008	0.000
	Untreated	2	-0.004	0.008	0.004
		3	-0.011	0.011	0.000
	G	1	-0.009	0.009	0.000
Redwood	Cement- Treated	2	-0.006	0.009	0.003
		3	-0.01	0.010	0.000
	<u> </u>	1	-0.005	0.008	0.003
	Salt- Treated	2	-0.007	0.007	0.000
		3	-0.007	0.010	0.003
	Untreated	1	-0.007	0.007	0.000
		2	-0.01	0.010	0.000
		3	-0.012	0.012	0.000
	Cement- Treated	1	-0.011	0.011	0.000
McGuire		2	-0.012	0.012	0.000
		3	-0.011	0.011	0.000
	Salt- Treated	1	-0.007	0.008	0.001
		2	-0.002	0.007	0.005
		3	-0.013	0.013	0.000
		1	-0.006	0.006	0.000
400 North	Untreated	2	-0.011	0.011	0.000
		3	-0.007	0.009	0.002

Table 4.9 Heave of Untreated and Treated Specimens

Mate	rial	Specimen	Average Thermal Gradient (°F/in.)
		1	0.73
	Untreated	2	0.71
		3	0.76
		1	0.78
Redwood	Cement- Treated	2	0.73
		3	0.77
		1	0.74
	Salt-Treated	2	0.69
		3	0.66
		1	0.74
	Untreated	2	0.74
		3	0.69
	Cement- Treated	1	0.70
McGuire		2	0.68
		3	0.75
		1	0.71
	Salt-Treated	2	0.59
		3	0.68
		1	0.66
400 N	Untreated	2	0.65
		3	0.70

 Table 4.10 Thermal Gradient in Untreated and Treated Specimens

Material		Specimen	Change in Water Content	Average Change in Water Content (%)
		1	1.22	(,,,,,
	Untreated	2	1.31	1.3
		3	1.28	
	Comont	1	1.07	
Redwood	Treated	2	1.08	1.1
		3	1.10	
		1	1.67	
	Salt-Treated	2	1.48	1.6
		3	1.51	
	Untreated	1	0.97	
		2	1.07	1.0
		3	1.08	
	Cement- Treated	1	1.22	
McGuire		2	1.33	1.3
		3	1.29	
		1	1.16	
	Salt-Treated	2	1.15	1.1
		3	1.13	
100 North		1	0.72	
Base	Untreated	2	0.75	0.7
Dase		3	0.74	

 Table 4.11 Change in Water Content of Untreated and Treated Specimens

4.2.11 Statistical Analysis

Statistical analyses were performed in this research to investigate possible relationships between frost susceptibility data and other soil characteristics determined in laboratory testing. The intent of the analyses was not to develop regression equations for general application but only to explore relationships between the sets of data measured in this research. Analyses were first performed on the data collected during frost susceptibility testing of untreated materials, which focused on evaluating the effects of moisture availability on the frost heave characteristics of untreated specimens, including the ingress and/or redistribution of moisture within the materials. Specifically, the dependent variable of frost heave was of interest in the correlations that were performed, and it was examined in terms of apparent heave, thermal contraction, and actual heave. Each of these three measurements was examined for each of the three water-availability conditions included in the experimentation.

For the specimens placed directly in the water bath and closed to water ingress, the independent variables that were determined in the statistical analyses to be correlated to at least one of the dependent variables included apparent specific gravity, initial water content, pore-size distribution index, air-entry potential, and electrical conductivity. As illustrated in Figures 4.34 to 4.48, an increase in specific gravity correlated with a decrease in apparent heave, actual heave, and thermal contraction; an increase in initial moisture content correlated with an increase in apparent heave, actual heave, and thermal contraction; an increase in apparent heave, actual heave, and thermal contraction; an increase in apparent heave, actual heave, and thermal contraction; an increase in apparent heave, actual heave, and thermal contraction; and an increase in electrical conductivity correlated with an increase in apparent heave, actual heave, and thermal contraction; and an increase in electrical conductivity correlated with an increase in apparent heave, actual heave, and thermal contraction; and an increase in electrical conductivity correlated with an increase in apparent heave, actual heave, and thermal contraction; and an increase in electrical conductivity correlated with an increase in apparent heave, actual heave, and thermal contraction.



Figure 4.34 Correlation between apparent heave and specific gravity for untreated specimens closed to water ingress.



Figure 4.35 Correlation between thermal contraction and specific gravity for untreated specimens closed to water ingress.



Figure 4.36 Correlation between actual heave and specific gravity for untreated specimens closed to water ingress.


Figure 4.37 Correlation between apparent heave and initial moisture content for untreated specimens closed to water ingress.



Figure 4.38 Correlation between thermal contraction and initial moisture content for untreated specimens closed to water ingress.



Figure 4.39 Correlation between actual heave and initial moisture content for untreated specimens closed to water ingress.



Figure 4.40 Correlation between apparent heave and pore-size distribution index for untreated specimens closed to water ingress.



Figure 4.41 Correlation between thermal contraction and pore-size distribution index for untreated specimens closed to water ingress.



Figure 4.42 Correlation between actual heave and pore-size distribution index for untreated specimens closed to water ingress.



Figure 4.43 Correlation between apparent heave and air-entry potential for untreated specimens closed to water ingress.



Figure 4.44 Correlation between thermal contraction and air-entry potential for untreated specimens closed to water ingress.



Figure 4.45 Correlation between actual heave and air-entry potential for untreated specimens closed to water ingress.



Figure 4.46 Correlation between apparent heave and electrical conductivity for untreated specimens closed to water ingress.



Figure 4.47 Correlation between thermal contraction and electrical conductivity for untreated specimens closed to water ingress.



Figure 4.48 Correlation between actual heave and electrical conductivity for untreated specimens closed to water ingress.

For the specimens placed directly in the water bath and open to capillary rise, the independent variables that were determined in the statistical analysis to be correlated to at least one of the dependent variables were apparent specific gravity, degree of saturation, pore-size distribution index, air-entry potential, and electrical conductivity. As illustrated in Figures 4.49 to 4.58, an increase in specific gravity correlated with a decrease in apparent heave and actual heave; an increase in degree of saturation correlated with an increase in apparent heave and actual heave; an increase in the pore-size distribution index correlated with an increase in apparent heave and actual heave; an increase in air-entry potential correlated with a decrease in apparent heave and actual heave; an increase in air-entry potential correlated with a decrease in apparent heave and actual heave; and an increase in electrical conductivity correlated with an increase in apparent heave and actual heave; and an increase in electrical conductivity correlated with an increase in apparent heave and actual heave; and an increase in electrical conductivity correlated with an increase in apparent heave and actual heave; and an increase in electrical conductivity correlated with an increase in apparent heave and actual heave; and actual heave

For the specimens suspended over the water bath and open to water vapor movement, the independent variables that were determined in the statistical analysis to be correlated to at least one of the dependent variables were percent of material finer than the No. 200 sieve and porosity. As illustrated in Figures 4.59 to 4.64, an increase in the percent of material finer than the No. 200 sieve correlated with an increase in apparent heave and actual heave and a decrease in thermal contraction, while an increase in porosity correlated with an increase in apparent heave and actual heave and a decrease in thermal contraction.



Figure 4.49 Correlation between apparent heave and specific gravity for untreated specimens open to capillary rise.



Figure 4.50 Correlation between actual heave and specific gravity for untreated specimens open to capillary rise.



Figure 4.51 Correlation between apparent heave and initial degree of saturation for untreated specimens open to capillary rise.



Figure 4.52 Correlation between actual heave and initial degree of saturation for untreated specimens open to capillary rise.



Figure 4.53 Correlation between apparent heave and pore-size distribution index for untreated specimens open to capillary rise.



Figure 4.54 Correlation between actual heave and pore-size distribution index for untreated specimens open to capillary rise.



Figure 4.55 Correlation between apparent heave and air-entry potential for untreated specimens open to capillary rise.



Figure 4.56 Correlation between actual heave and air-entry potential for untreated specimens open to capillary rise.



Figure 4.57 Correlation between apparent heave and electrical conductivity for untreated specimens open to capillary rise.



