MOISTURE MOVEMENT
IN
HIGHWAY PAVEMENT STRUCTURES
COUPLED WITH
SOIL-ATMOSPHERIC FLUXES

A thesis Submitted to the College of
Graduate Studies and Research
In Partial Fulfillment of the Requirements
For the Degree of Master of Science
In the Department of Civil and Geological Engineering
University of Saskatchewan
Saskatoon, Saskatchewan, Canada

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ABSTRACT

The overall performance of a standard highway pavement structure depends on moisture characteristics of the underlying soil layers. Increase in the moisture content of the sublayers caused by moisture fluxes across the soil-atmosphere interface, decreases the strength of the pavement structure, which may cause premature failure of the structure. Therefore, the ability to evaluate the surface fluxes may be helpful in understanding mechanisms, which may enhance or degrade highway pavement performance.

This research evaluates the application of the soil-atmosphere modelling software VADOSE/W as a tool for predicting the movement of moisture in highway pavement structures. VADOSE/W is a two-dimensional transient finite element program that simulates coupled heat and moisture migration in unsaturated soils with particular focus on fluxes across the soil-atmosphere interface. A typical standard highway pavement structure in Saskatchewan was chosen to evaluate coupled heat and moisture interactions between highway pavement structures and atmosphere, and the impact that design features may have on moisture movement.

A laboratory testing program was established to characterize the material properties of hot mix asphalt (HMA), which is used as a surface layer for the driving lane as well as on occasion for the shoulder of highway pavement structures. HMA was characterized by its saturated hydraulic conductivity, soil-water characteristic curve, vapour flux rate and air permeability. The saturated hydraulic conductivity defines the maximum rate at which water can infiltrate HMA in the absence of cracks. Drastic changes to the saturated hydraulic conductivity of HMA can significantly increase or decrease the amount of infiltration during critical storm durations. The volumetric water content of HMA decreases rapidly at relatively low values of suction suggesting that HMA is either
relatively hydrophobic or contains cracking of the internal structure such that it
demonstrates very low air entry values. The ‘pore spaces’ of the HMA are likely only
partially filled with water following drainage. The vapour flux rate of HMA defines the
maximum evaporation rate through HMA in the absence of cracks. HMA produces a
negligible amount of evaporation during the summer period compared with the amount of
infiltration. The measured and calculated air permeability results for HMA were quite
different indicating that problems might have occurred during the testing process. Some
of the possible problems include air bubbles in the manometer, air leakage, and not
allowing the flow meter to come into equilibrium.

A numerical modelling component evaluated the mechanisms of coupled heat and
moisture flux into the pavement structure when using six different design features, which
have the ability to either enhance or degrade performance. The six design features
include: varying the fluxes on the HMA surface; changing the shoulder conditions from
unpaved to paved shoulders; changing the steepness of the sideslope; using both good
and poor vegetation conditions; varying the initial suction conditions; and varying the
snow removal process during the winter season.

Each of the eight numerical simulations was simulated for twelve years in order to reach
long-term equilibrium conditions. The results of annual cumulative boundary flux for
each of the numerical simulations indicates that long-term equilibrium conditions for the
highway pavement system have yet to be established after the twelve years of simulation.
However, overall the cumulative flux rates are slowly changing from year to year.

Comparing each of the three divisions of the top boundary indicates that the flow through
the driving surface (i.e. driving lane and shoulder) is quite negligible. The granular area
of the sideslope is generally moving water into the system due to the granular materials
along the surface. The organic area of the sideslope (i.e. ditch area) is generally moving
water out of the system for all eight cases.
The modified numerical simulation design features can cause a positive or negative impact on the moisture and heat fluxes of highway pavement structures. The design feature that has a positive impact is applying a vapour diffusion rate to the paved surface (Case 5). Paving the shoulder (Case 2) also has a positive effect. These features produce less moisture to accumulate in the subgrade layer over the entire climatic season. The design features that create a negative impact include applying poor vegetation conditions to the sideslope and ditch areas (Case 3), steepening the sideslope (Case 6) and allowing snow to accumulate along the shoulder (Case 8). These three design features cause more moisture to accumulate within the subgrade layer in comparison to that of the base simulation case.

Further studies are required before the research is extended to engineering practice. The six most important recommendations include:

1. Addressing the convergence and water balance issues within the numerical models themselves.
2. Enhancing the mesh to provide more realistic highway pavement geometry.
3. Improving the initial conditions for a more realistic hydrogeologic setting.
4. Refining subgrade layer to include: (a) a difference in the recompacted layer from the compacted layer and (b) a separate upper layer that will be affected by freeze-thaw.
5. An indepth investigation into the ability to apply actual HMA properties from laboratory studies along with pertinent climate information required within VADOSE/W.
6. Laboratory testing of the pavement sublayers (i.e. base, subbase and subgrade layers).
ACKNOWLEDGEMENTS

I would like to acknowledge Saskatchewan Department of Highways and Transportation for their financial support throughout this research project. I would also like to sincerely thank my supervisors, Professor S. Lee Barbour and Professor Moir D. Haug, for their suggestions, guidance and patience. I couldn’t have got through this research project without them.

I would also like to acknowledge Mr. Jorge Antunes, and Mr. Ron Genest of Saskatchewan Department of Highways and Transportation for providing me with their expertise in highway pavement structures.

I am indebted to Mr. Alex Kozlow, Mr. Randall Osicki, Mr. Julian Gan and the staff at MDH Engineered Solutions Laboratory for their assistance and direction throughout the laboratory component of this research project. I am also indebted to Mr. Greg Newman of Geo-Slope International Limited for his patience, support and guidance throughout the numerous months of numerical modeling. His assistance in developing the model was greatly appreciated.

I would like to thank the members of the CANSIM center for the use of the computers throughout my modelling component. I would also like to thank my fellow graduate students for sharing their experiences and their support throughout the course of my graduate studies, especially during the preparation of my thesis. I would also like to thank any others I have missed who have contributed to the preparation of this manuscript.

Thanks to my friends and family for their support, patience and encouragement. Without them I would never have got to this point in my engineering career.
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Chapter 1 Introduction

The overall performance of a standard highway pavement structure is dependent upon the moisture characteristics of the underlying soil layers. If the moisture content in these layers increases, the stiffness and strength will decrease which may cause premature failure of the highway pavement structure (Elsayed and Lindly, 1996; Cyr and Chiasson, 1999; Rainwater et al., 2001).

This chapter reviews the problem of moisture ingress to standard highway pavement structures. The objectives and the methodology of the thesis research are also presented.

1.1 Introduction

Highway systems are designed to ensure a high level of service for its users. In 2004, the Saskatchewan provincial highway road network consisted of 26,255 km of roadway, divided between principal roadways and regional roadways. There are four types of road designs throughout the road network: gravel, thin membrane surface (TMS), granular pavements and standard pavements. Standard pavements make up the majority of the road network, with 8,875 km of roadway (SDHT, 2004a). A standard pavement is designed as a flexible pavement, generally consisting of a surface course, base course, subbase course and a subgrade.

Yoder (1964) and Watson (1989) describe a flexible pavement as being capable of retaining its structural integrity even when small vertical movements occur at surface. The load carrying capacity of a flexible pavement is dependent upon the load-carrying characteristics of the sublayers, which must be capable of resisting the aggressive effects of traffic and climate. The upper layer should have the strength to support traffic loads
and the ability to transmit the traffic loads to the underlying layers in such a way as to avoid premature failure.

A primary factor causing premature failure of a standard highway pavement structure is an increase in the moisture content within the base, subbase and subgrade layers. Climatic data for Saskatchewan for an average year is 400 mm of precipitation and 900 mm of evaporation (Environment Canada, 2004). Progressive wetting of the pavement sublayers may still occur even under Saskatchewan’s arid climatic condition. Barbour et al. (1995) found that the dominant mechanism causing increases in moisture within the pavement structure was saturated and unsaturated flow as driven by the surface fluxes at the soil/atmosphere interface.

Surface fluxes consist of infiltration and evaporation. Infiltration of moisture into the pavement structure can occur as a result of cracks in the asphalt layer; however, it can also occur through unpaved shoulders and ditches adjacent to the pavement structure. Water losses from the highway structure are primarily by drainage through the base and subbase towards the ditch, or as the result of transpiration from vegetation on the sideslopes and in the ditches. The hydraulic properties of the unsaturated base and subbase layers are such that they easily allow infiltration under precipitation events, but have limited capabilities for drainage or drying during extended evaporative cycles. This leads to a net ingress of moisture during each cycle of wet and dry climatic conditions (Lacher et al., 2004).

As moisture builds up in the pavement structure, its strength and stiffness begin to decrease. Reductions in strength due to increases in moisture can occur in a number of ways. The apparent cohesion can be reduced due to the fact that moisture increases are accompanied by a reduction in suction. Once the pavement structure becomes highly saturated, vehicle loads may cause erosion and pumping, disintegration of cement treated bases, stripping of asphalt coatings from bases and subbases and overstressing of the already weakened subgrade (Elsayed and Lindly, 1996; Cyr and Chiasson, 1999; Rainwater et al., 2001).
1.2 Research Objectives

The overall objective for this research is to evaluate the relative magnitudes of moisture and heat fluxes within a highway pavement system as well as the average annual water and energy balance. This overall objective is of importance because defining the typical 'order of magnitudes' of heat and moisture fluxes is essential in evaluating which mechanisms control the heat and moisture balance within the subgrade layer. This will also aid in identifying design features that may enhance or degrade highway pavement performance.

The overall objective is pursued through a series of more specific objectives. These specific objectives include: (1) evaluating key physical and hydraulic material properties of standard pavement structures; (2) simulating coupled heat and moisture interactions between a typical pavement structure and atmosphere in order to obtain approximate estimates of the long-term heat and moisture balances and flux rates within various elements of the pavement structure; and (3) evaluating the relative impact that design features such as paved shoulders, backslope angle, and asphalt material characteristics may have on these fluxes.

1.3 Research Methodology

The research methodology is divided into three components:

1. Field Component
2. Laboratory Component
3. Numerical Modelling Component

The field component involves selecting an appropriate standard pavement structure that is representative of a typical Saskatchewan highway pavement design; to collect the necessary core samples; and to obtain knowledge of the highway geometry, site conditions and highway pavement design. The highway pavement selected for this research is located along Highway 16 near Saskatoon, Saskatchewan, Canada.
The laboratory component involves testing of the collected core samples to determine various material properties such as the saturated hydraulic conductivity, the moisture retention characteristics and vapour diffusion characteristics.

The numerical modelling component involves simulating a series of typical cases to illustrate the type of moisture and heat flux that operate within a highway pavement system using VADOSE/W.
Chapter 2 Literature Review

Standard highway pavement structures are designed as a series of layers, with each layer having a specific function in maintaining serviceability requirements. The movement of moisture into these layers can be detrimental to overall pavement performance and can cause premature deterioration in road serviceability. This chapter presents background information into highway pavement structures and the effect that moisture ingress has on the performance of these structures.

2.1 Highway Pavement Structures

The design of highway pavement structures has changed significantly over the years. Watson (1989) stated that the first road builders were the Romans, to whom the ability to move quickly from one part of the Empire to another was important for military and civil reasons. A typical major Roman road consisted of several layers of material, increasing in strength from the bottom layer through intermediate layers to an upper layer. The bottom layer may have been rubble, the intermediate layers lime bound concrete and the upper layer flags or stone slabs grouted in lime. As time progressed the design of highway pavement structures improved, leading to the most recent design of flexible pavement systems (Watson, 1989).

A flexible asphalt concrete pavement consists of a series of several layers, with the highest quality materials at or near the surface. Figure 2-1 illustrates a typical flexible pavement design. The surface layer of a flexible pavement consists of a wearing surface and a shoulder, each which have different functions. The wearing surface, or driving lane, is a paved surface that is designed to be watertight to prevent the ingress of water into the pavement structure, but flexible enough so that it will not fail if deformation of
the subgrade or base course takes place. The wearing surface must also possess skid resistance, and the ability to resist load and non-load associated fractures and permanent deformations. The shoulder is the portion of the surface layer contiguous to the wearing surface that accommodates stopped vehicles for emergency use and is a lateral support for the base and wearing surface. The shoulder must be sloped to provide adequate runoff of surface water (Yoder, 1964; Yoder and Witczak, 1975; and Watson, 1989).

The base and subbase provide support to the wearing surface, while protecting the subgrade from loads which it cannot support. Both layers are used to increase the load supporting capacity of the pavement by providing stiffness and resistance to fatigue as well as building up relatively thick layers in order to distribute the load. Spreading the loads applied at the surface layer helps to limit shear and consolidation deformations within the subgrade.

The base layer must have the ability to provide drainage and give adequate protection against frost action. The materials used for the base layer must be of high quality to prevent failure due to high stress concentrations directly below the surface. The thickness of the base layer is also of importance. If the base layer is too thick the cost of the project increases, but if the layer is too thin it may fail to provide adequate protection to the unbound layers beneath. The base layer is also constructed to some distance beyond the edge of the pavement wearing surface in order to make certain that loads applied at the edge will be supported by the underlying layers.

![Figure 2-1: Typical Flexible Pavement Design (Yoder, 1964)](image-url)
The subbase layer insulates the subgrade against the action of cold weather. It allows for the building of relatively thick pavements at a lower cost to the project. The subbase layer is constructed with a suitably graded granular material of lower quality than that required for the base layer (Yoder, 1964; Yoder and Witczak, 1975; AASHTO, 1980; and Watson, 1989).

Yoder and Witczak (1975) found the most desirable properties of a subgrade to include strength, ease of compaction, permanency of compaction and permanency of strength. Proper compaction of the subgrade layer is an essential property affecting the characteristics of the subgrade. Subgrade strength properties are dependent upon the moisture content and density of the soil with increased density and lower moisture contents producing higher strength. With the proper subgrade characteristics, a stable platform is provided to construct the upper pavement layers.

2.2 Highway Pavement Evaluations

The presence of excess water within a pavement structure will adversely affect the overall performance of the underlying layers. Excess water is a primary factor for premature roadway failure because it causes reductions in the strength of the structural section and foundation materials as a result of a reduction in effective stresses within these materials. An evaluation of the mechanisms of moisture movement within each pavement layer is discussed within the following sections.

2.2.1 Base Layer

Water will drain from the base and subbase layers of a highway pavement system. In an ideal situation, where pavement structures are well drained and well maintained, the sublayers would be capable of reaching drained equilibrium conditions. However, the ideal situation is generally never the case in most geographic areas. Research on moisture movement in highway pavement structures in humid and semi-humid regions has focused on the drainage capacity of the base and subbase layers.
Drainage of water through the base layer occurs from the centerline towards the edge of the shoulder. Wallace (1978) evaluated the drainage pattern of a standard low cost pavement structure. Water is found to drain at the edge of the shoulder as long as water pressures remain above atmospheric pressure conditions. After drainage has ceased, the remaining water remains within the highway pavement structure, causing a slight increase in the moisture contents of the underlying layers. Drainage quantities are controlled by the properties of the drainage layer materials and by the design of the highway pavement system.

Moynahan and Sternberg (1974), Pessaran (1994), Elsayed and Lindly (1996), and Cote (2002) showed how the grain size distribution of the base and subbase material would affect the drainage conditions of the base and subbase layer. Elsayed and Lindly (1996) studied the effect of the larger size particles on the drainage conditions of the base layer. It was concluded that increasing the size of the larger size particles had no effect on drainage conditions and therefore is only to be considered during strength evaluations of the pavement structure. The amount of fines in the base layer can drastically affect the drainage conditions of the base and subbase layer.

Pessaran (1994) established that increasing the amount of fines in the granular layers of a highway pavement structure affects the air entry value (AEV) and the residual suction of the material’s soil water characteristic curve. Pessaran (1994) estimated the AEV using two points on the grain size distribution curve, $d_{75}$ and $d_{25}$, and used $d_{75}/d_{25}$ to define the slope of the grain size distribution curve. The residual suction is dependent of the amount of fines. Therefore, increasing the ratio of $d_{75}/d_{25}$ will increase the optimum density of the base and subbase, thus lowering its porosity. Lower porosities result in lowering the hydraulic conductivity, thereby affecting the materials drainage behaviour of the material.

Markow (1982) and Pessaran (1994) varied the hydraulic characteristics of the base and subbase material in order to evaluate their effects on drainage from highway pavement.
structures. Markow (1982) varied the hydraulic conductivity of the base material and determined that higher hydraulic conductivity values, in the range of 30.5 m/day to 3050 m/day, allowed for sufficient drainage quantities through the base layer, whereas hydraulic conductivities as low as 0.03 m/day lowered the drainage quantity. Lowering the drainage quantity of the base and subbase layer increases the amount of moisture accumulating within the highway pavement structure causing premature strength reductions and a reduction of its service life (Pessaran, 1994).

Highway pavement structures can be designed to include subsoil drainage systems to improve the drainage conditions of the base and subbase layer. Numerous subsoil drainage systems are available for highway pavement design. Some of the common subsoil drainage systems analyzed included perforated drainage pipes, edge drains, open-graded drainage layers, and drainage mats installed with perforated drainage pipes (Kaxmierowski et al., 1994; Cyr and Chiasson, 1999; Birgisson and Roberson, 2000).

All drainage systems analyzed were found to assist in removing water from within a highway pavement system. Some systems were found to remove more moisture than others. Perforated drainage pipes perform differently depending on the base material according to Cyr and Chiasson (1999). If a perforated drainage pipe is placed within a homogeneous base layer, the ability to drain water from within a highway pavement is effective for short drainage periods. If the perforated drainage pipe is placed in a highway pavement system that has a heterogeneous base layer, there is no improvement in the rate of saturation during a precipitation event compared to a no drain design (Cyr and Chiasson, 1999).

Edge drains have a limited effect on the amount of water removed from within a highway pavement system. The most beneficial area within the pavement system for installing an edge drain system is the shoulder. Birgisson and Roberson (2000) found that the outer wheel path is a problem area for an edge drain system. Water contents were found to be higher at the outer wheel path than at the centerline or shoulder area, indicating that the highway pavement system could either be wetting from the shoulder inward or the edge
The base layer has the ability to provide sufficient drainage of a highway pavement structure providing that this layer has good drainage characteristics. There are various methods available to obtain good drainage characteristics, for example, limiting the fines content of the granular material or installing a subsoil drainage system within the design of the structure (Pessaran, 1994; Cyr and Chiasson, 1999). The method(s) considered should assist in keeping the sublayers of the highway pavement structure from becoming saturated.

2.2.2 Subgrade Layer

Both external and internal factors influence the moisture content of the subgrade layer.
External Factors

External factors influencing subgrade moisture contents include precipitation, seasonal temperatures, the location and variation of the groundwater table, pavement and shoulder conditions, and pavement drainability. Seasonal temperature changes are one of the most well defined external factors influencing moisture contents in the subgrade (Hall and Rao, 1999).

Seasonal temperature changes allow for maximum temperatures in the summer and minimum temperatures in the winter. The correlation between temperature and moisture contents is more likely in the shallower depths of the subgrade layer than at greater depths. Vaswani (1975) and Hall and Rao (1999) found that temperature is negatively correlated to subgrade moisture content indicating that lower moisture contents in the subgrade are expected in winter because the available subgrade moisture is drawn to the frost front where ice lenses develop.

The moisture content of the subgrade layer are most influenced by moisture flow during the spring due to spring thaw. This increase in moisture content is dependent on the frost penetration depth and ice lense development in the subgrade layer during winter freezing conditions. Spring thawing of the frost front softens the subgrade, subbase and base layers causing approximately 90% of the overall damage seen on highway pavement structures (MacMaster et al., 1982; Bibbens et al., 1985; Medina, 1997; Janoo and Shepherd, 2000; Konrad and Roy, 2000; Tart, 2000; Watson and Rajapakse, 2000; ITC, 2002).

Three factors control the rate of thawing within the subgrade: (a) ground heat flux from underlying soils; (b) increased net radiation; and (c) variations in air temperature (Backstrom, 1999). Thaw normally begins around the beginning of April and can occur from the top and/or the bottom of the pavement structure. Thaw occurs from the top when the surface air temperature rises from below the freezing point to well above that point and remains there for an appreciable time. Excess pore water in the subgrade is
generated as moisture accumulates during freezing conditions. Thaw occurs from the bottom if the air temperature remains slightly below the freezing point for a sufficient length of time, leaving the outward conduction of heat from the earth’s interior to provide thaw in the subgrade. Thaw from the bottom causes the subgrade to carry the overburden pressure as the frost front retreats upward. The melt water is unable to escape due to this frost front and contributes to the increasing water content of the subgrade. After the frost front has completely thawed, any excess water in the highway pavement system can begin to migrate upwards and out of the system (Konrad and Roy, 2000).

