The Ottawa-Carleton Institute for Civil Engineering

**Experimental and Analytical Studies of Friction
Characteristics of Airfield Pavements**

Researched and written by:

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A thesis submitted to the
Faculty of Graduate Studies and Research
in partial fulfilment of the requirements for the degree of

Doctor of Philosophy

Department of Civil and Environmental Engineering
Carleton University
Ottawa, Canada
September 2003

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in partial fulfilment of the requirements for
the degree of Doctor of Philosophy

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September 2003

ABSTRACT

The main purpose of runway or road pavements is to carry traffic loads conveniently, economically, and safely during their design life. Pavement safety is usually evaluated through several components; among them the skid resistance and rutting are the most important indicators. They may cause a serious safety hazard in specific situations especially when water exists in any form. The main objectives of this thesis were to provide guidelines for the design of improved airfield mixes with higher quality both in terms of safety and performance and to develop prediction models concerning frictional characteristics and mechanical properties of airfield asphalt mixes. Nine different airfield asphalt mixes were utilized and tested for their frictional and mechanical properties. In addition to the laboratory evaluation, periodic friction measurements were conducted on two actual taxiway surfaces at Ottawa International Airport. The data and information obtained from the field and laboratory investigations were used to perform the analytical part of the thesis. The analytical investigation included performing statistical analysis to check the significance of the test results and developing several prediction models. The results showed that coarser aggregates could enhance surface friction of airfield pavements. In most cases, this improvement was achieved with better shear properties and other mechanical properties. The thesis provides simple prediction models that relate frictional quality to physical and mechanical properties of airfield asphalt mixes. The results and conclusions of the research provide a cornerstone toward the development of comprehensive design guidelines for airfield mixes with consideration of surface characteristics as one of their main objectives.

ACKNOWLEDGEMENTS

The author would like to express his debt to the following persons for their continuous support and encouragement throughout this thesis:

1. The Government of Egypt for their financial sponsorship during my Ph.D. study period.
2. My two co-supervisors Drs. Halim and Easa for their valuable guidance and scientific advice. The work presented in this thesis would not have been completed without the time and efforts invested by both supervisors and their confidence in my abilities.
3. Transport Canada, PWGSC and NSERC for the financial support.
4. Dr. Hassan, Mr. Farha, and the Staff of the Civil and Environmental Engineering Laboratory for the support provided throughout my experimental study.
5. My fellow students who spent long nights discussing and debating how a measurement could be made at a temperature lower than -10°C .
6. My parents and family for their spiritual support and prayers for successful completion of my studies.

Finally, I would like to acknowledge and offer special thanks to my wife, son (Ehab) and daughter (Aya) who suffered the many cold nights of the Canadian winter waiting for my late arrival to assure me that my hard work is worth the sacrifice for better future.

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ABBREVIATIONS

AC	Asphalt Content
ADT	Average Daily Traffic
AIR	Acid Insoluble Residue
ARHM-GG	Asphalt-Rubber Hot Mix Gap-Graded
ASTM	American Society for Testing and Materials
Av	Air Voids
BPN	British Pendulum Number
BPT	British Pendulum Tester
BRD	Bulk Relative Density
CISST	Carleton In-Situ Shear Strength Test
CRFI	Canadian Runway Friction Index
CRLA	Confined Repeated Load Axial
CTM	Circular Track Meter
DGAC	Dense-Graded Asphalt Concrete
DND	Department of National Defence
DOT	Department of Transportation
DSR	Dynamic Shear Rheometer

ERD	Electronic Recording Decelerometer
ETD	Estimated Texture Depth
FAA	Federal Aviation Administration
FEM	Finite Element Method
HMAC	Hot-Mix Asphalt Concrete
ICAO	International Civil Aviation Organization
IFI	International Friction Index
InSiSST	In Situ Shear Stiffness Test
IRFI	International Runway Friction Index
ITS	Indirect Tensile Strength
JBD	James Brake Decelerometer
JB I	James Brake Index
JDFT	Japanese Dynamic Friction Tester
JWRFMP	Joint Winter Runway Friction Measurement Program
LOD	Longitudinal Direction
LVDT	Linear Variable Differential Transducer
MPD	Mean Profile Depth
MRD	Maximum Relative Density
MTD	Mean Texture Depth

MuN	Mu Number
NASA	National Aeronautics and Space Administration
NCAT	National Centre for Asphalt Technology
NMS	Nominal Maximum Size
NRC	National Resources Canada
NS	Normal Stress
NSERC	Natural Sciences and Engineering Research Council of Canada
OFM	Out Flow Meter
OFT	Out Flow Time
OGEAM	Open Graded Emulsified Asphalt Mixtures
OGFC	Open Graded Friction Course
OIA	Ottawa International Airport
PIARC	World Road Association
PSV	Polished Stone Value
PWGSC	Public Works and Government Services Canada
RLA	Repeated Load Axial
RTAC	Roads and Transportation Association of Canada
SAS	Scandinavian Airlines System
SCRIM	Sideways-Force Coefficient Routine Investigation Machine

SFC	Side Force Coefficient
SFT	Surface Friction Tester
SHRP	Strategic Highway Research Program
SLP	Stationary Laser Profiler
SMA	Stone Mastic Asphalt
SN	Skid Number
SPSS	Statistical Package for Social Sciences
SPT	Sand Patch Test
SS	Shear Strength
SSD	Shear Stress Evaluated by Direct Shear Test
SST	Shear Stress Evaluated by Triaxial Test
STD	Standard Deviation
TC	Transport Canada
TD	Texture Depth
TFOT	Thin Film Oven Test
TRD	Transverse Direction
TSR	Tensile Strength Ratio
USAF	United States Air Force
VMA	Voids in Mineral Aggregate

VPPL	Cumulated Number of Vehicle Passes per Lane
WWT	Wheel Track Tester

CHAPTER 1

1. INTRODUCTION

1.1. General

The main purpose of airfield and/or road pavements is to provide safe and efficient transportation to the public and to promote economic growth during their design life (Flintsch et al., 2003). The safety of the pavement surface is usually evaluated through several components including (RTAC, 1977):

- Skid resistance.
- Rutting susceptibility.
- Pavement surface light reflectivity.
- Area demarcation.
- Debris of foreign objects and oil spillage.

Among these components, skid resistance and rutting of asphalt surfaces are the most important factors to control pavement safety. Pavement skid resistance, as defined by ASTM committee E17, is the "*retarding force generated by interaction between a pavement and a tire under locked non-rotating conditions*" (Kennedy et al., 1990). The lack of skid resistance can provide a serious safety hazard in specific situations, when water exists in any form (RTAC, 1977). During skidding, friction forces are much less than that at any time during wheel rolling (Thenoux et al., 1996). On the other hand, surface deformations of

asphalt surfaces such as rutting have been observed and reported since the late 1950's. It was found that rutting of flexible pavements is a common distress type (Deshpande and Cebon, 2000; Sybilski, 1994; Nikolaides, 2000). Flintsch et al. (2003) suggested that water ponds on the road surface due to rutting could cause deficiency in frictional characteristics of pavement surfaces. Federal Aviation Administration, FAA (1997) also concluded that pavement structural failure, such as rutting or ravelling, is a contributing factor to airfield friction losses.

Existence of rutting can restrict the use of pavement and cause serious safety problems when rut depth exceeds a certain limit (Al-Abdul Wahhab, 1994). If the amount of water on the pavement surface exceeds the combined drainage capacity of the tire tread and pavement macrotexture, dynamic hydroplaning occurs. In such case, the channelization or depression caused by the effect of the wheel loadings can create a serious safety problem. Sufficient inertial forces exist and separate the tires from the pavement surface (Browne and Whicker, 1993). Rutting and excess deformations allow puddles to form and consequently increase the hazard of skidding.

1.2. Problem Definition

Canadian airport agencies, owners and operators have recently reported problems related to the poor frictional properties of newly constructed hot-mix asphalt concrete (HMAC) mixes (PWGSC, 2002). For airfield pavements, friction characteristics are extremely important. They provide the spin-up of the wheels, which is required to operate the electronically controlled antiskid braking systems installed in most modern aircraft. In other words,

adequate runway surface friction is essential for braking and deceleration operations. A previous study (Hatherly and Young, 1976) emphasized the role of skid resistance to reduce the accident rates. It was found that lack of skid resistance was the reason or factor of more than a quarter of wet-road vehicle accidents in the United Kingdom (Kennedy et al., 1990). For airport accidents, the National Transportation Safety Board reported that runway condition was a cause or factor in 115 aircraft accidents between 1983 and 1987. In 1991, a brand new British aircraft skidded more than 1,000 m into the Beagle channel, Chile, during landing. Twenty passengers among sixty tourists on board died in the accident. Thenoux et al. (1996) conducted an investigation to determine why the aircraft skidded off the end of the runway. It was found that the runway had low skid resistance at the time of accident and the skidding occurred due to hydroplaning caused by the presence of water in the rutted area.

Since the lack of adequate friction is a safety concern, there is a need to review the current Canadian HMAC specifications for airport pavements and update them, if necessary. As mentioned previously, rutting can cause safety problems when it exceeds a certain limit (Al-Abdul Wahhab, 1994). In addition, when water exists, rutting allows puddles to form and then increases the hazard of skidding. Therefore, the key for improving the air traffic operations and optimizing the use of runways is to improve skid resistance and reduce rutting of asphalt surfaces without compromising the main mix properties.

1.3. Research Objectives

The main objectives of this research are as follows:

1. To carry out an extensive literature review to determine the main causes and parameters that govern surface characteristics of airfield pavements.
2. To perform an analytical and experimental investigation to quantify the contribution of the main parameters to the overall airfield mix properties.
3. To examine the possibility of improving the frictional characteristics as well as the mechanical properties of airfield mixes.
4. To use the findings of (1), (2), and (3) to provide guidelines for the design of improved mixes with higher quality both in terms of ensuring safety and long-term performance.
5. To develop a prediction model relating friction of airfield mixes to the main mix properties.
6. To develop a statistically based model predicting friction in the field from laboratory friction test results.
7. To enhance airfield pavement safety in terms of improving rutting resistance of the mix through accurate characterization of shear properties of airfield asphalt mixes. This can be performed by relating shear properties of airfield mixes evaluated using different test methods.

These objectives were expected to be realized through the research plan described in Chapter (3).

1.4. Thesis Organization

The thesis is organized in seven main chapters. In Chapter (1), problem definition, main research objectives and thesis organization are introduced. Extensive literature review on skid resistance and rutting of asphalt mixes is performed in Chapter (2). In this chapter, the main factors contributing to safety of airfield pavements are identified. In addition, test facilities to perform the experimental investigation are recognized. Based on the main findings of the literature review presented in Chapter (2), the research plan is designed to achieve the thesis objectives. Chapter (3) presents a discussion on the main phases of the research plan, which consists of three main phases. The first phase is the literature review. The output of this phase is used to formulate the experimental investigation presented in the second phase of the research plan, while the last phase is the analytical investigation. Chapter (4) presents the test facilities and description of test procedures to perform the experimental investigation. Preparation of test specimens and detailed testing program are included in this chapter as well. Results of experimental investigation are presented and discussed in Chapter (5). Relationships between frictional properties and other mix properties are examined as well. Based on the test results presented in Chapter (5), Chapter (6) provides the statistical analysis of these results. In addition, the analytical investigation presented in Chapter (6) results in three mathematical models concerning frictional and shear properties of airfield asphalt mixes. Finally, Chapter (7) includes the summary of work performed through the preparation of this thesis, the conclusions and the recommendations.

CHAPTER 2

2. LITERATURE REVIEW

2.1. Introduction

This chapter presents a comprehensive literature review of the most critical factors affecting safety of highway/runway pavements. They include skid resistance and surface rutting. The review of skid resistance covers; skid resistance measurements, the importance of good frictional properties to reduce the risk of traffic accidents and surface treatments to enhance skid resistance of asphalt pavements. On the other hand, rutting has been known to be the common form of pavement distress (Deshpande and Cebon, 2000). It can cause serious safety problems, if rut depth exceeds a certain limit (Al-Abdul Wahhab, 1994). The review of rutting includes the identification of the factors affecting it, and tests to evaluate rutting susceptibility of asphalt mixes. Models to predict skid resistance and rut depth are also included in this chapter.

2.2. Skid Resistance Development

As the frictional properties of pavement surfaces play an essential role in pavement safety, skid resistance is considered as the most important surface characteristic (Fulop et al., 2000; Chelliah et al., 2003). For safe road and runway operations, adequate level of skid resistance should be maintained at all surface conditions (Fulop et al., 2000).

Skid resistance of pavement surface is affected by the surface microtexture and macrotexture (Wambold et al., 1986; Henry, 2000). If water exists, microtexture provides direct tire-pavement contact while macrotexture enhances the drainage of surface water. Macrotexture allows for the rapid drainage of water from underneath the tires and consequently increases the contact between pavement surface and the tires (Henry, 2000; Fulop et al., 2000). Figure (2.1) presents the basic idea of microtexture and macrotexture (Fulop et al., 2000).

Figure 2.1: Microtexture and Macrotexture Illustration

(Source: Fulop et al., 2000)

Microtexture provides the fine-scale irregularities found on the surface of the pavement. It is defined as the surface irregularities between 0.005 and 0.3 mm. The value of 0.005 mm represents the smaller size of surface irregularities affecting wet friction (Forster, 1981; Forster, 1990). While the value of 0.3 mm represents the greater value of irregularities that can penetrate into the rubber material of tires. Microtexture's initial value depends on the initial roughness of aggregates in the presence of traffic loads polishing action (Jayawickrama et al., 1996). Then its value is a function of aggregate particle mineralogy,

weather, traffic and pavement age. Polishable materials yield low level of microtexture, which in turn reduces skid resistance and causes traffic accidents (Panagouli and Kokkalis 1998). The value of British Pendulum Number (BPN) provides a good surrogate of the size of pavement microtexture.

On the other hand, macrotexture contributes to the roughness of the pavement surface caused by the distribution of aggregate particles (Fulop et al., 2000). Macrotexture are the irregularities between 0.3 and 4.0 mm. The initial macrotexture depends on shape, size and gradation of coarse aggregates in addition to the construction technique used in the surface layer placement. Then its value is affected by the bituminous mix properties and the environmental factors such as temperature (Jayawickrama et al., 1996). Inadequate macrotexture, as a result of wear or improper placement of the surface layer, reduces the skid resistance especially at high speeds and thereby increases the accident risk. Sand patch test (SPT) method is believed to be the basic method to assess pavement macrotexture. The pavement surface is considered rough if the depth of macrotexture is more than 1.0 mm (Panagouli and Kokkalis, 1998). Figure (2.2) shows different combinations of microtexture and macrotexture (Hegmon, 1987).

Microtexture and macrotexture become very important when the pavement surface is wet. The water on the pavement surface should be removed to improve skid resistance on wet pavements. This objective can be achieved by using appropriate surface course mixes and cross slopes. In this case, caution should be paid to design the surface layer with good initial microtexture and macrotexture. The properties of aggregate are very important to provide adequate pavement skid resistance. Polished stone value (PSV) test is usually used

Surface	Macrotexture	Microtexture
Coarse	Harsh
Coarse	Polished
Smooth	Harsh
Smooth	Polished

Figure 2.2: Different Microtextures and Macrotextures

(Source: Hegmon, 1987)

to measure the ability of aggregates to develop good microtexture. In addition, the ability of aggregates to provide good macrotexture can be evaluated by the aggregate abrasion test (Rogers et al., 2003). On the other hand, grooving and tire tread can help to remove surface water from tire-pavement contact area (Thenoux et al., 1996).

Rough macrotexture provides channels that allow water to escape from the tire-pavement contact area (Henry, 2000; Fulop et al., 2000; Chelliah et al., 2003). However, at high speeds, less time is usually available for water flow through the macrotexture. Therefore, the macrotexture alone can not remove all water from the contact area (Thenoux et al., 1996). Good microtexture is required to provide additional skid resistance to supplement the resistance from macrotexture. Although macrotexture and surface grooving expel the majority of surface water, a thin water film may exist on the pavement surface. In

this case, rough microtexture provides dry points of contact for the tire (Thenoux et al., 1996; Henry, 2000; Chelliah et al., 2003).

If the surface water can not be expelled by the tire tread and pavement macrotexture, dynamic hydroplaning occurs and causes the separation of tires from the pavement surface (Browne and Whicker, 1993). Further, if rutting exists, puddles of water in the rutted pavements can increase the hazard of skidding. If a thin film of water covers the pavement surface, viscous hydroplaning occurs (Moore, 1975). This phenomenon can take place at any speed especially on surfaces with poor microtexture. Consequently, good microtexture is important at all driving speeds to provide dry contact between pavement surface and tires (Chelliah et al., 2003).

Figures (2.3) and (2.4) present examples of inadequate and adequate pavement friction (Harald, 1990). In general, regarding microtexture and macrotexture, pavement surface can be divided into four main classes as follows:

- Rough (coarse) and harsh surface, which has microtexture and macrotexture.
- Rough (coarse) and polished surface, which has macrotexture only.
- Smooth and harsh surface, which has microtexture only.
- Smooth and polished surface, which has neither microtexture nor macrotexture.

2.3. Factors Affecting Skid Resistance

There are many factors affecting the skid resistance of HMAC pavements. The skid resistance depends mainly on the friction of pavement surface in terms of coefficient of

Figure 2.3: Profile of a Pavement Section with Insufficient Coefficient of Friction

(Source: Harald, 1990)

Figure 2.4: Profile of a Pavement Section with Satisfactory Coefficient of Friction

(Source: Harald, 1990)

friction (f_c). A large number of factors affect the tire pavement friction (RTAC, 1977; Henery, 1986; Hegmon, 1987; Abdul-Malak et al., 1989; Wambold and Kulakowski, 1991; Thenoux et al., 1996; Chelliah et al., 2003). Among these factors are:

- *Surface conditions* of HMAC pavements such as rutting and the existence of water, thickness of water film and accumulation of contaminants. Contaminants may be found in many forms such as dust particles, oil spillage, jet fuel, ice, snow, and rubber deposits especially in down touch zones for runway pavements.
- *Mix properties* are of great importance to provide good pavement texture. The properties of HMAC surfaces depend on aggregate size, shape and gradation, aggregate type and characteristics such as polishing resistance, and percent of asphalt content. Excessive asphalt (flushing) on the pavement surface decreases skid resistance and may cause safety problem especially if surface is contaminated.
- *Traffic*; this parameter includes intensity of traffic, speed, load and load distribution, and tire pressure.
- *Vehicle/aircraft characteristics* such as tire tread pattern and depth, and braking system conditions.
- *Other parameters* may affect the frictional properties of HMAC pavements such as age of pavement, seasonal variations, pavement geometry, and driver experience.

Friction coefficient can be stated as (RTAC, 1977);

$$f_c = \frac{F_r}{L} \quad (2.1)$$

where:

f_c = coefficient of friction,

F_r = frictional resistance (kN), and

L = vertical load (kN).

Irregularities of the pavement surface should be characterized to study the skid resistance (Kennedy et al., 1990; Gargett, 1990; Wambold and Kulakoweski, 1991). The interaction between the tire and pavement surface is determined through two components. First component is the adhesion evolved between the tire material and surface aggregate. Second one is hysteresis of tires (Jayawickrama and Graham, 1995; Henry, 2000);

- *Adhesion* is the friction developed at the tire-pavement contact area caused by the horizontal shearing force. Its value depends on the microtexture that may be lost under the polishing action of traffic. Polished stone value (PSV) test measures losses in aggregate microtexture under traffic action.
- *Hysteresis* is the second component in determining the interaction between tire and pavement surface. When a tire moves on the pavement surface, its rubber material deforms. This phenomenon is known as hysteresis of tires. It provides additional friction between tire and pavement surface. Its value depends on pavement macrotexture, which is a function of the size, shape and gradation of coarse aggregates.