Figure 4.58 Correlation between actual heave and electrical conductivity for untreated specimens open to capillary rise.



Figure 4.59 Correlation between apparent heave and percentage of material finer than No. 200 sieve for untreated specimens open to water vapor movement.



Figure 4.60 Correlation between thermal contraction and percent of material finer than No. 200 sieve for untreated specimens open to water vapor movement.



Figure 4.61 Correlation between actual heave and percent of material finer than No. 200 sieve for untreated specimens open to water vapor movement.



Figure 4.62 Correlation between apparent heave and porosity for untreated specimens open to water vapor movement.



Figure 4.63 Correlation between thermal contraction and porosity for untreated specimens open to water vapor movement.



Figure 4.64 Correlation between actual heave and porosity for untreated specimens open to water vapor movement.

Statistical analyses were also performed on the data collected during frost susceptibility testing of untreated and treated materials, which focused on evaluating the effect of water vapor movement on frost heave characteristics of both untreated and treated specimens. Specifically, the dependent variables of frost heave and water ingress were of interest in the correlations that were performed. Frost heave was examined in the same three ways previously mentioned, and water ingress was computed as the percentage of water by weight of dry aggregate that entered a given specimen during testing.

For the untreated and treated specimens suspended above the water bath and open to water vapor movement, the independent variables that were determined in the statistical analysis to be correlated to at least one of the dependent variables were dry density, percent of material finer than the No. 200 sieve, percent of material finer than 0.02 mm, apparent specific gravity, absorption, initial water content, porosity, degree of saturation, hydraulic conductivity, and electrical conductivity. As illustrated in Figures 4.65 to 4.78, an increase in dry density correlated with a decrease in apparent heave and an increase in thermal contraction; an increase in thermal contraction; an increase in the percent of material finer than 0.02 mm correlated with a decrease in the percent of material finer than 0.02 mm correlated with a decrease in thermal contraction; an increase in specific gravity correlated with a decrease in apparent

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Figure 4.65 Correlation between apparent heave and dry density for untreated and treated specimens open to water vapor movement.



Figure 4.66 Correlation between thermal contraction and dry density for untreated and treated specimens open to water vapor movement.



Figure 4.67 Correlation between thermal contraction and percent of material finer than No. 200 sieve for untreated and treated specimens open to water vapor movement.



Figure 4.68 Correlation between thermal contraction and percent of material finer than 0.02 mm for untreated and treated specimens open to water vapor movement.



Figure 4.69 Correlation between apparent heave and specific gravity for untreated and treated specimens open to water vapor movement.



Figure 4.70 Correlation between water ingress and specific gravity for untreated and treated specimens open to water vapor movement.



Figure 4.71 Correlation between water ingress and absorption for untreated and treated specimens open to water vapor movement.



Figure 4.72 Correlation between water ingress and initial water content for untreated and treated specimens open to water vapor movement.



Figure 4.73 Correlation between water ingress and porosity for untreated and treated specimens open to water vapor movement.



Figure 4.74 Correlation between thermal contraction and porosity for untreated and treated specimens open to water vapor movement.



Figure 4.75 Correlation between water ingress and initial degree of saturation for untreated and treated specimens open to water vapor movement.



Figure 4.76 Correlation between water ingress and hydraulic conductivity for untreated and treated specimens open to water vapor movement.



Figure 4.77 Correlation between water ingress and electrical conductivity for untreated and treated specimens open to water vapor movement.



Figure 4.78 Correlation between actual heave and electrical conductivity for untreated and treated specimens open to water vapor movement.

heave and total water ingress; an increase in absorption correlated with an increase in total water ingress; an increase in initial water content correlated with an increase in total water ingress; an increase in porosity correlated with a decrease in total water ingress and thermal contraction; an increase in degree of saturation correlated with an increase in total water ingress; an increase in hydraulic conductivity correlated with a decrease in total water ingress; and an increase in electrical conductivity correlated with an increase in total water ingress and an increase in actual heave.

4.3 Field Testing

Field testing involved measuring several material properties during and after construction of the experimental pavement sections. Table 4.12 shows that the Young's modulus and dry density values were similar for the two pavement sections for the layers shared by both. Additional data are presented in the following sections, including temperature, moisture content,

Location	Young's Modulus (ksi)	Dry Density (lb/ft ³)
Subgrade		
Capillary Barrier	9.0	101.8
Dense-Graded Base		
Capillary Barrier	12.0	129.1
No Capillary Barrier	11.5	128.3
Asphalt		
Capillary Barrier	-	128.7
No Capillary Barrier	-	127.3

Table 4.12 Modulus and Dry Density Measured during Construction

water potential, elevation, and stiffness data. Numerical modeling was also performed on data collected from the subsurface sensors.

4.3.1 Temperature

Measured temperatures from the top and middle of the asphalt of the pavement section with a capillary barrier are shown in Figures 4.79 and 4.80. Temperature sensors were not placed in the asphalt layer of the pavement section without a capillary barrier, so only one set of asphalt temperatures is shown. Comparisons of measured subsurface temperatures from the sections with and without a capillary barrier are shown in Figures 4.81 through 4.84. The temperature data reflect both daily and seasonal fluctuations that occurred during the monitoring period, with a gradual decrease in fluctuation with increasing depth. During winter, although the base layer experienced temperatures at or below 32°F from about December 3 (elapsed day 47) to January 21 (elapsed day 96), the subgrade at this site never experienced freezing temperatures. The subgrade below the capillary barrier, suggesting that the capillary barrier acted as an insulating layer.



Figure 4.79 Measured temperature at top of asphalt.



Figure 4.80 Measured temperature at middle of asphalt.



Figure 4.81 Measured temperature at top of base.



Figure 4.82 Measured temperature at middle of base.



Figure 4.83 Measured temperature at bottom of base.



Figure 4.84 Measured temperature at subgrade.

4.3.2 Liquid Water Content

Measured volumetric liquid water contents for the pavement sections with and without a capillary barrier are displayed in Figures 4.85 through 4.88. Overall, water contents in the base layer of the pavement section without a capillary barrier were consistently lower than water



Figure 4.85 Measured volumetric liquid water content at top of base.



Figure 4.86 Measured volumetric liquid water content at middle of base.



Figure 4.87 Measured volumetric liquid water content at bottom of base.



Figure 4.88 Measured volumetric liquid water content at subgrade.

contents in the base layer of the pavement section with a capillary barrier. Because the moisture sensors detect only liquid water, the base material appears to have reduced moisture contents during the freezing period when part of the water changed to ice. After the freezing period, the water contents of both the base and subgrade materials in both pavement sections increased compared to the water contents recorded just prior to the onset of freezing temperatures.

Specifically, the water contents at the top, middle, and bottom of the base layer of the pavement section with the capillary barrier were 20, 1, and 14 percent higher after thawing on January 21 (elapsed day 96) than before freezing on December 3 (elapsed day 47), respectively, increasing from 0.204 to 0.245, 0.226 to 0.227, and 0.202 to 0.229. The subgrade material of the same section experienced an increase in water content from 0.106 to 0.153 between January 18 and 21 (elapsed days 93 and 96), presumably in response to downward flux of melt water or precipitation. Water contents at the top, middle, and bottom of the base layer of the pavement section without a capillary barrier were 20, 21, and 8 percent higher after thawing on January 21 (elapsed day 96) than before freezing on December 3 (elapsed day 47), respectively, increasing from 0.151 to 0.182, 0.173 to 0.209, and 0.158 to 0.171. The subgrade material of the same section experienced an increase in water content from 0.133 to 0.189 between January 18 and 21 (elapsed days 93 and 96). Numerical modeling was performed to determine the likely source of the additional water that must have entered the base layer during the freezing period.