Precipitation, in the form of rainfall during the various seasons, can affect the moisture contents of the subgrade layer. Hall and Rao (1999) found that precipitation and subgrade moisture content have a positive correlation; however, short rainfall periods, especially in arid regions, are insufficient to affect subgrade moisture contents. The rate at which precipitation infiltrates into a pavement structure affects the moisture content within the subgrade. Barbour et al. (1995) concluded that higher infiltration rates cause lower suctions to be produced within the pavement structure. Lower suctions result in higher moisture contents according to the soil-water characteristic curve of the soil.

The influence of pavement factors on moisture content has also been extensively studied. If the pavement surface layer has not deteriorated then the pavement surface is nearly impervious and minimal changes in subgrade moisture will occur. The only exception will be at the edges of the pavement structure (Russam, 1965). Van Ganse et al. (1973) described “edge effect” as a method in which water migrating from the sideslope interferes with the imperviousness of the road pavement along the shoulder of the roadway. This edge effect can cause a significant increase in the moisture content of the subgrade in the shoulder region. Repetitive increases in the moisture content of the base and subbase will gradually cause an increase in the subgrade moisture content. Water that is perched over the soil subgrade has the ability to slowly infiltrate into the compacted subgrade (Rainwater et al., 1999). Drainage conditions of the base layer can have a significant effect on the moisture content of the subgrade. Moderately well drained to well drained soils can reduce the water content of subgrade, but poor drainage
conditions can increase the amount of water remaining in the subgrade (Van Ganse et al., 1973; Hall and Rao, 1999).

The presence of shrinkage cracks in the subgrade soil is a possible explanation for wet conditions under pavements, especially in arid and semi-arid climates. Picornell and Abd Rahim (1991) simulated infiltration of rainfall through shrinkage cracks. Shrinkage cracks are found to fill with water after about four months of simulated precipitation. Water levels in the cracks remain high, except during a few short summer periods (Picornell and Abd Rahim, 1991).

**Internal Factors**

Internal factors affecting moisture contents within the subgrade layer include the subgrade soil type, compaction and density properties, depth of soil and soil suction. Soil suction is important in understanding unsaturated soil characteristics. The soil suction in a highway pavement will affect the shear strength, volume change and water flow through the subgrade.

Marjerison (2001) and Pufahl and Lytton (1991) monitored suction profiles within pavement subgrades. Horizontal and vertical suction profiles indicated different relationships between moisture content and suction. Horizontal suction profiles reveal greater suctions under the shoulder and side-slope regions of a pavement structure. The lower suctions are generated under the centerline of the pavement because the soil directly under the pavement is protected from the effects of evaporation and transpiration. However, suctions generated at the side-slope are generally greater than suctions generated at the shoulder. Higher suctions at the side-slope occur due to water not infiltrating as readily as it would at the shoulder and vegetation enhancing transpiration of moisture from the side-slope.

Vertical suction profiles indicate that greater suctions normally occur in the shallower depths because of the proximity of the soil to the soil-atmosphere interface. Seasonal
variations also produce variations in subgrade suction profiles. Minimum subgrade suction profiles are generally found in the spring when pavement strength is low. Larger suctions are therefore seen during fall and winter followed by a decrease in the spring.

Livneh and Shklarsky (1965) and Hall and Rao (1999) noted that subgrade moisture contents vary with depth. However, variations in subgrade moisture contents are site dependent. The moisture content of a subgrade can vary depending on whether the area of the highway pavement structure is founded on cut or fill (Livneh and Shklarsky, 1965).

2.2.3 Asphalt Surface Layer

The permeability of the asphalt surface layer increases due to the development of cracks. Water is capable of infiltrating through the cracks on the asphalt surface causing increases in water content in the structure.

Ridgeway (1976) evaluated pavement surfaces and found that the amount of water capable of entering through a crack is dependent upon four mechanisms:

1. The water carrying capacity of the crack,
2. The amount of cracking present on the asphalt surface,
3. The possible area that drains towards the crack, and
4. Rainfall patterns.

*Water Carrying Capacity of the Crack*

Theoretical analysis of the water carrying capacity of a crack indicate that cracks on the asphalt surface layer are capable of infiltrating all rainfall that reaches the pavement surface under normal precipitation conditions (Ridgeway, 1976). Ridgeway (1976) performed field investigations on dense-graded bituminous concrete (BC) pavements and portland cement concrete (PCC) pavements to evaluate the actual water carrying capacity of cracks in the field. There seems to be quite a difference between the water carrying capacity of a BC pavement and a PCC pavement. A BC pavement allowed for water to
infiltrate at a mean rate of 100 cm$^3$/h/cm of crack, where the PCC pavement had a mean rate of 37 cm$^3$/h/cm of crack. These values are based upon the assumption that flow is not restricted by the ability of the underlying layers to drain water.

During dry climatic periods, cracks are capable of being obstructed with sand, twigs, rocks and other forms of debris. Infiltration through a debris-filled crack depends on the precipitation conditions. For example, during a moderate intensity rainfall most debris-filled cracks receive more water than what they can carry. However, there are still a few debris-filled cracks on the pavement surface that are capable of carrying more water than what was supplied to them (Ridgeway, 1976).

Ridgeway (1976) observed that debris-filled cracks are capable of infiltrating the same quantity of water regardless of the underlying base materials gradation. However, it was observed that dense-graded base materials do not develop the same high flow rates as those observed in open-graded base materials.

*Amount of Cracking Present*

Cracks develop over the design life span of highway pavement structures. There are a variety of cracks that can be present on the asphalt surface. These cracks may occur from a variety of causes, including stresses from axle loads, temperature change in the surface layer, or moisture and temperature changes in the pavement sublayers. Some of the more common cracks that may occur on a highway pavement structure include fatigue cracks, longitudinal cracks, and thermal cracks.

Fatigue cracking is a load-associated failure and is identified by the closely spaced crack patterns on the surface of a highway pavement. It occurs when the pavement structure has been stressed to the limit of its fatigue life by repetitive axle load applications. Fatigue cracking is often associated with loads which are too heavy for the pavement structure or by a number of load repetitions greater than that for which it was designed.
Fatigue cracking is made worse by inadequate drainage, which contributes to the distress by allowing the pavement to become saturated and lose strength (Roberts et al., 1996).

Thermal cracks (transverse cracks) run perpendicular to the roadway centerline and is most often equally spaced. Thermal cracks occur when the temperature at the surface drops sufficiently to produce a thermally induced shrinkage stress in the surface layer that exceeds its tensile strength.

Longitudinal cracking consists of individual cracks that run parallel to the centerline of the roadway. Longitudinal cracking occurs at joints between adjacent lanes of the asphalt mixture or at the edges of the wheel paths in a rutted pavement. Water is capable of penetrating through the longitudinal cracks towards the underlying layers, possibly softening non-stabilized layers and accelerating the development of fatigue cracking. Longitudinal cracks can be introduced by low temperature, due to the density at the joint between paved lanes being low, reducing the tensile strength (Roberts et al., 1996).

*Area that Drains towards Crack*

The amount of water infiltrating through a crack in the asphalt surface depends on the amount of water falling on the crack, but also on the area that drains towards the crack. The amount of area draining towards each crack on the pavement surface relates to the cross-sectional design of the pavement structure. Pavement structures are either designed to be crowned level roadways or on grades with super elevated cross-sections. These designs are illustrated in Figure 2-2a and Figure 2-2b.
The area that drains towards cracks on a tangent crowned level pavement begins at the centerline of the pavement structure and moves towards the edge. The area draining totals one half of the total amount of rainfall falling on the pavement structure. Pavements constructed on grades with super elevated cross-sections allow for the entire pavement area to drain towards the cracks. Ridgeway (1976) determined that the total amount of rainfall infiltrating through a crack is equal to the amount of water that falls on the area draining towards the crack minus the amount of water that has infiltrated previous cracks on the drainage path.

**Rainfall Patterns**

Three pieces of information concerning local rainfall pattern are required when estimating the quantity of rainfall on a specific pavement that could possibly enter the cracks of a pavement structure. According to Liu and Lytton (1985) these three pieces of information concerning local rainfall pattern include:

1. The quantity of rain that falls in a given rainfall,
2. The intensity and duration of each rainfall, and
3. The random occurrence of sequences of wet and dry days.
The total quantity of rain that falls in each rainfall varies from one rainfall to the next. Knowing the quantity of rain that falls in a given rainfall will aid in predicting the amount of water available to infiltrate the highway pavement structure. Historical records indicate that the quantity of rain in any given rainfall follows a probability density function (Ridgeway, 1976).

Certain combinations of rainfall intensity and duration may limit the amount of water capable of entering cracks in the asphalt surface. Rainfall intensity is only significant up until the crack has reached its carrying capacity. Beyond this stage, duration becomes significant. Higher intensity, short duration rainfall is important if a crack has a large carrying capacity. However, if the carrying capacity of the crack is low, the duration of the storm tends to have more significance on the amount of water capable of infiltrating into the pavement system than intensity (Ridgeway, 1976).

The number of wet and dry days that occur over a certain time period are useful in predicting the sequence of days it will rain and days without rain. On the days when rain occurs, moisture is capable of infiltrating through cracks on the surface of the highway pavement structure causing saturation. On the days without rain, moisture has the ability to evaporate from the highway pavement structure allowing for the sublayers to desaturate (Ridgeway, 1976). However, all three pieces of information concerning rainfall pattern must be obtained for an accurate prediction of local rainfall patterns.

2.3 Literature Review Summary

There is a great deal of literature on highway pavement structures and the effect that moisture ingress has on its performance. Literature focused on the base and subbase layers of a highway pavement structure are concerned with the ability of these layers to drain. Moisture drainage through the base and subbase layer occurs at the edge of the shoulder region. Drainage occurs up until the vertical moisture profile at the shoulders edge equals the capillary fringe condition of the base and subbase material (Wallace, 1978). The number of fines within the base and subbase layers decreases the hydraulic
conductivity of these layers creating a decrease in the drainage ability of the base and subbase (Elsayed and Lindly, 1996). Fines also affect the soils air entry value and cause an increase in the degree of saturation after drainage has ceased (Pessaran, 1994). Numerous researchers have investigated the use of installing a subsoil drainage system to improve the drainage conditions of the base and subbase. The most common and beneficial subsoil drainage system researched includes drainage mats in combination with deep perforated drainage pipes (Cyr and Chiasson, 1999).

The moisture contents within the subgrade layer of a highway pavement structure are affected by numerous external and internal conditions. Climate is the most significant external factor increasing the moisture content within the subgrade layer. Spring thaw is a major concern in cold climate, like that of Saskatchewan. The amount of thawing seen during spring thaw is a factor of winter freezing conditions at the highway pavement site. Spring and summer rainfall can affect the moisture content of the subgrade layer. However, rainfall effects are a function of rainfall intensity and duration. In an arid place, like Saskatchewan, a high intensity short duration rainstorm is more likely to affect the subgrade layers moisture contents due to excess water ponding on the surface and within the ditches surrounding the pavement structure. Edge effect is also an external factor affecting the moisture contents of the subgrade layer. The affect that edge effect has on the subgrade layer moisture content is a function of the base and subbase layers drainage ability (Hall and Rao, 1999; and Russam, 1965).

Internal factors such as suction profiles within the subgrade layer can affect its moisture contents. Marjerison (2001) found higher suctions at the shoulder and side-slope than at the centerline of a highway pavement structure. These high suction areas indicate pathways for moisture to be released to the atmosphere. Vertical suction profiles indicate that suctions are greatest in the shallower depths of the subgrade due to the soils proximity to the soil-atmosphere interface.

The surface layer of highway pavement structures allows moisture to infiltrate the system in the development of cracks. The amount of moisture capable of infiltrating through the
cracks in the surface layer is dependent on the water carrying capacity of the crack, the amount of cracking present on the surface, the area draining toward the crack and rainfall conditions (Ridgeway, 1976).
Chapter 3: Theoretical Background

The numerical model used in this study to simulate moisture movement in highway pavement structures is VADOSE/W (Geo-Slope Int. Ltd., 1991). VADOSE/W is a two-dimensional transient finite element program that simulates coupled heat and moisture migration in unsaturated soils with particular focus on fluxes across the soil-atmosphere interface. VADOSE/W accounts for precipitation, surface evaporation and plant transpiration, ground freezing and thawing, and vapour flow. It is one of the first numerical modeling packages capable of calculating the actual evaporation based on physically based first principles.

VADOSE/W is formulated on the basis of the well-known principles of Darcy’s Law for the movement of liquid water, Fick’s Law for water vapour diffusion and Fourier’s Law for heat flow. The water and heat flow equations are derived from Richard’s equation (Richard, 1931) for transient flow in unsaturated soils with modifications provided by Wilson et al. (1994). Wilson et al. (1994) modified the Penman formulation (Penman, 1948) to couple the actual evaporation to soil suction and climatic conditions. The coupling of the soil profile to the atmosphere allows for the calculation of evaporation from a saturated or unsaturated soil surface.

Wilson’s (1990) soil-atmosphere model is an explicit finite difference scheme that utilizes the Dalton-type equation to calculate evaporation at the soil surface. Joshi et al. (1993) developed a finite element formulation of the original program. Geo-Slope International Ltd. (1991) modified Joshi et al. (1993) finite element formulation to develop a two-dimensional and axisymmetric numerical modelling package for use on personal computers. VADOSE/W is a result of these efforts. The method developed by Tratch (1996) to simulate plant transpiration as well as the
ground freezing routine of Newman (1995) is utilized within VADOSE/W.

This chapter represents the theoretical background for the VADOSE/W program used in this research.

3.1 Coupled Moisture and Heat Flow

Water migration near the ground surface can occur in the form of either liquid water flow or water vapour diffusion. The vapour pressure in the soil is a function of soil temperature and soil suction, therefore a system of equations is required to describe the flow of liquid water, water vapour and heat in the soil under transient conditions (Wilson et al., 1994).

Wilson (1990) developed a set of coupled partial differential equations to describe the transient processes of heat and water migration in saturated and unsaturated soils. The formulation was based on a continuum mechanics approach that used the well-known principles of Darcy's, Fick's and Fourier's laws.

3.1.1 Moisture Flow

Moisture flow through a porous medium can be described using equations developed by Richard (1931), Philip (1957), Lam and Fredlund (1984) and Wilson et al. (1994). The driving energy for the flow of liquid water in both unsaturated and saturated porous soil is a hydraulic head. The flow of liquid water can therefore be described using Darcy's law:

\[
q_w = -k_w \frac{\partial h_w}{\partial y}
\]

where \(q_w\) is the unit volumetric liquid water flux \((m^3/s/m^2)\), \(k_w\) is the hydraulic conductivity \((m/s)\), \(h_w\) is the hydraulic head \((i.e \ (u_w/p_wg)+y)\) \((m)\), \(u_w\) is the pore-water
pressure (kPa), \( y \) is the elevation (m), \( g \) is the acceleration due to gravity (m/s\(^2\)), and \( \rho_w \) is the density of liquid water (kg/m\(^3\)).

Water vapour can be transported by the diffusion of water molecules due to a gradient in the partial pressure of water vapour. In an unsaturated soil system water will flow between the liquid phase and the vapour phase across the air-water interface. This inter-phase liquid water flux must be accounted for to maintain a water balance. Fick’s law describes the diffusion of water vapor through the following expression:

\[
q_v = -D_v \frac{\partial P_v}{\partial y}
\]  

[3-2]

where \( q_v \) is the water vapour flux (kg/(m\(^2\)s)), \( P_v \) is the partial pressure due to water vapour (kPa) and \( D_v \) is the diffusion coefficient of the water vapour through soil (kgm/(kNs)) (Bruch, 1993; Wilson et al, 1994 and Swanson, 1995).

The combined transient flow of liquid water and water vapour in an unsaturated porous medium may result in a change in the volume of water stored in the soil. Combining Darcy’s law and Fick’s law with the constitutive relationship for water volume change results in Wilson’s (1990) equation for one-dimensional transient moisture flow:

\[
\frac{\partial h}{\partial t} = C_w \frac{\partial}{\partial y} \left( k_w \frac{\partial h_w}{\partial y} \right) + C_v \frac{\partial}{\partial y} \left( D_v \frac{\partial P_v}{\partial y} \right)
\]  

[3-3]

where \( C_w \) is the modulus of volume change with respect to the liquid phase (s\(^2\)kN/kg) and \( C_v \) is the modulus of volume change with respect to the vapour phase (s\(^2\)kN/(kgm\(^3\))). \( C_w \) and \( C_v \) are derived as follows:

\[
C_w = \frac{1}{\rho_w g m_2^w}
\]  

[3-4]
\[
C_v = \frac{1}{(\rho_w)^2 m^w_2} \left( \frac{P + P_v}{P} \right) \]  \hspace{1cm} [3-5]

where \( m^w_2 \) is the slope of the suction \((u_a - u_w)\) versus volumetric water content curve when \((\sigma_y - u_a)\) is zero \((m^2/kN)\), \(\sigma_y\) is vertical stress \((kPa)\), \((P + P_v)P\) is a correction factor for vapour diffusion, \(P\) is the total atmospheric pressure \((kPa)\), and \(P_v\) is the partial pressure in the soil due to water vapour \((kPa)\) (Wilson et al., 1994). Since vapour pressure is a function of temperature, equation [3-3] must be coupled to an equation describing heat flow.

### 3.1.2 Heat Flow

There are three possible mechanisms that contribute to the heat transfer in a soil profile. These mechanisms include conduction, convection and latent heat transfer due to phase changes. Bruch (1993) defined these mechanisms as follows:

- **Conduction**: the transfer of heat between soil particles and through the pore liquid.
- **Convection**: the transfer of heat energy through the movement of the fluid phase.
- **Latent heat transfer**: the heat that is absorbed or released as water changes between phases.

The Fourier diffusion equation (Eqn 3-6) describes heat flow using conductive and latent heat transfers, while assuming the convective heat transfer to be negligible. This equation is written as follows:

\[
C_h \frac{\partial T}{\partial t} = \frac{\partial}{\partial y} \left( \lambda \frac{\partial T}{\partial y} \right) - L_v \left( \frac{P + P_v}{P} \right) \frac{\partial}{\partial y} \left( D_v \frac{\partial P_v}{\partial y} \right) \]  \hspace{1cm} [3-6]

where \(C_h\) is the volumetric specific heat \((J/(m^3\, °C))\), \(T\) is the temperature \((K)\), \(t\) is the time, \(\lambda\) is the thermal conductivity \((W/(m\, °C))\) and \(L_v\) is the latent heat of vaporization for water \((J/kg)\) (Wilson et al., 1994).
3.1.3 Coupled Flow

It can be seen that in the governing equations for moisture and heat flow there are three unknown parameters: pressure, temperature and vapour pressure. Therefore a third relationship is required to solve these governing equations (Geo-Slope International Ltd., 1991; Joshi et al., 1993).

The soil vapour pressure term, $P_v$, is common to both the heat and moisture flow equations. Therefore, this soil vapour pressure term can be used for the coupling of moisture and heat flow. Edlefson and Anderson (1943) developed a relationship that allows the vapour pressure within a soil to be calculated on the basis of total suction in the liquid phase:

$$ P_v = P_{vs} h_r $$

[3-7]

where $P_v$ is the partial pressure of water vapour within the voids of the unsaturated soil (kPa), $P_{vs}$ is the saturation vapor pressure (kPa) of the soil water at the soil temperature, T, and relative humidity is defined as $h_r$. The saturation vapour pressure is expressed as

$$ \ln(P_{vs}) = \frac{C_1}{T} + C_2 + C_3 T + C_4 T^2 + C_5 T^3 + C_6 \ln(T) $$

[3-8]

where $T$ is the absolute temperature, $C_1$ equals −5.8002206E+03, $C_2$ equals 1.3914993E+00, $C_3$ equals −4.8640239E-02, $C_4$ equals 4.1764768E-05, $C_5$ equals −1.4452093E-08, and $C_6$ equals 6.5459673E+00. Relative humidity is expressed using equation 3-9.

$$ h_r = \frac{w_g W_v}{RT} $$

[3-9]
where \( \psi \) is the total potential in the liquid water phase (i.e. \((u_a - u_w) + \text{osmotic suction}\)) (m), \( W_v \) is the molecular weight of water (0.018 kg/mole), \( g \) is acceleration (m/s\(^2\)), \( R \) is the universal gas constant (8.314 J/mole-K), and \( T \) is absolute temperature (K) (Zhou, 1997; Dobchuk et al., 2004).