2.4. Change of Skid Resistance

The magnitude of skid resistance varies with time, traffic, and environmental effects. Variation of skid resistance with environmental effects is due to rainfall, icing, snow or seasonal changes. Figure (2.5) shows the variation of skid resistance with time due to rainfall. The figure illustrates that rainfall significantly changes the skid resistance. However, Figure (2.6) is an example for seasonal changes, it shows the change of skid resistance for long time period. The change of skid resistance with traffic is shown in Figure (2.7) for two pavement sections. The figure shows that skid resistance decreases rapidly with a decreasing rate as traffic increases (RTAC, 1977). This behaviour was further demonstrated by (Flintsch et al., 2003; Do et al., 2003). Bean and Pidwerbesky (1994) found a relationship between the skid resistance and traffic volume as shown in Figure (2.8). The road section was opened for traffic for more than 7 years before they conducted their study.

The effect of speed on skid resistance was the objective of many researchers (Kummer and Meyer, 1967; Meyer, 1991; Fulop et al., 2000; Flintsch et al., 2003). Generally, skid resistance decreases as speed increases. If macrotexture is available, this decrease will not be significant. On the other hand, to maintain high values of friction, microtexture is always required. Meyer (1991) developed a typical relationship between skid number (SN) and speed (V) for different combinations of microtexture and macrotexture. The relationship could be described by the following equation;

$$SN = SN_o e^{-(PNG/100).V} \quad (2.2)$$

Figure 2.5: Change in Skid Resistance due to Rain

(Source: RTAC, 1977)

Figure 2.6: Seasonal Change in Skid Resistance

(Source: RTAC, 1977)

Figure 2.7: Long Term Change in Skid Resistance of Two Pavement Sections

(Source: RTAC, 1977)

Figure 2.8: Effect of Traffic Volume on Skid Resistance

(Source: Bean and Pidwerbesky, 1994)

where:

SN_0 = the intercept at zero speed, its value depends on the microtexture of the pavement surface.

PNG = the slope of the curve between SN and V, it measures the macrotexture of the pavement surface, for coarser macrotexture, the slope (PNG) decreases.

V = test speed

Figure (2.9) illustrates the importance of good macrotexture to control skid resistance of pavement surfaces. The figure shows that the change of wet friction with speed decreases for good macrotexture (Flintsch et al., 2003).

It is obvious that wet friction and speed are inversely related, i.e., as speed increases, skid number decreases. At low speeds, the measured wet friction depends on the microtexture of the pavement surface. At such speeds, the rate of water expulsion from under the tire is low and the flow rate can be easily accommodated by tire tread and the macrotexture of the pavement surface. However at high speeds, the rate of water expulsion from under the tire increases. If the pavement has a smooth macrotexture, the channels on the pavement surface cannot accommodate the flow. Consequently, the thickness of water film underneath the tire increases and the measured skid number decreases (Flintsch et al., 2003).

The speed of rubber shoe of the British pendulum tester (BPT) is low, approximately 7 mph, when it contacts the pavement surface. The adhesion, which is a function of the microtexture, is dominant at such low speeds. However, SN measured by locked-wheel

Figure 2.9: Importance of Good Macrotexture to Control Skid Resistance

(Source: Flintsch et al., 2003)

trailer at 40 mph indicates both microtexture and macrotexture effects. As a result, the poor correlation between the laboratory PSV and the field skid number (SN_{40}) is due to the failure of PSV test to represent the importance of the macrotexture. However, other studies showed that better correlation could be obtained if the influence of microtexture and macrotexture are separated from each other (Wambold, a and b, 1988).

Since the skid resistance of pavement surfaces can change with time, traffic, and environmental effects, periodic measurements are needed to account for these changes in skid resistance. Measured skid resistance could be affected by surface wear, polishing of surface aggregates, rutting, bleeding and surface contamination. For airfield pavements, periodic measurements of skid resistance are usually performed with the objectives of (Antvik, 1997):

- Evaluating frictional properties of runways under winter conditions.
- Checking frictional properties of newly constructed runways.
- Counting for periodic changes in the slipperiness of wet runways.
- Determining the effect of drainage system on friction characteristics.

2.5. Skid Resistance Measurements

2.5.1. General

There are many methods to evaluate safety of pavement surfaces in terms of skid resistance. Some methods measure the force required to drag a locked-wheel. Others measure skid resistance in slip mode or yaw mode. As each device measures a different aspect of frictional interaction between tire and pavement, correlation between skid testing devices is not very accurate (RTAC, 1977).

2.5.2. Development of Measuring Devices

In the early of 1900's, although many runway surfaces were grass, it was recognized that there was a need to evaluate the frictional properties of these surfaces to assure safe aircraft operations. At this time, frictional properties were usually checked by a skid test performed by the airport manager. The flight activities were stopped if his skidding distance was too long. As accident numbers increased and aircraft became larger and heavier, the demand for better ways for measuring friction increased.

In 1946, Scandinavian airlines system (SAS) started to operate DC-4 aeroplanes. To avoid accidents, the Oslo airport manager used to check the friction of runway surface

before DC-4 operations. The friction test was performed using a big truck loaded with sand. The truck's full brakes were applied at speed of 30 km/h. Time (t) and/or distance (d) to full stop were recorded to represent the frictional properties of runway surface. The ratio V/t was calculated as the retardation (r). SAS found that the calculated r was approximately half the value of the DC-4 retardation. This method was known as Kollerud method for measuring friction (Antvik, 1997).

At that time, the United States Air Force (USAF) developed the James Brake Decelerometer (JBD) to measure the friction at American airports. The measured parameter is reported as James Brake Index (JBI), which is the deceleration in ft/sec/sec. JBI usually ranges between 0 and 32 (RTAC, 1977).

Later on, Kollerud method was developed and a decelerometer known as Tapely-meter was introduced. The test was performed by installing the Tapely-meter in a car. The car was accelerated to a selected speed and then the full brakes were applied. Some years later, the skiddometer was introduced and by late of 1960's surface friction tester (SFT) was developed (Antvik, 1997). The following section presents a discussion for some of the friction measurement methods.

2.5.3. Friction Measurement Methods

2.5.3.1. Field measurement methodologies

General

As the weight and landing speed of turbojet aircraft increased, frictional properties of runway surfaces are now considered as a safety problem especially under wet conditions. Occurring of hydroplaning or having a low level of friction may result in loss of directional

control (FAA, 1997). To assure safe aircraft operations, full-scale measuring devices were developed. They were categorised into four main groups: locked wheel, side force, fixed slip and variable slip methods. Each measuring method has its own advantages. The locked wheel method has the advantage to simulate the emergency braking without anti-lock brakes. In contrast, fixed slip and variable slip deal with the scenario of braking with anti-lock brakes. Finally, the side force method examines the directional control in horizontal curves (Henry, 2000).

Locked wheel method

This test method is used to measure the skid resistance of pavement surfaces using a full-scale automotive tire (ASTM E274-90, 1996). It is the most commonly used method to measure skid resistance in the field. Test method measures the friction force on locked wheel during dragging, under a constant load and at a constant speed, over a wetted pavement surface. Locked wheel testers operate at 100 percent slip speed. The slip speed is “*the relative velocity between the surface of the tire and the pavement surface*” (Henry, 2000). The equipment consists of an automotive vehicle with one or more test tires. The system contains a transducer, instrumentation, water supply and brake control for the test wheel. Skid resistance can be determined in terms of SN, which can be calculated as;

$$SN = 100 \frac{F_r}{L} \quad (2.3)$$

where:

F_r and L are the same as presented in Equation (2.1).

This method is known as Locked-Wheel Trailer Method. Skid numbers of 30 in low traffic and 35-38 in high traffic volume roads are considered adequate by most Departments of Transportation (DOTs) in the United States. However, some states use a higher value of 40. When SN ranges between 31 and 34, the pavement surface should be observed and cautions are taken such as placing alert signs and conducting periodic friction testing. SN below 30 is considered not safe by most DOTs (Jayawickrama et al., 1996). Earlier research indicated that locked wheel tester measurements performed using a ribbed tire are more susceptible to surface microtexture than macrotexture. In consequence, they give a good estimation for pavement microtexture (Leu and Henry, 1978).

Side force devices

To prevent test wheel from rolling freely, all side force devices keep a yaw angle (α) during the friction measurements. Yaw angle is the angle between the direction of motion and the plane of test wheel (Henry, 2000). Side force measurements are performed at a low speed, which is the test speed multiplied by $\sin\alpha$. As a result, all these devices are insensitive to macrotexture measurements. Mu-meter and sideways-force coefficient routine investigation machine (SCRIM) are examples for side force devices.

The Mu-meter (ASTM E670-94, 1996) has been developed in the United Kingdom (UK), it is an uncomplicated trailer towed by a vehicle that measures skid resistance in the yaw mode. The test is performed by running the Mu-meter at the preferred test speed on the wetted pavement surface and the resulting cornering friction force between the test wheels and the pavement surface is recorded. The cornering friction force is obtained and recorded

as Mu number (MuN). The International Civil Aviation Organization (ICAO) recommended the minimum acceptable MuN to be 0.4 Mu (Thenoux et al., 1996).

The SCRIM has been developed by Transport and Road Research Laboratory in UK and it is commonly used in UK and France. It also measures the skid resistance in the yaw mode, but it is considered more complicated than Mu-meter. SCRIM uses a yaw angle of 20 degrees on the test tire. The output measurement is known as the sideways-force coefficient (SFC), which can be calculated as (RTAC, 1977);

$$SFC = 100 f_c \quad (2.4)$$

where:

SFC = sideways-force coefficient.

f_c = coefficient of friction.

Although, SFC is numerically the same as SN mentioned before, SCRIM has advantage that it has a continuous record of skid resistance and it can run high operating speeds. In distinction, the relatively high cost is the main disadvantage of that machine (RTAC, 1977). Most SCRIMs now have a laser system installed on the front of the vehicle to provide macrotexture measurements (Henry, 2000).

Fixed slip devices

These measuring devices operate at a constant slip ranging from 10 to 20 percent. Similar to side force devices, fixed slip devices measure the friction at low speeds. Griptester is the most commonly used device runs at constant slip (Henry, 2000). The main component of

Griptester, as shown in Figure (2.10), is its rubber wheel that rolls under certain operating conditions such as constant load, constant slip, and constant speed (Cenek et al., 1999).

Variable slip devices

These devices operate under a predefined set of slip ratios. Many locked wheel testers have the capability to run under variable slip mode. The variable slip Norsemeter ROAR is an example of variable slip devices (Henry, 2000).

2.5.3.2. Laboratory methods

There are two devices to measure the friction in the laboratory. First device is the BPT and second one is the Japanese Dynamic Friction Tester (JDFT). Both devices can be used to evaluate the field friction as well.

British pendulum tester

This method utilizes the well-known pendulum type device developed by the British Transport and Road Research Laboratory as presented in ASTM, E303-93 (1996). The test is conducted by sliding a rubber shoe on the pavement surface. Test results are reported in terms of BPN. The BPN of 55 to 65 is considered to be sufficient and safe for special runway sections such as braking section (Thenoux et al., 1996). As mentioned before, the results of BPT do not correlate well with the locked-Wheel Trailer test results. This may be relating the relatively low speed of the shoe when contacting the pavement surface. The tester has many advantages such as ease, low cost, and effortless transportability. The only disadvantage of this test is that it needs to stop traffic during the field measurements (RTAC, 1977). A previous research demonstrated that friction values measured by BPT correlate well with both SCRIM and Griptester measurements (PIARC, 1995).

Figure 2.10: Findlay Irvine Griptester

(Source: Cenek et al., 1999)

Japanese dynamic friction tester

JDFT has three rubber sliders mounted on a disk. The device has the capability to measure the surface friction at a wide speed range of 0 to 90 km/h. Friction data are collected by a transducer and data are automatically saved to a notebook computer. The laboratory specimens should have minimum dimensions of 450×450 mm to be tested using the JDFT (Henry, 2000).

2.5.4. Texture Measurement

Schonfeld (1970, 1974) used colour stereo-photographs to analyse the texture of the pavement surface in terms of a texture code number. This number was correlated with SN. The advantages of this method are the low cost, ease and capability to test sharp curves and stop sign locations. However, it is very slow and depends on operator cleverness.

Microtexture profiles are too difficult to be measured directly in the field especially at high speeds. As mentioned before, there are some measuring devices that give a good surrogate for pavement microtexture. Among these devices are SCRIM, BPT and ribbed locked-wheel trucks. In contrast, macrotexture can be evaluated through several techniques. The basic method to measure pavement macrotexture is the sand patch method. Laser devices and out flow meter (OFM) are also used to measure the macrotexture of pavement surfaces.

2.5.4.1. Sand patch method

This method is used to determine the average macrotexture depth of pavement surface using a volumetric technique (ASTM, E965-87, 1996). The apparatus consists of a millimetre ruler, cylinder with volume of approximately 25,000 mm³, flat wooden disk of 65 mm diameter with a hard rubber base of 1.5 mm thickness, a 250,000 mm³ plastic container used for holding the sand and a soft brush. Test procedure involves distributing a known volume of sand on a clean dry pavement surface. *“The sand should pass a No. 50 sieve and be retained on No. 100 sieve. Natural sand with a round particle shape should be used”* (Wessex, 2001). The diameter of area covered with sand is measured and the mean texture depth (MTD) is calculated as follows;

$$MTD = \frac{Vol}{\pi R^2} \quad (2.5)$$

where:

Vol = volume of cylinder (mm)

R = radius of sand patch (mm)

MTD = texture depth (mm)

The International Civil Aviation Organization (ICAO) recommended a minimum texture depth value of 1.00 mm for safe macrotexture of runway surfaces (Thenoux et al., 1996; Antvik, 1997).

2.5.4.2. Laser devices

As a result of advances in laser technology, laser devices are now used to evaluate pavement macrotexture. Laser devices can be mounted on the front of measuring vehicle and macrotexture measurements can be conducted without any traffic diversions. Figure (2.11) shows a Stationary Laser Profiler (SLP) used in New Zealand. The profiler can provide macrotexture profiles with a very high resolution compared to profiles obtained using a laser device mounted on a vehicle. SLP consists of a laser device moving along a rail. The device has the capability to measure surface macrotexture with an accuracy of ± 0.03 mm (Cenek et al., 1999).

The Circular Track Meter (CTM) also uses the laser technology to measure pavement macrotexture. It was introduced in 1998 and it could be used in both laboratory and field measurements. CTM has a laser sensor to measure the texture profile in a circle of approximately 127 mm radius. Texture calculations are performed using a notebook computer attached to the device (Henry, 2000; Flintsch et al., 2003). Flintsch et al. (2003) developed a relationship between the mean profile depth (MPD) evaluated by the CTM and the sand patch measurements in terms of MTD. Figure (2.12) demonstrates that the correlation between MPD and MTD is excellent as shown by the coefficient of determination (R^2) of 0.943.

Figure 2.11: Transit New Zealand's Stationary Laser Profiler
(Source: Cenek et al., 1999)

Figure 2.12: Correlation between CTM and SPT
(Source: Cenek et al., 1999)

2.5.4.3. Out flow meter

Out flow meter (OFM) is also used to measure pavement macrotexture. The test can be performed by filling a transparent cylinder with water and placing it on the pavement surface. The cylinder should have a rubber annulus and a valve at the bottom end. The valve is opened and the time required for a specific amount of water to flow through the pavement surface is recorded. The inverse of out flow time (OFT) is used as a measure for pavement macrotexture. OFM measurements were also found to correlate well with the MTD measured by the sand patch test as indicated by R^2 value of 0.99 (Henry, 2000).

2.5.5. International Friction Index

To harmonize the different friction measurement methods, the world road association performed an international experiment in 1992. This experiment was the base to develop an international friction scale. The data obtained by this experiment were combinations of friction and macrotexture measurement values. As a result the international friction index (IFI) was developed (PIARC, 1995). IFI consists of two parameters, friction parameter and texture parameter. Friction parameter (F_{60}) is the friction of pavement surface, formerly denoted as SN, measured at 60 km/h. While, the texture or speed parameter (S_p) presents a measure for pavement texture according to the following equation (PIARC, 1995);

$$S_p = a + bT \quad (2.6)$$

where:

S_p = texture parameter of IFI (km/h)

T = pavement texture measurement (mm)

a, b = calibration constants for each texture measuring device.

If pavement texture is measured in terms of MTD, then a and b are equal to -11.6 and 113.6 respectively. While a and b are equal to 14.2 and 89.7 for pavement texture measured in terms of MPD.

The friction parameter F_{60} can be calculated as (PIARC, 1995);

$$F_{60} = A + BF_s e^{\left(\frac{S-60}{S_p}\right)} \quad (2.7)$$

where:

F_{60} = friction parameter of IFI, adjusted to 60 km/h base velocity.

A, B = empirically determined constants for each friction measuring device

F_s = friction measured at slip speed S

S = measuring slip speed (km/h)

S_p = texture parameter of IFI (km/h)

If ribbed tire is used in friction measurements, the texture measurement is then used to adjust F_{60} according to the following equation (PIARC 1995);

$$F_{60} = A + BF_s e^{\left(\frac{S-60}{S_p}\right)} + CT \quad (2.8)$$

where:

A, B, C = calibration constants for each friction measuring device

If the ASTM E274 skid trailer is used to measure surface friction then $A = 0.045$, $B = 0.925$, and $C = 0$ for smooth tire and $A = -0.023$, $B = 0.607$, and $C = 0.098$ for ribbed tire.

2.5.6. International Runway Friction Index

It was recognized that snow/ice contaminated areas in airfield pavements might cause safety problems during aircraft winter operations (Norheim et al., 2001). As a result, sufficient data about winter friction should be provided to airport agencies for more safe air transportation. At the same time, international format for runway conditions was needed to exchange information between airport agencies, owners and operators. Therefore, the demand for an international runway friction index (IRFI) to be developed was raised (Norheim et al., 2001). Providing aviation agencies with the necessary information to operate aircraft safely under all weather conditions, IRFI helps to reduce aircraft accidents and minimize delays at airports.

The National Aeronautics and Space Administration (NASA), Transport Canada (TC) and the Federal Aviation Administration (FAA) have conducted the joint winter runway friction measurement program (JWRFMP) to harmonize and standardize the ground vehicle friction values. To achieve this objective, aircraft and ground vehicle friction tests were performed to identify a common number representing friction of runway surface. This number was denoted as IRFI and it was independent of the country or the measuring device. IRFI could provide the pilot with accurate runway friction information to make go/no go decisions before aircraft operations (Yager et al., 2002).

Similar to the approach used by PIARC international experiment (PIARC, 1995), 10 aircraft and 44 ground vehicles were used to collect data at several airports. Figures (2.13) and (2.14) shows two different aircraft runs performed in Canada and USA. About 15,000 ground vehicle runs and 450 aircraft runs were conducted to correlate between aircraft braking performance and ground vehicle measurements.

Data collected from test runs were used to calculate the parameters (a and b) of IRFI for the different measuring devices. Table (2.1) presents the IRFI correlations for the different ground vehicles based on the data collected through 1999. The measured actual aircraft braking suggested that the IRFI could provide representative data for the actual aircraft performance. Similar index was developed for the Canadian airports and it was denoted as Canadian Runway Friction Index (CRFI). The first stage of the JWRFMP will be completed by 2005 with the aim of reducing the fatal aircraft accident rate by 80 percent. It is planned to perform aircraft friction measurements using bigger aircraft such as B77 and A330 in the next 2 years. By the end of the project in 2020, the fatal accident rate is expected to be reduced by 90 percent (Yager et al., 2002).