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4.3.3 Matric Potential

The matric water potentials measured in the sections with and without a capillary barrier are displayed in Figures 4.89 to 4.92. The plots show that, when the temperature of the subsurface pavement layers dropped enough to cause freezing of the pore water, water potential in the layers decreased dramatically. The largest decrease in water potential for the pavement section without the capillary barrier occurred in the beginning of December, when the coldest temperatures were experienced. This decrease in water potential is reasonable due to the fact that water potential decreases with decreasing liquid water content for a given soil, where the liquid water content may decrease from drying or from freezing. The measured matric potential may be excessively low during periods of freezing, however, due to the formation of ice in the ceramic disk within the matric water potential sensor. The pavement section with the capillary barrier did not experience the same reaction to the initial drop in temperature but experienced a relatively constant decrease in water potential throughout the winter season. Towards the end of January, when temperatures began steadily increasing again, the water potential in both pavement sections increased to practically negligible values because all the frozen water in the pavement section once again became liquid.



Figure 4.89 Measured matric potential at top of base.



Figure 4.90 Measured matric potential at middle of base.



Figure 4.91 Measured matric potential at bottom of base.



Figure 4.92 Measured matric potential at subgrade.

4.3.4 Frost Heave

The results of the elevation surveys performed to measure frost heave of the pavement sections are shown in Figure 4.93, and tabular data are provided in Appendix A. The data indicate that both pavement sections experienced thermal contraction during the cooling period that happened in the middle of November. The section of pavement with the capillary barrier subsequently exhibited 0.48 in. of apparent heave over the winter season, while the section of pavement without the capillary barrier remained at a relatively constant elevation, exhibiting only about 0.06 in. of apparent heave throughout the monitoring period. During the thawing period, both pavement sections experienced thaw subsidence, but the pavement section without the capillary barrier during the freezing period. The pavement section without the capillary barrier then gradually returned to its original elevation, possibly due to bulking in the fines of the dense-graded subbase caused by the slow drainage of melt water from the layer.



Figure 4.93 Frost heave of experimental pavement sections.

4.3.5 Stiffness

The results of the BAKFAA analysis performed on the PFWD data are shown in Figures 4.94 and 4.95 for the sections with and without the capillary barrier, respectively. The moduli of the asphalt and base layers increased significantly during the winter season, corresponding with the freezing period experienced at the site. Modulus ratios were computed to help analyze the changes in stiffness of the two pavement sections through time, where the modulus value for a given layer at a given time was divided by the initial modulus value of that layer corresponding to the beginning of the monitoring period, and the results are shown in Figures 4.96 and 4.97 for the sections with and without the capillary barrier, respectively. Although the pavement section with the capillary barrier had higher modulus values overall, both sections exhibited similar modulus ratios. Tabular data for both the Young's modulus values and modulus ratios are provided in Appendix A.

While the elevation data indicated the occurrence of thaw subsidence at the two pavement sections, the PFWD data provided no clear evidence of thaw weakening at either site during the spring thaw. Possible reasons may be that any thaw weakening was of short duration and occurred between site visits, the increase in water content during spring was not sufficient to cause a measurable reduction in stiffness, and/or the pavement sections were not susceptible to thaw weakening, with the newly constructed pavement having sufficient structural capacity to minimize PFWD deflections even during spring.



Figure 4.94 Modulus values for pavement section with capillary barrier.



Figure 4.95 Modulus values for pavement section without capillary barrier.



Figure 4.96 Modulus ratios for pavement section with capillary barrier.



Figure 4.97 Modulus ratios for pavement section without capillary barrier.

4.3.6 Numerical Modeling

The numerical modeling relied on the results of many of the laboratory tests previously reported. The specific inputs for the SHAW model are shown in Table 4.13. The air-entry potential of the base material was adjusted based on measured soil temperature and liquid water content using the approach outlined by Flerchinger et al. (*95*) so that computed liquid water contents using the soil hydraulic parameters more closely matched the measured values. This procedure resulted in the air-entry potential of the base material being decreased from -0.2 to - 2.1 J/kg, which was a reasonable compromise for both pavement sections. For comparison, the results of the simulation performed using only measured data, with no adjustments, are provided in Appendix B.

As a check of the validity of the SHAW model, the measured data and the simulated data are plotted together. Comparisons of measured and simulated temperature data for the sections with and without the capillary barrier are shown in Figures 4.98 to 4.109, respectively. Because the measured subgrade temperature was an input used for the lower boundary condition in the
Property	Asphalt	Dense- Graded Base	Dense- Graded Subbase	Open- Graded Subbase	Subgrade
Apparent Specific Gravity	2.63	2.68	2.66	2.67	2.63
Dry Density of In-Situ Layer (pcf)	128.7	128.7	-	_	101.8
Porosity of In-Situ Layer (%)	21.4	23.1	-	-	38.0
Maximum Dry Density (pcf)	-	-	135.5	96.6	-
Porosity Calculated Using Maximum Dry Density (%)	-	-	21.2	42.1	-
Saturated Hydraulic Conductivity (ft/day)	-	0.4790	0.1342	5417	12.19
Air-Entry Value (J/kg)	-	-2.06084	-0.68730	-0.00026	-0.00111
Index Value b	-	5.12	4.48	4.36	6.07
Sand (%)	99.00	88.15	90.97	98.48	94.92
Silt (%)	1.00	11.85	9.03	1.52	5.08

Table 4.13 Inputs for Numerical Modeling

model, the measured and simulated subgrade temperatures are identical. Comparisons of measured and simulated liquid water content data for the sections with and without the capillary barrier are shown in Figures 4.110 to 4.117, respectively, while comparisons of measured and simulated water potential data are shown in Figures 4.118 to 4.125, respectively. These figures show the ability of the SHAW model to satisfactorily simulate temperature, water potential, and liquid water content fluctuations over time at varying depths in a freezing pavement system.



Figure 4.98 Temperature at top of asphalt for pavement section with capillary barrier and permeable asphalt.



Figure 4.99 Temperature at middle of asphalt for pavement section with capillary barrier and permeable asphalt.



Figure 4.100 Temperature at top of base for pavement section with capillary barrier and permeable asphalt.



Figure 4.101 Temperature at middle of base for pavement section with capillary barrier and permeable asphalt.



Figure 4.102 Temperature at bottom of base for pavement section with capillary barrier and permeable asphalt.



Figure 4.103 Temperature at subgrade for pavement section with capillary barrier and permeable asphalt.



Figure 4.104 Temperature at top of asphalt for pavement section without capillary barrier and with permeable asphalt.



Figure 4.105 Temperature at middle of asphalt for pavement section without capillary barrier and with permeable asphalt.



Figure 4.106 Temperature at top of base for pavement section without capillary barrier and with permeable asphalt.



Figure 4.107 Temperature at middle of base for pavement section without capillary barrier and with permeable asphalt.



Figure 4.108 Temperature at bottom of base for pavement section without capillary barrier and with permeable asphalt.



Figure 4.109 Temperature at subgrade for pavement section without capillary barrier and with permeable asphalt.



Figure 4.110 Volumetric liquid water content at top of base for pavement section with capillary barrier and permeable asphalt.



Figure 4.111 Volumetric liquid water content at middle of base for pavement section with capillary barrier and permeable asphalt.



Figure 4.112 Volumetric liquid water content at bottom of base for pavement section with capillary barrier and permeable asphalt.



Figure 4.113 Volumetric liquid water content at subgrade for pavement section with capillary barrier and permeable asphalt.



Figure 4.114 Volumetric liquid water content at top of base for pavement section without capillary barrier and with permeable asphalt.



Figure 4.115 Volumetric liquid water content at middle of base for pavement section without capillary barrier and with permeable asphalt.



Figure 4.116 Volumetric liquid water content at bottom of base for pavement section without capillary barrier and with permeable asphalt.



Figure 4.117 Volumetric liquid water content at subgrade for pavement section without capillary barrier and with permeable asphalt.



Figure 4.118 Matric potential at top of base for pavement section with capillary barrier and permeable asphalt.



Figure 4.119 Matric potential at middle of base for pavement section with capillary barrier and permeable asphalt.



Figure 4.120 Matric potential at bottom of base for pavement section with capillary barrier and permeable asphalt.



Figure 4.121 Matric potential at subgrade for pavement section with capillary barrier and permeable asphalt.



Figure 4.122 Matric potential at top of base for pavement section without capillary barrier and with permeable asphalt.