### 3.2 Atmospheric Coupling

The net flux of water across the ground surface consists of surface infiltration and evapotranspiration values. These fluxes are determined by coupling the moisture and heat flow equations with actual climatic data for the region in question. Actual evaporation (AE) at the soil surface is determined from the Penman-Wilson method such that the actual evaporation is computed as a function of potential evaporation. For all soil moisture contents actual evaporation is independent of soil type and drying history being only a function of soil suction at the ground surface.

#### 3.2.1 Evaporation

Evapotranspiration from land surfaces is the combination of evaporation from plant and soil surfaces and water transpired from vegetation. In other words, it is the transfer of water from the earth back to the atmosphere. It is a continuous and variable function that is driven by a drying force the atmosphere exerts on the soil and plant surfaces. A typical evaporation curve for soil is shown in Figure 3-1. The maximum rate of evaporation is referred to as "potential evaporation (PE)." This maximum rate of evaporation is maintained as long as there is a sufficient supply of water at the soil surface. The rate of evaporation begins to decline once the soil surface begins to dry.
Bruch (1993), Wilson et al. (1994), and Swanson (1995) describe evaporation in three stages: (1) the initial constant rate stage; (2) the intermediate falling rate stage; and (3) the residual slow rate stage. The initial constant rate stage is the maximum or potential rate of drying that occurs when the soil surface is at or near saturation. The water that is available from storage and liquid phase flow to the surface (hydraulic conductivity) is sufficient to supply enough water to satisfy the evaporative demand. The evaporation from this stage of evaporation is similar to evaporation from a free water surface. This initial constant rate stage is controlled by the climatic conditions. The intermediate falling rate stage begins when the conductive properties of the soil can no longer supply enough water to the surface to maintain the maximum potential rate of evaporation. This stage of evaporation is also referred to as the soil-profile condition stage because soil conditions are now the dominant factor controlling the rate of evaporation. The third stage of evaporation gradually develops as the soil surface becomes dry. The primary mode of water movement near the surface in this stage of evaporation is due to vapour diffusion.

Transpiration is the evaporation counterpart for plants. Plants have the ability to absorb water through the root systems and release water to the atmosphere through leaf transpiration. The rate of transpiration depends on potential evaporation and the degree
of plant development as long as the soil water is not limiting. Transpiration is controlled by the same climatic conditions that control the initial constant rate stage of evaporation. In addition, transpiration is also affected by the plant type and density. The maximum amount of transpiration (i.e. the potential transpiration) is a function of the leaf area index. Transpiration increases non-linearly with increases in the leaf area index. The leaf area index measures the leaf surface that is exposed per unit of ground surface area. Increasing suction within the soil profile during drying causes the vegetation to begin to stress causing the amount of transpiration to decrease from its potential rate. Transpiration ceases once a plant's wilting point is reached. The wilting point is defined as the point at which the rate of water being transpired is greater than the root water uptake (Swanson, 1995; Viessman and Lewis, 1996)

Prediction of soil evaporative fluxes is more complex than that of infiltration because the actual rate of evaporation is a function of both the rate of potential evaporation and the suction in the soil at the ground surface. Potential evaporation is solely a function of climatic conditions (i.e. not dependent on soil conditions). A soil will evaporate at this rate as long as there is a sufficient supply of water to maintain saturated soil conditions. Potential evaporation is difficult to measure because it does not represent the actual transfer of water to the atmosphere. Many researchers have developed methods to estimate the potential rate of evaporation. Thornthwaite (1948) was one of the first researchers to develop a method for estimating potential evaporation. Other methods available for estimating potential evaporation include the Priestly-Taylor method (Priestly and Taylor, 1972), and the Penman method (Penman, 1948). Penman’s method for estimating potential evaporation is one of the most popular and accepted methods developed.

Penman’s method promotes an understanding of the physical process of evaporation from soil surfaces. This original formulation is simple, realistic and only requires routine meteorological data. The main assumption for Penman’s method is that the soil surface is saturated at all times. This assumption is also the main disadvantage of this method.
Wilson (1990) modified Penman’s method to calculate the rate of soil evaporative fluxes from any soil surface. The Penman-Wilson relationship is illustrated as follows:

\[
E = \frac{\Delta Q + vE_a}{\Delta + vA} \tag{3-10}
\]

where \( E \) is the vertical evaporation (mm/day), \( \Delta \) is the slope of the saturation vapour pressure versus temperature curve at the current temperature of the air (kPa/°C), \( Q \) is the net radiant energy available at the surface (mm/day), \( v \) is the psychrometric constant, \( E_a \) is equal to \( f(u)e_a(B-A) \), \( f(u) \) is the function dependent upon wind speed, surface roughness and eddy diffusion which is equal to \( 2.63(1+(0.537/3.6)U_a) \), \( U_a \) is the wind speed (km/h), \( e_a \) is the vapour pressure in the air above the evaporating surface (kPa), \( B \) is the inverse of the relative humidity of the air, and \( A \) is the inverse of the relative humidity of the soil surface.

The Penman-Wilson formulation (eqn 3-10) accounts for the net radiation, the wind speed and the relative humidity of both air and soil surfaces in calculating the rate of evaporation from an unsaturated soil surface. The Penman-Wilson formulation reduces to the Penman formulation when the soil surface is saturated. At saturation the relative humidity is equal to 100% and \( A \) is equal to unity. The actual evaporative flux can be calculated based on the potential evaporation of the current time step if there is no measured or estimated net surface radiation data, but measured potential evaporation data is available.

### 3.2.2 Infiltration

Infiltration is the downward movement of liquid water into a soil. When precipitation reaches the ground surface, it infiltrates at a rate that decreases with time. Horton (1933) found that there is a limiting curve that defines infiltration rate versus time for any given soil, as seen in Figure 3-2. The limiting curve is also described as the infiltration capacity for soil.
The initial flat portion of the curve indicates that the infiltration capacity is sufficient to accept the precipitation. The point on the limiting curve where the decline in infiltration rate begins is the point where ponding occurs. This point occurs because the infiltration capacity of the soil is less than the rate of precipitation. The infiltration capacity declines until it reaches a minimum infiltration rate. This minimum infiltration rate is equivalent to the saturated hydraulic conductivity of the soil in the case where there is no lower barrier to water movement.

There are two options for how precipitation can be applied in the model. Precipitation is applied in the form of a surface flux boundary condition that is equal to precipitation minus actual evaporation. If the amount of precipitation is less than the anticipated actual evaporation over the next step, the applied surface flux boundary conditions will be negative. The applied surface flux boundary is positive if the amount of precipitation is greater than the actual evapotranspiration anticipated over the next time step (Geo-Slope Int. Ltd., 1991).

![Typical Soil Infiltration Curve](image)

Figure 3-2: Typical Soil Infiltration Curve (after Swanson, 1995)
When a positive surface flux boundary condition is applied to the system the pressures at the ground surface can become positive in the model as the precipitation is forced to enter the model as infiltration. If the pressures do become positive values, the solution of that time step will converge to these positive pressures and cause the surface nodes to be changed to a head boundary condition. The time step will then be repeated in order to converge the surface nodes to this new head boundary condition. Once convergence of the head boundary condition is reached, the computed flux is checked against the original flux applied. If the computed flux is less than the original flux, runoff is calculated as:

\[ \text{Runoff} = \text{Precipitation} - AE - \text{Infiltration} \]  
\[ [3-11] \]

where AE is the actual evapotranspiration. Runoff is only applied as a negative value if previous surface runoff has been stored down slope and then reapplied as infiltration (Geo-Slope Int. Ltd., 1991).

### 3.2.3 Surface Temperatures

The surface temperature values are required for the solution of the moisture and heat flow equations. Surface temperature is estimated with the relationship proposed by Wilson (1990):

\[ T_s = T_a + \frac{1}{v f(u)} (Q - E) \]  
\[ [3-12] \]

where \( T_s \) is the temperature at the soil surface (°C), \( T_a \) is the temperature in the air above the soil surface (°C), \( v \) is the psychrometric constant (kPa°C), \( Q \) is the net radiant energy available at the surface (mm/day), \( E \) is the vertical evaporative flux (mm/day), \( f(u) \) is a function dependent upon wind speed (mm/day/kPa), surface roughness and eddy diffusion that is equal to \( 2.63(1 + (0.537/U_a)^3) \), \( U_a \) is the wind (km/hr).
3.3 Soil Properties

The hydraulic and thermal soil properties required for coupled heat and mass transfer are described in the following sections.

3.3.1 Hydraulic Properties

The hydraulic properties required for solving the moisture flow equation include the functions of volumetric water content and hydraulic conductivity expressed in terms of negative pore water pressure (i.e. suction).

Volumetric Water Content Relationship

The volumetric water content versus pore water pressure curve is also described as the soil-water characteristic curve or moisture retention curve. The soil water characteristic curve is fundamental to water flow formulations in the unsaturated zone because it defines the soils ability to retain water under changes in negative pore water pressure (Lam et al., 1987; Wilson et al., 1994 and Barbour et al., 1995; Swanson, 1995).

Figure 3-3 shows a typical storage function. The slope of the curve represents the volume of water retained or released by a change in pore water pressure. In the negative pore water pressure range the slope is referred to as $m_2^w$, and in the positive pore water pressure range the slope is equivalent to the soils coefficient of compressibility for one-dimensional consolidation, $m_v$ (Lam et al., 1987; Geo-Slope Int. Ltd., 1991; Barbour et al., 1995; Swanson, 1995). When the degree of saturation is 100% (zero suction), the volumetric water content is equal to the porosity of the soil. The suction corresponding to the point in the curve where there is a sharp drop in water content is referred to as the air entry value (AEV). The AEV indicates the suction value at which the soil will begin to desaturate. Applying suctions that are less than the AEV will produce only a change in the volumetric water content due to soil compressibility. Increasing suctions beyond the AEV causes the volumetric water content to decrease causing the soil to desaturate.
Desaturation due to drainage alone will continue until the suction value corresponds to the residual suction (Geo-Slope Int. Ltd., 1991; Swanson, 1995).

Measuring the volumetric water content function is not especially difficult in the laboratory, but can be quite time consuming and expensive. It is somewhat beneficial to use predictive methods to develop the necessary volumetric water content functions. Predictive methods can be completed using either a closed-form solution that requires user-specified curve fitting parameters or estimation methods that use a measured grain size distribution curve. VADOSE/W has four predictive methods available to develop volumetric water content relationships. Van Genuchten (1980) and Fredlund and Xing (1994) developed closed form solutions to represent the volumetric water content functions. The methods available based on predicting the volumetric water content function from grain size distribution are the Arya and Paris (1981) method and the modified Kovacs method (Aubertin et al., 2001).

Figure 3-3: Typical Soil-Water Characteristic Curve (after Swanson, 1995)
Hydraulic Conductivity Relationship

The hydraulic conductivity function describes a soil's ability to transmit water by liquid flow. Liquid flow in soils can be represented by water filled webs of interconnected but continuous conduits. Under saturated conditions all conduits within the soil profile are filled with water allowing all pathways to be available for moisture flow. The hydraulic conductivity of a soil is therefore at its maximum value under saturated conditions. Decreasing the water content has the ability to decrease the size and number of water filled conduits, thereby reducing the capacity to conduct water through the soil. Since the water content is progressively reduced by increases in suction, the larger conduits are emptied early in the process of desaturation and the smaller conduits are left full of liquid. As the soil is drying, the capacity for the soil to conduct moisture along the water filled conduits disappears. A conduit that is full of air is not effective as a conductor; it in fact becomes an obstacle. Moisture that was originally passed through the conduit that was full of moisture is now deflected around the air filled conduit. The introduction of these obstacles causes the hydraulic conductivity value to decrease (Geo-Slope Int. Ltd., 1991; Swanson, 1995).

![Graph of typical hydraulic conductivity function](image-url)

Figure 3-4: Typical Hydraulic Conductivity Function (after Geo-Slope Int. Ltd., 1991)
Measuring the hydraulic conductivity function can be a time consuming and expensive procedure. It is possible, however, to develop the function using predictive methods. VADOSE/W has three built-in predictive methods (Green and Corey, 1971; and Van Genuchten, 1980; Fredlund et al., 1994) available to estimate the hydraulic conductivity function once the volumetric water content function and the saturated hydraulic conductivity value have been defined. The saturated hydraulic conductivity can be determined through laboratory or field tests.

3.3.2 Thermal Properties

Thermal properties of soils are closely related to the water content of a soil. The thermal property functions required within VADOSE/W include the thermal conductivity and the specific heat as a function of volumetric water content. These thermal properties are required to solve the conductive heat transfer component of the coupled finite element equations. Thermal functions must be defined over a range of water contents (i.e. from complete saturation to complete dryness) because the thermal properties of a soil increase with increasing water content (Baver et al., 1972; Swanson, 1995).

**Thermal Conductivity Relationship**

The thermal conductivity function describes the ability of a soil to transmit heat, just as the hydraulic conductivity function describes a soil's ability to transmit water. The rate at which a soil transmits heat is dependent upon the temperature gradient and the thermal conductivity of the soil. Thermal conductivity can therefore be defined as the amount of heat that flows through a unit area of soil in a unit time, under a unit temperature gradient (Geo-Slope Int. Ltd., 1991; Swanson, 1995).

Various empirical or semi-empirical methods have been developed to predict the thermal conductivity relationship. The most general method developed was that of Johansen (1975). This method provides estimations of thermal conductivity for dry soils, saturated
soils and unsaturated soils. For dry, natural soils, the dry thermal conductivity, $k_{dry}$, can be estimated based on its dry density, $\gamma_d$ using:

$$k_{dry} = \frac{0.135\gamma_d + 64.7}{2700 - 0.947\gamma_d} \pm 20\%$$  \[3-13\]

For dry crushed rock materials, the dry thermal conductivity, $k_{dry}$, can be estimated based on its porosity, $n$:

$$k_{dry} = 0.039n^{-22} \pm 25\%$$  \[3-14\]

For saturated unfrozen soil, the saturated thermal conductivity, $k_{sat}$, is estimated based on the thermal conductivity of its components and their respective volume fractions:

$$k_{sat} = k_s (1-n)k_w n$$  \[3-15\]

where $k_s$ is the thermal conductivity of soil particles, and $k_w$ is the thermal conductivity of pore water. For saturated unfrozen soils containing some unfrozen water content, $w_u$, the saturated thermal conductivity, $k_{sat}$, becomes:

$$k_{sat} = k_s (1-n)k_i (n-w_u)k_w (w_u)$$  \[3-16\]

where $k_i$ is the thermal conductivity of ice. For unsaturated soils, the thermal conductivity, $k_{unsat}$ is estimated based on saturated conductivity, dry conductivity and the degree of saturation, $S$:

$$k_{unsat} = \left(k_{sat} - k_{dry}\right)K_e + k_{dry}$$  \[3-17\]

where $K_e = 0.7\log S + 1.0$ for unfrozen coarse soil, $K_e = \log S + 1.0$ for unfrozen fine soil and $K_e = \log S$ for frozen soil (Geo-Slope Int. Ltd., 1991).
Specific Heat Relationship

Specific heat is defined as the amount of stored heat that is required to change the temperature of a unit volume of soil by one degree Celsius. If specific heat is described on a unit weight basis, the quantity of heat is referred to as the specific heat capacity. If specific heat is expressed on a unit volume basis, the quantity of heat is referred to as the volumetric specific heat capacity. Volumetric specific heat is calculated as:

\[ C_v = C_m \rho_s \]  

[3-18]

where \( C_v \) is the volumetric specific heat, \( C_m \) is the mass specific heat and \( \rho_s \) is the mass density of the soil (Geo-Slope Int. Ltd., 1991; Swanson, 1995).

The volumetric heat capacity is approximated by the dry density of the soil and the sum of specific heat capacities of its different constituents (i.e. soil particles, water, ice and air). The air component is often neglected because it is such a small value compared to that of the other constituents. The general equation used for estimating unfrozen and frozen volumetric heat capacities is:

\[ C = c \gamma = \gamma_d \left[ c_s + c_w w_u + c_i w_f \right] \]  

[3-19]

where \( C \) is the volumetric heat capacity, \( c \) is the specific heat capacity, \( \gamma \) is the bulk density, \( \gamma_d \) is the dry density, \( c_s \) is the soil particle specific heat, \( c_w \) is the specific heat of water, \( c_i \) is the specific heat of ice, \( w_u \) is the unfrozen water content, and \( w_f \) is the frozen water content. If a phase change is involved, solving for volumetric specific heat is not as simple. The latent heat absorbed or released during the ice-water phase transformation must be included in estimating the volumetric water content when a phase change is included.
3.4 Vegetation Properties

Vegetation is significant to the overall evapotranspiration process. These effects are incorporated using a nodal vegetative uptake source term that is combined with a surface energy term based on canopy cover. Actual root water uptake depends on root depth and density and negative pore water pressures.

The anticipated root depth and root distribution is required for the duration of the growing season. It specifies the depth from which vegetation will extract water from the soil profile. How the energy at the surface is partitioned between that available for direct evaporation from the soil and that which is available to the plants in their attempt to transpire water, is controlled by the leaf area index (Geo-Slope Int. Ltd., 1991).

If there is a lack of plant water and/or a high evaporative demand, most plants will react by closing the stoma, which reduces the amount of transpiration and metabolic reactions. Plants will eventually reach their wilting point with continued and increasing stress. In order to determine the decrease in the ability of plants to draw water as the negative pore water pressures increase, a plant limiting moisture function is applied in the model.

Leaf area index (LAI) controls how the energy at the surface is divided between direct evaporation from the soil and plants ability to transpire water. Therefore LAI is used in VADOSE/W to reduce the amount of net radiation intercepting the soil surface, which in turn is reducing the computed Actual Evaporation (Krahn, 2004).

Once all the functions are defined and applied to a climate surface within VADOSE/W the Actual Evaporation is computed and modified by equation 3-20 such that a portion of the available energy is partitioned to the plants.

\[
AE = AE \times \left\{1 - \left(0.21 + 0.7\sqrt{LAI}\right)\right\} \tag{3-20}
\]

where \(LAI\) is the specified leaf area index for the day in question.
The modified evaporation value equals zero if the LAI value is greater than 2.7 and equals its full value if LAI is less than 0.1. Potential transpiration, PT, is determined through its relation to potential evaporation and will be equal to potential evaporation if the LAI value is greater than 2.7. PT is calculated using the following equation.

\[ PT = PE\left(-0.21 + 0.7\sqrt{LAI}\right) \]

where \(PE\) is the potential evaporation (mm/day).

Potential transpiration is defined as the amount of energy available to the plant. When the soil is saturated, all energy is applied to the roots according to the root depth and root shape functions. However, if the soil is partially saturated the actual transpiration value is reduced according to the plant moisture limiting function (Krahn, 2004).

\[ AT = PRU \times PML \]

\[ PRU = \frac{2PT}{R_T} \left(1 - \frac{R_n}{R_T}\right)A_n \]

where \(AT\) is the actual nodal transpiration, \(RT\) is the total thickness of the root zone, \(R_n\) is the depth to the node in question, \(A_n\) is the nodal contributing area of the node in question and \(PML\) is the plant moisture limiting function value at the current nodal soil negative pore water pressure.

### 3.5 Theoretical Background Summary

The numerical modeling package VADOSE/W has three vital requirements for simulating moisture movement in highway pavement structures. These requirements include:

1. Geometry
2. Material Properties, and
3. Boundary Conditions

The geometry must be a simplified version of the actual field geometry. The field geometry conditions of highway pavement structures are described in detail in Chapter 4: Site Characterization. The material properties required must include both soil and vegetative properties that are representative of field conditions.

VADOSE/W requires both hydraulic properties and thermal properties for each soil type. The hydraulic properties consist of a volumetric water content function and a hydraulic conductivity function. These properties can be entered in manually, or estimated. In order to estimate the volumetric water content of a soil material the measured grain size distribution curve must be available. The hydraulic conductivity function is estimated using the volumetric water content function and the saturated hydraulic conductivity. The thermal properties required for numerical modelling in VADOSE/W include the thermal conductivity and the volumetric specific heat. These functions can also be entered manually or estimated based on volumetric water content functions (Geo-Slope International Ltd., 1991).