2.6. Controlling of Skid Resistance

2.6.1. General

Generally, microtexture and macrotexture control the skid resistance of HMAC surfaces. Choosing of good quality, polish-resistant aggregates controls microtexture. While macrotexture depends on type and stability of mix and aggregate gradation (Jayawickrama et al., 1996; Chelliah et al., 2003). In the United States, controlling the quality of coarse

Figure 2.13: Falcon-20 Aircraft during Test Run at North Bay, Canada

(Source: Yager et al., 2002)

Figure 2.14: The NASA 757 Research Aircraft Makes a Test Run in Michigan, USA

(Source: NASA, 2000)

Table 2.1: Harmonisation of Friction to Develop IRFI

(Source: Yager et al., 2002)

Device Description	Tire Type	Constant a	Constant b	R ²	Standard Error of Estimate
Airport surface friction tester SAAB95*	Treleborg AERO 890 kPa (100 psi)	0.0565	0.4264	0.78	0.023
FAA Trailer BV-11	Treleborg T520 690 kPa (100 psi)	0.0445	0.7635	0.83	0.052
TC ERD in Chevrolet 1500 Truck		0.0879	0.8814	0.73	0.045
DND Griptester	ASTM E-1844	0.0001	1.1109	0.82	0.042
NASA Instrumented tire test vehicle	Aircraft tire 26 by 6 inches	0.0907	1.0231	0.92	0.048
FAA Runway friction tester**	ASTM E-1551 69D kPa (100 psi)	0.0226	0.7262	0.98	0.034
Norsemeter RUNAR	ASTM E-1551 207 kPa (30 psi)	0.0504	0.3923	0.77	0.030
TC Surface friction tester SAAB 1979	ASTM E-1551 69D kPa (100 psi)	0.0598	0.7589	0.92	0.034
Munich Airport surface friction tester	Treleborg AERO 69D kPa (100 psi)	0.1261	0.6727	0.67	0.044

* 1998 data only

** Small number of data points

ERD: Electronic Recording Decelerometer

DND: Department of National Defense (Canada)

aggregates can assure good skid resistance of the pavement surface course. Complete laboratory assessment of aggregates should be conducted to measure the quality of these aggregates. Frictional properties of aggregates can be evaluated by the either PSV test or acid insoluble residue (AIR) test.

2.6.2. Skid Resistance Control Categories

There are major differences in the real design methodology used by DOTs that take into account the skid resistance in their design procedures. For example, some DOTs consider that sufficient skid resistance is obtained through the suitable design procedures. Others control the quality of coarse aggregates used in the surface mix to get higher skid resistance. Table (2.2) presents the five categories of state DOTs skid control practices (Jayawickrama et al., 1996).

Category I is practical in the states that have high-quality aggregates. The state DOTs classified as category II control skid resistance through the appropriate mix design. DOTs included in category III consider that surface friction is covered in the design process of new pavements. DOTs highlight controlling the aggregate quality to gain the adequate skid resistance. As DOTs belong to the first three categories do not evaluate the aggregate frictional properties, regular field measurements of skid resistance of pavement surface are performed to make sure that a good skid resistance is retained. The states classified as category IV carry out laboratory tests, such as the AIR test, which aims to find a relationship between the frictional properties of aggregates and physical properties obtained by the test, to assess frictional properties of aggregates. Finally, for category V, DOTs use field

Table 2.2: Description of DOT Categories

(Source: Jayawickrama et al., 1996)

Category	Description
I	No specific guidelines to address skid resistance
II	Skid resistance is accounted for through mix design
III	General aggregate classification procedures are used
IV	Evaluate aggregate frictional properties using laboratory test procedures
V	Incorporates field performance in aggregate qualification

performance results in combination with laboratory tests to assess aggregates properties. An example for this category is the procedures followed by Texas DOT.

2.6.3. Texas DOT Procedures

Texas DOT found that some low PSV materials could give a high-quality field performance regarding skid resistance. In addition, some of the previous work conducted in Texas indicated that there was no good relationship between aggregate PSV and the measured skid properties (Dahir et al., 1976). Therefore, Texas DOT developed other procedures to evaluate aggregates based on their skid performance history. The skid performance history for a given aggregate source is evolved by measuring the skid number at 40 km/h (SN_{40}) on the pavement surface constructed using the same aggregate type. The cumulated number of vehicle passes per lane (VPPL) is also determined and plotted against SN_{40} to develop a typical relationship between SN_{40} and VPPL. For given traffic speed and surface type, Table

(2.3) provides guidelines for minimum satisfactory skid numbers. Using Table (2.3) in combination with the developed relationship between SN_{40} and VPPL, VPPL can be obtained for the preferred skid number. Using VPPL, directional distribution factor, lane distribution factor and average daily traffic (ADT), pavement service life can be calculated. If the calculated pavement life is greater than the design life of pavement, the aggregate is assumed to be acceptable (Jayawickrama and Graham, 1995).

Procedure described above has a major lack regarding changes associated with friction measurements. Although skid number measurements done on pavement sections built with the same aggregates type as the aggregates under evaluation, variability may happen due to worker error, changes in pavement macrotexture, seasonal changes, pavement distress, inappropriate calibration and traffic characteristics.

A minimum of 6 data points is required to build the skid performance history for an aggregate. If such data are available, the procedure suggested the utilization of 30 data points. To ensure appropriate distribution in data, the following cautions should be taken into consideration (Jayawickrama and Graham, 1995):

- The total range values for VPPL should be at least 250,000.
- At least one measurement should be made after a VPPL of 750,000, beyond which substantial deterioration of skid resistance can occur.

2.6.4. Polished Stone Value Test

There are many factors affecting the maximum skid resistance values of the pavement surfaces. These variables are; temperature, abrasion, dust, weathering, traffic, location

Table 2.3: Design Guidelines for Minimum Acceptable SN_{40} for Given Traffic Speeds and Surface Types

(Source: Jayawickrama and Graham, 1995)

Mean traffic speed	SN_{40}	
	Surface treatment	ACP
64 km/hr	33	33
80 km/hr	33	34
96 km/hr	33	35

and polishing (Bean and Pidwerbesky, 1994). Of them, only polishing resistance can be controlled at the time of construction. The relationship between the PSV and the future obtainable skid resistance of pavement surface was suggested and verified by many authors during the sixties and seventies (Shergold, 1963; Hatherly and Young, 1976). The PSV test is considered as a good indicator of the capability of a specific aggregate to give an appropriate surface skid resistance (harsh microtexture) that can reduce the number of accidents (Bean and Pidwerbesky, 1994).

An accelerated polishing machine is used to simulate the effect of traffic wheels on aggregates. The wheel of the machine is of 406 mm diameter. Machine can test 12 specimens and two control specimens of known PSV as a base line. 35-50 stones should be chosen to represent the whole aggregate sample. These stones are placed in a one layer, with their smooth faces downwards, in the test mould and arranged such that they reduce the voids in between. The test machine runs at 320 revolutions/min. (to simulate a surface speed

of 24.5 km/h). Two runs are required for a complete test of four specimens placed in pairs. At the end of the test, samples are removed, washed and tested for skid resistance by PBT.

The variability of the PSV test results can be reduced by (Bean and Pidwerbesky, 1994):

- Knowing the source of control stones.
- Replacing the tires of testing equipment after a specific test number to get reliable results.
- Completing the test with the same operator who should be qualified.

However, there is a large variation in aggregate nature even if they are delivered from the same source. Consequently, it is hard to obtain a representative sample of the entire source.

2.7. Skid Resistance-Traffic Accident Relationship

Pavement skid resistance is one of the most important parameters to reduce traffic accidents (Flintsch et al., 2003). A previous study concluded that there is a good correlation between skid resistance and accident rates (Hatherly and Young, 1976). This study emphasized the importance of skid resistance to decrease the accident rates. Skid resistance provides the needed friction between tires and pavement surface to control vehicle direction and speed. Low level of skid resistance leads to skidding and causes traffic accidents. It was found that on wet pavements, skidding problems were the reasons or at least factors of more than a quarter of roads accidents in the United Kingdom (Kennedy et al., 1990).

Andrey et al. (2003) has studied the effect of winter precipitation events on the accident rate in Ottawa, Ontario, Canada. He concluded that the collision risk have increased significantly during the winter season. Panagouli and Kokkalis (1998) found that, for constant traffic volume, the accident rate decreased significantly as the skid number increased. Hosking (1987) found that an enhancement of 10 % in the level of skid resistance resulted in 13 % reduction in the wet accident rate.

Other studies indicated that the microtexture and macrotexture have a substantial influence on accident rate especially at high speeds on wet pavements. As a general conclusion, they found that “*wet weather accident rates are a little bit more than twice those of dry pavement accidents*” (Hardwood et al., 1976; Wambold et al., 1986).

Chelliah et al. (2003) have developed a relationship between the number of wet accidents and the friction of pavement surface. The model was developed using all wet accident data for US highways occurring in 2001. The model is presented as;

$$WA = 4390.1e^{(-0.1776F+4.8567)} \quad (2.9)$$

where:

WA = number of wet accidents / year

F = friction of highway surface (SN₄₀).

For airfield pavements, “safe winter runway operations remain a challenge for airport operators, air traffic controllers, airlines and pilots who must coordinate their efforts under rapidly-changing weather conditions” (Yager et al., 2002). The National

Transportation Safety Board reported that “*runway condition was a cause or factor in 115 aircraft accidents between 1983 and 1987*” (Thenoux et al., 1996). Contaminated runway surface, by snow and/or ice, was also a factor in approximately 30 aircraft accidents between 1983 and 1995. Additionally, inaccurate data on runway friction conditions was a reason for many dangerous aircraft operations (NASA, 2000).

At Los Angeles International Airport, a jumbo jet DC-10 was completely lost due to lack of skid resistance of runway surface in 1970s. The touch down zone was very slippery at the time of accident (Antvik, 1997). Because of inaccurate runway friction information, a fatal aircraft accident occurred in Dryden, Ontario, Canada, in 1989 (Moshansky, 1992). Figure (2.15) shows another aircraft accident at a Mexican airport due to insufficient data on runway friction. The DC-9 aircraft was thoroughly damaged during a thunderstorm and 4 passengers were killed (Yager et al., 2002). In 1991, “*Although the weather conditions were good, a brand new British aircraft skidded more than 1,000 m into the Beagle channel, Chile, during landing. Among 60 tourists on board, 20 people died*”. An investigation was conducted to figure out why the aircraft skidded. It was found that the skid resistance of runway surface was low and the accident took place due to viscous hydroplaning (Thenoux et al., 1996).

2.8. Surface Treatments to Enhancing Skid Resistance

Polish-resistant aggregates should be used to enhance skid resistance of pavement surface. The aggregates should be angular, coarse and hard with hard mineral particles to have good frictional characteristics. In addition, they should be distributed uniformly over the surface.

Figure 2.15: Aircraft Accident due to Lack of Skid Resistance

(Source: Yager et al., 2002)

Under traffic action, the aggregates may be smoothed or polished and as a result the microtexture and macrotexture are reduced. Traffic compaction may insert the overhanging aggregates into the pavement surface. At higher temperatures, the depth of macrotexture is reduced, and tire treads will be responsible to expel the surface water. It is obvious that higher friction is needed at critical sections such as curves and traffic lights (Kennedy et al., 1990).

Roque et al. (1995) conducted a field study to enhance the frictional characteristics for bituminous wearing course mixtures. They found that keeping field air-void content above a certain value was the solution to control skid resistance of pavement surfaces. The results showed that these values were 3.4 % for 12.5 mm maximum aggregate size mixes and 3.0 % for 25.4 mm maximum aggregate size mixes. In this study, the Texas gyratory compactor was found to produce more representative asphalt mixes than Marshall

compactor, which did not simulate field conditions under traffic loads. The study indicated that coarse aggregates play a major role to enhance the skid resistance for the asphalt surfaces where higher SN was resulted from using 25.4 mm maximum aggregate size. It was also found that, a small increase in asphalt content resulted in a considerable decrease in air void content that could yield lower skid resistance level.

Applying antiskid surfacing to gain skid resistance is of great importance to reduce accidents. Kennedy et al. (1990) found that, a 35% reduction in wet road accidents was obtained by applying antiskid surfacing. In addition, this reduction in accidents gave a net return of 54% of the expenses.

Slurry seals are usually used to give better riding quality and protection against wear for asphalt pavement surfaces. As they improve the surface texture, slurry seals are typically used on runway surfaces in USA (FAA, 1997). Cautions should be paid when applying slurry seals to runways. Too high asphalt content leads to excessive asphalt on the surface that can cause lower skid resistance level. In contrast, too low asphalt content separates the aggregates from the seal and may cause a safety problem for aircraft engines due to the debris.

Having a high void content, open-graded friction course (OGFC) can enhance the friction of pavement surface (Cooley et al., 2000). It is used frequently in airfield pavements to allow surface water to be expelled through the asphalt layer and thus maintain a high level of friction for pavement surface (FAA, 1997). However, in 1998, the National Centre for Asphalt Technology (NCAT) conducted a survey to study the performance of OGFC. The survey showed that OGFC could be used successfully by using coarser aggregate gradations

(Kandhal and Mallick, 1998). More improvement in the performance of OGFC could be achieved using polymer-modified binders (Watson et al., 2003)

For airfield pavements, both NASA and FAA concluded that cutting transverse grooves on the runway surface could enhance pavement surface friction. These grooves should be closely spaced such that it permits water to escape from the contact area between pavement surface and aircraft tires (FAA, 1997).

Open graded emulsified asphalt mixtures (OGEAM) also have high air void content. Crushed stone or gravel should be used in OGEAM. The aggregate gradation is usually controlled by the percentages passing number 10 sieve and number 200 sieve. These percentages should be less than 10 percent and 2 percent for number 10 sieve and number 200 sieve, respectively. OGEAM could be used to improve the skid resistance and reduce the risk of hydroplaning. However, for airfield pavements, some quality control precautions should be taken to assure a good performance after construction (FAA, 1981).

2.9. Estimation of Skid Resistance

Many researchers related the skid resistance in terms of skid numbers with the pavement texture parameters as explained below.

The following equations were proposed to relate SN_{40} to BPN and MTD (Wambold and Kulakowski, 1991; Henry, 1986; Descornet, 1989);

$$SN_{40} = 0.884(BPN) + 5.16(MTD) - 17.8, R^2 = 0.86 \quad (2.10)$$

where:

BPN = British pendulum number

MTD = Texture depth (mm)

$$SN_v = SN_o^* e^{\frac{V(PNG)}{100}}, R^2 = 0.86 \quad (2.11)$$

where:

SN_v = skid number at speed V in km/h

SN_o = zero speed intercept of SN-speed graph, calculated as;

$$SN_o = 9.44 \sqrt{\frac{\sum m_i^2}{n}} - 44.4, R^2 = 0.76 \quad (2.12)$$

where:

$\sum m_i$ = the sum of pavement microtexture (μm).

n = the number of m_i values.

PNG = the slope of the curve between SN and V, it measures the macrotexture of the pavement surface, for coarser macrotexture, the slope (PNG) decreases.

PNG can be calculated as;

$$PNG = 0.35 \left(\frac{\sum m_a^2}{n} \right)^{-0.26}, R^2 = 0.84 \quad (2.13)$$

where:

$\sum m_a$ = the sum of pavement macrotexture (mm).

n = the number of m_a values.

Forster (1981, 1990) studied the correlation between SN_{40} and the wavelength of the pavement texture. He proposed the following equation;

$$SN_{40} = 18.6 \frac{100/\lambda}{28.5 + \left(\frac{145 - m_a}{m_a - 3.9}\right)^{1.7}} + 12.6, R^2 = 0.71 \quad (2.14)$$

where:

λ = distance between adjacent asperities.

m_a = depth of pavement macrotexture (mm).

However Kokkalies (1998) proposed a relationship between SN_{40} and all descriptors of pavement surface texture as follows;

$$SN_{40} = \frac{9.4m_i - 38}{4.25\lambda^{0.1} e^{(0.14/\lambda^{0.72})}}, R^2 = 0.89 \quad (2.15)$$

or

$$SN_{40} = -12.3 + 1.365m_i + 12.7m_a, R^2 = 0.88 \quad (2.16)$$

where:

m_i , m_a and λ are the same as defined before.

Stroup-Gardiner and Brown (2000) studied the pavement macrotexture using ROSAN laser measurements. The pavement macrotexture was measured in terms of estimated texture depth (ETD). They proposed a model to predict the pavement macrotexture from surface course aggregate properties according to the following equation;

$$ETD = 0.0198 \times MS - 0.004984 \times P_{4.75} + 0.1038 \times C_c - 0.004861 \times C_u \quad (2.17)$$

where:

MS = maximum aggregate size (mm)

$P_{4.75}$ = percentage passing 4.75 mm sieve

C_C = coefficient of curvature = $(D_{30})^2/D_{10}D_{60}$

C_U = coefficient of uniformity = D_{60}/D_{10}

D_{10} = the sieve size associated with 10% passing (mm)

D_{30} = the sieve size associated with 30% passing (mm)

D_{60} = the sieve size associated with 60% passing (mm).

Davis et al. (2002) concluded that surface macrotexture could be estimated as a function of VMA and nominal maximum aggregate size (NMS). The following equation shows this relationship, which has an excellent R^2 value of 0.965;

$$EPD = -2.896 + 0.2993 \times NMS + 0.0698 \times VMA \quad (2.18)$$

where:

NMS = nominal maximum aggregate size (mm)

VMA = voids in the mineral aggregate (%).

2.10. Development of Rutting of Asphalt Pavements

Rutting of flexible pavements is defined as “*the permanent deformation noticed under the wheel paths*” (Sybilski, 1994). Permanent deformation results from the action of heavy traffic loads at a high pavement temperature. This in turn produces horizontal and lateral

displacements of binder and mineral particles in the pavement surface under the effect of shear stresses.

Rutting of flexible pavements may take place in all of the asphalt layers due to densification, shear deformation or both of them. To develop mix design methods that would effectively reduce rutting, it is important to understand and measure the contributions of the two mechanisms, densification and shear (Harvey and Popescu, 2000). Densification is the secondary compaction occurring in the surface layer of pavement due to traffic load applications. Rutting developed by densification contributes to a small portion of rut depth (Huber, 1999). It was found that poorly compacted mixes experienced the most densification. However, The dense graded mixes that had good compaction during construction stage did not suffer much densification even under heavy traffic loads and higher temperatures. These findings emphasized the importance of good compaction to reduce the amount of rutting caused by densification (Eisenmann and Hilmer, 1987; Harvey and Popescu, 2000).

If the densification happens vertically only, the shear flow causes the movement of material into the humps and away from the wheel-path. A larger thickness change can occur as a result of shear flow if densification takes place in the lateral directions as well. The shear deformation is the lateral deformation occurring in the pavement layers. Previous studies indicated that the shear deformation was the main contributor to rutting of asphalt pavements, especially if adequate stiff lower layers existed (Abd El Halim et al., 1998; Epps et al., 1998). Harvey and Popescu (2000) reported a similar conclusion. They concluded

that, at higher temperatures, higher tire pressures could increase the contribution of shear deformation to rutting compared to that of densification.

2.11. Factors Affecting Rutting of Asphalt Pavements

There are many factors affecting the rutting of asphalt pavements. These factors can be divided into two main categories: external factors and internal factors.

2.11.1. External Factors

These factors include traffic characteristics and environmental effects (Emery, 1990; Al-Abdul Wahhab and Balghonaim, 1994; Atkins, 1996; Archilla and Madanat, 2000). Heavy traffic loads and higher tire pressures increase rutting susceptibility. Environmental factors are also important to characterize rutting performance of asphalt pavements. Higher pavement temperatures cause traditional asphalt to be more plastic and consequently rutting occurs. Seasonal variation, freezing and thawing cycles, and excessive moisture may also cause rutting.

2.11.2. Internal Factors

These factors are related to asphalt concrete mix properties. Generally, rutting occurs due to low air voids (A_v) and voids in mineral aggregate (VMA), low binder viscosity, low aggregate angularity, and high asphalt content (AC) (Al-Dhalaan, 1990).

Air voids is one of the most important factors that affect rutting behaviour of asphalt mixes. For each load application, a tiny displacement and rearrangement of the aggregate particles occur. This displacement and rearrangement can be accommodated within the existing voids without causing any volumetric changes. If the percent of air voids of the

asphalt mix is low, the displacement will result in a volumetric change after the first load application and the asphalt layer will deform fast (Nikolaides, 2000; Archilla and Madanat, 2001).