Figure 4.123 Matric potential at middle of base for pavement section without capillary barrier and with permeable asphalt.



Figure 4.124 Matric potential at bottom of base for pavement section without capillary barrier and with permeable asphalt.



Figure 4.125 Matric potential at subgrade for pavement section without capillary barrier and with permeable asphalt.

The middles of the base layers in the sections with and without the capillary barrier were quite moist (0.213 and 0.193, respectively) at the start of the simulation. The comparatively high air-entry potential of the capillary barrier prevented water from draining from the base material in that section, thus causing the moisture content of the base layer to continue to increase. The unsaturated hydraulic conductivity of the capillary barrier is essentially zero until the water potential nears the air-entry potential of the capillary barrier (-0.00026 J/kg). However, with an air-entry potential of -2.1 J/kg, the base material is saturated long before it reaches a water potential comparable to the air-entry potential of the capillary barrier. Thus, ironically, the extremely coarse capillary barrier actually inhibits drainage from the base layer until the base layer nearly reaches positive pore pressures. The section without the capillary barrier had a temporary increase in moisture content, but hydraulic contact with the dense-graded subbase allowed the dense-graded base to drain more efficiently.

Based on the simulation, Figure 4.126 shows how water vapor flux varied with changes in the thermal gradient in the middle of the capillary barrier. Positive vapor flux numbers indicate water vapor flowing from the base layer into the capillary barrier, while negative vapor flux numbers indicate water vapor flowing from the capillary barrier into the base layer. A positive thermal gradient indicates that the top of the pavement was colder than the deeper layers, while a negative thermal gradient indicates that the top of the pavement was warmer than the deeper layers. The trend shown in Figure 4.126 indicates that, as the temperature at the bottom of the base layer decreased, water vapor was drawn up through the capillary barrier. Thus, during winter, cold temperatures create an ideal environment for water vapor to travel upward from the warm subgrade soil below the frost line, through the capillary barrier, and into the base material, possibly contributing to frost heave. According to this relationship, if this base material were to experience a sustained thermal gradient of 1°F/in. for a period of 3 months, which is the length of a typical winter in northern Utah, it would experience an increase in water content of approximately 1 percent. While 1 percent is not a large increase, a base material already at or above OMC could experience measurable reductions in stiffness as a result of this additional moisture.

The total simulated volumetric water contents of the top, middle, and bottom of the base layer and the subgrade are shown in Figure 4.127 for the pavement section with a capillary

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Figure 4.126 Simulated water vapor flux in capillary barrier as affected by thermal gradient for pavement section with permeable asphalt.



Figure 4.127 Simulated total volumetric water contents of base and subgrade in pavement section with capillary barrier and permeable asphalt.

barrier; in the figure, the total volumetric water content includes both liquid water and ice. The total volumetric water content of the bottom of the base material was higher than that of the middle and top of the base material during non-freezing periods, with a decrease in the water content of the bottom of the base during the freezing period that corresponds to an increase in the water contents of the top and middle of the base material. All depths within the base layer exhibited an increase in moisture content corresponding to thawing of the base layer and a rain event that occurred on January 21 (elapsed day 96), at which point the middle and bottom of the base layer became saturated. These depths remained saturated or nearly saturated for the remainder of the simulation, and the top of the base layer eventually became saturated as well.

The simulated dynamics of water flow at the top and bottom of the base layer for the pavement section with a capillary barrier are presented in Figure 4.128, where negative numbers indicate an upward movement of water. Vapor flux into the bottom of the base layer was steady but accumulated only 0.060 in., a 0.24 percent increase in moisture, over the course of the simulation. The cause of this flux was the combined temperature and water potential gradients between the base layer and capillary barrier. Only 0.03 in., a 0.12 percent increase in moisture, of water vapor accumulated in the 45 days prior to the onset of freezing temperatures. The cumulative liquid water flux at the top of the base layer suggests steady evaporation of water out of the top of the base layer for the majority of the simulation period, interrupted by brief periods of downward liquid water flow coinciding with the onset of thawing and rain events. Drainage of water from the bottom of the base layer was negligible, suggesting that all moisture entering the base through vapor flow or rain events was being trapped in the base layer.

The total simulated volumetric water contents of the top, middle, and bottom of the base layer and the subgrade are shown in Figure 4.129 for the pavement section without a capillary barrier. Again, in the figure, the total volumetric water content includes both liquid water and ice. The total volumetric water contents of the pavement layers in this section fluctuated less than those within the pavement section with a capillary barrier, and no depths within the base layer ever became completely saturated. The subgrade in this pavement section accumulated significantly more water than the pavement section with the capillary barrier, which essentially had no change in water content throughout the simulation, reinforcing the idea that the capillary barrier essentially trapped moisture in the base layer.

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section with capillary barrier and permeable asphalt.



Figure 4.129 Simulated total volumetric water contents of base and subgrade in pavement section without capillary barrier and with permeable asphalt.

The simulated dynamics of water flow at the top and bottom of the base layer for the pavement section without a capillary barrier are presented in Figure 4.130, where negative numbers again indicate an upward movement of water. Vapor flow at the top and bottom of the base layer was negligible, even though the pavement sections were not completely saturated for the majority of the simulation period. Cumulative liquid water flux at the top and bottom of the base layer were very similar and showed dramatic downward flow during spring thaw and with every rainfall event, including the major one that occurred on January 21 (elapsed day 96) and the series of rainfall events that started on February 20 (elapsed day 126). Unlike the pavement section with a capillary barrier, this pavement section experienced capillary rise from the dense-graded subbase into the dense-graded base.

Although the SHAW model does not simulate expansion of the soil matrix, it does allow the total water content of a layer to exceed the porosity due to expansion of the water upon freezing. Figure 4.131 shows the predicted heave of the base layer in the pavement section with



Figure 4.130 Simulated cumulative water fluxes at top and bottom of base in pavement section without capillary barrier and with permeable asphalt.



Figure 4.131 Predicted heave based on simulated liquid water and ice contents in pavement section with capillary barrier.

a capillary barrier based on simulated liquid water and ice contents; a maximum heave of 0.05 in. was predicted. No heave was predicted for the pavement section without a capillary barrier. While the predicted heave does not exactly match the measured heave, the pavement section with a capillary barrier was predicted to heave more than the section without a capillary barrier, which does match the observed trends. Greater heave in the field than that predicted in the numerical modeling would be expected due to entrapment of air in the base layer during freezing, which is a phenomenon that has been previously documented in the literature (*108*).

To investigate the behavior of the pavement sections with a properly compacted asphalt layer, an additional simulation was performed. Inputs for the SHAW model all remained the same except for the hydraulic conductivity of the asphalt, which was changed to 0. The original simulated data and the new simulated data were plotted together to reveal what most likely would have occurred in the layers of the pavement system had the asphalt been compacted properly. The results of the simulations comparing the effects of impermeable and permeable asphalt are presented in Figures 4.132 to 4.175, which include temperature, volumetric liquid water content, matric potential, total volumetric water content, and cumulative water fluxes for the pavement sections with and without a capillary barrier.



Figure 4.132 Simulated temperatures at top of asphalt for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.133 Simulated temperatures at middle of asphalt for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.134 Simulated temperatures at top of base for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.135 Simulated temperatures at middle of base for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.136 Simulated temperatures at bottom of base for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.137 Simulated temperatures at subgrade for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.138 Simulated temperatures at top of asphalt for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.139 Simulated temperatures at middle of asphalt for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.140 Simulated temperatures at top of base for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.141 Simulated temperatures at middle of base for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.142 Simulated temperatures at bottom of base for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.143 Simulated temperatures at subgrade for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.144 Simulated volumetric liquid water contents at top of base for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.145 Simulated volumetric liquid water contents at middle of base for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.146 Simulated volumetric liquid water contents at bottom of base for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.147 Simulated volumetric liquid water contents at subgrade for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.148 Simulated volumetric liquid water contents at top of base for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.149 Simulated volumetric liquid water contents at middle of base for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.150 Simulated volumetric liquid water contents at bottom of base for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.151 Simulated volumetric liquid water contents at subgrade for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.152 Simulated matric potentials at top of base for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.153 Simulated matric potentials at middle of base for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.154 Simulated matric potentials at bottom of base for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.155 Simulated matric potentials at subgrade for pavement section with capillary barrier and impermeable or permeable asphalt.