The vegetative properties required within VADOSE/W include the leaf area index, root depth and limiting moisture functions. The leaf area index is a function of the plants growing season and grass quality. The root depth function is dependent upon the surface cover region applied in the numerical analysis. The limiting moisture function depicts the transpiration ability of the vegetation available at the site.

The boundary conditions applied for numerical analysis generally include both hydraulic and thermal conditions in association with actual climate data. The boundary conditions applied to the analysis of highway pavement structures are discussed in Chapter 6: Numerical Modelling.
Chapter 4 Site Characterization

This chapter presents the highway site chosen to represent a typical standard highway pavement structure for Saskatchewan, Canada. The soil gradations selected by the Saskatchewan Department of Highways and Transportation (SDHT) for the hot mix asphalt surface layer, granular base layer and granular subbase layer are also presented. The vegetation in the sideslope and ditch areas of the site are typical of those used on other major roadways in Saskatchewan.

Local climate normals are also presented for Saskatchewan in and around the Saskatoon area, including average precipitation, maximum and minimum air temperatures, maximum and minimum relative humidities, average wind speeds, and average monthly net solar radiation.

A description of the fieldwork completed on Highway 16 is also presented within this chapter.

4.1 Site Description

The highway site is located within the Central Region of Saskatchewan, Canada. The selected highway site is Highway 16 Control-Section 20 (Highway 16 - 20), beginning just west of the town of Clavet, Saskatchewan and continuing into Saskatoon, Saskatchewan. Only a small portion of this section was chosen for the research study. This section is located just past the Boychuck Drive exit in Saskatoon continuing for about 10 kilometres east of the city. Figure 4-1 shows the location of the site.
Highway 16 - 20 is classified as a standard highway pavement structure in the Saskatchewan road network. This section of highway has two westbound lanes and two eastbound lanes separated by a median ditch, as seen in Figure 4-2. Standard highway pavements in Saskatchewan consist of a hot mix asphalt concrete surface layer, a granular base and subbase material, and a compacted subgrade (SDHT, 2004b).

The main reason for selecting this site is that it represents a typical design of a standard pavement structure. Standard pavement structures are of particular interest to this research because in recent years SDHT have found that these highways are not lasting to their potential design life. It has been found that moisture ingress and accumulation within the system is causing a significant amount of structural surface damage. This increase in structural surface damage is resulting in increasing maintenance costs within the first few years of service (Pessaran, 1994).
4.2 Highway Pavement Details

A simplified geometry and boundary conditions were developed for the numerical analyses based on the details of a typical highway pavement structures. It must be understood that the purpose is to establish as simple a model as possible which still accurately represents the mechanisms and processes of interest in the ‘real’ cross-section. Using extraneous details will only create an overly complex model, creating difficulties and often impossibilities when interpreting and/or obtaining results.

The simulation model of the highway pavement structure was developed with a view to answering the specific objectives as discussed in Chapter 1. These focused on evaluating the coupled heat and moisture interactions between typical standard highway pavement structures and atmosphere as well as evaluating the relative impact of design changes such as shoulder paving and steepening of the sideslope.

The design of standard highway pavement structures in Saskatchewan has changed over time. In the 1960s, standard highway pavement structures were typically undivided sections of road. The cross-section design of these highway pavement structures consisted of 3.7 m driving lanes, 3 m shoulders and 3:1 sideslopes, as seen in Figure 4-3. SDHT moved away from this design of highway pavement structures due to safety concerns with the steepness of the sideslope. The current guidelines for sideslopes are as follows:

1. 6:1 on four lane roads,
2. 5:1 on high volume two lane roads and
(3) 4:1 on all other two lane roads.

The majority of the current standard highway pavement structures are designed as divided sections of road (i.e. they are designed as four lane roads) due to increases in traffic volume. Therefore, the cross section design of current standard highway pavement structures includes two 3.7 m driving lanes, a 3 m exterior shoulder and a 1 m interior shoulder with a 6:1 sideslope, as seen in Figure 4-4.

The design of Highway 16 - 20 is representative of typical standard highway pavement structures in Saskatchewan and consequently the specific pavement structure geometry for this site will be used in the model. The design of this site in the 1960s consisted of a driving lane with a 102 mm hot mix asphalt (HMAC) surface layer, a 152 mm granular base layer and a 165 mm granular subbase layer. The shoulder was unpaved with a 152 mm granular base layer and a 165 mm granular subbase layer. The current geometry at this site has a driving lane with a 110 mm HMAC surface layer, a 220 mm granular base layer and a 600 mm granular base layer. The shoulder, however, is paved with a 50 mm hot mix asphalt surface layer overlaying a 280 mm granular base layer and 600 mm granular subbase layer (SDHT, 2004b).

![Figure 4-3: Older Highway Pavement Structure Design in Saskatchewan, Canada (SDHT, 1988)](image-url)
4.2.1 Soil Properties

Numerous alternatives in the selection of the gradation of the various soil layers within the pavement structure are available. For example, Tables 4-1, 4-2 and 4-3 indicate the SDHT gradations for HMAC surface layers, Granular Base layers and Granular Subbase layers, respectively. Each pavement layer has an ideal gradation and numerous less ideal gradations, in decreasing order. When choosing appropriate gradations for the design of a highway pavement structure two factors are considered. The first, and most important factor is the economics of the available material with the material located at the nearest gravel pit being given primary consideration. The second factor considered is the design purpose for the material, for example, what type of highway structure is being constructed, and what layers this highway structure requires. SDHT then determines the most ideal and cost economical gradation mix available for the construction of their highway pavement design (SDHT, 1988).

Typical pavement structures in Saskatchewan are generally constructed using an HMAC Type 70 gradation with asphalt content of 5.4% by weight of aggregate. The minimum amount of voids in the mineral aggregate (VMA) is 14% for the HMAC surface layer. The base course gradation commonly used is a Type 33. A Type 10 subbase gradation was selected for this research study. Figure 4-5 represents the gradation curves of each layer of a typical Saskatchewan pavement structure (SDHT, 1988).
Table 4-1: Saskatchewan’s Department of Highways and Transportation Soil Gradations of Hot Mix Asphalt Concrete surface layers (SDHT, 1988)

<table>
<thead>
<tr>
<th>Sieve Designation</th>
<th>Percent by Weight Passing Canadian Standard Sieves</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type</td>
</tr>
<tr>
<td>18.0 mm</td>
<td>100</td>
</tr>
<tr>
<td>16.0 mm</td>
<td>100</td>
</tr>
<tr>
<td>12.5 mm</td>
<td>75 - 100</td>
</tr>
<tr>
<td>9.0 mm</td>
<td>75 - 100</td>
</tr>
<tr>
<td>5.0 mm</td>
<td>40 - 65</td>
</tr>
<tr>
<td>2.0 mm</td>
<td>25 - 46</td>
</tr>
<tr>
<td>900 μm</td>
<td>17 - 33</td>
</tr>
<tr>
<td>400 μm</td>
<td>13 - 25</td>
</tr>
<tr>
<td>160 μm</td>
<td>7 - 15</td>
</tr>
<tr>
<td>71 μm</td>
<td>3 - 9</td>
</tr>
</tbody>
</table>

Table 4-2: Saskatchewan’s Department of Highways and Transportation Soil Gradations of Granular Base layers (SDHT, 1988)

<table>
<thead>
<tr>
<th>Sieve Designation</th>
<th>Percent by Weight Passing Canadian Metric Sieve Sizes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type</td>
</tr>
<tr>
<td>40.0 mm</td>
<td>100</td>
</tr>
<tr>
<td>31.5 mm</td>
<td>95 - 100</td>
</tr>
<tr>
<td>25.0 mm</td>
<td>90 - 100</td>
</tr>
<tr>
<td>18.0 mm</td>
<td>70 - 90</td>
</tr>
<tr>
<td>12.5 mm</td>
<td>65 - 83</td>
</tr>
<tr>
<td>5.0 mm</td>
<td>35 - 60</td>
</tr>
<tr>
<td>2.0 mm</td>
<td>22 - 38</td>
</tr>
<tr>
<td>160 μm</td>
<td>6 - 12</td>
</tr>
<tr>
<td>71 μm</td>
<td>5 - 10</td>
</tr>
</tbody>
</table>

Plasticity Index  0 - 6  0 - 6  0 - 6  0 - 6  0 - 6  0 - 6  0 - 6  0 - 6  0 - 4
Table 4-3: Saskatchewan’s Department of Highways and Transportation Soil Gradations of Granular Subbase layers (SDHT, 1988)

<table>
<thead>
<tr>
<th>Sieve Designation</th>
<th>Percent by Weight Passing U.S. Standard Sieves</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type</td>
</tr>
<tr>
<td>50 mm</td>
<td>100</td>
</tr>
<tr>
<td>71μm</td>
<td>0 - 20</td>
</tr>
<tr>
<td>Plasticity Index</td>
<td>0 - 6</td>
</tr>
</tbody>
</table>

Figure 4-5: Pavement Layer Gradations for Highway 16 Cross Section 20

4.2.2. Vegetation

The sideslope and ditch areas are host to a mixture of vegetation. The vegetation is provided to make the construction zone seem aesthetically pleasing to traffic users and provide a more efficient form of evapotranspiration for the highway pavement structure. The seed mixture that exists at the site is given in Table 4-4. The seed mixture consists of five grasses proportioned equally over the sideslope and ditch areas (SDHT, 2004c).
The growing season for most of these grasses starts around the middle of May and continues through to the middle of August. The root depths for the grasses in the grass mixture defined in Table 4-4 vary. The root depth for Brome Grass (*Bromus Inermis*) can extend to 2 m. The root of Orchard Grass (*Dactylis Glomerata*) can reach depths of 1.3 m. Dahurian Wild Rye (*Elymus Dauricus*) has a shallow root system no more than 15 cm in depth. Timothy Grass (*Phleum Pratense*) has a very shallow root depth, where Crested Wheat Grass (*Agropyron Desertorum*) has a deep extensive root system (Loudon, 1866; Beal, 1896; Cathey, 2002). Krahn (2004) indicates that the depth over which the root depth function is applied can not exceed the total thickness of the cover nodes within the mesh defined within VADOSE/W. Therefore, the root depth function for the grass mixture can be seen in Figure 4-6, which is dependent upon vegetation type.

The ability of the plant to draw water from the surrounding areas relies on its moisture limiting function. The moisture limiting function indicates when the ability of the grass to draw water begins to shutdown and when it can no longer draw water from the soil system. The moisture limiting function for the seed mixture seen on Saskatchewan highways is illustrated in Figure 4-7, and dependent upon climate (Loudon, 1866; Beal, 1896; Cathey, 2002).

<table>
<thead>
<tr>
<th>Breed</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brome Grass</td>
<td>20%</td>
</tr>
<tr>
<td>Timothy</td>
<td>20%</td>
</tr>
<tr>
<td>Crested Wheat Grass</td>
<td>20%</td>
</tr>
<tr>
<td>Dahurian Wild Rye</td>
<td>20%</td>
</tr>
<tr>
<td>Orchard Grass</td>
<td>20%</td>
</tr>
</tbody>
</table>
Figure 4-6: Root Depth Function of Vegetation

Figure 4-7: Moisture Limiting Function of Vegetation
4.3 Climate Conditions

Local climate normals are presented in the following section. These values provide a general characterization of the climatic conditions seen in Saskatchewan. Two weather stations in close proximity were used to develop a typical climate data set for Saskatchewan. Two weather stations were required due to the difficulty in obtaining annual average daily information on maximum and minimum air temperature, maximum and minimum relative humidity, wind speed, precipitation and net solar radiation for one weather station. The two weather stations used are located in Outlook and Saskatoon, Saskatchewan.

Environment Canada (2004) reports that Saskatchewan has an annual precipitation of 360 mm including 260 mm of rainfall and approximately 100 mm of snowfall. The annual potential evaporation for Saskatoon is approximately 900 mm, which is much greater than the average annual precipitation. The average monthly precipitation is shown in Figure 4-8(a). The amount of frost-free days in Saskatchewan varies with location. For example the Saskatoon area has about 111 frost-free days where Regina sees 123 frost-free days and the valley lands along the South Saskatchewan River ranges from 150 to 160 frost-free days (PFRA, 2000).

Average monthly air temperatures for Saskatchewan are shown in Figure 4-8(b). An average maximum air temperature of 25 °C occurs in August. The average minimum air temperature occurs in December at a temperature of −22 °C. The highest air temperatures are generally experienced in the months of June, July and August, while the lowest air temperatures are seen in the months of December, January and February.

The monthly values of maximum and minimum relative humidity are shown in Figure 4-8(c). The maximum monthly relative humidity occurs in July at a value of 87%. The minimum monthly relative humidity is 33% in September. The maximum monthly relative humidity varies little over the year, whereas the minimum monthly relative humidity can vary over the year. The minimum relative humidity values decrease from
Figure 4-8: Climate Data (a) Average Monthly Precipitation (b) Monthly Air Temperature (c) Monthly Relative Humidity and (d) Monthly Average Wind Speed (Environment Canada, 2004)
January to September. For the last three months, the minimum relative humidity increases.

The monthly average wind speeds for Saskatchewan are shown in Figure 4-8(d). Wind speed varies between the months of January and December. Between January and June the monthly wind speed values are greater than those in July through December. The wind speed values between January and June see vary little variation, with an average value of 8.5 m/s. The wind speed values for the months of July through December see a great deal of variation. The wind speed decreases from 5.7 m/s in July to 2.9 m/s in December. The average wind speed value over the entire year is 6.76 m/s.

Figure 4-9 illustrates the average monthly net solar radiation for Saskatchewan. Net solar radiation is a measure of the incoming radiation incident on the earth's surface minus the outgoing energy radiated by the earth itself (Geo-Slope International Ltd., 1991). The average monthly net solar radiation is greatest during July and August (0.5 MJ/m²), due to warm air temperatures and longer sunshine periods. Average monthly net solar radiation is lowest during the December and January, -0.08 MJ/m².

Figure 4-9 Monthly Average Net Radiation Values for Saskatchewan (Environment Canada, 2004)
4.4 Sample Collection

SDHT extracted four-inch diameter core samples from various locations along the highway site. The samples are used to determine relevant hydraulic properties of each layer within the highway pavement structure as input to the simulation. Figure 4-10 shows the extraction equipment used by SDHT to collect the necessary cores. The equipment used to extract asphalt cores was a portable drill. The drill is equipped with a power source suitable for driving the core barrel. The drill is equipped with a source of air or water supply to cool the barrel during drilling operations (SDHT, 1995).

The samples were collected at three locations along Highway 16 - 20. The locations were divided between older and newer constructed sections of the highway site with two newer sections and one older section being chosen. The two newer constructed sections included an eastbound section and a westbound section. Samples were extracted from four locations along each of the three sections. The sample locations included the inner wheel path, outer wheel path, and between the wheel paths and shoulder, as seen in Figure 4-11. An extra sample from the inner wheel path was also collected at the older section. A total of thirteen samples were collected along Highway 16 - 20.

Figure 4-10: Saskatchewan's Department of Highways and Transportation Coring Equipment
The thirteen samples were collected during the winter season to ensure that the ground was frozen. Frozen conditions were required to ensure that the base and subbase samples remained relatively intact, ready for laboratory testing. The eight samples extracted from the newer section of Highway 16 were extracted January 6, 2004. The five samples extracted from the older section of Highway 16 were extracted on March 9, 2004. By this time in March the base and subbase layers of the highway pavement structure were already thawed. This made it difficult to collect base and subbase cores. Only one bagged sample of base material was extracted for testing.

The intent was to test all highway layers for their hydraulic properties, however it was decided that due to time constraints the hydraulic properties of the base and subbase would be estimated based on their appropriate gradation curves. The hydraulic properties of the HMAC surface layer material was tested in the laboratory program described in Chapter 5.

4.5 Summary

Highway 16, just southeast of Saskatoon, is representative of typical standard highway pavement structures in Saskatchewan. The design of typical highway pavement structures has changed over time. Older standard highway pavements are typically undivided road networks where current standard highway pavements are generally
divided road networks. Other changes in the design of standard pavements structures over time include changes to the steepness of the sideslope and the condition of the shoulder area. Older highway pavement structures were designed with a 3:1 sideslope. The design of current highway pavement structures consists of paved shoulders and a 6:1 sideslope.

The material gradations used by SDHT for the selected site include Type 70 HMAC surface layer, Type 33 granular base layer and Type 10 granular subbase layer. The subgrade layer of highway pavement structures in Saskatchewan is typically compacted clayey silt material. The typical vegetation planted along the ditch and backslope is a mixture of Brome Grass, Timothy, Crested Wheat Rye, Dahurian Wild Rye and Orchard Grass.

The local climatic norms for Saskatoon include an annual potential evaporation of 900 mm, much greater than the annual precipitation of 350 mm. Temperature is minimal during the months of December through February and is greatest during June through August. The average annual wind speed for Saskatoon, Saskatchewan is 6.76 m/s.

Thirteen core samples were extracted along Highway 16 - 20 at the inner wheel path, outer wheel path, between wheel path and shoulder. These thirteen cores were extracted to represent materials used during old and current highway pavement construction. The HMAC surface layer of each of the thirteen extracted samples were tested for their hydraulic properties, as discussed in Chapter 5.
Chapter 5 Laboratory Testing

This chapter describes the laboratory component of this research. The laboratory testing procedures as well as the results are discussed. The hot mix asphalt (HMA) cores collected in the field component were tested for various hydraulic material properties. The measured material properties included the saturated hydraulic conductivity, the soil-water characteristic curves, vapour diffusion parameters and air permeability values.

5.1 Hydraulic Conductivity

The hydraulic conductivity of a soil is the property controlling the rate at which water can move through a soil. The hydraulic conductivity is a function of void ratio and degree of saturation (Huang et al., 1998). The maximum rate at which water can move through a soil occurs at saturation. As a soil becomes unsaturated, the hydraulic conductivity decreases as a function of suction.

5.1.1 Testing Procedures

Various methods are available to measure the hydraulic conductivity of a soil. The use of a flexible wall triaxial permeameter, as shown in Figure 5-1, is one of the more common methods available.
Figure 5-1: Triaxial Permeameter Cell (Osicki, 2004)
The triaxial cells are fabricated from stainless steel and are a modification of the stress-controlled isotropic cell developed by Ho (1988). The interchangeable base and top cap are spirally grooved, 2 mm wide by 2 mm deep, to ensure a uniform distribution between the entering permeating liquid (influent) and exiting permeating liquid (effluent) collection. The sample is placed between two porous stones and is enclosed with a flexible membrane. The flexible membrane is attached to the base and top cap using O-rings. The effluent line is connected to the base and top cap. All pressure lines are passed through the base of the cell. Transducers are located at the base of the permeameter to monitor the pressures applied to the sample. A differential transducer is used to measure the pressure head across the sample. The influent and effluent volumes are measured with the conventional twin burette system connected between the reservoir and the sample, as seen in Figure 5-2 (Osicki, 2004). The conventional twin burette volume change indicators are graduated 25 ml or 10 ml burettes with a precision of 0.05 ml or 0.02 ml, respectively. Influent and effluent volume changes provoke changes to the elevation of the water/kerosene interfaces in the graduated burettes (Osicki, 2004).

Figure 5-2: Triaxial Permeameter Control and Measurement System (Osicki, 2004)
Hydraulic conductivity tests are performed by passing influent upwards into the sample under pressure. The effluent exits the top of the sample where the backpressure can be maintained at a lower level to create a gradient. Air leakage is a concern through the gaps between the rubber membrane and the cap as well as through the pedestal into the specimen. Long term leakage observations have determined that with tight O-rings the triaxial permeameter can be used to measure hydraulic conductivity values as low as 5x10^{-11} m/s for a saturated soil, with an error of approximately ± 18% (Huang et al., 1998). A confining cell pressure, greater than the influent pressure, is applied to press the flexible membrane firmly against the sample. The confining pressure prevents sidewall leakage, and in conjunction with the influent and effluent pressures imposes an effective stress on the sample (Huang et al., 1998; Osicki, 2004).

In a typical test, distilled water is passed through the sample to establish a stable initial hydraulic conductivity before the test permeant liquid is introduced. Hydraulic conductivity, effluent volume and time increments are reported for each volume of effluent passed through the sample. These values are then used to plot the hydraulic conductivity versus pore volumes of permeant (Osicki, 2004).