As the asphalt binder viscosity increases, a reduction of resilient strain takes place and rutting decreases (Al-Abdul Wahhab, 1994). The Strategic Highway Research Program (SHRP) parameter ($G^*/\sin\delta$) is usually used to classify binders based on their rutting susceptibility. This parameter can be determined by the dynamic shear rheometer (DSR) test. Generally, binders with high $G^*/\sin\delta$ ratio should be used to get asphalt mixes with low rutting susceptibilities for a certain nominal maximum aggregate size. Styrelf and Novophalt are examples for binders of high $G^*/\sin\delta$ (Stuart and Mogawer, 1997).

It is important to use a good aggregate gradation in asphalt mixes to obtain higher rutting resistance. In addition, NMS was found to have a considerable effect on the rutting resistance of asphalt mixes. Rutting was significantly decreased when NMS increased. It is important to mention that an increase in NMS causes the optimum asphalt content to decrease (Stuart and Mogawer, 1997).

As the filler content increases, the asphalt film thickness decreases and a consequent drop on durability occurs. This effect is not noticed obviously for asphalt mixes of coarser gradation such as base courses. It was found that the increase in filler content caused a decrease in the rutting resistance due to the reduced particle-to-particle contact (Al-Abdul Wahhab, 1994).

Harvey et al. (2000) found that the resistance to shear deformation increased considerably with increasing compaction. In other words, proper compaction during construction could enhance the rutting resistance of asphalt pavements.

2.12. Role of Aggregate and Binder in Rutting Resistance

Many researchers are concerned with problems related to rutting in asphalt pavements. Resistance to permanent deformation of asphalt mixes can be caused by the binder stiffness and the structure of the asphalt mix such as distribution and shape of the mineral particles, and bituminous binder content.

Brown and Cooper (1984) found that there was a significant effect of the binder on the measured rut depths. They also emphasized the importance of good aggregate gradation to provide long-term rutting resistance. Recently, an experimental study was conducted to study the role of aggregate and binder in offering rutting resistance. In this study, wheel-tracking test was used. Test results showed better performances, according to average rut depths and transverse surface profiles, for sections containing the modified binder (Brown and Gibb, 1996).

Generally, for conventional dense mixes, the role of aggregate in resisting rutting becomes important when the binder stiffness drops below about 100 kPa, while at the highest stiffness (8,000 kPa), the state is reversed (Brown and Gibb, 1996). For cases where very low speeds are expected and at the highest temperature, good aggregate-based rutting resistance is needed (Brown and Gibb, 1996).

Harvey et al. (2000) concluded similar findings. They found that the effect of binder ageing had a significant effect on permanent shear resistance at lower temperature because the binder stiffness increased considerably at such temperatures. While at higher temperatures, the aggregate structure had the main role in shear resistance to permanent deformation.

2.13. Tests Evaluating Rutting Susceptibility

Rutting susceptibility of asphalt concrete mixes are usually evaluated in the laboratories. There are three main categories of laboratory tests to measure rutting susceptibility; the wheel track tests, the uniaxial and triaxial tests and the diametrical tests (SHRP, 1991). However, many researchers tried to characterize rutting performance utilizing easier tests. Some of the previous work in this area is presented in section (2.13.4).

2.13.1. Wheel Track Tests

Wheel track testers (WTT) characterize rutting performance of asphalt mixes by applying a repeated load to a test specimen until it fails. These testers do not measure the mechanical properties of asphalt mixes under evaluation. WTT relate the rutting performance to the number of load repetitions before failure. Generally, the main component of any WTT is the rubber wheel that applies a repeated load to the test specimen placed on a moving table. Rut depth is usually measured utilizing a displacement-measuring device (Oliver and Tredrea, 1998). *The French pavement rutting tester, Georgia loaded-wheel tester and Hamburg wheel-tracking device* are examples for WTT (Stuart and Mogawer, 1997). However, these types of testers are not available in most asphalt material laboratories. In addition, it is

difficult to conduct a wheel track test for ordinary mix design. Oliver and Tredrea (1998) found that the SHRP parameter ($G^*/\sin\delta$) had a good correlation with WTT results for unmodified binders rather than polymer modified binders.

2.13.2. Uniaxial and Triaxial Tests

Uniaxial and triaxial tests are performed on cylindrical asphalt concrete samples. In general, this test category gives a good indication of the rutting susceptibility of asphalt concrete mixes. Many researchers have utilized this category of rutting susceptibility tests. Brown and Gibb (1996) conducted a *repeated load axial (RLA) test*. They found that RLA test could rank asphalt mixes correctly according to their rutting susceptibility. Deshpande and Cebon (2000) conducted a *simple compression test* to study the rutting performance of asphalt mixes. Stuart and Mogawer (1997) found that RLA test and WTT could rank different asphalt concrete mixes similarly.

Previous studies concluded that the application of confinement has a significant positive influence on permanent deformation resistance especially for continuously graded materials, which depend on aggregate interlock and friction (Monismith and Tayebali, 1988; Brown, 1995). Based on this finding, Brown and Gibb (1996) modified the RLA test by applying partial internal vacuum to provide confining stress. The modified test facility was known as *confined repeated load axial (CRLA) test*. Brown and Scholz (1998) conducted a series of RLA and CRLA tests using different bitumen types. Test results indicated that a relatively low confining pressure (20 kPa) appeared to be sufficient to significantly improve permanent deformation resistance. The polymer modified bitumen caused a significant reduction in deformation for the unconfined tests but it had no effect when applying

confining stress. They suggested that the confining stresses mobilized resistance to permanent deformation from the aggregate and reduced the contribution from the bitumen.

To replicate field conditions accurately, the *triaxial test* is usually used to provide confined tests on axially loaded specimens. Because the test is expensive, complex and time consuming, it has been limited to research studies (Brown and Cooper, 1984).

2.13.3. Diametrical Test

This test is commonly known as the indirect tensile strength (ITS) test. The test is performed by loading a cylindrical specimen with compressive load acting along the vertical diametral plane. More details about the test are presented in Chapter (4). The test is used successfully to evaluate the resilient modulus and ITS. However, the correlation between these properties and rutting performance in the field is not available at the time and the test is considered unsuitable for permanent deformation characterization (SHRP,1991)

2.13.4. Other Tests

Al-Abdul Wahhab (1994) tried to rank asphalt mixtures using other relatively easier tests, including Marshall, creep and Hveem tests. Then he compared the results of these elementary tests to that obtained from the wheel-tracking test to determine the test that gave the best ranking of asphalt mixes similar to WTT. Test results showed that, the *Hveem stability test* gave the best ranking of asphalt mixes according to their rutting susceptibility for different aggregate gradations. This could be due to the sensitivity of Hveem stability to the angle of internal friction, which was influenced by gradation. However, for mixes that have similar aggregate gradations, the *resilient modulus test* gave best ranking of mixes. In

this case, Hveem stability did not vary appreciably because the resilient modulus is more sensitive to variation of asphalt binder viscosity.

Sybilski (1994) studied the rheological properties of the binder to correlate them with the rutting resistance of asphalt mixes, in terms of number of load passes to rut depth of 10 mm (N_{10}) using the WTT facility. He carried out many tests to measure the rheological properties of conventional road bitumen, and polymer bitumen, containing (terpolymer ethylene-propylenediene EPDM, copolymer styrene-butadiene-styrene SBS and polyolefinic plastomer PO). These tests were: needle penetration at 25°C, 15°C, 10°C, and 5°C, ring and ball (R&B) softening point, and dynamic viscosity at 60°C, 90°C, and 135°C. All tests were carried out before and after the bitumen binders were aged utilizing the thin film oven test (TFOT). Test results showed that the rutting resistance of bituminous mixtures was proportional to the absolute viscosity of the bitumen binder. Multiple regression analysis was performed between N_{10} as the dependent variable and three bituminous binder properties chosen as independent variables. The best fit was achieved when N_{10} was correlated with penetration index, $\ln$ (viscosity at 60°C) and penetration at 25°C.

2.14. Enhancing Rutting Resistance

2.14.1. General

If the subgrade and granular layers are compacted well and have proper strength, rutting is caused by the permanent deformation of the asphalt courses, especially the upper layers. This deformation of the asphalt concrete layers is due to the visco-elastic behaviour of the bituminous binder, the low shear strength of the asphalt mixture or a combination of both. If

the asphalt mixture has adequate shear strength, rutting will happen after a long time. On the other hand, if the shear strength of asphalt mixture is low, rutting will take place within a short period of time after construction (Nikolaides, 2000). In general, increasing the shear strength of the mixture and/or using modified binders can enhance the rutting resistance of asphalt mixes. The shear strength can be increased by choosing an aggregate gradation that causes particle-to-particle contact or by using a stiffer binder (Nikolaides, 2000).

2.14.2. Shear Properties of Asphalt Mixes

Results of research performed as a part of the SHRP project led to the conclusion that shear properties of asphalt concrete are the most important factor in characterizing rutting performance (Weissman et al., 1998). Shear properties of asphalt mixes, in terms of shear strength and shear modulus, were usually evaluated using the torque machine (Higdon et al., 1985). To simulate the field condition, the triaxial test facility was found to be the best test to evaluate shear properties accurately (Weissman et al., 1998).

Researchers at Carleton University have developed a new test facility to measure shear properties of asphalt pavements in the field. The work was started in the early of 1990s (Abd El Nabi, 1995; Abd El Halim et al., 1995 a; 1997; 1998) to develop a new in-situ shear tester known as Carleton In-Situ Shear Strength Test (CISSST) facility, as shown in Figure (2.16). The principal idea of the design of this machine was to apply direct torque to the pavement surface. An electric motor was attached to a gear reducer and a gearbox for load application. The gearbox was responsible to transmit the torque through a vertical shaft to a pre-glued circular steel plate on the pavement surface. Torque cell was attached to the vertical shaft to measure the torque at failure. Angle of twist was measured using a

Figure 2.16: The Carleton In-Situ Shear Strength Tester (CISSST)

protractor. The shear strength of the mix under study was calculated by resolving the torque at failure across the resulting failure surface. The preliminary test results showed that the CISSST facility yielded higher SS values compared to the laboratory evaluated SS using the torque machine. Figure (2.17) shows the loading method and the typical failure pattern imposed by CISSST.

Abdel Nabi et al. (1995) conducted a study using the CISSST facility to investigate the effect of road geometry on the behaviour of asphalt concrete. They concluded that the shear strength decreases significantly in horizontal curves compared to shear strength evaluated in straight sections. Similar conclusion was reported by El-Desouky (1998). He concluded that asphalt pavements fail earlier in horizontal section than in straight sections

Figure 2.17: Load Application and Typical Failure Pattern of CISSST

(Source: Goodman et al., 2002)

due to the effect of horizontal forces developed between vehicle tires and the pavement surface at horizontal sections.

The CISSST facility was modified for easier mobility and more accurate test results (Goodman 2000) and (Abd El Halim a and b, 2001). The new machine is known as In-Situ Shear Stiffness Test (InSiSST) facility. Figure (2.18) shows the new facility, which consists of a cart-like chassis on small wheels. The loading system is similar to that used in CISSST machine. The measurements of torque and angle of twist were collected and controlled by a laptop computer attached to the tester. The motor and gearbox combination was mounted vertically on a platform to allow movement in both longitudinal and transverse directions. The entire test frame was attached to the main chassis such that the loading system could move up and down through four jacks. The mathematical models used to calculate shear strength and shear stiffness are beyond the scope of this research. These models are valid

Figure 2.18: InSiSST Adopted and Developed at Carleton University

only for elastic, homogenous and isotropic materials. A detailed overview of the development of the mathematical models may be found in (Reissner and Sagoci, 1944). Further development and analytical investigation in InSiSST mathematical models are underway to account for the actual behaviour of asphalt concrete as a heterogeneous material.

Bekheet et al (2001) used the finite element method (FEM) to simulate the boundary conditions, geometry and applied load for tests performed by InSiSST facility. The developed finite element model yielded very close results compared to both the mathematical models and field results evaluated using InSiSST facility. However, the researchers at Carleton University suggested that different environmental condition might give different results.

Bekheet (2002) continued this work to model the asphalt concrete as a linear visco-elastic material using the FEM. He concluded that shear stiffness of asphalt mixes was significantly affected by the visco-elastic properties. In other words, the torque at failure was found to decrease, as viscous properties became more dominant. The results of this study suggested that the capacity of InSiSST facility provided limited loading rates that caused the failure to occur in the zone where viscous properties of the mix affecting the evaluated shear stiffness.

2.14.3. Effect of Bitumen Type on Rutting Resistance

At low temperature and high frequencies (short loading time), bitumen behaves as an elastic solid. While, at high temperature and low frequency, it acts as a viscous fluid and results in an irrecoverable strain. Between the two extremes, bitumen's behaviour is visco-elastic. The material recovers but there is a small permanent deformation. Modified polymers are used to enhance this visco-elastic performance. Some polymers are used to increase the stiffness, so the total visco-elastic reaction is reduced. Others increase the elastic component, hence reducing the viscous element and the irrecoverable strain (Atkins, 1996).

Many researchers concluded that using the polymer-modified bitumen is the best solution to improve rutting resistance of flexible pavements (Al-Abdul Wahhab, 1994; Abd El Halim et al., 1995 b; Brown and Gibb, 1996; Atkins, 1996; Stuart and Mogawer, 1997; Oliver and Tredrea, 1998; Deshpande and Cebon; 2000). Generally, polymer modified bitumen could prevent severe rutting for up to seven years (Atkins, 1996). If the aggregates have poor quality, the rutting resistance can be increased using a suitable amount of stiff binder. For high-quality aggregates, e.g., crushed rock, the binder role is insignificant.

However, higher binder content of the stiffer binder causes some reduction in strain. Higher stiffer binder content causes the accumulated strain to decrease and shows the helpful contribution of the bitumen (Brown and Gibb, 1996). The following are examples for polymer-modified bitumen;

- Novophalt: *“the Novophalt binder was formulated by blending the Lagoven AC-10 asphalt with 6.5 % low-density polyethylene by mass”* (Al-Abdul Wahhab, 1994; Stuart and Mogawer, 1997).
- Styrelf 1-D: *“the Styrelf 1-D binder was formulated by reacting the Lagoven AC-20 asphalt with 4 % styrene-butadiene by volume”* (Stuart and Mogawer, 1997).
- Cariphalt DM: *“Cariphalt is polymer-modified bitumen which offers greater elasticity than conventional asphalt, through a rubbery mid-section sandwiched between rigid end blocks of polystyrene, making the surface less susceptible to rutting and to reflective cracking when laid over concrete”* (Atkins, 1996).
- Polybelt is a chemical addition that can decrease the rutting susceptibility of asphalt mixes. This can be achieved as the result of the increase in the asphalt cement stiffness and the decreased temperature susceptibility (Al-Abdul Wahhab, 1994).
- EVA: *“EVA is an ethylene vinyl acetate copolymer binder”*. It can experience a phase modification dependent on the thermal history of the material. Asphalt mixes prepared with an EVA binder are easier to be compacted than those made of materials, which do not undertake a phase change. However, once the binder temperature decreases to about 40°C and a structure is developed, it will be more

resistant to rutting even at the maximum anticipated service temperature of 70°C (Oliver and Tredrea, 1998).

2.14.4. Surface Treatments to Enhance Rutting Resistance

Stone mastic asphalt (SMA) is an example of asphalt mixtures that have high rutting resistance. It was developed in Germany in the mid-1960s. Because of its high-quality performance, durability, and surface characteristics, many countries now use SMA mixes. SMA is a gap-graded asphalt mix that is considered as a good solution for use in areas of heavy traffic (Nikolaides, 2000). It has comparatively high voids partially filled with binder-rich mastic mortar. The shape, number of crushed faces and the surface texture of the aggregate are essential to obtain a stable aggregate structure and high shear strength. It is important also to use binder with relatively high viscosity at in-service temperatures to reduce deformation (Nikolaides, 2000). On the other hand, it was concluded that, the use of crushed rock fines as a substitute of natural sand decreases the workability of the mixture and increases the rutting resistance. Thus, crushed rock fines are suggested to be used for all heavy-duty pavements. The improper compaction of the underlying layers may yield considerable increase in rut formation during the hot summer months (Nikolaides, 2000).

Other examples for asphalt mixes that have a good rutting resistance are the Dense-Graded Asphalt Concrete (DGAC) and the Asphalt-Rubber Hot Mix Gap-Graded (ARHM-GG). They were used effectively as overlays to control rutting and cracking for several years (Harvey and Popescu, 2000; Shatnawi, 2001).

2.15. Estimation of Rutting and Shear Properties

Abd El Halim et al. (1995 b) conducted a study to investigate rutting resistance for different asphalt mixes. Test results indicated that there was so-called critical asphalt content at which the rut depth was minimum. Figure (2.19) shows this behaviour for three different binder types. The figure illustrates the effect of binder type on rut depth where using polymer binder reduces the rut depth significantly. Figure (2.20) shows the effect of asphalt type on shear strength of asphalt mixes, where polymer binder yields higher SS for all AC greater than 5 %. In addition, as AC increases, the SS increases until it reaches its maximum value. Beyond this value, any increase in AC results in lower SS.

Goodman et al. (2002) proposed two models to predict shear stiffness and shear strength from mix properties. They used the test results and mix properties from previous research (Zahw, 1995) conducted at Carleton University to develop the two following models:

$$G = 560 - 2.7 \times PVR + 232 \times C_U + 344 \times C_{bin} - 14135 \times ARD + 10680 \times VMA \quad (2.19)$$

$$\tau = -66 - 0.9 \times PVR + 50 \times C_U + 65 \times C_{bin} - 2410 \times ARD + 2552 \times VMA \quad (2.20)$$

where:

G = shear stiffness, (kPa)

τ = shear strength, (kPa)

PVR = ratio of penetration (mm) to viscosity at 60°C (Pa.s), (mm/Pa.s)

Figure 2.19: Effect of Asphalt Cement Type on the Rut Depth of Asphalt Concrete Mixes

(Source: Abd El Halim et al., 1995 b)

Figure 2.20: Effect of Asphalt Cement Type on the Shear Strength of Asphalt Concrete Mixes

(Source: Abd El Halim et al., 1995 b)

C_U = coefficient of uniformity (D_{60}/D_{10})

C_{bin} = presence of crushed coarse aggregate in mix (1 for yes and 0 for no)

ARD = average rate of densification (ratio of mix density to square root of the number of blows with Marshall compactor)

VMA = voids in mineral aggregate, (%)

The R^2 values were 0.83 and 0.88 for Equation (2.19) and Equation (2.20), respectively. In addition, they related the shear properties of the mix to its resistance to permanent deformation in terms of rut depth. A series creep test was performed at three stress levels of 0.1 MPa, 0.2 MPa, and 0.3 MPa. The following equations present the rut depth as a function of shear stiffness and shear strength for the different stress levels, respectively:

$$RD = 245 \times G^{-0.742} \quad (2.21 \text{ a})$$

$$RD = 18.25 \times \tau^{-0.515} \quad (2.21 \text{ b})$$

$$RD = 4301 \times G^{-1.037} \quad (2.22 \text{ a})$$

$$RD = 62.92 \times \tau^{-0.626} \quad (2.22 \text{ b})$$

$$RD = 3E6 \times G^{-1.820} \quad (2.23 \text{ a})$$

$$RD = 11797 \times \tau^{-1.383} \quad (2.23 \text{ b})$$

where:

RD = rut depth (mm)

G, τ are the same parameters as in Equations (2.19) and (2.20).

The proposed models suggested that a large amount of variability observed in rut depth could be explained by the shear properties. The shear stiffness values evaluated by

InSiSST facility were more highly correlated with the permanent deformation observed in the field than the laboratory results using the torque machine (Goodman et al., 2002).