Figure 4.156 Simulated matric potentials at top of base for pavement section without capillary barrier and impermeable or permeable asphalt.



Figure 4.157 Simulated matric potentials at middle of base for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.158 Simulated matric potentials at bottom of base for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.159 Simulated matric potentials at subgrade for pavement section without capillary barrier and with impermeable or permeable asphalt.



Figure 4.160 Simulated total volumetric water contents at top of base for pavement sections with capillary barrier.



Figure 4.161 Simulated total volumetric water contents at middle of base for pavement sections with capillary barrier.


Figure 4.162 Simulated total volumetric water contents at bottom of base for pavement sections with capillary barrier.



Figure 4.163 Simulated total volumetric water contents at subgrade for pavement sections with capillary barrier.



Figure 4.164 Simulated total volumetric water contents at top of base for pavement sections without capillary barrier.



Figure 4.165 Simulated total volumetric water contents at middle of base for pavement sections without capillary barrier.



Figure 4.166 Simulated total volumetric water contents at bottom of base for pavement sections without capillary barrier.



Figure 4.167 Simulated total volumetric water contents at subgrade for pavement sections without capillary barrier.



Figure 4.168 Simulated cumulative water vapor fluxes at top of base in pavement section with capillary barrier.



Figure 4.169 Simulated cumulative water vapor fluxes at bottom of base in pavement section with capillary barrier.



Figure 4.170 Simulated cumulative liquid water fluxes at top of base in pavement section with capillary barrier.



Figure 4.171 Simulated cumulative liquid water fluxes at bottom of base in pavement section with capillary barrier.



Figure 4.172 Simulated cumulative water vapor fluxes at top of base in pavement section without capillary barrier.



Figure 4.173 Simulated cumulative water vapor fluxes at bottom of base in pavement section without capillary barrier.



Figure 4.174 Simulated cumulative liquid water fluxes at top of base in pavement section without capillary barrier.



Figure 4.175 Simulated cumulative liquid water fluxes at bottom of base in pavement section without capillary barrier.

Several important differences between impermeable and permeable asphalt are apparent in the figures. While the simulated temperatures remain mostly unchanged in the base layer and subgrade, the asphalt pavement surface temperatures are generally more variable for the impermeable asphalt than the permeable asphalt for both types of pavement sections; that is, when the asphalt is assumed to be impermeable, the predicted daily high temperatures are higher and the low temperatures are lower than those associated with permeable asphalt. Although the effect is most pronounced in the pavement section with a capillary barrier, both pavement sections experience increases in the volumetric liquid water content and total volumetric water content of the base layer during the period leading up to the onset of freezing temperatures when the asphalt is assumed to be impermeable. However, during the thawing period, both the volumetric liquid water content and the total volumetric water content of the base layer are lower when the asphalt is assumed to be impermeable instead of permeable. The effect of the impermeable asphalt is clearly to reduce the occurrence of water evaporation out of the top of the base layer during fall while also protecting the base layer from downward infiltration of water during spring rain events, for example. A comparison of the subgrade water contents shows that the effect of the impermeable asphalt is much more apparent in the pavement section without a capillary barrier, in which the water content over time is much more stable when the asphalt is assumed to be impermeable instead of permeable. In the pavement section with a capillary barrier, the subgrade water content remains unchanged when the asphalt is assumed to be impermeable, and it is also substantially lower than the water content of the subgrade in the pavement section without a capillary barrier; indeed, as evidenced by the fact that the bottom of the base layer in the pavement section with a capillary barrier remains saturated for the majority of the simulation period, water is unable to drain out of the base layer into the capillary barrier, which reinforces the idea that the capillary barrier essentially traps moisture in the base layer. The matric potential data also support these observations.

No apparent differences between impermeable asphalt and permeable asphalt are evident with respect to the simulated cumulative water vapor fluxes at the top or bottom of the base layer; only at the bottom of the base layer within the pavement section with a capillary barrier did any water vapor flux occur, and it was not influenced by the permeability of the asphalt layer. However, the simulated cumulative liquid water fluxes are strongly influenced by the asphalt permeability. When the asphalt is assumed to be impermeable, the cumulative liquid

water fluxes at the top of the base layer in the pavement section with a capillary barrier and at the top and bottom of the base layer in the pavement section without a capillary barrier are greatly reduced, with only limited upward movements of water occurring in conjunction with freezing events during the winter months; the cumulative liquid water fluxes at the bottom of the base layer in the pavement section with a capillary barrier were negligible for both impermeable asphalt and permeable asphalt.

4.4 Summary

The results of the laboratory and field testing performed on the Redwood, McGuire, and 400 North materials are specific to the materials tested and may therefore not be applicable to other material types. The results of the laboratory testing, which included gradations, Atterberg limits, soil classifications, specific gravity and absorption values, electrical conductivity values, moisture-density relationships, SWCCs, moisture-stiffness curves, hydraulic conductivity values, and frost susceptibility assessments, were used to characterize each material and enable subsequent statistical analyses. Testing of both untreated and treated materials enabled investigation of a wide variety of material properties. The results of the field testing, which included temperature, moisture content, water potential, elevation, and stiffness data over time, provided the basis for comparing pavement sections with and without capillary barriers and established the framework for numerical modeling.

Laboratory frost susceptibility testing of the untreated materials focused on evaluating the effects of moisture availability on the frost heave characteristics of untreated specimens, including the ingress and/or redistribution of moisture within the materials. The results of this testing showed that shrinkage of the McGuire and 400 North aggregate base materials was observed in all of the mold configurations. This shrinkage can be attributed to thermal contraction upon cooling associated with the coefficient of thermal expansion and also to decreases in matric potential arising from higher surface tension at the air-water interfaces. The specimens suspended above the water bath also exhibited net shrinkage, suggesting that the amounts of incoming water vapor were not sufficient in this case to cause heaving of the materials. Concerning the specimens that were placed directly in the water bath, which were exposed to lower chamber air temperatures than those to which the specimens suspended above

the water bath were exposed, measurable frost heave was exhibited by the Redwood material in both cases. In the specimen without the possibility of water ingress, the frost heave was due exclusively to the redistribution of water in the specimen. The redistribution of water was most pronounced at the highest thermal gradient imposed on the specimen, which occurred at the end of the testing after the specimens suspended above the water table had already been removed. The Redwood specimen suspended above the water bath may have yielded similar results if a higher thermal gradient could have been applied.

Laboratory frost susceptibility testing of the untreated and treated materials focused on evaluating the effects of cement and salt treatments on water vapor movement during freezing. While all of the specimens evaluated in this testing experienced reductions in height during testing, the data show that selected untreated and salt-treated Redwood specimens and salttreated McGuire specimens were experiencing slow rates of frost heaving by the end of the testing. Over a longer testing time, these specimens may have eventually heaved above their original heights; however, in this testing, the heaving did not exceed the thermal contraction experienced by the specimens. The moisture contents of all the specimens increased during testing. Because the specimens were suspended over the water bath, the increase in moisture content can be attributed solely to water vapor movement. While the measured increase in moisture contents may not have been sufficient to markedly decrease the stiffness of the specimens, continuing accumulation of moisture over a longer period would be expected to have detrimental effects. For example, if the laboratory testing period had been extended to 3 months, which is the typical length of a winter season in Utah, the water content would have been expected to increase from 1.5 to 3.1 percent, depending on the material, at the same thermal gradient of approximately 0.75°F/in., and those changes in water content could be sufficient to cause frost heave and reductions in stiffness.

The applied treatments had different effects on the frost susceptibility of each material. For the Redwood material, specimens treated with calcium chloride experienced the largest increase in moisture content, while those treated with cement experienced the smallest increase in moisture content. For the McGuire material, specimens treated with cement experienced the largest increase in moisture content, while the untreated specimens experienced the smallest increase in moisture content. One likely cause for these inconsistencies is the specific interaction

occurring between the treatments and the materials themselves, in which gradation and mineralogy most likely play a role, and another possible cause pertains to the amount of cement or calcium chloride that was added. For example, the McGuire material may require a higher percentage of cement than the Redwood material to slow or stop the movement of water vapor.