Four-inch diameter flexible wall permeameters were used to test HMA samples using distilled water as the permeant. The HMA test specimens were cut using a diamond saw, as shown in Figure 5-3. The diamond saw consists of a diamond tip blade that is capable of cutting rock specimens. The field cores are positioned on the sliding metal guide of the saw. The metal guide is used to gently and accurately advance the specimen to the blade. The soil end of the sample was wrapped in plastic to keep the core intact. The sample was positioned to obtain a HMA sample approximately 5 cm in height. A water coolant was sprayed out from the edge of the blade to reduce the amount of friction produced between the specimen and the blade. The specimen is gently but firmly guided through the blade to make the appropriate cut (Hollocher, 2002).
Figure 5-3: Diamond Saw (a) Small Diamond Saw and (b) Large Diamond Saw (Hollocher, 2002)

Four-inch diameter flexible wall permeameters were used in order to reduce the effect of sidewall leakage and to increase the total flow through the specimens. The increased total flow through the specimens combined with the use of low gradients and a precise flow in and out measuring apparatus was intended to provide accuracy in test values and confirmation that equilibrium has been achieved or at least approached (Osicki, 2004).

5.1.2 Testing Results

The hydraulic conductivity of the HMA provides an approximation of the maximum infiltration rate in the absence of cracks through this material. Thirteen HMA samples were tested for saturated hydraulic conductivity, with results shown in Figure 5-4. The figure divides the samples between the age of the highway pavement structure and the location at which the samples were collected across the structure. The five samples collected at an older highway pavement section are represented in solid black, where the samples collected at newer pavement section are represented by blue outlined symbols. A detailed summary of the hydraulic conductivity results is presented in Appendix A.
Figure 5-4 shows that the hydraulic conductivity values group between the younger section of highway pavement and older section of highway pavement. The saturated hydraulic conductivity of the younger samples and the older samples differ by two orders of magnitude. The samples collected at the younger section of highway pavement seem to congregate around a saturated hydraulic conductivity of $1.1 \times 10^{-6}$ cm/s, whereas the samples from older pavement sections congregate around a hydraulic conductivity of $1.1 \times 10^{-8}$ cm/s. This may indicate that the newer HMA layers have the ability to allow more moisture to infiltrate into the sublayers of a highway pavement structure than older HMA layers.

The relationship of saturated hydraulic conductivity in granular soils, at a given temperature, and void ratio can be written as

$$k = \frac{C e^2}{1 + e}$$  \hspace{1cm} [5-1]$$

or

$$k = C' e^2$$  \hspace{1cm} [5-2]$$
where \( k \) is the saturated hydraulic conductivity of soil, \( e \) is equal to void ratio, and \( C \) and \( C' \) are equal to constants of proportionality. Equation 5-2 differs from equation 5-1 through the use of a soil's hydraulic radius (Taylor, 1948).

A parametric study completed by Kanitpong et al. (2001) found that the hydraulic conductivity of HMA is primarily controlled by void ratio. The relationship between void ratio and saturated hydraulic conductivity found by Kanitpong et al. (2001) is shown in equation 5-3.

\[
k = 6 \times 10^{13} e^{0.56}
\]  

Separating the results of Figure 5-4 into their location at which the samples were collected across Highway 16 introduces a trend between the hydraulic conductivity and void ratio. The general trend is that the saturated hydraulic conductivity increases as void ratio of the HMA samples increases by the relationship indicated below.

\[
k = 1 \times 10^{-6} e^{0.57}
\]  

The above relationship and the relationship determined by Kanitpong et al. (2001) indicate that there is more variation in saturated hydraulic conductivity than void ratio accounts for. Kanitpong et al. (2001) indicated that gradation and specimen thickness were also sensitive to hydraulic conductivity results.

The amount of scatter in the saturated hydraulic conductivity values for the HMA samples creates complexity in determining the exact saturated hydraulic conductivity of HMA. A mean saturated hydraulic conductivity value is therefore used to represent the saturated hydraulic conductivity of HMA. The geometric mean value of the thirteen HMA samples is \( 1.85 \times 10^{-7} \text{ cm/s} \).
An illustration of the amount of moisture capable of infiltrating through the HMA layer of a highway pavement structure in the absence of cracks is conducted using the geometric mean saturated hydraulic conductivity and the amount of precipitation seen during the summer period in Saskatchewan, Canada. The summer precipitation period is taken from the local climatic norms presented in Chapter 4 for the months of April through September. Six storm durations were used for this analysis, including a 24 hour storm, a 12 hour storm, a 6 hour storm, a 1 hour storm, a 30 minute storm and a 15 minute storm. Figure 5-5 illustrates the amount of precipitation that an HMA surface layer of a standard highway pavement structure will infiltrate over the six-storm duration for varying saturated hydraulic conductivities. For example, a 24 hour storm will allow 7.2 mm of precipitation to infiltrate into the sublayers of the highway pavement structure, whereas a 15 minute storm duration only allows 1.01x10^-2 mm of infiltration in the absence of cracks at a saturated hydraulic conductivity of 1.85x10^-7 cm/s. A detailed summary of the illustration calculations can be found in Appendix B.

Figure 5-5: Precipitation Infiltration Analysis for Hot Mix Asphalt with varying Saturated Hydraulic Conductivity Values
For design of structures in Saskatchewan the critical storm duration is approximately one hour (Smith, 2005). For this critical storm duration the mean saturated hydraulic conductivity of HMA, $1.85 \times 10^{-7}$ cm/s, will allow 0.162 mm of precipitation to infiltrate into the sublayers of a standard highway pavement structure in the absence of cracks. The amount of infiltration will change in direct relation to the saturated hydraulic conductivity, as shown in Figure 5-5. For example having an HMA surface layer with a saturated hydraulic conductivity of $1 \times 10^{-5}$ cm/s for a typical rainstorm in Saskatchewan allows 19 mm of rain to infiltrate into the sublayers. However, slightly changing the saturated hydraulic conductivity to $1 \times 10^{-9}$ cm/s will allow only $4.74 \times 10^{-6}$ mm of rain to infiltrate into the sublayers. Therefore, a slight change in the saturated hydraulic conductivity of HMA can drastically affect the amount of moisture infiltrating into the sublayers of a highway pavement structure even in the absence of cracks. Figure 5-5 also suggests that as saturated hydraulic conductivity decreases the infiltration becomes more controlled by rainfall intensity rather than the HMA saturated hydraulic conductivity.

### 5.2 Soil-Water Characteristic Curves

A moisture retention or soil-water characteristic curve defines the amount of water that is capable of being stored in a soil or similar porous medium. The soil-water characteristic curve relates gravimetric water content, volumetric water content or degree of saturation with suction. Most soil-water characteristic curves described in the literature are for sandy or loamy soils frequently encountered in agricultural and soil science disciplines. It is quite rare to find a soil water characteristic curve for HMA.

#### 5.2.1 Testing Procedures

Soil-water characteristic curves up to suctions of 1000 kPa are often measured using a pressure plate extractor. Osmotic desiccators are then used to determine the volumetric water content at suctions above 1000 kPa.
The pressure plate extractor was developed based on the fundamental definition of matric suction. Matric suction, $\psi$, is defined as the pressure difference across the air-water interface, namely $(u_a - u_w)$, where $u_a$ is the pore air pressure and $u_w$ is the pore water pressure. In a pressure plate extractor, the pore water pressure is maintained constant, usually at zero pressure, while the air pressure is elevated to produce the desired matric suction (Fredlund and Rahardjo, 1993; Tinjum et al., 1997).

Pressure plate extractors are cylindrical cells that are available in varying diameters. A saturated sample is placed into the pressure plate extractor; however, the sample must be in direct contact with the porous ceramic disk. The ceramic disk is located at the bottom of the inner chamber and is a high air entry disk (HAED) that allows water to readily flow through the ceramic disk, while restricting airflow up to a certain pressure; in other words it is separating the air phase from the water phase, as shown in Figure 5-6. The air pressure inside the cell is increased in increments causing pore water to be pushed out through the ceramic disk and air to enter the previously water filled pores. It is important that the air entering the pores in the sample be from the chamber and not from diffusion through the ceramic disk. The pressure required to cause air to flow through the disk is called the air entry pressure of the HAED being used. For example, HAED can have air entry pressures of 100 kPa, 200 kPa, 500 kPa or 1500 kPa (Geo-Slope International Ltd., 1991; Leong and Rahardjo, 1997; Tinjum et al., 1997).

The air pressure is increased in increments and maintained until the sample drainage is constant. This results in an incremental decrease in the water content of the sample. Higher air pressure increments are applied by using a compressed air system, which supplies air to the system from an inlet tube on top of the pressure plate extractor. In order to have more control of suctions, when the material has a low air entry value, the air pressure is kept equal to one atmosphere and matric suction is applied by applying a negative water pressure to the base of the cell through the use of a hanging column of water. This method is used for pressure increments up to 10 kPa, as shown in Figure 5-7 (Pessaran, 1994). At the end of the test, changes in weight of the cell, the dry weight of
the sample and the volume of the sample are used to compute the volumetric water content at each applied suction.

Figure 5-6: Pressure Plate Extractor (Tinjum et al., 1997)

Figure 5-7: Pressure Increment Procedure for Suctions up to 10 kPa
Although the measurement of the soil-water characteristic curves is conceptually simple, practical problems are quite common when using pressure plate extractors. Air leakage is of serious concern with pressure plate extractors. For pressure plate extractors like that shown in Figure 5-6, air leakage may take place along the outer edge or at the bottom of the ceramic plate with air bypassing the O-ring seal. Air leakage occurring along the edge of the plate between the membrane and the ceramic plate is also possible. Air leakage will result in the removal of water as vapour rather than by drainage, thereby affecting the accuracy of the soil-water characteristic curve (Wang and Benson, 2004).

Three pressure plate extractors were used to obtain the lower suction range of the soil-water characteristic curve for the HMA samples. Each pressure plate extractor contained a different HAED. The air entry values for the air entry disks used were 200 kPa, 500 kPa and 1500 kPa. The HMA samples used in the pressure plate extractors were saturated using one of two methods. Samples previously tested for their saturated hydraulic conductivity were near saturation, therefore they were immediately weighed for their saturated mass and then placed in the pressure plate extractor. Samples that had not been tested for saturated hydraulic conductivity needed to be saturated before being placed in the pressure plate extractor. It was difficult to maintain saturation of the HMA samples. Therefore, these HMA samples were placed in a well-sealed container filled with water and left to saturate for a minimum of one week. After the saturation period was complete, the samples were weighed and placed in the pressure plate extractors. Since a vacuum was not applied to this saturation procedure error is introduced into the level of saturation of these particular samples.

The testing period of the HMA samples varied between 2.5 and 5 months depending on the air entry disk used. Throughout the testing period, the pressure plate extractors were monitored to ensure air leakage was at a minimum. Minimum air leakage is required to ensure accuracy in the outflow measurements.

For suctions greater than 1000 kPa, osmotic rather than matric suction is used to produce drying of the sample. Conventional osmotic desiccators, like the one shown Figure 5-8,
Figure 5-8: Osmotic Desiccators

are used. An osmotic desiccator is a glass filled crucible with an electrolyte solution in the lower reservoir. The suction in the soil samples comes to equilibrium with the vapour in the chamber, which in turn, is in equilibrium with the osmotic pressure of the electrolyte solution. The osmotic pressure can be calculated using the expression for thermodynamic equilibria of solvents as follows:

$$\pi = RTC$$  \hspace{1cm} [5-3]

where $\pi$ is the osmotic pressure or suction (kPa), $R$ is the universal gas constant (8.32 L kPa/K mol), $T$ is absolute temperature (K), and $C$ is the sum of the molar concentrations in solution (mol/L) (Barbour and Fredlund, 1989).

Small samples are placed in the desiccators and are left to come to equilibrium with the electrolyte solution for a minimum of four weeks. There is a small air space separating the samples and the electrolyte solution, which acts as a semi-permeable membrane. The changes in mass of the samples are continually monitored to ensure equilibrium is attained after the testing period (Wilson et al., 1997).
Five osmotic desiccators were used to obtain the higher suction range of the soil-water characteristic curve. The electrolyte solutions of the desiccators included: Lithium Chloride, Magnesium Chloride, Magnesium Nitrate, Sodium Chloride and Potassium Sulphide. Table 5-1 provides a summary of the electrolyte solutions used and the corresponding values of osmotic suction. The samples used were prepared by breaking off small samples from the HMA samples used during the pressure plate extractor testing. An attempt was made to use pieces representative of a compacted HMA sample (i.e. samples that include both aggregate and asphalt cement). The samples were placed in the osmotic desiccators and left to achieve equilibrium. Equilibrium is generally reached after a minimum of four weeks of testing. The amount of time it takes to reach equilibrium depends on the size of the sample placed within the osmotic desiccator. Smaller samples have a shorter time to reach equilibrium compared to larger samples.

Table 5-1: Summary of Saturated Salt Solution and Osmotic Suctions used for the Vacuum Desiccators at 20°C

<table>
<thead>
<tr>
<th>Electrolyte Solution</th>
<th>Osmotic Suction (kPa X 10^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lithium Chloride [LiCl H₂O] and [LiCl 2H₂O]</td>
<td>292.4</td>
</tr>
<tr>
<td>Magnesium Chloride [MgCl₂ 6H₂O]</td>
<td>152.4</td>
</tr>
<tr>
<td>Magnesium Nitrate [Mg(NO₃)₂ 6H₂O]</td>
<td>84.0</td>
</tr>
<tr>
<td>Sodium Chloride [NaCl]</td>
<td>38.6</td>
</tr>
<tr>
<td>Potassium Sulphate [K₂SO₄]</td>
<td>4.19</td>
</tr>
</tbody>
</table>
5.2.2 Test Results

The soil water characteristic curve of HMA provides information on the moisture storage ability of this material. Thirteen HMA samples were tested using pressure plate extractors to determine the soil water characteristic curve up to suctions of 1000 kPa.

Testing HMA within pressure plate extractors presents numerous challenges in comparison to a granular soil. There were two major challenges during the testing of the HMA samples. The first problem encountered was saturating the HMA sample. When saturating a granular soil, a saturation period of 24 hours is normally sufficient to ensure all pores in the sample are filled with water. However, a 24-hour saturation period does not allow for all pores in the sample to fill with water. To ensure that the HMA samples were near their saturation point, samples were saturated for a minimum of one week in an airtight container. A second challenge was the combination of sample size and the starting point of the test. The diameters of the cores collected were smaller than the interior diameter of the pressure plate extractors used. This difference in size left the edge around the sample as a void where water is capable of filling when completing the testing procedure, as seen in Figure 5-9. For a granular soil, the soil fills the entire interior diameter of the pressure plate extractor. This allows for the starting point of the test to be at the top of the sample height. However, due to the void created in the pressure plate extractor with the HMA samples, the starting point of the test needs to be at the base of the sample. This will prevent water from wanting to fill the void between the edge of the sample and the interior wall of the pressure plate extractor if the water column elevation was started at the top of the sample.

After resolving the main challenges in the testing procedure, the soil water characteristic curves of the thirteen HMA samples up to a suction of 200 to 300 kPa were produced, as seen in Figure 5-10. The volumetric water content at saturation (i.e. measured porosity) was assumed to be at a suction of 0.1 kPa due to the near impossibility of measuring volumetric water content at such a low suction. A detailed summary of these results can be found in Appendix C.
Figure 5-9: Void within Pressure Plate Extractor

Figure 5-10: Soil-Water Characteristic Curves for Hot Mix Asphalt Samples
Figure 5-10 shows that the soil-water characteristic curve testing results of the thirteen HMA samples grouped into two sections. Grouping the HMA samples by age, as seen in Figure 5-11, indicates that the younger HMA samples have lower porosities and the older HMA samples have higher porosities. Porosity is calculated based on the volume of voids and the total volume of sample. This calculation excludes pores that are not large enough to contain water molecules and those that are not interconnected. Therefore the ability for the younger HMA samples to contain lower porosities then the older samples is that the younger samples contain smaller pores and fewer interconnected pores.

Another possibility for the HMA samples grouping into two sections is the difference in their method of saturation, as illustrated in Figure 5-12. Figure 5-12(a) illustrates that the samples saturated in the triaxial permeameter prior to being placed in the pressure plate extractor for testing tend to group within the same range of porosities (0.02 to 0.025).

However, samples saturated in an airtight container without a vacuum seal, as illustrated in Figure 5-12(b), shows that there is some variability in the porosity of the HMA samples, which is expected due to the unknown level of accuracy in saturation within this method.

Grouping the HMA samples into locations at which they were collected along the pavement structure, as seen in Figure 5-13, shows that there is no trend in porosity at the various locations. Figure 5-13 does illustrate that the amount of moisture stored in HMA is quite low and that the volumetric water content of HMA decreases with suction. The air entry value of HMA is practically negligible because it is located at a very low suction making it difficult to measure within a pressure plate extractor. Figure 5-13 also illustrates that the drop in volumetric water content is immediate for most HMA samples from porosity values (0.1kPa) to the first suction reading (approximately 0.3kPa). This suggests that they are actually hydrophobic and therefore are incapable of demonstrating capillarity so all that is being measured is ‘trapped’ moisture.
Figure 5-11: Age Relationship of SWCC (a) Newer Cores and (b) Older Cores
Figure 5-12: Method of Saturation Relationship between Soil-Water Characteristic Curves of Hot Mix Asphalt (a) Saturated in Triaxial Permeameter and (b) Saturated in Airtight Container
Figure 5-13: Location Relationship between Soil-Water Characteristic Curves of Hot Mix Asphalt (a) Inner Wheel Path, (b) Between Wheel Paths, (c) Outer Wheel Path and (d) Shoulder
Five small representative HMA samples were tested using osmotic desiccators to determine the soil water characteristic curve for suctions greater than 1000 kPa. Figure 5-14 represents the results of the soil water characteristic curve for suctions greater than 1000 kPa. Detailed results of the osmotic desiccator tests can be found in Appendix C. Combining the results of the osmotic desiccator tests and representative results from pressure plate extractor testing produces a soil-water characteristic curve for the HMA over the full suction range, as seen in Figure 5-15. Figure 5-15 shows that the volumetric water content at a pressure of 4120 kPa does not fit the linear relationship of HMA. The possible explanation for the volumetric water content at this pressure to not fit the linear relationship is that the sample may not be a representative HMA sample.

![Figure 5-14: Osmotic Desiccator Results](image-url)
Figure 5-15: Soil-Water Characteristic Curve of Hot Mix Asphalt Over Entire Suction Range

5.3 Vapour Diffusion

Moisture movement in an unsaturated soil occurs in both liquid and vapour phases. Research on liquid movement is more common; however, it is becoming more common that geotechnical engineers are being faced with moisture migration problems in which vapour diffusion is significant (Dobchuk et al., 2004). A column test can be constructed to determine the vapour diffusion through the HMA samples.

5.3.1 Testing Procedures

Vapour diffusion occurs in response to a gradient in vapour pressure. Due to this existing gradient, vapour molecules diffuse at random because of their own thermal energy producing a net flow of moisture. Vapour diffusion occurs in accordance to Fick’s First Law. Fick’s First Law describes the mass flux of water vapour as a function of a vapour concentration gradient.
\[ q_v = -n_a D^* \frac{\partial C}{\partial z} \]  \hfill \text{[5-4]}

\[ D^* = n^{2/3} D_o \]  \hfill \text{[5-5]}

\[ D_o = 0.229 \times 10^{-4} \left[ \frac{T}{273} \right]^{1.75} \]  \hfill \text{[5-6]}

where \( n_a \) is the air porosity of the soil, \( D^* \) is the diffusion coefficient, \( \partial C/\partial z \) is the concentration gradient, \( D_o \) is the diffusion coefficient of water vapour in air (m\(^2\)/s), \( n \) is total porosity and \( T \) is absolute temperature (K) (Zhou and Barbour, 1997; Dobchuck et al., 2004). If the soil is assumed to be ‘dry’ so that all pores are open and free from liquid phase than total porosity, \( n \), is equal to the air porosity, \( n_a \), of the soil. A simple column test was been fabricated for testing HMA cores.

The HMA samples were dried in an oven at 60°C and weighed every three hours to ensure that the bond between aggregate and binder was not lost. Dry HMA samples were used in order to ensure that all pores are open and free from a liquid phase, which is capable of blocking gas diffusion. The initial weight was recorded and the weight of the assembled column was measured with time.