2.16. Summary

A comprehensive literature review on skid resistance and rutting of asphalt pavements was presented in this chapter. They represent the most important factors contributing to surface-related safety. Based on the literature review, several of conclusions follow;

Skid resistance of pavement surface is developed from the macrotexture and microtexture of pavement surface. Coarse aggregates play an important role to enhance the skid resistance for the asphalt mixes. Microtexture is a function of aggregate particle mineralogy, weather, traffic and pavement age. The value of British Pendulum Number gives a good approximation of pavement microtexture. The initial macrotexture is determined by shape, size and gradation of coarse aggregates in addition to the construction technique used in the surface layer placement. Sand patch method is the basic method to measure pavement macrotexture. On the other hand, maintaining field air-void content above certain value is also important to control skid resistance of asphalt surfaces. However, excessive asphalt on the surface results in poor frictional performance.

Rutting takes place due to low aggregate angularity, high asphalt content, relatively low air voids at time of construction, low percentage of voids in mineral aggregate, low binder viscosity and inappropriate compaction. Inadequate compaction of the underlying layers yields considerable increase in rut formation during the hot summer months. Shear properties of asphalt concrete are the most important factor in determining rutting

performance. Rutting will take place after a long time if the asphalt mix has adequate shear strength. Choosing an aggregate gradation that develops particle-to-particle contact and has a high degree of internal friction improves the shear strength of the mix and as a result increases its rutting resistance. To simulate field conditions correctly, triaxial cell is usually used to estimate shear properties of asphalt mixes.

In general, the review shows that there is a safety hazard due to lack of skid resistance and rutting resistance of asphalt pavement. Skid resistance and traffic collisions are highly correlated. The loss of surface friction could provide a serious safety problem. At the same time, rutting of asphalt surface is known as one of the common distress types. Rutting can cause loss of surface friction especially when water exists. The review showed also that there is lack in predictive models dealing with frictional properties of asphalt mixes.

CHAPTER 3

3. RESEARCH PLAN

3.1. General

A research plan was designed to achieve the objectives of this research and it was divided into a number of phases. The first phase dealt with literature review. It presented a comprehensive background on the factors governing safety of pavement surfaces. Based on the results of this review, the main parameters to be included in this thesis were selected and the second phase was formulated. It included the design of experimental investigation to quantify the effect of each parameter to the characteristics of airfield mixes. Laboratory evaluation of frictional and mechanical properties of selected airfield mixes was performed. In this phase, certain factors were changed and the effects of these changes on the surface friction as well as the mechanical properties of the mixes were evaluated. In addition, friction measurements on actual airfield surfaces were performed in the field. The third phase included the analytical investigation where significance of test results was checked, the contribution of each parameter was quantified and guidelines to improve surface safety were provided. This phase included the data analysis to develop several prediction models. Based on the analytical investigation, the last phase was completed by providing the conclusions and recommendations.

The results of this thesis provide sufficient data and information to design airfield mixes with proper frictional and mechanical properties. Figure (3.1) shows the flowchart for the research plan.

3.2. Summary of the Literature Review

The main objectives for the literature review were:

- To study the impact of skid resistance on of highway/runway safety.
- To identify the variables and factors affecting the skid resistance of pavement surfaces.
- To select the appropriate test methods and test facilities to evaluate frictional properties of asphalt surfaces both in the laboratory and in the field.
- To identify the factors affecting rutting of asphalt mixes and study the influence of mechanical properties on rutting susceptibility.
- To select the parameters to be included in the experimental investigation.

3.2.1. Skid Resistance of Asphalt Concrete Pavements

This part of the literature review, as described in Chapter (2), presented the state-of-the-art of the previous research work carried out in the field of skid resistance and frictional properties of asphalt concrete pavements. In general, this work included skid resistance development, factors affecting skid resistance, change of skid resistance, and measurements of skid resistance using different test methods. Skid resistance-traffic accident relationship and surface treatments to enhance the skid resistance were also discussed.

Figure 3.1: Research Plan

3.2.2. Rutting of Asphalt Surfaces

This part covered the reasons and causes of rutting as one of the most important indicators to assess pavement safety. Then, it reviewed the factors affecting rutting and a comprehensive discussion on role of aggregate and binder in developing the rutting resistance of asphalt mixes. Rutting evaluation and previous work carried out to control rutting were also included in this part of the literature review. In addition, surface treatments to enhance rutting resistance and models to estimate rutting and shear properties were introduced at the end of this background.

3.2.3. Main Findings of the Literature Review

Based on the literature review, the following are considered to formulate the experimental investigation;

- Frictional properties of asphalt mixes are evaluated using the BPT and SPT.
- Shear properties of asphalt concrete are the most important factors in determining rutting performance. Shear test by the torque machine, triaxial test, and direct shear test are chosen to evaluate shear properties for asphalt mixes.
- Testing program is formulated to study the effects of the following parameters on the frictional properties of asphalt mixes: aggregate gradation, aggregate type, asphalt content, percent of air voids, ageing of asphalt mixes, weather, traffic, and compaction technique.
- Periodic field and laboratory measurements are required to take the effects of ageing of asphalt mixes, weather, and traffic into consideration.

3.3. Experimental Investigation

In this phase of the thesis development, a brief prescription of test methods was introduced. Based on the literature review, the variables to be included in this study were selected, tests to evaluate the properties of airfield HMAC were identified and the testing program was formulated. Experimental investigation included laboratory investigation and field measurements. To achieve the objectives of this thesis, six different mixes for airfield pavements were prepared in the laboratory using Marshall mix design method. In addition, three different asphalt mixes were obtained from the field during the construction of two taxiways at Ottawa International Airport (OIA). The experimental investigation included the evaluation of frictional properties, shear properties and other mechanical properties of the selected airfield mixes. Frictional properties were characterized periodically in the laboratory for two field mixes. At the same time, field measurements included the evaluation of friction of two actual taxiway surfaces at OIA.

3.4. Analytical Investigation

This phase of study included two main parts. The first one was the statistical analysis performed to check the significance of test results. Based on this analysis, the contribution of each parameter affecting the airfield mix properties was quantified and guidelines to improve airfield pavement mixes in terms of safety and performance were concluded. The second part was to develop several prediction models dealing with the frictional properties and shear properties of airfield mixes. The SPSS software was used to calibrate the linear

regression models. The models concerning the frictional properties of airfield mixes were provided for two levels of analysis.

At the microscopic analysis level, surface friction was related to the main mix properties. The developed model provides a good tool for airport owners and operators to pre-estimate the frictional properties of HMAC airfield surfaces prior to construction. In other words, the developed model can be used by pavement engineers to pre-check the frictional properties and, if required, make any modifications to obtain the desired friction level.

At macroscopic level of analysis field friction was related to the friction evaluated in the laboratory. The proposed model can be used to provide a simple tool to evaluate surface friction without any site visits especially during the freezing conditions of the winter season. The predicted airfield surface friction can be used for maintenance planning and pilot advisories, in the emergency cases.

Finally, the models dealing with shear properties were developed by relating shear properties evaluated using three different test techniques. Figure (3.2) shows the criterion of this stage of analytical investigation. The figure shows three test facilities; the torque-testing machine, triaxial test facility, which is used to simulate field conditions accurately, and direct shear test facility. The developed models can be used to evaluate shear properties with confidence utilizing any available shear test facility. Furthermore, the models can be used in the future to correlate the laboratory test results with in-situ values.

Figure 3.2: Criterion for Accurate Shear Properties Evaluation

CHAPTER 4

4. EXPERIMENTAL PROGRAM

4.1. Introduction

This chapter presents a description of test methods performed to achieve the objectives of this research. Preparation of test specimens preparation and experimental program are also introduced in this chapter. The experimental program includes the determination of bulk relative density, maximum relative density, air voids and voids in mineral aggregate, frictional properties, shear properties, indirect tensile strength, and moisture sensitivity. Description of test procedures is presented in section (4.2). Specimen preparation and testing program are discussed in section (4.3). In addition, section (4.4) presents a discussion on the basic characteristics of surface friction.

4.2. Description of Test Procedures

4.2.1. Determination of Physical Properties

The bulk relative density (BRD) was determined, for all core specimens before performing any test, according to ASTM D2726-90 (1996). The BRD was required to assure that all samples were compacted to the same level. In addition, loose asphalt mixes were tested for their maximum relative density (MRD) according to ASTM D2041-95 (1996). BRD and MRD were essential to determine the mix volumetric properties in terms of air voids (Av) and voids in mineral aggregate (VMA).

Percent of Av in a compacted asphalt mix was calculated as (ASTM D3203-94, 1996);

$$Av = 100 \times \left(1 - \frac{BRD}{MRD}\right) \quad (4.1)$$

where:

Av = percent of air voids (%).

BRD = bulk relative density (gm/cm³).

MRD = maximum relative density (gm/cm³).

While VMA could be calculated as follows;

$$VMA = 100 - \left[\frac{BRD \times (100 - AC)}{G_{Agg}} \right] \quad (4.2)$$

where:

VMA = percent voids in mineral aggregate (%).

AC = asphalt content (%).

G_{Agg} = average specific gravity of aggregate (gm/cm³).

The average specific gravity of aggregate blend was calculated as (ASTM C127, 1996);

$$G_{Agg} = \frac{1}{\frac{P_1}{100G_1} + \frac{P_2}{100G_2} + \dots + \frac{P_n}{100G_n}} \quad (4.3)$$

where:

G₁, G₂...G_n = specific gravity values for each aggregate size fraction.

P₁, P₂...P_n = weight percentages of each aggregate size fraction presents in the aggregate blend.

4.2.2. Friction Test using the British Pendulum Tester

The British pendulum tester (BPT) is a dynamic pendulum tester used to measure the energy loss when a rubber slider shoe strikes the asphalt surface under study. The measured values are recorded in terms of BPN, which represents the frictional properties of pavement surfaces. Figure (4.1) shows the different components of the BPT. The apparatus consists of the pendulum with slider and slider mount of 1500 ± 30 gm. The spring and lever arrangement give a normal slider weight of 2500 ± 100 gm. The slider assembly consists of an aluminium plate connected to a $6 \times 25 \times 76$ mm rubber strip for testing flat surfaces. The distance between centre of gravity of the pendulum and centre of oscillation is 411 ± 5 mm. The tester can be adjusted vertically to give a slider contact path of 125 ± 1.6 mm for tests on flat surface and 77 ± 1 mm for tests on polishing-wheel specimens. The required contact path length is measured using the marked ruler provided with the tester. The minimum dimensions of laboratory test samples are 89×152 mm.

The BPT was used in this thesis to measure the frictional properties of the asphalt mixes according to ASTM (E303-93, 1996). Instrument was levelled using the levelling screws until the bubble was centred in the spirit level. After levelling, the pendulum was adjusted such that its free swing carried pointer to zero as shown in Figure (4.2). Zero-adjustment was performed by raising the pendulum mechanism to allow slider to swing free of test surface. Locking knob was then tightened and the pendulum was placed in release position. Pendulum was released and pointer reading was noted. Adjustment procedures were repeated until the pendulum free swing carried pointer to zero.

Figure 4.1: British Pendulum Tester

Figure 4.2: Preparation of British Pendulum Tester, Zero Adjustment

With pendulum hanging free, pendulum was lowered such that the edge of slider just touched the pavement surface. Contact path length was adjusted using a marked ruler provided with the pendulum as shown in Figure (4.3). Enough water was applied to cover the test area completely. One swing was executed but the reading was not recorded. After that four more swings were carried out with the surface rewetted each time and results were recorded.

4.2.3. Sand Patch Test

The sand patch test (SPT) was used in this research to measure the macrotexture of airfield pavement surfaces. The elements of sand patch kit and test procedures were discussed in section (2.5.4.1). Figure (4.4) shows the different components of the sand patch kit.

4.2.4. Shear Test using the Torque Machine

The torque machine has two jaws to fasten the plates to which the specimen is glued. Figure (4.5) shows the torque machine and a fastened sample. A scale to measure the angle of twist and a dial gauge to record the maximum torque are attached to the machine. The dial gauge capacity used in shear test is 1100 N.m with a sensitivity of 2 N.m.

The shear test using the torque machine was performed to determine the shear strength (SS) of laboratory compacted specimens. First the specimens were glued to the steel plates and left in room temperature for 24 hours to dry. Specimens were fastened to the machine jaws using the arms of the steel plates. The dial gauge was set to zero and the torque was applied at constant rate of 18 degrees twist angle per minute until the failure of the specimen occurred. The shear strength was calculated according to (Bowes et al. 1990);

Figure 4.3: Adjustment of Contact Path Length

Figure 4.4: Sand Patch Test Apparatus

Figure 4.5: Torque Testing Machine

$$\tau_{\max} = \frac{TD}{2J} \quad (4.4)$$

and

$$J = \frac{\pi D^4}{32} \quad (4.5)$$

where:

$\tau_{\max}$ = maximum shear stress(N/m²).

T = torque moment (N.m).

J = polar moment of inertia (m⁴).

D = diameter (m).

4.2.5. Triaxial Test

The triaxial compression test is the most commonly used shear strength test for soils (Whitlow, 1990). The test can be used to simulate field conditions accurately for asphalt mixes. Triaxial test was used in this thesis to estimate the shear strength parameters for different airfield mixes. Figure (4.6) shows the different components of the triaxial test facility used in this research. The apparatus consists of a 4500 kN rock mechanics test system. The system includes load unit control pod and TestStar II digital controller, cascade temperature controller, temperature display, 4500 kN actuator/load frame, and a triaxial chamber. The triaxial chamber has a 2500 kN load cell, three linear variable differential transducers (LVDT), and three circumferential extensometers for load and strain measurements. The system is fully computerized and strain/displacement measurements can be monitored via two strain indicators attached to the system (NRC, 2001).

The test was carried out by applying three different confining pressures to the test specimens and increasing the normal load up to failure. Then Mohor circle for each peak stress was plotted. A common tangent to the failure circles was then drawn to represent the shear envelope, from which the coefficient of cohesion (C) and angle of internal friction (ϕ) could be determined (Whitlow, 1990).

The sides of test specimen were encapsulated using heat shrink polyolefin tubing. Then, the specimen was sealed by wire wrapping around the polyolefin. Test specimen was placed and centred on the test system load cell, and a circumferential extensometer was installed around it at mid height. All LVDT signals were set to zero and the triaxial chamber

Figure 4.6: Triaxial Test Facility

was closed. Confining pressure was applied by filling the triaxial chamber with a confining fluid. The vertical load was applied until failure of the specimen occurred (NRC, 2001).

4.2.6. Direct Shear Test

The direct shear test facility was used in this study to characterize shear properties of airfield asphalt mixes. The apparatus consists of shear testing machine that has the capability to apply vertical and shear loads on the test specimens. PC computer and data acquisition system are attached to the machine for data control and collection. Sample is locked in its position using a vice before being tested. Load and displacement are measured and recorded by means of load cell and two LVDT. Figure (4.7) shows the different components of the direct shear test facility used in this study (NRC, 2001).

The test was carried out by applying a normal load to the specimen and shearing it at a constant rate until failure occurred. Test procedure was repeated for three specimens. The

Figure 4.7: Direct Shear Test Facility

values of normal stresses were plotted versus shear stress values. The line that best fitted the plotted stresses was the shear envelope from which C and ϕ could be calculated (Whitlow, 1990). First, the sample was weighed and placed in the top plate mould. A hydrostone grout was prepared by adding water to the hydrostone with ratio of 1:4. The grout was poured into the top plate mould and left for one hour to be dried. Then the top plate mould was lowered into position on the direct shear-testing machine. Grout was then poured into the bottom plate mould to immerse the lower half of the test specimen. The sample was left for an hour to allow grout to dry, and then it was tested for its shear properties.

4.2.7. Indirect Tensile Strength Test

Instron Model TT Floor Standing Universal Testing Machine was utilized to perform indirect tensile strength (ITS) test. The machine consists of a screw drive system with speed

range of 0.051 to 50.8 mm/min. The system provides a maximum load capacity of 89.2 kN. The load is applied through two vertical screw posts. A strain gauge load cell is attached to the system to measure applied loads. Displacement is measured by means of a rotary transducer attached to the load screw. Control and data collection is performed by a data acquisition system custom-configured for the Instron Model TT (Instron, 1967). The test facility is shown in Figure (4.8).

Indirect tensile strength (ITS) test was performed to investigate the mechanical properties of all mixes. To perform the test, specimens were loaded with compressive load acting along the vertical diametral plane. A steel-loading strip was used to apply the load uniformly. The loading rate was 50.8 mm/minute. For each specimen, the ITS was calculated according to the following equation (ASTM D4123-82, 1996)

$$ITS = \frac{2P_{ult}}{\pi h D} \quad (4.6)$$

where:

ITS = indirect tensile strength (MPa).

P_{ult} = ultimate applied load at specimen failure (N).

h = height of specimen (mm).

D = diameter of specimen (mm).

4.2.8. Moisture Sensitivity Test

Moisture sensitivity test was performed to evaluate the resistance of asphalt mixes to stripping, i.e., to determine the effect of water on the tensile strength of the paving material. Testing procedures followed ASTM D4867-92 (1996). Samples to be tested were divided

Figure 4.8: Instron Model TT Testing Machine

into two groups such that the mean BRD of each group was approximately the same. First group of samples was placed in a $25^{\circ}\text{C} \pm 0.3^{\circ}\text{C}$ water bath for one hour to adjust its temperature. Then, the group was removed quickly and tested for indirect tensile strength. The other group was placed in a vacuum cell containing water at $25^{\circ}\text{C} \pm 1^{\circ}\text{C}$ and pressure was reduced to 30 ± 1 mm Hg for a period of one hour. The cell was agitated periodically to release the air. Then, samples were removed and immersed in a water bath at temperature of $60^{\circ}\text{C} \pm 1^{\circ}\text{C}$. After 24 hours, samples were immersed in the $25^{\circ}\text{C} \pm 0.3^{\circ}\text{C}$ water bath to adjust their temperature for one hour then, they were tested for their ITS. Test procedures, equipment, and equation for calculating ITS were the same as described in section (4.2.7).

For each mix, tensile strength ratio (TSR) was calculated according to ASTM D4867-92 (1996) as follows:

$$TSR = \left(\frac{ITS_m}{ITS_d} \right) \times 100 \quad (4.7)$$

where:

TSR = tensile strength ratio (%).

ITS_m = tensile strength of moisture condition set (MPa).

ITS_d = tensile strength of the dry set (MPa).

4.3. Experimental Program

4.3.1. General

The experimental program was formulated to test nine different airfield asphalt mixes, six from laboratory and three from the field, for their frictional properties, shear properties and other mechanical properties. The laboratory mixes were prepared based on Marshall mix design method. These mixes were referred to as Mix1 to Mix6. In this phase of study, the effects of aggregate gradation, AC, Av, and VMA on the properties of asphalt mixes were studied. The field mixes were obtained during the construction of two taxiways at Ottawa International Airport (OIA). These Mixes were referred to as Mix7, Mix8, and Mix9. Mix7 and Mix8 were two different surface courses for Taxiway “F” and Taxiway “B” at OIA. However, Mix9 was the base course for Taxiway “B”. The difference between Mix7 and Mix8 was that the limestone aggregates were used in Mix7 while dolomitic sandstone aggregates were utilized in Mix8 to enhance the frictional characteristics of the taxiway surface. In this phase of experimental program, the effects of aggregate type, gradation,

compaction technique, ageing, weather and traffic on asphalt mix properties were investigated.

4.3.2. Laboratory Mixes

4.3.2.1. Tests performed

The tests performed for the laboratory mixes are shown in Figure (4.9). The experimental program included testing slab samples for their frictional properties. In addition, core specimens were tested for their:

1. Bulk relative density.
2. Air voids and voids in mineral aggregate
3. Indirect tensile strength
4. Moisture damage (stripping)
5. Shear strength.

4.3.2.2. Mix design

This section describes the mix design procedures according to Marshall mix design method. Mix design included choosing aggregate gradation that fit into the given specifications of the asphalt surface courses. Then, core specimens were prepared using five different AC for each aggregate gradation. Dimensions were measured and the specimens were tested for their Marshall stability and flow, according to ASTM D1559-89 (1996), BRD, and MRD. Av and VMA were calculated and the optimum asphalt content (AC) was determined for each aggregate gradation. The following paragraphs present the detailed procedures and results for the mix design.