The statistical analyses performed in this research investigated possible relationships between frost susceptibility data and other soil characteristics determined in laboratory testing. The dependent variables of particular interest included frost heave and water ingress, where frost heave was examined in terms of apparent heave, thermal contraction, and actual heave. For the specimens placed directly in the water bath and closed to water ingress, the independent variables that were determined in the statistical analyses to be correlated to at least one of the dependent variables included apparent specific gravity, initial water content, pore-size distribution index, air-entry potential, and electrical conductivity. For the specimens placed directly in the water bath and open to capillary rise, the independent variables that were determined in the statistical analysis to be correlated to at least one of the dependent variables were apparent specific gravity, degree of saturation, pore-size distribution index, air-entry potential, and electrical conductivity. For the untreated specimens suspended over the water bath and open to water vapor movement, the independent variables that were determined in the statistical analysis to be correlated to at least one of the dependent variables were percent of material finer than the No. 200 sieve and porosity. For the untreated and treated specimens suspended above the water bath and open to water vapor movement, the independent variables that were determined in the statistical analysis to be correlated to at least one of the dependent variables were dry density, percent of material finer than the No. 200 sieve, percent of material finer than 0.02 mm, apparent specific gravity, absorption, initial water content, porosity, degree of saturation, hydraulic conductivity, and electrical conductivity.

Survey data collected at the field site indicate that both experimental pavement sections experienced thermal contraction during the cooling period that happened in the middle of November. The section of pavement with the capillary barrier subsequently exhibited 0.48 in. of heave over the winter season, while the section of pavement without the capillary barrier remained at a relatively constant elevation, exhibiting only about 0.06 in. of heave throughout

the monitoring period. During the thawing period, both pavement sections experienced thaw subsidence.

While the elevation data indicated the occurrence of thaw subsidence at the two pavement sections, the PFWD data provided no clear evidence of thaw weakening at either site during the spring thaw. Possible reasons may be that any thaw weakening was of short duration and occurred between site visits, the increase in water content during spring was not sufficient to cause a measurable reduction in stiffness, and/or the pavement sections were not susceptible to thaw weakening, with the newly constructed pavement having sufficient structural capacity to minimize PFWD deflections even during spring.

The measured field data plotted together with the results of numerical modeling showed the ability of the SHAW model to satisfactorily simulate temperature, water potential, and liquid water content fluctuations over time at varying depths in a freezing pavement system. The comparatively high air-entry potential of the capillary barrier prevented water from draining from the base material in that section, thus causing the moisture content of the base layer to continue to increase. Thus, the extremely coarse capillary barrier actually inhibited drainage from the base layer until the base layer nearly reached positive pore pressures. Examination of how water vapor flux varied with changes in the thermal gradient in the middle of the capillary barrier revealed that, during winter, cold temperatures create an ideal environment for water vapor to travel upward from the warm subgrade soil below the frost line, through the capillary barrier, and into the base material, possibly contributing to frost heave. If the material were to experience a sustained thermal gradient of 1°F/in. for a period of 3 months, it would experience an increase in water content of approximately 1 percent. Although the frost heave predicted using the SHAW model did not exactly match the measured heave, the pavement section with a capillary barrier was predicted to heave more than the section without a capillary barrier, which did match the observed trends.

To investigate the behavior of the pavement sections with a properly compacted asphalt layer, an additional simulation was performed. Although the effect was most pronounced in the pavement section with a capillary barrier, both pavement sections experienced increases in the volumetric liquid water content and total volumetric water content of the base layer during the period leading up to the onset of freezing temperatures when the asphalt was assumed to be impermeable. However, during the thawing period, both the volumetric liquid water content and the total volumetric water content of the base layer were lower when the asphalt was assumed to be impermeable instead of permeable. The effect of the impermeable asphalt was clearly to reduce the occurrence of water evaporation out of the top of the base layer during fall while also protecting the base layer from downward infiltration of water during spring rain events, for example. In the pavement section with a capillary barrier, the subgrade water content remained unchanged when the asphalt was assumed to be impermeable, and it was also substantially lower than the water content of the subgrade in the pavement section without a capillary barrier; indeed, as evidenced by the fact that the bottom of the base layer in the pavement section with a capillary barrier remained saturated for the majority of the simulation period, water was unable to drain out of the base layer into the capillary barrier, which reinforces the idea that the capillary barrier essentially trapped moisture in the base layer.

Simulated cumulative liquid water fluxes were strongly influenced by the asphalt permeability. When the asphalt was assumed to be impermeable, the cumulative liquid water fluxes at the top of the base layer in the pavement section with a capillary barrier and at the top and bottom of the base layer in the pavement section without a capillary barrier were greatly reduced, with only limited upward movements of water occurring in conjunction with freezing events during the winter months; the cumulative liquid water fluxes at the bottom of the base layer in the pavement section with a capillary barrier were negligible for both impermeable asphalt and permeable asphalt.

5.0 CONCLUSION

5.1 Summary

Six objectives were developed for this research:

- 1. Measure the extent to which water vapor movement results in water accumulation in freezing base materials.
- 2. Evaluate the effect of soil stabilization on water vapor movement in freezing base materials.
- 3. Determine if the corresponding changes in water content are sufficient to cause frost heave during winter.
- 4. Determine if the corresponding changes in water content are sufficient to cause reductions in stiffness during spring.
- 5. Evaluate relationships between selected material properties, freezing conditions, and the occurrence and impact of water vapor movement.
- 6. Numerically simulate heat and water movement in selected pavement design scenarios.

The research involved extensive laboratory and field testing, statistical analyses, and numerical modeling.

5.2 Conclusions

Several conclusions corresponding to the research objectives can be derived from this work. In a pavement section with a capillary barrier underlying the base layer, water vapor movement from the subgrade through the capillary barrier may be expected to increase the water content of the base layer by 1 to 3 percent during a typical winter season in northern Utah for base materials similar to those studied in this research. During winter, cold temperatures create

an ideal environment for water vapor to travel upward from the warm subgrade soil below the frost line, through the capillary barrier, and into the base material.

The effect of soil stabilization on water vapor movement in freezing base materials depends on the properties of the stabilized soil. Gradation, mineralogy, and stabilizer type and concentration are some of the factors that can affect the material properties. As demonstrated in this work, treatments with either cement or calcium chloride can lead to increased or decreased amounts of water vapor movement into materials similar to those studied in this research.

Frost heave does not generally result exclusively from water vapor movement, mainly because the rate at which water vapor can move through a freezing pavement system is comparatively slow at thermal gradients characteristic of natural conditions. However, the accumulation of water from long-term water vapor movement into frost-susceptible base materials underlain by a capillary barrier can lead to frost heave of the base layer as it approaches saturation; water available in the layer can be redistributed upwards to create ice lenses upon freezing. Infiltration of water through a permeable surface course would be expected to exacerbate frost heave problems. While a permeable surface layer permits evaporation of water from the base layer into the atmosphere, it also permits the infiltration of water during rain or thawing events, for example, which can lead to higher water contents throughout the full pavement structure.

While the accumulation of water from long-term water vapor movement into a frostsusceptible base material underlain by a capillary barrier would be expected to reduce the stiffness of the base material compared to its stiffness in a drier condition, the incremental increase in total water content that may occur exclusively from water vapor movement during a single winter season in northern Utah would not be expected to cause measurable increases in thaw weakening of the base layer during spring. Indeed, assuming that excess water in the base material could always readily drain into the capillary barrier, conditions of super-saturation typical of thaw weakening could potentially persist only as long as the thawing front was located within the base layer. Thus, a base layer that is only several inches thick, as in the experimental pavement sections studied in this research, may have positive pore water pressures for only a short time before continued thawing allows drainage of the excess water through the bottom of

the layer. However, because water in a base layer overlying a capillary barrier cannot drain until nearly reaching positive pore pressures, the base layer will remain indefinitely saturated or nearly saturated as demonstrated in this research. Conversely, when the base layer is in hydraulic contact with an underlying subbase or subgrade, the average water content of the base layer is lower due to improved drainage.