Each clear glass jar was filled one third full of water. The sample is placed on the top of the jar, as seen in Figure 5-16, creating an interior boundary condition of relative humidity approximately equal to 100%. The HMA core was enclosed in a flexible membrane attached to a clear glass jar with O-rings. The flexible membrane was used to ensure that water vapour only escape through the top of the column. The top of the column is left open to atmospheric conditions, creating an upper boundary condition. This upper boundary condition is assumed equivalent to the relative humidity of the room in which the column is located.
HMA samples were tested over a period of one and a half months to ensure a constant vapour diffusion rate was reached. A blank case was tested using a non-porous steel cap to monitor the possibility of vapour leakage through the system. Any air leakage found in this blank case will be used to modify the results of the HMA vapour diffusion rates.

5.3.2 Testing Results

The vapour diffusion rate of HMA material provides an approximation of the maximum evaporation rates through this material. Six HMA samples were tested for their vapour diffusion rates, as seen in Figure 5-17. A detailed summary of the vapour diffusion results can be found in Appendix D.

Figure 5-17 shows a linear relationship between the amount of water escaping from the asphalt and time. The linear relationship between all six samples has minimal variation over the testing period, indicating the vapour flux rate of HMA is relatively consistent.
between location and age of HMA specimen. HMA sample F has a rapid increase in weight around day 19 (27,360 minutes) of the testing period. This may indicate that the sample in the column test may have cracked due to the pressure from the O-ring, as seen in Figure 5-18. Using the remaining five HMA samples, the average measured vapour flux rate is $8.5 \times 10^{-5} \text{ g/min} (1.42 \times 10^{-9} \text{ kg/s})$.

![Figure 5-17: Vapour Diffusion Rates of Hot Mix Asphalt](image)

![Figure 5-18: Failed Hot Mix Asphalt Sample](image)
A non-porous steel cap was tested for potential vapour leakage in the column system. The column system used for the non-porous steel cap was identical to that used for the HMA samples. The vapour diffusion rate of a non-porous material, like that of the steel cap, should be almost negligible. However, the vapour diffusion rate of the steel cap, as seen in Figure 5-17, is comparable to that of the HMA material. The vapour flux rate of the non-porous steel cap was \(6 \times 10^{-5}\) g/min. The diffusion rates after correcting the vapour leakage are presented in Figure 5-19.

The measured and adjusted HMA vapour flux rates can be compared to calculated vapour flux rates through Fick's First Law, which is a function of material porosity. Table 5-2 compares the calculated and measured vapour flux rates for five HMA samples, detailed calculations can be found in Appendix D. Comparing the calculated results to the measured results shows that the measured values are about ten times greater than the calculated values. A possibility for the difference is that of vapour through the triaxial permeameter flexible membrane used in the test set up. It is known that the flexible membrane protects the system from moisture leaking through the membrane in a flexible triaxial permeameter test, but knowledge of vapour leakage through the membrane is uncertain.

![Graph showing adjusted vapour diffusion rates of Hot Mix Asphalt](image-url)
Comparing the average adjusted HMA vapour flux rate, $2.4 \times 10^{-5} \text{ g/min}$, to the average calculated HMA vapour flux rate, $6.8 \times 10^{-6} \text{ g/min}$, indicates that the measured values are only four times greater than the calculated results. Therefore, there is a possibility of vapour leakage through the flexible membrane used in the column test as well as the possibility of a difference in tortuosity factors between HMA and soil.

The vapour flux rate of HMA can be used to calculate the amount of evaporation produced through HMA samples. An HMA vapour flux rate of $2.4 \times 10^{-5} \text{ cm}^3/\text{min}$ produces 0.384 mm of evaporation over the summer period of April through September or 0.640 mm over a one-year period. This would not likely be a considerable 'loss' from the pavement structure and therefore ignored in the simulation of moisture movement in the structure.

### 5.4 Air Permeability

An important parameter in gas flow simulations is air permeability. It is of particular interest to soil and agricultural scientists because it correlates gas exchange between soils and the atmosphere as well as affects the movement of vadose zone water. A common method used for determining the air permeability of unsaturated rock, soil, and other materials is measuring the steady state flux through the sample (Springer et al., 1998; Hailong et al., 2004).
5.4.1 Testing Procedure

Air permeability is defined as the flow of gas through a porous medium governed by a gas pressure gradient. The air permeability of porous materials is dependent on the material’s physical properties, such as void ratio, degree of saturation or percent and direction of compaction. Darcy’s law is valid for the flow of gas through a porous material if the flow occurs horizontally; gravitational terms are neglected and saturation does not change over time (ASTM, 2000b).

Darcy’s law, however, is not integrated in the same fashion for gases as it is for liquids, because the volume of air per unit time passing the unit cross sectional area is no longer constant. The volume of air per unit time passing the unit cross sectional area now increases as the gas approaches the low pressure end of the porous material and the air expands with the decrease in pressure. The integration of Darcy’s law for gas flow is performed under the assumption that the mass flux along the column will, in the steady state, be constant, and the flow is isothermal. Therefore,

\[ Q_{av} = \frac{K_p \Delta P A}{L \mu} \]  

[5-7]

where \( Q \) is the flow of air through porous material (\( m^3/s \)), \( K_p \) is the pneumatic permeability of the porous medium (\( m^2 \)), \( A \) is the cross sectional area of the porous medium (\( m^2 \)), \( \Delta P \) is the pressure drop over the porous medium (Pa), \( L \) is the length of the porous medium (m) and \( \mu \) is the viscosity of air (Pa*s) (Kirkham, 1946; ASTM, 2000).

An air permeameter system is one of many methods available to determine the air permeability of porous materials. The air permeameter system deals with porous materials with both gaseous and liquid mobile fluids. The liquid phase of porous materials is much less compressible, has a higher viscosity and is more bound to the solid phase by chemical forces than the gaseous phase. An air permeameter system applies only to one-dimensional laminar flow of air in porous materials. Single-phase flow is
assumed valid because the test gradient used ensures that laminar flow conditions are met. The air permeameter system assumes that the rate of mass flow through the porous material is constant with time (ASTM, 2000).

In this study a flexible wall permeameter cell was used. The interchangeable base and top cap have concentric groves to improve the uniform dispersion of airflow to the sample. The specimen is placed between two porous end pieces and encased with a flexible membrane. The porous end pieces provide the dual purpose of (1) uniformly diffusing/collecting the flow of air across the end faces of the specimen, and (2) providing an efficient buffer zone between the static and flow lines. The flexible membrane is attached to the sample with one or more O-rings at each of the base and top cap. The tubing is then attached to the top cap and the permeameter cell is assembled in accordance to ASTM D5084. The permeameter transfer lines, valves, end pieces and fittings shall be dry and free of foreign matter. The permeameter cell transfer lines are attached to the pneumatic system. A flowmeter is then attached to the exit of the outlet valve, as shown in Figure 5-20 (ASTM, 2000).

Air permeability tests are performed by passing air in through the specimen under pressure. Air leakage is possible through the gaps between the rubber membrane and the cap as well as through the pedestal into the specimen. An air leakage test is performed on the air permeameter system and any leakage found in the system must be corrected before commencing the testing process. Leakage is determined by increasing the back pressure on the system and observing transient flow and pressure changes as the system pressurizes. If the mass flow meter and differential pressure readings do not return to their initial readings the system is exposed to air leakage. Air leakage is a concern in the permeameter system because the test effluent is air and any leakage would cause significant error in the results (ASTM, 2000).
Four-inch diameter flexible wall permeameters were used to test HMA for its pneumatic permeability. The degree of saturation of the HMA samples must be low enough to avoid the internal movement of pore water, which may alter the continuity of air voids under the applied air pressure gradients. Twelve HMA samples were tested by MDH Engineered Solutions Corporation of Saskatoon, SK, for air permeability. These samples
were those prepared for the determination of saturated hydraulic conductivity described above in Section 5.1: Hydraulic Conductivity.

The correlation between results obtained with this test method and in situ field measurements has only been partially established. The small laboratory specimen used in this method may not be representative of the distributed condition on site due to vadose zone fluctuations, or changes in soil stratigraphy. For this reason, laboratory test results should be applied to field situations with caution (ASTM, 2000).

5.4.2 Testing Results

The coefficient of permeability due to airflow through HMA is an indication of the amount of air capable of moving through this material and also provides an indication of the water permeability as well. The results of the air permeability testing as a function of void ratio shown in Figure 5-21. A detailed summary of the results is presented in Appendix E.

Figure 5-21 illustrates a non-linear relationship between air permeability and void ratio. The air permeability of the sample with a void ratio of 0.026 does not agree with the general trend of the remaining eleven samples. This sample had an air permeability two orders of magnitude greater than the other samples. This could be due to the possibility of cracks in the HMA sample or air leakage in the pneumatic permeameter system.
The mean air permeability of eleven HMA samples is $2.05 \times 10^{-7} \text{cm}^2$ (approximately 20.5 darcy). Comparing the HMA air permeability values of air permeability presented by Freeze and Cherry (1979), Table 5-3 indicates that the HMA material is in the range of clean sand, which is quite high for an impermeable material. The high range of air permeability of the HMA samples could be due to cracking within the sample. A more reasonable range for the air permeability of HMA is that of a clay material. This range is more consistent with the low saturated hydraulic conductivity of HMA.

The air permeability, $k$, of hot mix asphalt can be related to the saturated hydraulic conductivity, $K_{sat}$, of hot mix asphalt using the following expression:

$$K_{sat} = \frac{\rho g k}{\mu}$$  \[5-8\]

where $\rho$ is the density of water (kg/m$^3$), $\mu$ is the dynamic viscosity of water (kg/s$\cdot$m) and $g$ is the acceleration due to gravity (m/s$^2$). The measured air permeability of HMA was used to calculate its saturated hydraulic conductivity, as illustrated in Figure 5-22.
Comparing the calculated saturated hydraulic conductivity to the measured saturated hydraulic conductivity of the HMA samples indicates that the calculated saturated hydraulic conductivity is on average five orders of magnitude greater than the measured saturated hydraulic conductivity. The difference in saturated hydraulic conductivity results indicates that the samples tested were exposed to cracking allowing more air to infiltrate through the sample. Figure 5-22 also illustrates a similar trend in the measured saturated hydraulic conductivity and calculated saturated hydraulic conductivity versus void ratio. The trend is that as void ratio increases the saturated hydraulic conductivity increases. The calculated saturated hydraulic conductivity relationship as seen in Figure 5-22 is indicated in equation 5-9

\[ K_{sat} = 0.1134e^{0.39} \]  

[5-9]

where \( e \) is the void ratio of Hot Mix Asphalt.

The results of the air permeability tests on HMA indicate that problems occurred during the testing process. Some of the possible problems could include air bubbles in the manometer, possibility of air leakage in the system that could not be corrected prior to testing, and the possibility of not letting the flow meter come into equilibrium, which can affect the pressure gradient across the HMA sample.

Table 5-3: Typical Permeability Values for Unconsolidated Geological Materials (Freeze and Cherry, 1979)

<table>
<thead>
<tr>
<th>Geologic Material</th>
<th>Range of Permeability (Darcy)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gravel</td>
<td>( 10^2 - 10^5 )</td>
</tr>
<tr>
<td>Clean Sand</td>
<td>( 10^1 - 10^3 )</td>
</tr>
<tr>
<td>Silty Sand</td>
<td>( 10^2 - 10^4 )</td>
</tr>
<tr>
<td>Silt</td>
<td>( 10^3 - 1 )</td>
</tr>
<tr>
<td>Unweathered Marine Clay</td>
<td>( 10^{-7} - 10^{-3} )</td>
</tr>
</tbody>
</table>
The laboratory component of this research tested HMA for saturated hydraulic conductivity, soil-water characteristics curve, vapour diffusion rate and air permeability. The saturated hydraulic conductivity, $K_{sat}$, was tested using a flexible wall triaxial permeameter. Thirteen HMA samples were tested and the mean saturated hydraulic conductivity of all thirteen samples is $3.28 \times 10^{-7}$ cm/s. It was found that the age of the HMA samples can affect its saturated hydraulic conductivity. The older HMA samples tended to have higher values of saturated hydraulic conductivity. Hydraulic conductivity versus void ratio is not in general agreement with soils as developed by Taylor (1948) but is consistent with the relationship produced by Kanitpong et al. (2001) in that hydraulic conductivity of HMA is controlled by void ratio, but is also affected by other material properties such as gradation, and specimen thickness.

The soil-water characteristic curve of HMA over a full suction range (0.1 kPa to $1 \times 10^6$ kPa) was determined using pressure plate extractors and osmotic desiccators. The
soil-water characteristic curve of HMA material decreases with suction although the
volume of moisture storage is very low which is due to the amount of interconnected
pores within the HMA samples. The results of the pressure plate extractor tests show that
the age of HMA material affects its porosity. The older HMA samples tended to have
greater porosity. This may be due to an increase in porosity over time as the asphalt ages
and microcracks develop in response to loading.

The vapour diffusion rate of HMA was measured using a simple fabricated column test.
The vapour diffusion rate of HMA was found to be 2.4x10^{-5} g/min. The vapour diffusion
rate was corrected for vapour leakage, as determined through testing of a non-porous
steel cap. The measured vapour diffusion rate is comparable to the vapour diffusion rate
calculated using Fick’s First Law. There is however, still some difference between the
measured and calculated results.

The air permeability of HMA was measured using an air permeameter system. The mean
air permeability was found to be 2.05x10^{-7} cm² (approximately 20.5 darcy), which is
rather large for a material that has a low saturated hydraulic conductivity. The measured
pneumatic permeability results were also used to calculate saturated hydraulic
conductivity. The mean calculated saturated hydraulic conductivity is 2.68x10^{-2} cm/s.
The difference between the measured saturated hydraulic conductivity and the calculated
saturated hydraulic conductivity is five orders of magnitude. This indicates that problems
occurred within the air permeability testing process. These problems may have included:

- air bubbles in the manometer,
- the possibility of air leakage in the system that could not be corrected for prior to
testing, and
- the possibility of not letting the flow meter to come into equilibrium.

Each of these problems can affect the pressure gradient across the HMA samples.
Chapter 6 Numerical Modelling

The numerical modelling component of this research was undertaken in order to;

- compare alternatives,
- evaluate the relative significance of various input parameters and understand the key processes and mechanisms associated with moisture and heat fluxes within standard highway pavement structures.

The numerical modelling component is divided between preliminary modelling and sensitivity modelling. This chapter describes the preliminary modelling and sensitivity modelling used in this research.

6.1 Preliminary Modelling

Preliminary modelling was undertaken to evaluate potential modelling approaches in developing a numerical model of standard highway pavement structures. The process in developing numerical models is to start simple and add complexity in stages. This permits one to establish when sufficient complexity has been used within the simulation. Adjustments throughout the preliminary modelling included adjusting the mesh design, the material properties, the analysis settings, the initial conditions, the boundary conditions, and the climatic properties.

Mesh Design

The finite element mesh represents the physical dimensions of typical highway pavement structures. The mesh design was altered throughout the preliminary modelling period. Alterations included resolving any material breaks within the mesh, altering element size, and increasing/decreasing the number of finite elements.
Material breaks are caused by incompatibilities created between adjacent elements. Incompatibilities generate problems in obtaining a solution and can be identified by viewing "material boundaries." An example of a material break can be seen in Figure 6-1.

Element size can also affect the solution time of the numerical model. Elements should resemble squares if using quadrilateral elements or isosceles triangles when using triangular elements. Elements that are too small increase the time of each simulation without any increase in accuracy. Elements that are too large can affect the overall mass balance, particularly if the height of the element is greater than the air entry value of the soil because the model has difficulty defining the unsaturated soil properties across the element. Modifying the element size can increase or decrease the number of finite elements within the mesh design (Geo-Slope Int. Ltd., 1991).

Figure 6-1: Material Break within the Mesh Design
Material Properties

The material properties used within numerical simulations control the hydraulic and thermal behavior of each material within the analysis. Four different material types were initially defined: the asphalt layer, the base layer, the subbase layer and the subgrade layer.

Numerous changes were made to the material properties over the preliminary modelling period. The majority of the modifications included the addition of soil layers to the highway pavement structure. For example, a 0.2 m thick organic layer was provided along parts of the sideslope and the ditch area to provide a more suitable soil for vegetation growth. Also, the upper soil layer condition was modified to create an ‘artificial’ soil layer in order to improve the simulation of infiltration.

Figure 6-2: Soil Layer Modifications
Analysis Settings

The analysis settings define the type of analysis being completed within the numerical model. The convergence properties and time stepping properties were adjusted throughout the preliminary modelling stage. Convergence is defined as the “repeated solving of the nodal differential equations until the computed solution does not change by more than a specified amount on successive iterations” (Geo-Slope Int. Ltd., 1991). There are many factors that affect the convergence of the numerical model solution such as finite element size, material properties, or boundary conditions.

The convergence controls were modified over the preliminary modelling period by changing the maximum number of iterations performed, reducing the tolerance, and varying the maximum hydraulic conductivity change parameters. The maximum number of iterations is the amount of possible iterations completed on a time step to reach convergence. The tolerance is the target difference between two successive iterations. The maximum hydraulic conductivity change parameter was varied to limit the changes in the hydraulic conductivity function between iterations (Geo-Slope Int. Ltd., 1991).

The method of controlling time steps affects both accuracy and the ease of convergence. Changes to the time stepping function made during the preliminary modelling period included adjusting the frequency of data output and adding an adaptive time-stepping scheme. Adjusting the number of times at which results are saved reduces the running time and file size of the numerical model. Adaptive time stepping increases the number of time steps between the specified daily time steps, allowing convergence to be reached at these additional time steps. The adaptive time steps were calculated from vector norms because the vector norm approach is more rapid than the head norm approach for larger meshes with two-dimensional water flows (Geo-Slope Int. Ltd., 1991).
Initial Conditions

An essential component of transient analysis is the use of initial conditions. The initial conditions applied to the numerical model may have a significant effect on its solution with unrealistic conditions leading to unrealistic results.

The initial conditions applied to the numerical model must define the initial head or temperature conditions for all nodes. If the temperature condition is not supplied a zero value is assumed, which in most circumstances is incorrect. The initial head conditions can be specified using two options; (1) drawing an initial water table from which an initial hydrostatic pressure distribution is estimated, or (2) creating an initial condition file from a steady state analysis. The use of an initial water table was used in the model and was adjusted over the preliminary modelling period. The adjustments included varying the maximum negative hydrostatic pressure head in order to apply appropriate initial head conditions for all nodes within the numerical model.

Boundary Conditions

The key component of a numerical analysis is specifying conditions along the problem boundaries. VADOSE/W uses hydraulic (head or moisture flux) and thermal (temperature or heat flux) boundary conditions as well as climatic boundary conditions. Hydraulic boundary conditions are required for the seepage analysis while thermal boundary conditions apply to the thermal analysis of the problem. Climatic boundary conditions are used to apply the heat and moisture flux rates and calculate the potential evaporation and transpiration rates of the simulation.

The hydraulic and thermal boundary conditions applied to the lower boundary of the simulation model were adjusted throughout the preliminary modelling stage. Initially the lower and side boundaries were specified as zero flux boundaries, which caused a "bath tub" effect. Moisture was infiltrating through the surface and the only means by which moisture could escape from the system was by evapotranspiration through the surface.
Since the lower boundary should allow water to move vertically in or out of the hydrogeologic system, it was decided to set a pressure head along the lower boundary to represent some external hydrogeologic control on the water table below the highway structure. A constant temperature condition that matched the initial temperature condition was also used along this lower boundary.

Climatic Data

The essential climate input data required within VADOSE/W include daily maximum and minimum temperature, maximum and minimum relative humidity, daily average wind speed, the amount of daily precipitation and the precipitation period (start and end times). This information along with latitude allows VADOSE/W to calculate the net radiation and potential evaporation/transpiration for that particular data set. VADOSE/W also permits the user to apply user specified net radiation data or potential evaporation data if available. The climate data were first applied with VADOSE/W estimating net radiation and potential evaporation. It was found that the estimated potential evaporation results differed from the average annual potential evaporation by over 200 mm. The climate data set was reapplied within VADOSE/W to include user specified net radiation, thereby improving the accuracy in the potential evaporation results. Surface fluxes are more accurate when net radiation or potential evaporation data are supplied.

6.2 Sensitivity Modelling

Eight numerical simulations were investigated within this research of which seven are modifications of a base simulation case. Table 6-1 summarizes the eight numerical simulation cases. These cases include six main features or properties that are varied throughout each of the eight simulations. The main parameter is highlighted by bold text. The six properties include varying the fluxes on the asphalt surface, changing the shoulder conditions from unpaved to paved conditions, changing the steepness of the sideslope, using both good and poor vegetation conditions, varying the initial suction
conditions and varying the snow removal process during the winter season. Each of the eight cases will be described in the following sections.