Figure 4.9: Experimental Program For Laboratory Mixes

The aggregates used in designing the mix were obtained from Beaver Co. Ltd., Ottawa, Ontario. Five aggregate types were delivered to the Civil and Environmental Engineering Laboratories, Carleton University. They were HL4 stones, HL3 stones, dust, sand, and mineral filler. The HL4 stones were sieved first on sieve 12.7 mm and the passing portion was sieved again for its gradation and used in this study. Table (4.1) presents the properties of all aggregates used in mix design process.

The aggregate samples were prepared according to ASTM (C702-93, 1996) and graded by sieve analysis according to ASTM (C136-95a, 1996) for fine and coarse aggregates and ASTM (D546-94, 1996) for mineral fillers. Table (4.2) presents the results of sieve analysis for each aggregate type used in this study. To blend the five aggregates in one gradation that fit in the Canadian Standards (ASG06, 1996), many trials were performed. Based on these trials, two aggregate gradations were chosen for the mix design stage. They were referred to as Gradation (I) and Gradation (II).

Figure (4.10) illustrates the gradation of chosen aggregates in addition to the specification limits. The figure shows that Gradation (II) is coarser, as indicated by its lower percentage passing a given sieve size compared to Gradation (I). The Asphalt cement used

Table 4.1: Aggregate Properties

Designation	C-2	C-3	F-1	F-2	F-3
Description	HL4 Stone	HL3 Stone	Dust	Sand	Mineral Filler
Specific gravity	2.765	2.762	2.757	2.667	2.739
Absorption %	0.765	0.82	0.841	0.855	N.A.

Table 4.2: Gradations of Delivered Aggregates

Sieve size (mm)	Gradation (% passing)				
	HL4	HL3	Sand	Dust	M. Filler
12.7	99.7	100	100	100	100
4.75	7.8	15.9	100	95.0	100
2.00	5.7	7.6	98.7	61.4	99.9
0.425	4.1	5.2	70.1	26.5	96.3
0.180	3.4	3.9	16.5	17.4	84.3
0.075	2.3	2.4	1.6	9.3	75.1

Figure 4.10: Required Specifications and Aggregate Gradations for Mix Design

to prepare the asphalt mixes was 150/200-penetration asphalt supplied by Beaver Co. Ltd., Ottawa, Ontario. The specific gravity was 1.03. The mixing temperature was 165°C and the compaction temperature was 140°C.

The Marshall compactor was used to prepare the core samples for mix design. It allows two samples to be compacted at the same time. The compactor consists of two compaction hammers each of flat, circular tamping face and a sliding weight of $4,536 \pm 9$ gm, a concrete compaction pedestal, and a specimen mould holder mounted on the pedestal to centre the compaction mould and hold it during the compaction. The compactor was placed in a special room to decrease noise during the compaction process.

Aggregates were combined and placed in the oven for four hours to attain the mixing temperature. Binder was placed in the oven and heated to a temperature that would reach the proper viscosity (165°C). The mixing bowl was also heated to the same temperature to prevent temperature loss during the blending process. Then, the binder was added to the hot aggregates according to the selected AC, in percent of total mass, of the mix. For mix design, asphalt content was chosen to be 4.5 %, 5.0 %, 5.5 %, 6.0 % and 6.5 %. After that the mixture was mixed until the binder completely and uniformly coated all aggregate particles. About three kilograms from each mix was kept in its loose condition to be tested for its MRD.

Preheated moulds of 4 inches in diameter were used to make the core samples. Two filter papers were used during compaction to avoid sample sticking to the mould and compactor hammer and to get smooth sample surfaces. According to ASG-06 (1996), the specimens were compacted with 50 blows on each side. The compaction temperature was

140°C. Once the compaction was completed, the filter papers were removed and the specimens were left at room temperature for 24 hours. Specimens were then extracted from moulds and the BRD was determined for each specimen.

Asphalt mixture made up from aggregate Gradation I was referred to as Mix1 while it was referred to as Mix2 for aggregate Gradation II. For each mix, five different asphalt contents of 4.5 % to 6.5 % with a step of 0.5 % were used. Tables (A.1) to (A.10), Appendix A, present the results of BRD, MRD, A_v and VMA for Mix1 and Mix2. G_{Agg} was calculated for Gradation I and Gradation II according to Equation (4.3). Tables (A.11) and (A.12), Appendix A, present the Marshall stability values for Mix1 and Mix2, respectively. The measured stability values were corrected by multiplying these values by the stability correction ratios as documented in ASTM (D1559-89, 1996). The average flow values for Mix1 and Mix2 are presented in Tables (A.13) and (A.14), Appendix A, respectively. Generally, Mix1 resulted in higher BRD compared to Mix2. However, Mix2 yielded higher A_v % and VMA %. Test results showed that, both mixes had almost equal Marshall stability. However, Mix1 had relatively higher flow values than those of Mix2.

Mix design criteria of the Marshall mix design method are represented by Figures (A.1) to (A.5), Appendix A, for Mix1 and Figures (A.6) to (A.10), Appendix A, for Mix2. Based on Figures (A.1) to (A.10) and the required specifications of surface courses of airports (ASG-06, 1996), the optimum AC for Mix1 was found to be 4.5 % while it was found to be 5.0 % for Mix2.

To study the effect of AC on the properties of airfield hot-mix asphalt concrete (HMAC) surfaces, Mix3 and Mix4 were prepared from Gradation II. For these mixes, AC

were selected to be 4.5 % and 6.0 %, respectively. The effect of coarser aggregate gradation on the frictional properties of HMAC surfaces was evaluated by preparing two coarser mixes, Mix5 and Mix6. For both of them, AC of 5.0 % was chosen while two coarser gradations, Gradation III and Gradation IV were used, respectively. Table (A.15), Appendix A, presents the aggregate proportions used to blend aggregates into the four aggregate gradations utilized in this research. G_{Agg} values according to Equation (4.3) were also included in this table. Table (4.3) summarises the aggregate gradations utilized for all laboratory mixes. Sufficient amounts of the different bituminous mixtures for runway surfaces were fabricated. Core and slab samples for each mix were prepared and tested as discussed in the following sections to investigate the frictional properties, shear properties and other physical and mechanical properties for the candidate mixes.

Table 4.3: Aggregate Gradations for Laboratory Mixes

Sieve size (mm)	Gradation (% passing)				Specification	
	I	II	III	IV	Max.	Min.
12.7	6.5	3.8	4.7	4.4	8	3
4.75	13.3	10.6	10.3	9	20	5
2.00	27.4	29	23.5	19.1	30	15
0.425	47.8	45.3	38.8	31.4	55	35
0.180	64.7	55.2	50.5	41.5	75	55
0.075	100	100	99.9	99.9	100	100

4.3.2.3. Preparation of test specimens

For each laboratory mix, ten asphalt cores were manufactured using the Marshall compactor. Besides these specimens, a slab of dimensions $600 \times 130 \times 75$ mm was prepared and compacted using the plate compactor. The slabs were labelled from S1 to S6 and were tested for their frictional properties using the BPT. A ramp was prepared at the end of the wooden form to allow the compactor to move back and forth on top of asphalt slabs without damaging the wooden edges of the form. The form and ramp are shown in Figure (4.11). The compaction process is shown in Figure (4.12). After testing for their frictional properties, five cores were extracted from each slab to be tested for their BRD, and ITS.

4.3.3. Field Mixes

4.3.3.1. Testing program and field Measurements

The experimental program was designed to evaluate the frictional properties, shear properties and other physical and mechanical properties for field mixes. The frictional properties of field mixes were mainly evaluated using the BPT. In addition, SPT was used to assess the pavement texture. Shear properties were evaluated using three different test procedures including; shear test using the torque machine, triaxial test and direct shear test. The other mix properties included BRD, Av, VMA, ITS and moisture sensitivity tests.

As mentioned in Chapter (2), the frictional properties may change due to traffic and environmental factors such as temperature variations. Field measurements were conducted at OIA periodically to study the changes in frictional properties. The city of Ottawa is located in south-eastern Ontario, Canada a little north of 45°N and west of 75°W . On average, it receives 702 mm of rainfall and 225 cm of snow yearly (Andrey et al., 2003). Figure (4.13)

Figure 4.11: The Wooden Forms and Ramp, Laboratory Mixes

Figure 4.12: Compaction of the Asphalt Concrete Slabs, Laboratory Mixes

Source: Meteorological Service of Canada, 1961-90 Canadian Climate Normals,
<http://www.cmc.ec.gc.ca/climate/normals/eprovwmo.htm>

Figure 4.13: Weather Data for OIA

illustrates the long-term mean weather data for Ottawa International Airport. The figure shows that, for 30 years, the temperature has been ranged from -15°C to 25°C . The field measurements were performed to cover a wider range of temperature of -18°C to 32°C along one year. A summary of the testing program and field measurements is shown in Figure (4.14).

4.3.3.2. Construction of field slabs and preparation of test specimens

To construct testing slabs from Mix7 and Mix8, two large-scale wooden forms of $1500 \times 1000 \times 75$ mm were built at OIA. To allow the rollers to move back and forth on top of asphalt slabs without damaging the wooden edges of the forms, two ramps were constructed at ends of the wooden form. The forms and the ramps are shown in Figure (4.15). For each

Figure 4.14: Experimental Program For Field Mixes

Figure 4.15: The Wooden Forms and Ramp, Field Mixes

wooden form, an asphalt layer of approximately 85 mm was laid on top of the wooden base and levelled off manually. Each of the constructed slabs was compacted using steel roller and pneumatic roller as shown in Figures (4.16) and (4.17). The compaction was completed after 8 passes per slab. Desired densities were checked using the nuclear gauge density meter as shown in Figure (4.18). Following the completion of the construction stage, the asphalt slabs were left at the site for a period of three days before cutting each one into smaller size slabs for transportation to the Civil Engineering Laboratories at Carleton University. Each of the large slabs was cut into six smaller slabs with the dimensions of $500 \times 500 \times 75$ mm. These dimensions ensured that the test samples could be carried and handled without difficulty by two persons. After cutting the larger slabs as shown in Figure (4.19), the smaller slabs were transported to the laboratory for testing. Each slab was given a code to identify its type and location before cutting. The slabs were cleaned and completely

Figure 4.16: Compaction Process, Steel Roller

Figure 4.17: Compaction Process, Pneumatic Roller

Figure 4.18: Density Measurement using Nuclear Gauge

Figure 4.19: Cutting of Large Slabs

checked to make sure that they were free of any pre-testing damage. Half of the slabs were tested for their frictional properties using BPT and SPT, whereas the other half was used in another study conducted at Carleton University. For the field Mixes, Mix7, Mix8, and Mix9, loose samples were taken to prepare core specimens for testing according to the experimental program. Tables (A.16) and (A.17), Appendix A, present the mix design formula for the field mixes. In addition, two small slabs of $600 \times 130 \times 75$ mm were prepared in the laboratory according to the procedures discussed in section (4.3.2.3). The first slab was manufactured from Mix7, the surface course of Taxiway “F”, OIA. The second slab was fabricated from Mix9, the base course of Taxiway “B”, OIA. As the dolomitic sandstones were used in the surface course of Taxiway “B”, Mix8, no small slab was prepared from this mix. The small slabs were tested for their frictional properties using BPT. Then, five cores were extracted from each slab and tested for their BRD and ITS. Mix7 and Mix9 in addition to laboratory mixes were referred to as limestone mixes.

4.4. Basic Characteristics of Surface Friction

4.4.1. Introduction

The understanding of the factors and variables that govern and influence the frictional characteristics of a given surface is very important to improve the safety of traffic on that surface. In this section, the results of a basic laboratory simulation of several surfaces are presented and discussed to develop the required understanding. One of the main objectives of this thesis was to examine the possibility of enhancing the frictional characteristics of paved surfaces while improving their mechanical properties. Clearly, one of the most

desired mechanical properties in asphalt mixes is the tensile strength. In order to establish a range for friction values, glass was selected for the lower limit while loose large aggregates were selected to determine the upper limit. Based on this premise, the laboratory evaluation of surface friction included a sheet of glass and six different loose aggregate surfaces. Subsequently, six slabs of $300 \times 150 \times 100$ mm were prepared, each simulating different surface textures. Each slab contained a surface made of a compacted loose aggregate with a pre-specified gradation. The six slab specimens were prepared and tested for their frictional properties using the BPT. Figures (4.20) and (4.21) show the friction test set-up for the fine aggregate surface and coarse aggregate surface, respectively. In addition to these slabs, a glass sheet of dimensions 300×300 mm was tested as well for its surface friction as shown in Figure (4.22). The aggregates were selected to provide average aggregate sizes ranging from sand for very fine to a coarser surface made of HL-4 stone. Sand and HL4 stone were used directly to prepare two slabs. The gradations of sand and HL4 stone were presented in Table (4.2). The other four slabs were prepared using the aggregates retained on different sieve sizes. Table (4.4) presents a description for aggregates used in this study.

4.4.2. Test Results

Table (4.5) presents the test results for the seven surfaces under study. Mean friction values in terms of BPN, and STD are also introduced in the table. The gradation of sand and HL4 stone as presented in Table (4.2) were used to calculate the average aggregate size of each aggregate type. Table (C.1), Appendix C, presents the detailed calculations of average aggregate size of sand and HL4 stone. The average aggregate size for sand was found to be 0.255 mm while it was 4.454 mm for HL4 stone.

Figure 4.20: Friction Test on Fine Aggregate Surface, Sand

Figure 4.21: Friction Test on Coarse Aggregate Surface, HL4 Stone

Figure 4.22: Friction Test on Glass Surface

Table 4.4: Description of Aggregate Used in Loose Aggregate Slabs

Aggregate	Description
Sand	Gradation is presented in Table (4.2)
HL4	Gradation is presented in Table (4.2)
Agg _{12.7}	Aggregate passing sieve 16.0 mm and retained on sieve 12.7 mm
Agg _{4.75}	Aggregate passing sieve 12.7 mm and retained on sieve 4.75 mm
Agg _{2.00}	Aggregate passing sieve 4.75 mm and retained on sieve 2.00 mm
Agg _{0.425}	Aggregate passing sieve 2.00 mm and retained on sieve 0.425 mm

Table 4.5: Friction of Loose Aggregate Surfaces (BPN)

	Surface						
	Glass	Sand	HL4	Agg _{12.7}	Agg _{4.75}	Agg _{2.00}	Agg _{0.425}
	17	70	101	100	115	110	85
	18	71	110	110	120	110	89
	20	69	105	115	105	105	86
	18	70	112	113	110	107	87
	18	70	108	118	115	109	90
	19	68	100	122	109	106	87
	20	71	95	108	113	108	88
	19	70	102	109	117	110	84
	18	71	97	116	121	112	89
	17	70	106	124	118	106	84
Mean	18.4	70.0	103.6	113.5	114.3	108.3	86.9
STD	1.075	0.943	5.562	7.122	5.100	2.263	2.132

Figure (4.23) illustrates the relationship between aggregate size and surface friction. The figure shows that surface friction increases as the aggregate size increases until the friction reaches a value after which any increase in the size of the aggregates does not produce any significant increase in the measured friction. On the other hand, although glass has a relatively high tensile strength, it yields the lowest friction value of 18.4 compared to loose aggregates with very low tensile strength. Clearly, this relationship suggests that

Figure 4.23: Relationship between Aggregate Size and Surface Friction

improving the surface friction of a given material might not necessarily result in improving its mechanical properties.

4.4.3. Basic Understanding

The results presented in this section are very important in terms of the understanding of the factors and variables that govern the characteristics of surface friction. The laboratory simulation showed that surfaces of materials with higher tensile strength are not necessarily associated with surfaces with higher friction values. Furthermore, optimum friction could be obtained through careful consideration of the main variables that contribute most to the phenomenon. As shown in the figure, increasing the coarser aggregates will not always result in higher surface friction. Therefore, it is important that prediction models based on

well-planned field and laboratory studies be developed and used to design surfaces with optimum friction. This section represented the basic framework for the experimental and analytical investigation of this thesis.

4.5. Summary

This chapter presented a description of test facilities, test procedures, and specifications used to perform the experimental investigation of this research. Nine different asphalt mixes were prepared to perform the experimental investigation. Of them, three mixes were obtained from the field during the construction of two taxiways at OIA. The laboratory mixes were prepared in the laboratory based on Marshall mix design. The preparation of test specimens included preparation of slab samples and core specimens. Some test slabs were compacted in the laboratory using the small plate compactor, whereas other slabs were compacted in the field utilizing the steel and pneumatic heavy rollers. Slab and core samples were labelled and checked for any pre-damage before performing the testing program. Slab samples were tested for their frictional properties while core specimens were tested for their mechanical properties. This chapter also presented the basic characteristics of surface friction. It was found that increasing the aggregate size would not always result in higher surface friction. In addition, improving the surface friction is not necessarily a pre-requisite for better mechanical properties.

CHAPTER 5

5. RESULTS OF EXPERIMENTAL INVESTIGATION

5.1. Introduction

This chapter presents the results of the experimental investigation discussed in Chapter (4). Slab and core samples were prepared using nine different asphalt mixes. Six mixes were prepared in the laboratory and the other three mixes were obtained from the field during the construction of two taxiways at OIA. The slabs were tested for their frictional properties while core specimens were tested for their physical and mechanical properties. In addition to the laboratory work, field measurements were taken on the actual pavement surface at OIA. The following sections present the results and main findings of the performed tests.

5.2. Physical Properties

5.2.1. Bulk Relative Density

Before performing any test, dimensions and BRD, according to ASTM D2726-90 (1996), were determined for each core sample. Tables (B.1) to (B.6), Appendix B, present the dimensions and BRD for laboratory mixes. The mean BRD and standard deviation (STD) values are also introduced in these tables. The mean BRD ranges between 2.345 gm/cm^3 and 2.519 gm/cm^3 . The STD varies between 0.009 and 0.022. The small range of STD is attributed to the same compaction level for all core specimens. For field mixes, Tables (B.7) to (B.9), Appendix B, show the dimensions and BRD. The values of BRD ranges from 2.329

gm/cm³ to 2.452 gm/cm³, while the STD ranges from 0.020 to 0.037. Based on their BRD, core samples were classified into groups, each of approximately equal density, to perform the experimental program.

In addition to core specimens compacted using the Marshall compactor, five cores were extracted from each small slab of 600 × 130 × 75 mm after performing the friction test. Tables (B.10) to (B.17), Appendix B, present the dimensions and BRD of the extracted cores for all limestone mixes (slabs of laboratory mixes and small slabs built of Mix7 and Mix9). As expected, the tables show that the different compaction techniques result in different BRD. In other words, the mean BRD values for extracted cores are different from the BRD of core specimens compacted using the Marshall compactor. The range of mean BRD for laboratory mixes is from 2.285 gm/cm³ to 2.470 gm/cm³ and the STD range is from 0.004 to 0.013, which indicates that a uniform compaction level is achieved for all specimens prepared from the same mix. For limestone field mixes (Mix7 and Mix9), the ranges are from 2.346 gm/cm³ to 2.397 gm/cm³ and from 0.007 to 0.027 for mean BRD and STD, respectively. Extracted cores were tested for their ITS. The mean BRD values were used to evaluate the volumetric properties of mixes under study in terms of Av and VMA.

5.2.2. Maximum Relative Density

For all limestone mixes (all mixes except Mix8), loose samples were prepared to be tested for their MRD according to ASTM D2041-90 (1996). The MRD of Mix1, Mix2 and Mix3 were determined during the mix design process, section (4.3.2.2). They were found to be 2.541 gm/cm³, 2.505 gm/cm³, and 2.533 gm/cm³, respectively. Table (B.18), Appendix B,

presents the determination of MRD for limestone mixes. From this table, the MRD ranges from 2.483 gm/cm³ to 2.541 gm/cm³.