For materials similar to those studied in this research, potentially important material properties related to the occurrence of water vapor movement during freezing include dry density, percent of material finer than the No. 200 sieve, percent of material finer than 0.02 mm, apparent specific gravity, absorption, initial water content, porosity, degree of saturation, hydraulic conductivity, and electrical conductivity. The rate at which water vapor movement occurs is also dependent on the thermal gradient within the given material, where higher thermal gradients are associated with higher amounts of water vapor movement.

When all relevant material properties and environmental conditions can be quantified, numerical modeling using the SHAW model can be performed to examine water vapor movement in freezing aggregate base materials and investigate the influence of factors such as asphalt permeability on the water contents that develop in the pavement structure. In this research, the results of the numerical modeling clearly demonstrated that the asphalt layer at the experimental pavement sections was not properly compacted and was allowing water infiltration. The numerical modeling also supported the field observations that the capillary barrier effectively trapped moisture in the overlying base material, causing it to remain saturated or nearly saturated throughout the monitoring period. Numerical modeling can also be used to investigate the relationship between water vapor flux and thermal gradient.

5.3 Recommendations

Several recommendations based on this research are offered. While water vapor has been largely neglected in pavement engineering, long-term water vapor movement into base materials underlain by capillary barriers should be considered as a viable mechanism by which water ingress occurs in pavement structures. Furthermore, because capillary barriers effectively trap moisture in the base layer, causing it to remain indefinitely saturated or nearly saturated,

sufficient water may be available in the base material to cause frost heave of the layer through the redistribution of the accumulated water. Therefore, only non-frost-susceptible aggregate base materials should be specified for use in cold climates in conjunction with capillary barriers. Furthermore, if testing of the base material is specified to determine a stiffness value to be used in pavement design, it should be evaluated in a saturated condition to reasonably simulate the expected average field condition. In the pavement design process, the base material should be assumed to remain in a saturated or nearly saturated condition during the entire service life of the pavement, regardless of the potential of the surface course to prevent water infiltration during rain or thawing events, for example. If the base materials available for a given project are not expected to provide satisfactory performance under these conditions, appropriate soil stabilization should be performed to achieve target strength and durability levels. The benefits of using a capillary barrier may then be achieved while simultaneously mitigating the potentially negative consequences of water vapor movement into the overlying base material.

Further study is recommended on water vapor movement in freezing aggregate base materials. Laboratory and/or field testing in conjunction with numerical modeling is recommended to investigate the rates of water vapor movement in materials having properties different than those studied in this research and subjected to different thermal gradients. The effects of different types and concentrations of stabilizers on water vapor movement in freezing aggregate base materials specifically warrants further study. Longer monitoring or simulation periods more representative of more extreme northern climates are also recommended. Continued monitoring of the experimental pavement sections established in this research may also provide valuable information about the long-term performance of the pavements with respect to frost heave and thaw weakening, for example. Further investigations of correlations between field performance and specific material properties that can be easily measured in the laboratory may lead to the development of improved specifications in this respect. Finally, additional research on methods and products for reducing the permeability of pavement surface courses would also be beneficial in minimizing water-related damage to subsurface pavement layers.

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APPENDIX A

The tabular stiffness data collected using free-free resonance testing at each of five moisture contents for the Redwood, McGuire, and 400 North materials are presented in Tables A.1 through A.3, respectively. The temperature profiles and thermal gradients for frost susceptibility testing on the untreated materials, which focused on evaluating the effects of moisture availability on the frost heave characteristics of untreated specimens, including the ingress and/or redistribution of moisture within the materials, as well as the temperature history for the water bath, are presented in Figures A.1 through A.11. The temperature profiles and thermal gradients for frost susceptibility testing on the untreated and treated materials, which focused on evaluating the effects of cement and salt treatments on water vapor movement during freezing, as well as the temperature history for the water bath, are presented in Figures A.12 through A.26. The tabular field frost heave data collected using a rod and level, modulus values collected using a PFWD and analyzed using BAKFAA, and modulus ratios are presented in Tables A.4 through A.6, respectively.

Material		Untreated			Cement-Treated			Salt-Treated		
		Specimen			Specimen			Specimen		
		1	2	3	1	2	3	1	2	3
Round 1	Moisture Content (%)	9.6	9.7	9.6	8.2	8.2	8.3	9.1	9.1	9.2
	Young's Modulus (ksi)	9.3	12.6	17.1	86.4	40.0	22.6	0.7	1.0	0.8
Round 2	Moisture Content (%)	8.0	8.2	8.1	7.7	7.6	7.7	7.8	7.9	8.0
	Young's Modulus (ksi)	46.3	77.5	64.1	146.4	130.7	74.3	23.5	27.5	23.6
Round 3	Moisture Content (%)	6.2	6.4	6.2	7.1	7.0	7.1	6.2	6.3	6.5
	Young's Modulus (ksi)	125.3	116.7	91.2	209.9	91.1	111.4	78.4	71.1	80.2
Round 4	Moisture Content (%)	3.8	4.0	3.8	6.0	6.0	6.0	4.0	3.9	4.1
	Young's Modulus (ksi)	110.3	122.4	111.4	125.1	82.1	104.0	77.5	84.9	68.2
Round 5	Moisture Content (%)	2.5	2.7	2.5	5.2	5.2	5.3	2.8	2.7	3.0
	Young's Modulus (ksi)	110.5	108.0	118.7	112.9	82.5	103.4	102.9	49.7	48.8

Table A.1 Redwood Laboratory Stiffness Data

Material		Untreated			Cement-Treated			Salt-Treated		
		Specimen			Specimen			Specimen		
		1	2	3	1	2	3	1	2	3
Round 1	Moisture Content (%)	5.8	5.8	5.8	4.8	4.8	4.8	5.6	5.7	5.7
	Young's Modulus (ksi)	4.3	3.6	5.2	54.0	42.1	63.2	1.8	1.3	1.4
Round 2	Moisture Content (%)	4.6	4.9	4.6	3.9	3.8	3.9	4.8	5.0	4.9
	Young's Modulus (ksi)	11.3	4.9	5.6	150.6	201.0	147.0	2.5	2.5	2.6
Round 3	Moisture Content (%)	3.8	3.8	3.9	2.9	2.9	2.9	3.7	3.7	3.7
	Young's Modulus (ksi)	40.1	19.6	15.0	204.4	69.3	139.7	8.2	10.0	11.4
Round 4	Moisture Content (%)	2.7	2.6	2.7	2.0	2.0	1.9	2.9	2.9	3.0
	Young's Modulus (ksi)	77.7	48.8	55.2	176.3	141.0	105.7	20.3	26.3	25.0
Round 5	Moisture Content (%)	2.1	2.0	2.1	1.2	1.1	1.1	1.9	1.9	2.0
	Young's Modulus (ksi)	83.3	39.8	96.4	129.1	171.9	99.0	56.7	50.5	23.1

Table A.2 McGuire Laboratory Stiffness Data

Material		Dense-Graded Base			Dense-Graded Subbase			Subgrade		
		Specimen			Specimen			Specimen		
		1	2	3	1	2	3	1	2	3
Round 1	Moisture Content (%)	4.8	5.2	5.3	6.7	6.7	6.8	5.2	5.2	5.2
	Young's Modulus (ksi)	4.7	4.4	4.0	1.5	1.7	2.1	4.8	5.5	6.1
Round 2	Moisture Content (%)	3.9	4.3	4.3	5.4	5.5	5.4	4.4	4.3	4.4
	Young's Modulus (ksi)	3.5	2.9	2.3	1.2	1.1	1.0	0.8	1.0	1.1
Round 3	Moisture Content (%)	3.0	3.5	3.4	4.3	4.2	4.2	3.5	3.4	3.4
	Young's Modulus (ksi)	42.7	37.5	35.8	13.3	16.4	24.5	7.8	10.0	11.0
Round 4	Moisture Content (%)	2.2	2.5	2.5	2.9	2.8	2.8	2.9	2.9	2.9
	Young's Modulus (ksi)	95.1	86.4	78.3	77.0	51.2	71.0	15.3	18.7	21.8
Round 5	Moisture Content (%)	1.4	1.7	1.7	1.6	1.7	1.7	2.1	2.1	2.0
	Young's Modulus (ksi)	46.0	29.8	24.8	28.2	100.3	50.2	10.2	11.8	8.8

Table A.3 400 North Laboratory Stiffness Data



Figure A.1 Temperature history for untreated Redwood specimen closed to water ingress.