6.2.1 Base Case

The base simulation case developed for the analysis is described in detail with reference to mesh design, material properties, analysis settings, initial conditions and boundary conditions.

Mesh Design

Regions were used to describe the geometry of the base simulation case and assist in developing the discretization of the pavement structure. A total of 51 regions were used to create the mesh. The regions are divided between two base regions and 49 surface regions, as illustrated in Figure 6-3. A total of 903 finite elements were generated from these 51 regions.

Table 6-1: Numerical Modelling Simulation Cases (Bold Indicating Property Altered)

<table>
<thead>
<tr>
<th>Property Condition</th>
<th>Simulations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Base Case</td>
</tr>
<tr>
<td>A</td>
<td>Without Asphalt Fluxes</td>
</tr>
<tr>
<td>B</td>
<td>Unpaved Shoulders</td>
</tr>
<tr>
<td>D</td>
<td>Good Grass Conditions</td>
</tr>
<tr>
<td>E</td>
<td>Initial Suction Conditions</td>
</tr>
<tr>
<td>F</td>
<td>Snow Removal</td>
</tr>
</tbody>
</table>
Regions one and two are base regions consisting of a structured quadrilateral mesh, as seen in Figure 6-4. Each finite element within the base regions is characterized by its thickness, integration order and the existence of secondary nodes. The thickness of each element is one with an integration order of four. No secondary nodes exist within the base region. The material characteristics assigned to both regions one and two are those assigned for the subgrade material.

The remaining 49 regions create the surface region of the numerical simulation. A surface region applies finer discretization to the mesh design. It is required to comply with the rapid and dramatic climate boundary changes existing at the ground surface (Geo-Slope Int. Ltd., 1991). The surface region is a multi-layer region along the entire ground surface. The multi-layer regions include an ‘artificial’ soil layer (3 cm in depth across the entire pavement structure), a hot mix asphalt layer (110 mm thick at the centerline), a base layer (220 mm thick at the centerline and 330 mm thick at a distance of 6.7 m from the centerline), a subbase layer (600 mm thick at the centerline and at a distance of 6.7 m from the centerline), a thin subgrade layer, and an organic soil layer (0.2 m thick from the surface starting at a distance of 12.4 m from the centerline). The surface region consists of structured quadrilateral and triangular regions. The finite
elements within the surface region are characterized by the same thickness and integration order as the base region.

**Material Properties**

VADOSE/W requires both hydraulic and thermal properties to solve the coupled heat and moisture flow equations. The base simulation case is designed with six soil layers. The soil layers include:

- Hot Mix Asphalt
- Base
- Subbase
- Subgrade or natural ground
- Organic Layer
- Infiltration Soil Layer

These six soil layers make up the different material layers used in the construction of standard highway pavement structures. Figure 6-5 indicates the location of each of these soil layers within the base case simulation model.

![Figure 6-4: Mesh Discretization of Base Case Simulation](image-url)
The hot mix asphalt material layer is located along the driving lane in the base case simulation. The base case simulation utilizes a null material for this layer in order to replicate a situation in which no moisture fluxes cross the asphalt over the driving lane. A null material is a material that contains no hydraulic and thermal properties but it still discretized within the numerical simulation model.

The base material is the top granular soil material used in the construction of standard highway pavement structures. The base simulation case consists of unpaved shoulders, therefore exposing the base material on the shoulder surface. The subbase material is another granular layer placed above the subgrade layer. The material properties of the base and subbase materials were estimated using the built-in functions available within VADOSE/W. The volumetric water content function of the base and subbase layers was estimated from the grain size distribution curve using the Arya and Paris (1981) method. The grain size distributions applied were assumed to match SDHT specification Type 33 for the base and Type 10 for the subbase. The base and subbase hydraulic conductivity functions were estimated using the Fredlund and Xing (1994) method using the
volumetric water content function. The hydraulic properties of the base and subbase are illustrated in Figure 6-6. The thermal properties of the base and subbase, as represented by the thermal conductivity and specific heat functions, were estimated using their volumetric water content functions, as seen in Figure 6-7.

The subgrade material is assumed to be the same as the natural soil, which was assumed to be a clayey silt material common in Saskatchewan. The organic layer was added as a topsoil layer in the base simulation case. The organic layer is placed on the natural ground of the sideslope and ditch areas to increase the vegetation growth at these locations. The organic layer used in the base simulation case is representative of a silt loam material. The hydraulic properties of the subgrade and organic material layers were imported from the material database available within VADOSE/W, as illustrated in Figure 6-6.

The top 3 cm soil layer is the infiltration soil layer applied across the entire ground surface region, excluding the paved surface. It is composed of multiple soil materials including the base material, the subbase material and the organic layer material. The infiltration soil layer consists of material properties similar to those of the underlying soil layer but with a modified hydraulic conductivity function. For example, along the granular shoulder the unique soil layer has the same volumetric water content, thermal conductivity and specific heat functions as the base material. However, the hydraulic conductivity drops only a few orders of magnitude over the entire suction range to $1 \times 10^6$ kPa. This material modelling technique avoids difficulties in obtaining convergence when high intensity summer rainfall is applied to very dry and consequently low unsaturated hydraulic conductivity soils.
Figure 6-6: Hydraulic Properties for Various Material Layers in the Model (a) Hydraulic Conductivity and (b) Soil-Water Characteristic Curve
Figure 6-7: Thermal Properties for Various Material Layers in the Model (a) Thermal Conductivity and (b) Volumetric Specific Heat
Analysis Settings

Analysis settings defined for numerical simulations are divided into control, convergence and time stepping parameters. The control parameters of the base case simulation included vegetation properties, ground freezing and the ability for surface water to pond on the climate boundary condition. The vegetation properties defined were of good grass quality which assumes a maximum LAI of 2.0 during the growing season, as shown in Figure 6-8. Ground freezing allows ice to build up in the soil resulting in a reduction in hydraulic conductivity.

The convergence properties defined within the base simulation case include a maximum of 20 iterations with a tolerance of 0.1. The base simulation case uses the direct equation solver. The conductivity change parameters used to control the convergence of the finite element simulation include a minimum conductivity change of 0.0001, and a maximum conductivity change of 1 with a maximum conductivity change rate of 1.1.

Time step parameters control the length of time in which the base simulation case is analyzed. The base simulation case is run for six-years (i.e. 2190 days). The base simulation case limits the amount of data saved over the 2190-day time frame. Data are collected on day one and every fifteenth day after including day 2190. The base simulation case uses adaptive time stepping calculated from vector norms with a maximum 10% change in head per time step. The increment size of the adaptive time stepping scheme is between 15 minutes to two hours.
Initial Conditions

An initial water table condition was used to define the initial head in all models. In the base case the initial water table was drawn 3 m below the base of the model with a maximum negative pressure head of 1 m, a suction of 10 kPa, as seen in Figure 6-9. The initial water table was selected based on information found in Cyr and Chiasson (1999) and therefore, may not be representative for the selected study site. This produced a uniform suction across all soils. An initial temperature of 6°C was also applied to all nodes within the base simulation case.

Boundary Conditions

No flow boundary conditions were applied to the vertical centerline and outside edges of the base simulation case. The boundary conditions applied to the base of the model were a pressure head of –3 m (30kPa suction), representing the presence of a water table at elevation 8 m below the asphalt surface, and a temperature of 6 °C. Figure 6-10 illustrates these boundary conditions as well as a surface climate boundary condition.

Figure 6-8: Leaf Area Index Function for Vegetation of Good Grass Quality
VADOSE/W has the ability to apply different climate data sets as surface climate boundary conditions. The climate boundary condition applied to the base simulation case includes different climate data sets. The different climate data sets used within the base case simulation create different microclimates for the shoulder, sideslope and ditch regions. September 1\textsuperscript{st} was set as Day 1 within VADOSE/W for each climate data set. Table 6-2 describes the variation of the climate data sets used within the base simulation case as well as the seven modified simulations. The climate data used within the numerical simulations is representative of the study area and was described in Chapter 4.
In order to simulate appropriate thermal conditions at the base of the asphalt along the driving lane of the base case simulation, a one-dimensional simulation case was simulated in TEMP/W to obtain the soil temperatures at the base of the hot mix asphalt layer. The one-dimensional analysis in TEMP/W was performed to obtain realistic temperatures and heat flux values without affecting water flux. Some of the characteristics of the one-dimensional simulation case are similar to those applied to the

<table>
<thead>
<tr>
<th>Case</th>
<th>Driving Lane</th>
<th>Shoulder</th>
<th>Slope with Granular Surface Material</th>
<th>Slope with Organic Surface Material</th>
<th>Ditch</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base</td>
<td>None</td>
<td></td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Plus Precipitation from Driving Lane</td>
<td>Plus snow precipitation from driving lane and shoulder plus Vegetation</td>
<td>Plus Vegetation</td>
</tr>
<tr>
<td>2</td>
<td>None</td>
<td>None</td>
<td>OriginalClimate Data</td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Plus Precipitation from Driving Lane</td>
<td>Plus snow precipitation from driving lane and shoulder plus Vegetation</td>
<td>Plus Vegetation</td>
</tr>
<tr>
<td>3</td>
<td>None</td>
<td></td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Plus Precipitation from Driving Lane</td>
<td>Plus snow precipitation from driving lane and shoulder plus Vegetation</td>
<td>Plus Vegetation</td>
</tr>
<tr>
<td>4</td>
<td>None</td>
<td></td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Plus Precipitation from Driving Lane</td>
<td>Plus snow precipitation from driving lane and shoulder plus Vegetation</td>
<td>Plus Vegetation</td>
</tr>
<tr>
<td>5</td>
<td>Vapour flux rate</td>
<td></td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Plus Precipitation from Driving Lane</td>
<td>Plus snow precipitation from driving lane and shoulder plus Vegetation</td>
<td>Plus Vegetation</td>
</tr>
<tr>
<td>6</td>
<td>None</td>
<td></td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Plus Precipitation from Driving Lane</td>
<td>Plus snow precipitation from driving lane and shoulder plus Vegetation</td>
<td>Plus Vegetation</td>
</tr>
<tr>
<td>7</td>
<td>Vapour Flux Rate</td>
<td></td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Vapour Flux Rate</td>
<td>Plus Precipitation from Driving Lane</td>
<td>Plus snow precipitation from driving lane and shoulder plus Vegetation</td>
<td>Plus Vegetation</td>
</tr>
<tr>
<td>8</td>
<td>None</td>
<td></td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
<td>Original Climate Data</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Plus Precipitation from Driving Lane</td>
<td>Plus Vegetation</td>
<td>Plus Vegetation</td>
</tr>
</tbody>
</table>
base simulation case. The differences in the one-dimensional simulation case include an upper climate boundary condition applied to the HMA surface layer, as seen in Figure 6-11, and the application of the HMA surface layer material properties. The climate boundary condition applied in the one-dimensional simulation case includes maximum and minimum air temperature, maximum and minimum relative humidity and wind speed. Precipitation is not included in the climate boundary condition because the precipitation that reaches the HMA surface layer is assumed to runoff due to the "impermeable" nature of the HMA layer. The hydraulic material properties for the HMA surface layer included constant values of hydraulic conductivity and soil-water characteristic curve functions. The hydraulic conductivity value was significantly low to provide zero flux conditions. The thermal material properties for the HMA surface layer are illustrated in Figure 6-12.

The temperature time profile obtained from the one-dimensional TEMP/W simulation was applied as a temperature boundary function at the top of the base layer, illustrated in Figure 6-13.
Figure 6-12: Thermal Material Properties for HMA Surface Layer (a) Thermal Conductivity Function and (b) Volumetric Specific Heat
Figure 6-13: Thermal Boundary Condition Representing Thermal Conditions at Base of Hot Mix Asphalt Layer for One Climate Year
6.2.2 Cases 2 Through 7

The base case simulation was modified to create seven additional simulation cases:

1. Case 2: Shoulder Properties
2. Case 3: Vegetation Properties
3. Case 4: Initial Suction Condition
4. Case 5: Asphalt Vapour Diffusion
5. Case 6: Sideslope
6. Case 7: Combination Case
7. Case 8: Snow Application

Each of these seven simulation cases help to illustrate the influence of various design or field performance characteristics on the long-term heat and moisture balances and flux rates of typical standard highway pavement structures.

Case 2: Shoulder Properties

Case 2 modifies the shoulder surface layer of the base case simulation model. The top 50 mm of the granular shoulder is replaced by hot mix asphalt concrete (HMA). The HMA used on the shoulder has the same material characteristics as the HMA used on the driving lane described within the base simulation case.

The shoulder surface layer of the base simulation case was modified because paved shoulder conditions on standard pavement structures are common throughout Saskatchewan’s road network. A paved shoulder also creates a wider “impermeable” surface on the highway pavement structure, which reduces the amount of granular surface area capable of infiltration and evaporation.
Case 3: Vegetation Properties

In Case 3 the vegetation properties are modified to exhibit poor grass conditions instead of the good grass conditions exhibited in the base case simulation. In order to exhibit poor grass conditions the LAI function of the vegetation properties is changed. Figure 6-15 provides the modification to the LAI function for Case 3.

Altering the grass condition from good to poor changes the amount of evapotranspiration from the sideslope and ditch areas. The amount of energy available to the plants in their attempt to transpire water is decreased and the amount of energy available for direct evaporation at the soil surface is increased when shifting from good to poor grass conditions (Geo-Slope Int. Ltd, 1991). The LAI function was estimated to be poor quality by using the estimation functions provided within VADOSE/W which defines $\text{LAI}_{\text{max}}$ equal to 1.0.
Case 4: Initial Suction Condition

Case 4 modifies the initial suction condition used for the base simulation case. The initial suction conditions are increased from 10 kPa to 100 kPa by changing the conditions applied to the initial water table. The location of the initial water table is changed from 3m below the base of the simulation model to 10m below the base of the simulation model, as illustrated in Figure 6-16. Changing the initial suction condition in this case causes flow to initially be moving into the system instead of out of the system as in the base case simulation. Flow would be moving into the system since the numerical simulation is drier than that of the base case simulation.
Case 5: Asphalt Vapour Diffusion

Case 5 modifies the role of the HMA layer of the driving lane used within the base simulation case. The function of the HMA layer is modified to permit moisture accumulated in the highway pavement structure to be released through the driving surface by vapour diffusion. Allowing vapour to be released through the HMA layer has the ability to decrease the amount of moisture accumulated within the highway pavement structure.

The effect of vapour release through the HMA layer is achieved by applying a flux boundary condition of 0.0312 mm/day over the summer period, at the nodes located along the base of the driving lane. The total flux applied to the nodes was selected based on the results of the vapour diffusion rates found for HMA during laboratory testing. Figure 6-17 illustrates the location of the new boundary condition applied to Case 5 simulation.
Case 6: Sideslope

Case 6 modifies the steepness of the sideslope of the base case simulation. The sideslope is modified from a 6:1 slope to a 3:1 slope. Modifying the actual slope of the sideslope changes the width of the highway pavement structure. Figure 6-18 illustrates the mesh difference between the base case simulation and Case 6.

Over the past 40 years Saskatchewan Department of Highways and Transportation (SDHT) has moved away from the steeper sideslopes due to safety concerns. However, SDHT has found that many of the standard highway pavement structures designed with 3:1 sideslopes appear to have better performance over their service life than many of the current 6:1 sideslope designs, which is probably due to the increased amount runoff and therefore, decreased amount of surface infiltration.

Figure 6-17: Location of Total Flux Boundary Condition in Case 5
Case 7: Combination Case

Case 7 combines four features used within the base case simulation. The four features include; paved shoulder, initial suction conditions, HMA layer conditions and steepness of the sideslope. Combining these four property modifications creates a simulation model in which the various features are working in concert to influence the long-term heat and moisture balance and flux rates of the highway pavement structure.

The shoulder properties of Case 7 are modified from granular shoulders to paved shoulders as in Case 2. The driving lane and shoulder properties are adjusted to allow vapour to be released through the HMA material as in Case 5. The steepness of the sideslope is modified from 6:1 to 3:1 as in Case 6. The initial suction condition is modified as described in Case 4. All of these modifications should promote reduced moisture fluxes into the highway pavement structure.

Case 8: Snow Application

Case 8 modifies the snow removal process used within the base simulation case. In this case it is assumed that snow is not removed and is allowed to accumulate on the unpaved
shoulder surface over the winter period. This is modified within the climate data applied to the simulation model. Snow precipitation is included on the unpaved shoulder surface to identify whether or not snow removal is important to the long-term heat and moisture balance and flux rates of the highway pavement structure.

6.3 Sensitivity Modelling Results

The eight numerical simulations investigated within the sensitivity modelling, as presented in Table 6-1, assist in evaluating the relative magnitude of moisture and heat fluxes within standard highway pavement structures. Evaluating moisture and heat fluxes of highway pavement structures included looking at the time to reach equilibrium. The mechanisms applied to each of the eight numerical simulations are also compared for their effects on long-term moisture and heat fluxes.

6.3.1 Time to Reach Equilibrium

Each of the eight numerical simulation cases was simulated for a period of twelve years in an attempt to establish long-term equilibrium conditions. The simulations were run on single processor Pentium 4 CPUs (2.6GHz) with 1GB of DDR RAM and 100GB SATA harddrive. The simulation time of twelve years required run times in the order of 48 hours. In order for long-term equilibrium conditions to be achieved the annual cumulative flux into the domain (i.e. a positive slope) must be equivalent to the annual cumulative flux out of the domain (i.e. a negative slope).

The annual boundary flux for the base case simulation is illustrated in Figure 6-19. The top boundary flux for the base case is divided into three sections; flux along the driving surface (i.e. driving lane and shoulder), flux along the granular area of the sideslope and flux along the organic area of the sideslope (i.e. ditch). The driving surface section has an annual flux for year 12 was 0.02 out of the domain. The annual flux for year 12 along the granular area of the sideslope was 0.86 into the domain. The annual flux for year 12 along the organic area of the sideslope was 2.14 out of the domain. The base of the
model has an annual flux for year 12 of 0.17 into the domain. Therefore, the total flow into the system is equal to 1.03 and the total flow out of the system is equal to 2.16. The results for the annual boundary fluxes in and out of the system for Cases 2 through 7 are presented in Table 6-3.

![Figure 6-19: Annual Cumulative Boundary Flux for Base Case](image-url)
<table>
<thead>
<tr>
<th>Simulation Case</th>
<th>Annual Flux</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Σ in</td>
<td>Σ out</td>
</tr>
<tr>
<td>Base Case</td>
<td>1.03</td>
<td>-2.16</td>
</tr>
<tr>
<td>Case 2 - Paved Shoulders</td>
<td>1.68</td>
<td>-2.63</td>
</tr>
<tr>
<td>Case 3 - Altering Vegetation Condition</td>
<td>1.08</td>
<td>-2.04</td>
</tr>
<tr>
<td>Case 4 - Altering Initial Suction Condition</td>
<td>1.47</td>
<td>-1.94</td>
</tr>
<tr>
<td>Case 5 - Applying Asphalt Vapour Diffusion Rate</td>
<td>0.86</td>
<td>-2.34</td>
</tr>
<tr>
<td>Case 6 - Steepening Sideslopes</td>
<td>0.71</td>
<td>-3.78</td>
</tr>
<tr>
<td>Case 7 - Combining Several Mechanisms</td>
<td>1.40</td>
<td>-2.70</td>
</tr>
<tr>
<td>Case 8 - Snow Accumulation on Shoulder</td>
<td>1.72</td>
<td>-2.71</td>
</tr>
</tbody>
</table>

Equilibrium requires that the annual flow into the system must equal the annual flow out of the system. The results of annual cumulative boundary flux for each of the numerical simulations indicates that long-term equilibrium conditions for the highway pavement system have yet to be established after the twelve years of simulation. However, overall the cumulative flux rates are slowly changing from year to year.

Comparing each of the three divisions of the top boundary indicates that the flow through the driving surface (i.e. driving lane and shoulder) is quite negligible. The granular area of the sideslope is generally moving water into the system due to the granular materials along the surface. The organic area of the sideslope (i.e. ditch area) is generally moving water out of the system for all eight cases.