5.2.3. Air Voids and Voids in Mineral Aggregate

BRD and MRD were used to calculate Av and VMA for all limestone mixes. The values of Av were calculated according to Equation (4.1), whereas VMA was calculated using Equation (4.2). Av ranges between a minimum of 2.35 (for Mix6) and a maximum of 10.07 (for Mix1). Similarly, VMA ranges between a minimum of 14.61 (for Mix6) and a maximum of 20.42 (for Mix1). Table (5.1) presents a summary for the physical properties of all limestone mixes. The table shows that Av and VMA could approximately rank candidate mixes in a similar order. Relationships between physical properties and surface friction will be discussed in the following sections.

Table 5.1: Physical Properties of Limestone Mixes

Mix	BRD (gm/cm ³)	MRD (gm/cm ³)	Av (%)	VMA (%)
1	2.285	2.541	10.07	20.42
2	2.303	2.505	8.03	19.93
3	2.285	2.533	9.79	20.13
4	2.417	2.486	2.78	16.85
5	2.460	2.539	3.10	14.79
6	2.470	2.529	2.35	14.61
7	2.346	2.483	5.52	17.57
9	2.397	2.518	4.81	18.86

5.3. Friction Test Results

5.3.1. General

The friction test was performed on all laboratory and field mixes using the BPT. The aggregates used in the preparation of all asphalt mixes were limestone aggregates except that for Mix8, dolomitic sandstones were utilized. Therefore, for all limestone mixes (all mixes except Mix8), the friction test was performed on small slabs of $600 \times 130 \times 75$ mm. The slabs were compacted in the laboratory using the small plate compactor. The main purpose for this phase of testing program was to study the effects of mix properties on the frictional properties of airfield asphalt pavements. In addition, large slabs of $500 \times 500 \times 75$ mm built of Mix7 and Mix8 were tested periodically for their frictional properties. The slabs were compacted in the field using the heavy steel and pneumatic rollers. The main goal of that stage of testing program was to study the effect of aggregate type on the surface friction of airfield pavements.

5.3.2. Laboratory Compacted Mixes

This section presents the friction test results for all laboratory compacted mixes. These mixes included all mixes where limestone aggregates were utilized in mix preparation. These mixes contained the six laboratory mixes in addition to Mix7 and Mix9 obtained from the field. For each mix, a slab of $600 \times 130 \times 75$ mm was prepared in the laboratory using the small plate compactor and tested for its frictional properties using the BPT. For each slab, five different locations were marked to be tested, these locations were labelled from L1 to L5. The friction test was performed in the direction of compaction (longitudinal direction

LOD) at all locations. In addition and to investigate the effect of compaction direction on the frictional properties of asphalt mixes, the test was performed in the transverse direction (TRD) at locations L1 and L2. The friction was measured four times at each location and the results were recorded as R1 to R4. The average BPN was calculated at each location and the mean BPN for each mix was determined in both LOD and TRD. Tables (C.2) to (C.9), Appendix C, present the results of friction test in terms of BPN for mixes Mix1 to Mix9 (except Mix8), respectively. Mean values and STD are also included in these tables. The mean BPN values in LOD vary from a minimum of 72.7 (Mix1) to a maximum of 96.6 (Mix9). The STD ranges between 0.730 and 2.429. Table (5.2) summarises the results of friction test for all laboratory compacted mixes in both LOD and TRD. The change in friction due to compaction direction is presented in terms of percentage of difference between friction values in TRD and those in LOD.

Generally, the friction in TRD is found to be greater than that the friction in LOD. For most cases (7 out of 8), the percentage of increase varies between 4.8 % and 7.3 %. The minimum percentage of increase of 2.6 % is recorded for Mix4, while the maximum percentage of increase of 7.3 % is recorded for Mix5. This phenomenon can be explained as the plate compactor is not able to compact the asphalt slabs in TRD to the same compaction level achieved in LOD, which in turn results in rougher surface in the TRD. In all developed figures and relationships through the rest of the thesis, the term friction indicates the friction value in LOD in terms of BPN.

Table 5.2: Friction Test Results for Laboratory Compacted Mixes

Mix	LOD	TRD	% Difference *
1	72.7	76.3	5.0
2	76.3	79.9	4.8
3	73.1	78.1	6.9
4	73.1	75.0	2.6
5	77.5	83.1	7.3
6	79.7	83.8	5.1
7	82.8	88.4	6.8
9	96.6	103.4	7.0

* Percentage of difference between friction values in TRD and those in LOD

5.3.2.1. Effect of physical properties on friction of airfield surfaces

Figure (5.1) illustrates the relationship between BRD, for extracted cores, and friction in terms of BPN. The figure shows that the friction increases as the BRD increases until it reaches its maximum value at a so-called critical BRD. Beyond this value, any further increase in BRD yields lower surface friction. Similar trends are noticed for the relationships between friction and both of Av and VMA as shown in Figures (5.2) and (5.3). These Figures show that, there are critical values of Av and VMA, beyond them any increase in Av or VMA results in lower friction.

Figure 5.1: Relationship between Surface Friction and BRD

Figure 5.2: Relationship between Surface Friction and Percent of Av

Figure 5.3: Relationship between Surface Friction and Percent of VMA

5.3.2.2. Effect of asphalt content on the friction of airfield surfaces

The effect of asphalt content (AC) on the frictional properties of HMAC surfaces is investigated by comparing the friction test results of Mix2, Mix3 and Mix4. The three mixes have the same aggregate gradation (Gradation II). The only difference during the preparation of these mixes was the ratio of asphalt content. The ratios were 5.0 %, 4.5 %, and 6.0 % for Mix2, Mix3, and Mix4, respectively. In Figure (5.4), the friction values on the vertical axis are plotted versus the percent of AC on the horizontal axis. The figure shows that the optimum AC (5.0 %) yields the highest friction. The friction of Mix2, in terms of BPN, is 76.3 while both of Mix3 and Mix4 yield BPN of 73.1. The percentage of increase in BPN is 4.3 %. This finding emphasises the role of proper mix design to enhance the frictional

Figure 5.4: Effect of AC on the Frictional Properties of Airfield Pavements

performance of airfield HMAC surfaces. Further statistical analysis to check whether the differences in BPN are significant or not is introduced in Chapter (6).

5.3.2.3. Effect of aggregate gradation on the friction of airfield surfaces

Effect of aggregate gradation on the friction of airfield pavement surfaces is evaluated by comparing the test results of Mix2, Mix5, and Mix6. The same asphalt content ratio of 5.0 % was used for the three mixes. However, different aggregate gradations were utilized; Gradation II, Gradation III and Gradation IV, respectively. Table (5.2) shows that Mix6 of coarse aggregate gradation yields the highest friction value of 79.7 followed by Mix5 (BPN = 77.5) and finally Mix2 gives the lower friction value (BPN = 76.3) among the three mixes. The friction values in terms of BPN are plotted versus percent passing different sieve as

shown in Figure (5.5). In general, friction increases as aggregate gradation gets coarser, the coarser gradations are characterized by low percentages passing a specific sieve size. The results shown in Figure (5.5) support the previous observation that using coarser mixes would improve the friction of the surface. Also, it is evident that as the aggregate size approaches 0.180 mm, the measured friction appears to be less affected by the fine aggregate. This finding leads to a suggestion that the gradation of coarser aggregate sizes is more important to improve the friction of airfield HMAC. The statistical analysis to test the hypothesis that two friction means are equal, is discussed in Chapter (6). Based on this analysis, one can verify if the differences in BPN for Mix2, Mix5, and Mix6 are significant at a given significance level (α).

5.3.3. Field Compacted Mixes

For Mix7 and Mix8, two large slabs, $1500 \times 1000 \times 75$ mm, per each mix were prepared to be tested using the BPT. They were referred to as SL1 and SL2. Each large slab was cut into six smaller slabs of $500 \times 500 \times 75$ mm. Three small slabs (out of six) numbered 1, 2 and 3 were tested for their frictional properties. Slabs were tested at three different locations (LOC) in both LOD and TRD to figure out the effect of compaction direction on frictional properties over the pavement age. These locations were marked as L1, L2, and L3. Similar to the friction tests for laboratory mixes, four friction readings were recorded and the average reading (Rav.) was determined at each location. Then the average BPN (Avg.) for each small slab was calculated and the average BPN for each large slab was obtained in both directions. For each direction, the mean BPN was calculated as the average of BPNs of SL1

Figure 5.5: Effect of Aggregate Gradation on the Frictional Properties of Airfield Surfaces

and SL2. The friction test was performed periodically over a period of one year. Nine friction tests were performed for each mix during this period.

Table (5.3) presents the results of the first friction test for Mix7 in both LOD and TRD. The average BPN for SL1 in the LOD is 76.9, while for SL2 it is 80.5. In the TRD, these values are 81.3 and 83.2 for SL1 and SL2, respectively. The mean values of BPNs are 78.7 and 82.3 in LOD and TRD, respectively. Similar to the laboratory compacted mixes, BPN in the TRD is higher than that in the LOD. The percentage of increase is found to be 4.6 %.

Table 5.3: Results of Friction Test for Field Mixes, Mix7, after 0.4 Month**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	78	79	80	79	79.0	77.8	L1	84	84	85	85	84.5	82.4
	L2	81	80	79	80	80.0		L2	82	78	77	75	78.0	
	L3	73	74	74	76	74.3		L3	87	85	85	82	84.8	
2	L1	75	77	76	76	76.0	75.9	L1	79	80	81	82	80.5	81.3
	L2	76	74	77	74	75.3		L2	83	81	80	80	81.0	
	L3	80	75	75	76	76.5		L3	84	81	82	82	82.3	
3	L1	78	77	75	78	77.0	77.0	L1	82	80	77	77	79.0	80.3
	L2							L2	80	80	78	76	78.5	
	L3							L3	86	83	82	82	83.3	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	83	83	83	81	82.5	82.2	L1	85	84	84	84	84.3	84.6
	L2	82	83	81	81	81.8		L2	85	85	83	85	84.5	
	L3	83	82	82	82	82.3		L3	85	85	85	85	85.0	
2	L1	81	80	80	80	80.3	80.2	L1	84	83	82	83	83.0	83.7
	L2	79	78	78	78	78.3		L2	84	83	82	81	82.5	
	L3	82	82	82	82	82.0		L3	85	86	86	85	85.5	
3	L1	80	79	81	78	79.5	79.2	L1	83	80	80	82	81.3	81.3
	L2	80	79	79	79	79.3		L2	80	80	80	81	80.3	
	L3	79	78	79	79	78.8		L3	85	83	81	81	82.5	

	LS1	LS2	Mean
LOD	76.9	80.5	78.7
TRD	81.3	83.2	82.3

The results of the first friction test for Mix8 are presented in Table (5.4). The only difference between Mix7 and Mix8 was that, dolomitic sandstones were used in Mix8 instead of the limestone aggregates. The BPNs in the LOD are 83.2 and 82.8 for SL1 and SL2, respectively. Higher numbers of 83.7 and 85.2 are yielded in the TRD for SL1 and SL2, in order. The mean BPN in LOD is 83.0, while it increases to be 84.5 in TRD. The percentage of increase is 1.8 %. The results show that Mix8 results in higher initial BPNs than those of Mix7. The percentages of increase are 5.5 % and 2.7 % in LOD and TRD, respectively. The significance of these changes in the friction values is discussed in Chapter (6).

About two months later, the frictional properties were evaluated again for the two mixes. Tables (C.10) and (C.11), Appendix C, present the test results in terms of BPN for Mix7 and Mix8 respectively. The tables show that, Mix7 results in slightly higher mean friction values in both LOD and TRD than those of Mix8. The percentage of increase is 1.3 % in both directions. The overall BPN of Mix7 is 78.5, which is 1.3 % higher than the overall BPN of Mix8 (77.5). The difference between friction values in TRD and LOD for Mix7 decreases to 2.3 % against 4.6 % for the initial measurement. Conversely, this difference somewhat increases to 2.4 % against 1.8 % for the initial friction test of Mix8. Periodic friction tests were continued for one year. The test results are available in details in Appendix (C), Tables (C.12) to (C.25).

Tables (5.5) and (5.6) summarise the friction test results for Mix7 and Mix8 respectively. Comparing the two tables, one can conclude that after few days Mix8 provides better initial friction values than those measured on the surface of Mix7. However this

Table 5.4: Results of Friction Test for Field Mixes, Mix8, after 0.2 Month**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	83	83	82	84	83.0	84.3	L1	86	85	85	85	85.3	84.6
	L2	85	85	85	84	84.8		L2	86	85	86	86	85.8	
	L3	85	85	85	86	85.3		L3	82	84	83	82	82.8	
2	L1	83	83	83	84	83.3	83.3	L1	83	84	82	81	82.5	83.4
	L2	81	81	80	82	81.0		L2	83	83	84	83	83.3	
	L3	87	86	86	84	85.8		L3	84	85	85	84	84.5	
3	L1	83	83	83	82	82.8	81.9	L1	84	85	84	83	84.0	83.2
	L2	83	82	81	81	81.8		L2	83	83	82	83	82.8	
	L3	81	81	82	81	81.3		L3	84	84	82	81	82.8	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	86	86	85	85	85.5	85.0	L1	88	88	86	86	87.0	86.9
	L2	86	85	87	84	85.5		L2	88	88	89	87	88.0	
	L3	84	84	84	84	84.0		L3	86	86	86	85	85.8	
2	L1	82	81	81	80	81.0	81.9	L1	83	82	83	83	82.8	83.3
	L2	82	82	82	81	81.8		L2	84	84	83	83	83.5	
	L3	84	85	82	81	83.0		L3	84	84	84	83	83.8	
3	L1	81	81	81	80	80.8	81.6	L1	86	85	85	85	85.3	85.3
	L2	84	83	82	81	82.5		L2	84	85	84	83	84.0	
	L3	83	81	82	80	81.5		L3	87	87	87	86	86.8	

	LS1	LS2	Mean
LOD	83.2	82.8	83.0
TRD	83.7	85.2	84.5

Table 5.5: Friction Test Results (BPN) for Field Mixes, Mix7

Time (month)	LOD	TRD	% Increase *
0.4	78.7	82.3	4.6
2.8	77.6	79.4	2.3
3.5	73.7	74.2	0.7
6.7	74.1	73.9	-0.3
7.4	71.8	72.2	0.6
8.3	70.3	71	1.0
9.3	69.2	68.4	-1.2
10.3	68.6	68.1	-0.7
11.5	67.8	67.4	-0.6

* Percentage of increase in friction in TRD over that in LOD

advantage seems to be lost over a relatively short period of time where either the order of mixes reverses or the differences in friction values are deteriorated with time. Similar findings can be concluded for the differences in friction values between TRD and LOD. The initial measurements showed that friction values in TRD were higher than those in LOD. This phenomenon has been diminished over time where either the differences decrease or the order reverses.

Table 5.6: Friction Test Results (BPN) for Field Mixes, Mix8

Time (month)	LOD	TRD	% Increase *	% Improvement **	
				LOD	TRS
0.2	83.0	84.5	1.8	5.5	2.7
2.5	76.6	78.4	2.4	-1.3	-1.3
3.2	74.6	74.9	0.4	1.2	0.9
6.4	73.4	73.0	-0.5	-0.9	-1.2
7.1	71.8	71.8	0.0	0.0	-0.6
8.0	71.4	71.5	0.1	1.6	0.7
9.0	69.6	69.0	-0.9	0.6	0.9
9.9	68.9	68.8	-0.2	0.4	1.0
11.2	68.2	68.2	0.0	0.6	1.2

* Percentage of increase in friction in TRD over that in LOD

** Percentage of increase in friction of Mix8 compared to friction of Mix7

The effect of compaction on the surface texture is quite apparent as can be seen by comparing the results of Mix7 in Table (5.2), for small slab compacted in the laboratory using the plate compactor, and the results of the same mix in Table (5.5), for larger slabs compacted in the field using the steel and pneumatic rollers. It is clear that the compaction using the laboratory plate compactor produces higher initial friction values.

5.4. Sand Patch Test Results

The test was performed, as described in section (2.5.4.1), on $500 \times 500 \times 75$ mm slabs built of Mix7 and Mix8. The volume of the cylinder used in this thesis was given as $25,410 \text{ mm}^3$. After spreading the sand on the slabs surface, the diameter of the sand patch was measured four times and the average was calculated. Then, the texture depth (TD) was calculated according to Equation (2.5). Periodic texture measurements were taken over one year. Tables (C.26) and (C.27), Appendix C, present the SPT results for Mix7 and Mix8, respectively.

Table (5.7) summarizes the sand patch test results for both mixes. TD is plotted against time as shown in Figure (5.6). Mix8 results in higher initial mean TD of 0.206 mm compared to that of 0.174 mm for Mix7. The figure shows that this advantage is lost over a short period of time (about 4 months). For both mixes, the mean TD increases with time until it reaches its maximum value then it starts to stabilize with time. This behaviour suggests that pavement surfaces take a few months to gain their final texture characteristics. The friction test results and texture test results presented in this section show that, the use of dolomitic sandstones instead of limestone aggregates to improve skid resistance does not always yield better results. The significance of test results is presented in Chapter (6).

5.5. Field Measurements

Field measurements were performed on two taxiway surfaces, OIA, built of Mix7 and Mix8. The measurements included friction evaluation using BPT and surface texture evaluation using SPT. Figure (5.7) shows a plan of OIA in which the two taxiways are circled.

Table 5.7: Sand Patch Test Results

Mix	Time (month)	TD (mm)
7	1.0	0.174
	6.3	0.250
	7.5	0.240
	8.2	0.238
	8.5	0.221
	11.7	0.244
8	0.7	0.206
	6.0	0.234
	7.2	0.225
	7.9	0.219
	8.1	0.212
	11.4	0.228

5.5.1. Friction Measurement Results

A set of eight field friction measurements, as shown in Figure (5.8), was conducted on Taxiway “F” (Mix7) and Taxiway “B” (Mix8). Taxiway “F” was tested at nine different locations while Taxiway “B”, of shorter length, was tested at five different locations. The distance between any two successive locations was 10 m and the offset from taxiway centerline was 2.5 m. Table (5.8) presents the results of the first field friction test for the two taxiways. The average field temperature was 31°C during taking the measurements on

Figure 5.6: Change of MTD with Time for Field Surface Mixes

Figure 5.7: Plan for Ottawa International Airport

Figure 5.8: Field Friction Measurements, OIA

Taxiway “F”. When measurements were taken on Taxiway “B”, the average field temperature was 32°C.

For Taxiway “F” (Mix7), BPN is found to be 68.0 and 71.4 in LOD and TRD, respectively. The percentage of increase in friction measured in TRD is 5.0 %. For Taxiway “B” (Mix8), where dolomitic sandstones have been used, higher initial friction is resulted in for both directions. The BPNs are 71.6 and 74.9 in LOD and TRD, respectively. Like Taxiway “F”, the friction in TRD is 4.8 % higher than that in LOD. Clearly, using dolomitic sandstones increases the initial friction.

About three months later, the second field friction measurement was taken. The field temperature was 1°C for Taxiway “F” and 3°C for Taxiway “B”. The results are presented

Table 5.8: Field Friction, 1st Measurement**Taxiway "F", Mix7****Time: 1.2 months****Average temperature: 31°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	70	69	69	71	69.8	74	71	71	72	72.0
L2	71	69	70	69	69.8	73	73	72	72	72.5
L3	69	69	68	66	68.0	69	70	70	68	69.3
L4	69	69	68	69	68.8	73	71	71	70	71.3
L5	69	66	69	68	68.0	75	73	73	73	73.5
L6	70	67	66	65	67.0	72	71	71	71	71.3
L7	69	66	67	67	67.3	72	72	71	72	71.8
L8	66	66	68	65	66.3	70	69	69	69	69.3
L9	68	69	67	66	67.5	73	72	72	72	72.3
Mean	68.0					71.4				

Taxiway "B", Mix8**Time: 0.6 month****Average Temperature: 32°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	74	74	73	73	73.5	76	76	76	76	76.0
L2	73	72	72	71	72.0	75	75	75	75	75.0
L3	73	72	71	72	72.0	77	78	77	77	77.3
L4	70	70	70	70	70.0	71	70	70	71	70.5
L5	71	70	70	71	70.5	76	74	76	76	75.5
Mean	71.6					74.9				

in Table (C.28), Appendix C. Although the taxiways were opened for traffic for almost four months, the friction values of both taxiways increased. Comparing the friction values of the two taxiways, Taxiway “F” yields higher friction than Taxiway “B”. This confirms the laboratory results as shown in Tables (5.5) and (5.6) where Mix7 results in higher friction than Mix8 after the second friction test. Based on this finding, one can conclude that the degradation in friction of Mix8 is faster than that of Mix7.