Figure A.2 Temperature history for untreated Redwood specimen open to capillary rise.



Figure A.3 Temperature history for untreated Redwood specimen open to water vapor movement.



Figure A.4 Temperature history for untreated 400 North specimen closed to water ingress.



Figure A.5 Temperature history for untreated McGuire specimen open to capillary rise.



Figure A.6 Temperature history for untreated McGuire specimen open to water vapor movement.


Figure A.7 Temperature history for untreated 400 North specimen open to water vapor movement.



Figure A.8 Thermal gradients for untreated specimens closed to water ingress.



Figure A.9 Thermal gradients for untreated specimens open to capillary rise.



Figure A.10 Thermal gradients for untreated specimens open to water vapor movement.



Figure A.11 Temperature history for water bath during testing of untreated specimens.



vapor movement.



Figure A.13 Temperature histories for cement-treated Redwood specimens open to water vapor movement.





Figure A.15 Temperature histories for untreated McGuire specimens open to water vapor movement.





Figure A.17 Temperature histories for salt-treated McGuire specimens open to water vapor movement.





Figure A.19 Thermal gradients for untreated Redwood specimens open to water vapor movement.



Figure A.20 Thermal gradients for cement-treated Redwood specimens open to water vapor movement.



Figure A.21 Thermal gradients for salt-treated Redwood specimens open to water vapor movement.



Figure A.22 Thermal gradients for untreated McGuire specimens open to water vapor movement.



Figure A.23 Thermal gradients for cement-treated McGuire specimens open to water vapor movement.



Figure A.24 Thermal gradients for salt-treated McGuire specimens open to water vapor movement.



Figure A.25 Thermal gradients for untreated 400 North specimens open to water vapor movement.



Figure A.26 Temperature history for water bath during testing of untreated and treated specimens.

Elapsed Days	Heave (in.)				
	Capillary Barrier	No Capillary Barrier			
13	0.00	0.00			
27	0.00	0.00			
39	0.00	0.00			
55	0.12	0.00			
83	0.48	0.06			
96	0.00	0.00			
111	0.00	0.00			
125	0.00	0.00			
139	0.00	0.00			
153	0.00	0.00			
167	0.00	0.00			

Table A.4 Field Frost Heave Data

	Modulus Value (ksi)							
Elapsed Days		Capillary Barrier			No Capillary Barrier			
	Asphalt	Dense-Graded Base	Open-Graded Subbase/ Subgrade	Asphalt	Dense-Graded Base	Dense-Graded Subbase/ Subgrade		
13	527	118	13	437	86	15		
27	553	127	13	429	95	15		
39	631	147	13	469	122	15		
55	1064	2134	49	1666	2312	49		
83	3416	4803	27	1419	765	31		
96	644	163	13	541	125	17		
111	647	145	14	558	126	17		
125	716	159	13	528	114	17		
139	658	188	13	566	135	15		
153	323	57	12	314	53	16		
167	544	129	13	472	101	17		

Table A.5 Field Modulus Values

Elapsed Days	Capillary Barrier			No Capillary Barrier			
	Asphalt	Dense-Graded Base	Open-Graded Subbase + Subgrade	Asphalt	Dense-Graded Base	Dense-Graded Subbase + Subgrade	
13	1.7	1.9	0.9 1.6 1.7		1.7	1.0	
27	1.7	2.1	0.9	1.7	1.8	1.0	
39	1.8	2.6	0.9	1.9	2.1	1.0	
55	6.6	50.1	3.0	3.2	29.8	3.8	
83	5.6	16.6	1.9	10.3	67.1	2.1	
96	2.1	2.7	1.0	1.9	2.3	1.0	
111	2.2	2.7	1.0	1.9	2.0	1.1	
125	2.1	2.5	1.0	2.2	2.2	1.0	
139	2.2	2.9	0.9	2.0	2.6	1.0	
153	1.2	1.1	1.0	1.0	0.8	0.9	
167	1.9	2.2	1.0	1.6	1.8	1.0	

Table A.6 Field Modulus Ratios

APPENDIX B

The results of numerical modeling using only measured values for inputs, with no adjustment of the air-entry potential for the base material, are presented in this appendix. The specific inputs for the SHAW model are shown in Table B.1. For the purposes of modeling, the asphalt layer was assigned a saturated hydraulic conductivity of 1.0 cm/hr to allow infiltration through the asphalt. Other assumed input values for the asphalt were 0.20 J/kg and 5 for the air-entry value and pore-size index value (*b*), respectively. Percentages of sand and silt, used primarily for soil thermal conductivity calculations, were set to 99 and 1 percent, respectively.

Comparisons of measured and simulated temperature data for the sections with and without the capillary barrier are shown in Figures B.1 to B.12, respectively. Because the measured subgrade temperature was an input used for the lower boundary condition in the model, the measured and simulated subgrade temperatures are identical. Comparisons of measured and simulated liquid water content data for the sections with and without the capillary barrier are shown in Figures B.13 to B.20, respectively, while comparisons of measured and simulated water potential data are shown in Figures B.21 to B.28, respectively. These figures demonstrate the need to apply an adjustment to the air-entry potential of the base material to achieve improvements in the match between especially the measured and simulated water liquid water content data.

Property	Asphalt	Dense- Graded Base	Dense- Graded Subbase	Open- Graded Subbase	Subgrade
Apparent Specific Gravity	2.63	2.68	2.66	2.67	2.63
Dry Density of In-Situ Layer (pcf)	128.7	128.7	-	-	101.8
Porosity of In-Situ Layer (%)	21.4	23.1	-	-	38.0
Maximum Dry Density (pcf)	-	-	135.5	96.6	-
Porosity Calculated Using Maximum Dry Density (%)	-	-	21.2	42.1	-
Saturated Hydraulic Conductivity (ft/day)	-	0.4790	0.1342	5417	12.19
Air-Entry Value (J/kg)	-	-0.20433	-0.68730	-0.00026	-0.00111
Index Value b	-	5.12	4.48	4.36	6.07
Sand (%)	99.00	88.15	90.97	98.48	94.92
Silt (%)	1.00	11.85	9.03	1.52	5.08

 Table B.1 Inputs for Numerical Modeling (Data Based on Measured Air-Entry Potential)



Figure B.1 Temperature at top of asphalt for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.2 Temperature at middle of asphalt for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.3 Temperature at top of base for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.4 Temperature at middle of base for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.5 Temperature at bottom of base for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.6 Temperature at subgrade for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.7 Temperature at top of asphalt for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.8 Temperature at middle of asphalt for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.9 Temperature at top of base for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.10 Temperature at middle of base for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.11 Temperature at bottom of base for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.12 Temperature at subgrade for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.13 Volumetric liquid water content at top of base for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.14 Volumetric liquid water content at middle of base for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.15 Volumetric liquid water content at bottom of base for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.16 Volumetric liquid water content at subgrade for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.17 Volumetric liquid water content at top of base for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.18 Volumetric liquid water content at middle of base for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.19 Volumetric liquid water content at bottom of base for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.20 Volumetric liquid water content at subgrade for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.21 Matric potential at top of base for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.22 Matric potential at middle of base for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.23 Matric potential at bottom of base for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.24 Matric potential at subgrade for pavement section with capillary barrier and permeable asphalt (simulated data based on measured air-entry potential).



Figure B.25 Matric potential at top of base for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.26 Matric potential at middle of base for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.27 Matric potential at bottom of base for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).



Figure B.28 Matric potential at subgrade for pavement section without capillary barrier and with permeable asphalt (simulated data based on measured air-entry potential).