The fact that the cumulative flows in and out of the models did not balance after twelve years of a constant climatic condition suggests that it is unlikely that equilibrium conditions will ever be reached within the pavement structure. The drying and wetting of the pavement structure over time will be intrinsically transient and always tied to climatic variability, likely lagging significantly behind the current climatic conditions.
VADOSE/W provides a useful tool in understanding the reasonableness of the results. This tool is the overall cumulative water balance error for the entire mesh, which is the sum of all external boundary fluxes, minus the sum of the computed change in storage within all elements. Ideally, the cumulative water balance error for the entire mesh should equal zero, which would indicate that all water entering or leaving the system is accounted for as a gain or loss within the system (Geo-Slope Int. Ltd., 1991). The cumulative water balance error for the entire mesh for each numerical simulation is indicated in Table 6-4.

Table 6-4 indicates that the Base Case, Case 2, Case 3, Case 5 and Case 6 produce a substantial amount of cumulative water balance error (approximately 20% of water stored) within the highway pavement system. Case 4 and Case 8 are still quite significant; however, the amount of water balance error (approximately 7% of water stored) is closer to the ideal situation of zero as described by Geo-Slope Int. Ltd. (1991). Case 7 produces an amount of water balance error (3% of the water stored) that is less significant and closer to zero than the other 7 cases. Overall, the cumulative water balance error for each of the numerical simulation cases is of concern because the water balance error changes drastically during certain time steps, indicating that the accuracy of the results produced are not entirely accurate. The drastic changes also indicate that numerical simulation cases may not be converging as expected.

<table>
<thead>
<tr>
<th>Numerical Simulation Case</th>
<th>Cumulative Water Balance Error at Year 12 (m^3/% of water stored)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base Case</td>
<td>6.48/23%</td>
</tr>
<tr>
<td>Case 2 – paved shoulder condition</td>
<td>5.13/18%</td>
</tr>
<tr>
<td>Case 3 – altering vegetation condition</td>
<td>4.12/15%</td>
</tr>
<tr>
<td>Case 4 – altering initial suction condition</td>
<td>1.38/5%</td>
</tr>
<tr>
<td>Case 5 – altering vapour diffusion rate condition</td>
<td>5.22/19%</td>
</tr>
<tr>
<td>Case 6 – steepening sideslope condition</td>
<td>-7.94/28%</td>
</tr>
<tr>
<td>Case 7 – combination condition</td>
<td>-0.86/3%</td>
</tr>
<tr>
<td>Case 8 – snow accumulation on shoulder condition</td>
<td>2.30/8%</td>
</tr>
</tbody>
</table>
6.3.2 Mechanism Analysis

There are several mechanisms capable of affecting the moisture and heat fluxes of highway pavement structures. These mechanisms, as discussed in Table 6-1, include: varying the fluxes on the asphalt surface (Case 5); changing the shoulder surface conditions from unpaved to paved conditions (Case 2); changing the steepness of the sideslope (Case 6); changing vegetation conditions from good to poor (Case 3); varying the initial suction conditions (Case 4); and varying the snow removal process during the winter season (Case 8). The volume of water stored in the subgrade, subbase and base of a highway pavement structures with time at the centerline, interface of the driving lane and shoulder and the shoulder edge for the even simulations are compared to that of the base case to evaluate the impact of each design feature on the moisture fluxes of highway pavement structures.

The base case, as seen in Figure 6-20, shows that the volume of water stored in the base layer of the highway pavement structure is constant under the paved driving lane for all 12 years of simulation. Along the shoulder of the base layer the volume of water stored in this layer is increasing. The volume of water stored in the subbase layer of the highway pavement structure increases from the centerline to the shoulder edge. However, the volume of water stored in the subgrade layer of the highway pavement structure remains constant from the centerline to the shoulder edge. Both the base and subbase layers show that the volume of water stored in the layer seems to start out the same during the first year of simulation and then rapidly separates by position (i.e. centerline, driving lane/shoulder interface, and shoulder edge) with positions near the shoulder becoming wetter. The volume of water then cycles through these positions with relatively little difference on an annual basis. The volume of water stored in the subgrade does not appear to cycle over the twelve years of simulation.
The volume of water stored in the base layer of a highway pavement structure when the shoulder is paved (Case 2), as seen in Figure 6-21, remains generally constant from the centerline to the shoulder edge. The volume of water stored in the subgrade layer also remains constant from the centerline to the shoulder edge. However, the volume of water stored in the subbase layer increases from the centerline to the edge of the shoulder. The volume of water stored in the base layer seems to start out the same during the first four years of simulation and then rapidly separates by position (i.e. centerline, driving lane/shoulder interface, and shoulder edge). The volume of water then cycles through these position with relatively little difference on an annual basis. The volume of water stored in the subbase layer rapidly separates by position during the first year of simulation. The volume of water at the centerline seems to cycle with less difference on an annual basis than the other two positions for the first seven years of simulation. However, for the last five years of simulation the centerline position seems to cycle with
a greater difference on an annual basis than the other two positions. The volume of water stored in the subgrade does not appear to cycle over the twelve years of simulation.

The volume of water stored in the base layer of a highway pavement structure when the vegetation condition is altered (Case 3), as seen in Figure 6-22, increases slightly over the paved driving lane and continues to increases along the shoulder. The subbase layer also shows an increase in the volume of water stored in the layer from the centerline to the edge of the shoulder. The volume of water stored in the subgrade layer, however, remains quite constant across the paved driving lane and shoulder regions. Both the base and subbase layers show that the volume of water stored in the layer seems to start out the same during the first year of simulation and then rapidly separates by position (i.e. centerline, driving lane/shoulder interface, and shoulder edge). The volume of water then cycles through these positions with relatively little difference on an annual basis. The volume of water stored in the subgrade does not appear to cycle over the twelve years of simulation.

Figure 6-21: Volume of Water versus Time for Case 2 – Paved Shoulder Condition
Figure 6-22: Volume of Water versus Time for Case 3 – Altering Vegetation Condition

Altering the initial suction condition (Case 4), as seen in Figure 6-23, shows that the volume of water stored in the base layer of the highway pavement structure remains constant across the paved driving lane region and increases along the shoulder region. The subbase layer also shows an increase in the volume of water stored in the layer in both the driving lane and shoulder regions. However, the subgrade layer shows that the volume of water stored in this layer is constant across both the driving lane and shoulder. Both the base and subbase layers show that the volume of water stored in the layers starts out the same during the first year of simulation and then rapidly separates by position (i.e. centerline, driving lane/shoulder interface, and shoulder edge). The volume of water at the centerline and driving lane/shoulder interface positions of the base layer decreases, while the shoulder edge volume of water increases. The volume of water then cycles through these positions with relatively little difference on an annual basis. The volume of water in the subbase layers cycles through these positions with varying degrees of
difference on an annual basis. For example, the driving lane/shoulder interface cycles with a much greater difference than the other two positions. The centerline, however, cycles with the least amount of difference on an annual basis. The volume of water stored in the subgrade does not appear to cycle over the twelve years of simulation.

The volume of water stored in the base, subbase and subgrade layers of a highway pavement structure when applying an asphalt diffusion rate to the paved surface (Case 5), as seen in Figure 6-24, remains constant across the paved driving lane region as well as across the shoulder region. The volume of water in all three layers (i.e. base, subbase and subgrade layers) seems to start out at the same after the first year of simulation and the following eleven years of simulation. Each position (centerline, driving lane/shoulder interface, and shoulder edge) cycles with relative little difference on an annual basis.

Figure 6-23: Volume of Water versus Time for Case 4 – Altering Initial Suction Condition
Steepening the sideslope of the highway pavement structure (Case 6), as seen in Figure 6-25, shows that the volume of water stored in the base layer slightly increases across the driving lane and shoulder regions. The subbase layer shows that the volume of water stored in the layer remains constant across the driving lane and increases across the shoulder region. The volume of water stored in the subgrade layer remains constant across both the driving lane and shoulder regions. Both the base and subbase layers show that the volume of water stored in the layer seems to start out the same during the first year of simulation and then rapidly separates by position (i.e. centerline, driving lane/shoulder interface, and shoulder edge). The volume of water then cycles through these positions with relatively little difference on an annual basis. The volume of water stored in the subgrade does not appear to cycle over the twelve years of simulation.
Combining several mechanisms (Case 7), as seen in Figure 6-26, shows that the volume of water stored in the base and subbase layers of the highway pavement structure slightly increase across the driving lane and continue to increase along the shoulder region. The subgrade layer of the highway pavement structure shows that the volume of water stored in the layer remains constant across the driving lane and shoulder regions. Both the base and subbase layers show that the volume of water stored in the layers seems to start out the same during the first year of simulation and then slightly separates by position (i.e. centerline, driving lane/shoulder interface, and shoulder edge). The volume of water then cycles through these positions with relatively little difference on an annual basis. The volume of water stored in the subgrade does not appear to cycle over the twelve years of simulation, however, the volume of water increases from the first year of simulation to the forth year of simulation where it remains constant.
Figure 6-26: Volume of Water versus Time for Case 7 – Combining Several Mechanisms

Allowing snow to accumulate on the shoulder (Case 8), as seen in Figure 6-27, shows that the volume of water stored in the base layer of the highway pavement structure increases across the driving lane and remains generally constant across the shoulder region. The subbase and the subgrade layers indicate that the volume of water stored in the layers remain constant across both the driving lane and shoulder regions. Both the base and subbase layers show that the volume of water stored in the layers seems to start out the same during the first year of simulation for each position (i.e. centerline, driving lane/shoulder interface, and shoulder edge) and continues to remain the same for the next eleven years of simulation. The volume of water stored in the subgrade does not appear to cycle over the twelve years of simulation.
Figure 6-27: Volume of Water versus Time for Case 8 – Allowing Snow to Accumulate on Shoulder

The impact that design features may have on heat fluxes of highway pavement structure can be evaluated through the temperature variation with time from the centerline at the subbase/subgrade interface.

The base case, as seen in Figure 6-28, shows that temperature remains constant with position under the driving lane, with the warmest temperatures occurring in August and the coolest temperatures in February. However as the temperature profile reaches the driving lane/shoulder interface the temperature begins to change. In the spring and summer (May to September) temperatures are decreasing drastically over the shoulder width and reaching a gradual decrease in temperature at the shoulder/sidelslope interface. In the winter months (December through March) temperatures are decreasing gradually along the width of the shoulder and begin to increase at the shoulder edge to temperatures
nearing $0^\circ C$ along the sideslope. In October and November temperatures are decreasing along the shoulder and sideslope. The temperature variation for Case 3 (altering the vegetation condition), Case 4 (Alterning the initial suction condition), and Case 5 (Applying a vapour diffusion rate to the paved surface) are similar to that of the base simulation case.

Paving the shoulder area of a highway pavement structure (Case 2) alters the temperature profile in comparison to the base case simulation. The temperature now remains constant along both the driving lane and shoulder regions, as shown in Figure 6-29. Therefore, temperature begins to change at the shoulder/sideslope interface. Temperature profiles for the months of April until November are decreasing along the sideslope region. The temperature profile in March and December remain practically constant at a temperature of $0^\circ C$. The temperatures profiles in the winter (January and February) when the shoulder is paved are increasing along the sideslope to reach temperature values of $0^\circ C$.

![Figure 6-28: Lateral Temperature Profile of the Base Case](image-url)
Steepening the sideslope of a highway pavement structure as in Case 6 also alters the temperature profile in comparison to the base case simulation. Constant temperatures are still seen along the driving lane with a change in temperature occurring at the driving lane/shoulder interface as seen in Figure 6-30. The temperature profile change seen in the summer months of May through September begins with a drastic decrease in temperature along the shoulder followed by a drastic increase in temperature along the beginning of the sideslope with a drastic decrease in temperature occurring at the midpoint of the sideslope. In other words, a temperature cycling pattern is occurring along the shoulder and sideslope regions. In the winter months of December through March a gradual decrease in temperature occurs from the driving lane/shoulder interface followed by a gradual increase in temperature beginning at the shoulder/sideslope interface to reach temperatures of 0°C. Temperatures in April are decreasing along the shoulder region to temperatures of 0°C along the sideslope region. The month of October decreases drastically along the shoulder region to a temperature of 12°C where it drastically decreases again along the sideslope region. The temperature profile for the

Figure 6-29: Lateral Temperature Profile for Case 2 – Paved Shoulder Condition
month of November gradually decreases along the shoulder as well as the first half of the sideslope region where it decreases significantly over the remaining 2 m of the sideslope region.

Allowing snow to accumulate along the shoulder (Case 8) alters the temperature profile in comparison to the base case simulation especially in the winter months (December to March), as seen in Figure 6-31. Constant temperature profiles are again found along the paved portion of the highway pavement structure with temperatures changing at the driving lane/shoulder interface. Between the months of June to November temperatures are significantly decreasing along the shoulder region and gradually decreasing along the sideslope region. Temperatures are increasing drastically along the shoulder region and gradually increasing along the sideslope region for the winter months (December to February). However, the month of March temperatures are remaining constant at 0°C. In the spring (April and May) temperatures decrease along the shoulder region and then begin to increase at the shoulder/sideslope interface.

Figure 6-30: Lateral Temperature Profile for Case 6 – Steepening Sideslope Condition
Figure 6-31: Lateral Temperature Profile for Case 8 – Allowing Snow to Accumulate on Shoulder

Comparing each of the mechanisms to the base case simulation illustrates that certain design features have more of an impact on moisture and heat fluxes than other design features. Paving the shoulder (Case 2) impacts the moisture flux by providing less of separation in the volume of water stored in the layers over the twelve years of simulation. The impact on heat flux is a constant temperature under both the driving lane and shoulder. Applying poor vegetation quality (Case 3) impacts the moisture flux through the difference in the amount of volume of water seen on an annual basis within the sublayers. Applying a vapour diffusion rate to the paved surface (Case 5) impacts the moisture flux by allowing the volume of water in the layers to remain the same in each of the three positions (i.e centerline, driving lane/shoulder interface and shoulder edge) over the twelve years of simulation. There is no impact on heat flux when applying poor vegetation to the sideslope and ditch areas or when applying a vapour diffusion rate to the paved surface. Steepening the sideslope (Case 6) impacts moisture flux as well as heat flux. The moisture flux is impacted due to the volume of water at the centerline and
driving lane/shoulder interface remaining the same while the volume of water in the shoulder edge separates with position. The heat flux is altered along the sideslope causing temperature to fluctuate with a temperature difference of approximately 20°. Allowing snow to accumulate along the shoulder (Case 8) also alters moisture and heat flux. The moisture flux is impacted in the sense that the volume of water in the sublayers is cycling at a greater difference on an annual basis than that of the Base Case. The heat flux is altered in the sense that the lateral temperature profile of the shoulder is insulated keeping the temperature cooler and more constant than the Base Case.

6.4 Numerical Modelling Summary

The numerical modelling component of this research evaluated standard highway pavement structures to understand the moisture balances and fluxes and to identify key design features that influence pavement performance. Preliminary modelling was performed to develop a base simulation case whose results are somewhat realistic.

From the base case simulation developed through the preliminary modelling component, seven other simulation cases were developed. These seven cases are modifications to the base case, as discussed in Table 6-1. Each of the eight numerical simulations were simulated for twelve years in an attempt to reach long-term equilibrium conditions. After simulating each of the eight numerical simulations the annual cumulative boundary flux identified that each of the simulations were producing stable conditions at each of the top and base boundaries, but were far from reaching equilibrium.

The modified numerical simulation design features can cause a positive or negative impact on the moisture and heat fluxes of highway pavement structures. The design feature that has a positive impact is applying a vapour diffusion rate to the paved surface (Case 5). Paving the shoulder (Case 2) also has a positive effect. These features produce less moisture to accumulate in the subgrade layer over the entire climatic season. The design features that create a negative impact include applying poor vegetation conditions to the sideslope and ditch areas (Case 3), steepening the sideslope (Case 6) and allowing
snow to accumulate along the shoulder (Case 8). These three design features cause more moisture to accumulate within the subgrade layer in comparison to that of the base simulation case.
Chapter 7 Conclusion & Recommendations

The ability to evaluate surface fluxes is essential in understanding the mechanisms controlling the heat and moisture balance of the subgrade layer as well as the key design features which may enhance or degrade highway pavement performance. This chapter provides the conclusions, with reference to the objectives, of the research. Recommendations are also provided for future research on this subject matter.

The overall objective for this research is to evaluate the relative magnitudes of moisture and heat fluxes within a highway pavement system as well as the average annual water and energy balance. This overall objective is of importance because defining the typical ‘order of magnitudes’ of heat and moisture fluxes is essential in evaluating which mechanisms control the heat and moisture balance within the subgrade layer. This will also aid in identifying design features that may enhance or degrade highway pavement performance.

The overall objective is pursued through a series of more specific objectives. These specific objectives include: (1) evaluating key physical and hydraulic material properties of standard pavement structures; (2) simulating coupled heat and moisture interactions between a typical pavement structure and atmosphere in order to obtain approximate estimates of the long-term heat and moisture balances and flux rates within various elements of the pavement structure; and (3) evaluating the relative impact that design features such as paved shoulders, backslope angle, and asphalt material characteristics may have on these fluxes.
7.1 Conclusions

Based on the results of the laboratory testing undertaken in this study of hot mix asphalt (HMA), the following conclusions have been made:

1. Fresh HMA has the ability to permit more moisture to infiltrate into the sublayers of highway pavement structures as compared to older HMA if only the matrix or unfractured condition of the HMA is considered. The amount of infiltration into the sublayers of highway pavement structures changes with the saturated hydraulic conductivity, \( K_{sat} \) of HMA. The lower the \( K_{sat} \) of HMA the lower the amount of infiltration into the sublayers in the absence of cracks.

2. HMA has a linear relationship of volumetric water content with suction, dropping rapidly from porosity values (0.1kPa) to the first suction reading (approximately 0.3kPa). The drastic drop indicates that the air entry value (AEV) of HMA is almost negligible suggesting that the 'pore space' in the HMA is likely only partially filled with water following drainage likely due to the hydrophobic nature of asphalt.

3. Fick’s First Law can be used to estimate the vapour flux rate of HMA. However, the difference in measured versus calculated vapour flux rates could be from the difference in tortuosity between soil and HMA.

4. The air permeability, \( k_{air} \), of HMA could not be used as a rapid measure of the saturated hydraulic conductivity of HMA due to problems in the \( k_{air} \) procedure.

5. Current HMA surface layers do not provide a great deal of evaporation during the summer period compared with the amount of infiltration.

Based on the results of the numerical modeling sensitivity analysis on standard highway pavement structures, the following conclusions have been made:

1. Flow in and out of the highway pavement structure is approaching stable conditions from year to year. However, the pavement structure as a whole is unlikely to reach equilibrium condition under the application of constant climatic conditions.
2. The drying and wetting of the pavement structure over time will be intrinsically transient and always tied to climatic variability, likely lagging significantly behind the current climatic condition.

3. Paving the shoulder area of a highway pavement structure, steepening the sideslopes and allowing snow to accumulate on the shoulder of a highway pavement structure impacts both the moisture and heat fluxes of a highway pavement structure.

4. Altering the vegetation conditions applied to the sideslope and ditch, and applying a vapour diffusion rate to the paved surfaces only impact the moisture fluxes of a highway pavement structure.

7.2 Recommendations

The overall objective of this research was to evaluate the relative magnitude of moisture and heat fluxes within a highway pavement system as well as average annual water and energy balance. Although this objective has been achieved, further studies are required before the research is to be extended to engineering practice. Further refinement of the model is likely required before further work is done with the model. This might include:

1. Convergence and water balance issues within the numerical models themselves need to be addressed. Both convergence and water balance affect the cumulative annual flux in and out of the pavement structure. Therefore, improving the accuracy of the convergence has the ability to improve the water balance of the numerical model.

2. Enhancing the discretization of the mesh design is required. Improving the mesh discretization would improve the number of elements creating more realistic geometry of the highway pavement structure.

3. The initial conditions used within the numerical simulation model require a more realistic hydrogeologic application. Refining the initial conditions used within the numerical simulation model may improve the initial moisture conditions developed within the highway pavement structure.
4. Refining subgrade layer to include: (a) a difference in the recompacted layer from the compacted layer and (b) a separate upper layer that will be affected by freeze-thaw

General recommendations that would improve the results obtained with the numerical simulation include the following:

1. A more indepth investigation is required into the ability to apply actual HMA properties from laboratory studies along with the pertinent climate information required within VADOSE/W. This would provide more accuracy in the results obtained when analyzing the moisture and heat fluxes and balances within highway pavement structures.

2. Laboratory testing of the pavement sublayers (i.e. base, subbase and subgrade layers) would provide the model with more accuracy in the material properties defined for the soil layers applied within the numerical model.
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