Periodic field measurements continued for one year at a wide range of temperatures, 32°C to -18°C. The measurements covered a full year period starting in summer of 2002 and ending in the summer of 2003. As the temperature dropped to sub zero °C, it was very difficult to use water on the pavement surface during the friction test. Once the water reached the taxiway surface, it froze forming a thin layer of ice on the pavement surface. This ice layer caused the friction of the surface to decrease drastically. As a result, a special fluid was used to perform the friction test in the field during the winter season when temperature went down below 0°C. The field friction test results are presented in Tables (C.29) to (C.34), Appendix C.

Tables (5.9) and (5.10) summarize the results of friction test performed in the field for Taxiway “F” and Taxiway “B”, respectively. These results confirm the laboratory results where Taxiway “B” (Mix8) provides better initial friction values in both directions than those measured on the surface of Taxiway “F” (Mix7). However, this advantage seems to be lost over a short time period. The friction of Taxiway “B” lost rapidly under traffic and environmental changes. For Taxiway “B”, the BPN in LOD decreases by 14.2 % in one year

Table 5.9: Field Measurements, Friction Test Results (BPN) for Taxiway “F” (Mix7)

Time (month)	Temp. (°C)	LOD	TRD	% Increase *
1.2	31	68.0	71.4	5.0
3.8	1	82.3	83.2	1.1
6.8	-13	75.4	75.1	-0.4
7.3	-6	73.1	73.7	0.8
8.3	-5	70.4	71.4	1.4
9.2	5	68.8	69.7	1.3
10.3	14	67.0	68.1	1.6
11.5	18	64.7	66.0	2.0

* Percentage of increase in friction in TRD over that in LOD

while this decrease is 4.9 % for Taxiway “F”. The effect of temperature on the frictional properties is apparent. The friction increases as the temperature decreases to 1°C/3°C, after that the decreased temperature causes the friction to decrease slightly under traffic action. On other words, after the second measurement, the friction values obtained for both taxiway surfaces are deteriorated with time.

The test results of both taxiways show that field friction is generally lower than the friction of laboratory tested slabs. The reason is that the use of the conventional heavy rollers in the field can lead to smoother finished surface than the ones compacted either using the much lighter plate compactor or even the heavy rollers but with fewer passes.

Table 5.10: Field Measurements, Friction Test Results (BPN) for Taxiway “B” (Mix8)

Time (month)	Temp. (°C)	LOD	TRD	% Increase*	% Improvement**	
					LOD	TRS
0.6	32	71.6	74.9	4.5	5.3	4.8
3.5	3	79.5	79.5	0.1	-3.5	-4.5
6.3	-17	75.4	73.5	-2.5	-0.1	-2.2
7.0	-8	69.6	67.8	-2.5	-4.8	-8.0
7.9	-7	67.6	65.5	-3.2	-4.0	-8.3
8.8	5	65.1	67.8	4.2	-5.5	-2.8
9.9	13	63.1	65.7	4.1	-5.8	-3.5
11.2	20	61.5	61.4	-0.1	-5.1	-7.0

* Percentage of increase in friction in TRD over that in LOD

** Percentage of increase in friction of Taxiway “B” compared to friction of Taxiway “F”

5.5.2. Texture Measurement Results

The sand patch test as shown in Figure (5.9) was performed on some selected locations along the two actual taxiway surfaces, OIA. Table (5.11) presents a summary for texture measurements performed on both Taxiway “F” and Taxiway “B”. Detailed test results and texture calculations are available in Tables (C.35) and (C.36). Similar to the laboratory performance, field texture measurements show that Taxiway “B” yields higher initial mean TD compared to Taxiway “F”. However, Figure (5.10) shows that this benefit seems to be

Figure 5.9: Field Texture Measurements, OIA

Table 5.11: Field Texture Measurement Results

Surface	Time (month)	Temperature (°C)	TD (mm)
Taxiway “F” (Mix7)	1.2	31	0.198
	3.8	1	0.270
	6.8	-13	0.301
	10.3	14	0.342
	11.5	18	0.302
Taxiway “B” (Mix8)	0.6	32	0.342
	3.5	3	0.381
	6.3	-17	0.381
	9.9	13	0.271
	11.2	20	0.247

Figure 5.10: Change of Field Surface Texture with Time, OIA

lost in the early stage of pavement life (almost nine months). The figure confirms the results of SPT performed in the laboratory. Taxiway “B” built of dolomitic sandstones acquires its highest TD in about four months. However, the ageing time for Taxiway “F” to obtain its texture is nine months.

The field texture measurements agree with friction measurements presented in Tables (5.9) and (5.10). For Taxiway “F”, the initial friction was 68.0 and the field temperature was 31°C. As temperature dropped to 1°C, the friction increased to 82.3. Reviewing Table (5.11) one can observe a similar behavior where the TD increases from 0.198 mm to 0.270 mm. Same trend is observed for Taxiway “B” where the increase in friction from 71.6 to 79.5 is accompanied with an increase in TD from 0.342 mm to 0.381 mm.

The effect of temperature on TD is further shown in Figure (5.11), where the first three TD measurements are plotted versus time. The figure shows that the TD increases linearly with time as temperature decreases from 31°C/32°C to 1°C/3°C and then to -13°C/-17°C. It can be deduced that the effect of temperature change on the pavement texture is much higher than the effect of time. This may suggest that the time required for the actual pavement surface texture to be stabilized is longer than that in the laboratory.

5.6. Shear Test Results

5.6.1. General

Shear properties were evaluated for all the mixes investigated in this research. For the six laboratory mixes, shear properties were evaluated at room temperature (25°C) utilizing the torque machine and shear properties were evaluated in terms of shear strength. The main purpose of this phase of testing program was to study the shear properties of airfield asphalt mixes and their effects on both safety and performance. In other words, this phase of experimental investigation was carried out to evaluate the possible enhancement of both surface friction and rutting resistance (in terms of obtaining better shear properties) through the proper mix design.

For the three field mixes, shear properties were evaluated using three different shear tests; shear test by torque machine, triaxial test, and direct shear test. Shear test using the torque machine was performed at three different temperatures; 25°C, 35°C and 45°C and the shear properties were evaluated in terms of shear strength. Triaxial test and direct shear test were conducted at room temperature (25°C) and the shear properties were obtained in terms

Figure 5.11: Increase in Texture Depth with Time as Temperature Decreases

of shear strength parameters; coefficient of cohesion and angle of internal friction. In addition for one mix only (Mix7), the tests were performed at higher temperatures of 35°C and 45°C. This part of experimental investigation was performed with the purpose of relating shear properties obtained utilizing different test methods to each other. In turn, shear properties can be evaluated confidently using any available test facility. In addition, relating shear properties assessed by different test procedures can be used in the future to relate in-situ shear properties to other laboratory evaluated shear properties.

5.6.2. Laboratory Mixes

For each laboratory mix, four core specimens were tested for their shear strength using the torque machine. The test was performed at room temperature (25°C) and the loading rate

was 18°/min. Figure (5.12) shows a failed sample after the shear test, while Figure (5.13) illustrate a typical plot of applied torque against the elapsed time. Table (5.12) presents the shear strength (SS) results for all laboratory mixes. From the table, the mean shear strength values vary between 0.468 MPa and 0.969 MPa while the STD ranges from 0.040 to 0.246. The maximum shear strength is recorded for Mix5 built of coarse aggregates (Gradation III). However, Mix6 built of coarser aggregates (Gradation IV) results in a lower shear strength value of 0.642 MPa. This could be due to lack of sufficient AC that might cause the shear strength and other mechanical properties to be less than acceptable.

On the other hand, the effect of asphalt content on shear strength could be evaluated by comparing test results of Mix2, Mix3, and Mix4. In these mixes, the same aggregate gradation was used with different asphalt content of 5.0 %, 4.5 %, and 6.0 %. Figure (5.14) shows that the optimum asphalt content of 5.0 % used in Mix2 yields the highest shear strength value of 0.88 MPa. Mix3 of 4.5 % asphalt content shows the worst shear strength (0.468 MPa) while Mix4 of 6.0 % asphalt content results in slightly lower shear strength (0.851 MPa) compared to Mix2. The significance of differences in shear strength values will be discussed in Chapter (6).

The relationship between surface friction and shear strength is shown in Figure (5.15). The figure shows that the correlation between friction and shear strength is very weak as indicated by R^2 value of 0.0214. Having the coarsest gradation among laboratory mixes, Mix6 yields the highest friction of 79.7. However, due to lack of sufficient AC, the same mix results in a low shear strength value. Mix5 yields the highest shear strength values and highest friction as well if Mix6 has been excluded.

Figure 5.12 Failed Sample after Shear Test using the Torque Machine

Figure 5.13: Typical Plot of Applied Torque versus Elapsed Time

Table 5.12: Shear Strength Test Results for Laboratory Mixes

Mix	Sample No.	D (mm)	T (N.m)	SS (MPa)	Mean	STD
1	3	101.6	206.0	1.000	0.821	0.197
	5	101.7	160.0	0.774		
	6	101.8	196.0	0.946		
	7	101.6	116.0	0.563		
2	2	101.5	106.0	0.516	0.880	0.246
	3	101.5	214.0	1.042		
	6	101.7	196.0	0.949		
	7	101.5	208.0	1.013		
3	1	101.6	128.0	0.621	0.468	0.128
	2	101.7	78.0	0.378		
	4	101.6	72.0	0.350		
	10	101.6	108.0	0.524		
4	1	101.5	150.0	0.730	0.851	0.083
	5	101.5	189.0	0.920		
	7	101.5	178.0	0.867		
	10	101.5	182.0	0.886		
5	4	101.5	208.0	1.013	0.969	0.040
	6	101.5	190.0	0.925		
	7	101.7	196.0	0.949		
	10	101.6	204.0	0.990		
6	3	101.7	102.0	0.494	0.642	0.105
	5	101.7	139.0	0.673		
	7	101.6	136.0	0.660		
	8	101.5	152.0	0.740

Figure 5.14: Effect of Asphalt Content on Shear Strength of Airfield Mixes

Figure 5.15: Correlation between Surface Friction and Shear Strength, All Laboratory Mixes

The aggregate gradation used in Mix5 was slightly coarser than the one recommended by the Canadian Specifications ASG-06 (1996). In addition, Mix2 built of aggregate (Gradation II) along with the optimum AC of 5.0 % yielded the highest friction and shear strength values after those of Mix5. The obtained shear test results suggest that using coarser aggregates might improve both the surface friction and shear strength. Figure (5.16) shows that there is a good correlation between friction values and shear strength values for all laboratory mixes if Mix6 is excluded. The goodness of the relationship is shown by the R^2 value of 0.8183.

Figure 5.16: Correlation between Surface Friction and Shear Strength, Laboratory Mixes except Mix6

5.6.3. Field Mixes

5.6.3.1. Shear test using the torque machine

For the three field mixes, five core specimens from each mix were tested to evaluate their shear properties using the torque machine. As asphalt mixes are more susceptible to rutting at higher temperature, the test was performed at room temperature (25°C), 35°C, and 45°C, respectively. At higher temperature of 35°C, and 45°C, the test was carried out utilizing a special temperature chamber at the Laboratories of Civil and Environmental Engineering, Carleton University as shown in Figure (5.17).

Figure 5.17: Shear Test at 45°C

A summary of shear test results for all field mixes is shown in Table (5.13). Based on their mean shear strength values, Mix7 yields the highest shear strength followed by Mix9 while Mix8 built of dolomitic sandstones results in the lowest shear strength. This finding is valid for the selected range of temperature of 25°C to 45°C. Tables (D.1) to (D.3), Appendix D, present the detailed calculation and test results for Mix7, Mix8, and Mix9, respectively.

Figure (5.18) shows the change of shear strength with temperature for field mixes. Generally, shear strength is found to decrease linearly as the temperature increases. For Mix7, the mean shear strength is 1.285 MPa at room temperature. This value decreases to 0.868 MPa with percentage of decrease of 32.0 %, when performing the test at 35°C. At

Table 5.13: Shear Strength Test Results for Field Mixes

Mix	Temp. (°C)	SS (MPa)
7	25	1.285
8		0.748
9		0.839
7	35	0.868
8		0.563
9		0.735
7	45	0.510
8		0.444
9		0.502

Figure 5.18: Change of Shear Strength with Temperature, Field Mixes

45°C, the mean shear strength value is 0.510 MPa. The effect of an increase, in temperature, of 20°C from 25°C to 45°C results in a loss of about 60.0 % in mean shear strength value. Same trend is noticed for Mix8 where mean shear strength values are 0.748 MPa, 0.563 MPa and 0.444 MPa at 25°C, 35°C and 45°C, respectively.

Mix8 yields lower shear strength values compared to those of Mix7. The percentages of decrease are about 42.0 %, 35.0 %, and 13.0 % at 25°C, 35°C and 45°C, respectively. For Mix9, the mean shear strength value evaluated at 25°C is 0.839 MPa. This shear strength value drops to 0.735 MPa and 0.503 MPa as the temperature increases to 35°C and 45°C respectively. Although Mix7 yields higher mean shear strength values compared to Mix8, the results in Figure (5.18) illustrates that Mix7 shows high susceptibility to temperature. This observation could be explained by the effect of high AC and the use of limestone

aggregates in Mix7. Clearly, the design should yield a more stable mix when subjected to a wide range of changes in temperature.

The test results show that although Mix9, base course, yields the highest friction among all mixes, it results in lower shear strength values than those of Mix7. This finding confirms a similar conclusion in section (5.6.1) for Mix6 where too coarse aggregates show lower values of shear strength. Figures (5.19) and (5.20) show that there is poor to moderate correlation between surface friction and shear strength for limestone mixes if one of the coarser mixes, Mix6 and Mix9, is included in the relationship. However, Figure (5.21) shows that there is a good correlation as indicated by R^2 value of 0.7209, if these two mixes are excluded.

Figure 5.19: Correlation between Surface Friction and Shear Strength, All Limestone Mixes except Mix6

Figure 5.20: Correlation between Surface Friction and Shear Strength, All Limestone Mixes except Mix9

Figure 5.21: Correlation between Surface Friction and Shear Strength, All Limestone Mixes except Mix6 and Mix9

5.6.3.2. Triaxial test results

For each laboratory mix, a set of three core specimens was tested using the triaxial machine at room temperature. The test was performed by applying a pre-load of 0.669 kN to hold the sample in its position. Confining pressure (σ_3) of 0.345 MPa was applied to the sample with a loading rate of 0.414 MPa/min. The loads were held for thirty minutes and then the vertical stress was increased gradually with a rate of 2.45 mm/min until failure occurred. The maximum vertical stress at failure (σ_1) was recorded and the test was repeated using confining pressures of 0.689 MPa and 2.758 MPa. Figure (5.22) shows typical plots of normal stresses versus elapsed time after three triaxial test runs.

Three Mohr circles were plotted and the shear strength parameters (C and ϕ) were obtained by drawing the shear envelope as the common tangent for the three circles. The intercept of shear envelope with y-axis determined the coefficient of cohesion (C) while the slope represented the angle of internal friction (ϕ). Using the shear strength parameters, the shear stress (τ) could be determined for a given normal stress (σ_n) as;

$$\tau = c + \sigma_n \tan \phi \quad (5.1)$$

where:

τ = shear stress (MPa)

σ_n = normal stress (MPa)

C = coefficient of cohesion (MPa)

ϕ = angle of internal friction ($^\circ$)

Figure 5.22: Typical Plot of Normal Stress versus Elapsed Time, Triaxial Test

To evaluate the sensitivity of triaxial test to temperature variation, two sets of Mix7 were tested for their shear parameters at 35°C and 45°C. The desired testing temperature was obtained utilizing the cascade temperature controller. The testing temperature was reached by heating the fluid surrounding the sample and the temperature was monitored by three temperature indicators attached to the triaxial cell as shown in Figure (5.23). Figures (D.1) to (D.5), Appendix D, show the determination of shear strength parameters for the five sample sets tested using the triaxial test facility.

Table (5.14) summarises the triaxial test results for all field mixes. The table shows that ϕ is approximately the same for all mixes. The differences in shear properties are caused by the change in C values. The triaxial test ranks the field mixes in the same order obtained by the torque test machine. Mix7 gives higher shear parameters as shown by the higher C

Figure 5.23: Temperature Control, Triaxial Test

values, then Mix9 and finally Mix8. For Mix7, the change of temperature causes the cohesion coefficient C to decrease while ϕ is almost the same. The value of C is 1.514 MPa at 25°C, while it decreases to 1.207 MPa and 0.837 MPa as temperature increases to 35°C and 45°C, respectively.

Using the shear strength parameters presented in Table (5.14), shear stresses were calculated at three different normal stress levels of 0.689 MPa, 1.034 MPa, and 1.379 MPa. The effect of temperature on shear stresses evaluated by triaxial test for Mix7 is shown in Figure (5.24). This figure illustrates that the relationship between the calculated shear stresses and the temperature of the test is linear. Detailed calculations of shear stresses using Equation (5.1) are presented in Table (D.4), Appendix D.

Table 5.14: Triaxial Test Results for Field Mixes

Mix	Temp. (°C)	Sample No.	σ_3 (MPa)	σ_1 (MPa)	C (MPa)	ϕ (°)
7	25	56	0.345	8.685	1.514	45
		66	0.689	11.147		
		37	2.758	22.309		
	35	55	0.345	7.352	1.207	44
		68	0.689	9.065		
		5	2.758	20.512		
	45	34	0.345	5.652	0.837	45
		45	0.689	8.661		
		10	2.758	19.543		
8	25	31	0.345	7.910	1.284	44
		26	0.689	10.510		
		41	2.758	21.237		
9	25	20	0.345	8.712	1.396	44
		25	0.689	10.907		
		24	2.758	21.721

Figure 5.24: Effect of Temperature on Shear Stress Evaluated by Triaxial Test, Mix7

5.6.3.3. Direct shear test results

The test was performed at room temperature for all field mixes. In addition, the test was carried out at two higher temperatures of 35°C and 45°C for Mix7 only. For elevated temperatures, the sample was put into the oven for three hours to be heated to the desired testing temperature as shown in Figure (5.25). A thermocouple was attached to the bottom half of the sample to monitor the temperature during the progress of the test. The test procedures were as discussed in section (4.2.6). Three different vertical stresses of 0.689 MPa, 1.034 MPa, and 1.379 MPa were used to determine the shear envelope of field mixes. First the vertical stress was applied with a loading rate of 0.414 MPa/min. The stress was held for half an hour and then the shear load was applied with a rate of 2.54 mm/min until

Figure 5.25: Adjusting Testing Temperature, Direct Shear Test

failure occurred. The maximum shear stress at failure (τ) was calculated as the maximum shear load at failure divided by the cross sectional area of core sample.

The values of τ were plotted versus σ_n and the shear envelope was obtained. Figure (5.26) shows typical plots of shear loads versus time after three direct shear test runs. The test results are summarized and presented in Table (5.15). For most cases the direct shear test ranks the mixes in the same order as that obtained using the torque machine and triaxial test facility. Mix7 has the best shear properties followed by Mix9 and then Mix8. The only exception is that Mix8 gives highest shear stress at $\sigma_n = 1.379$ MPa. For Mix7, the shear stresses decreases for most cases as temperature increases. However, at 35°C and $\sigma_n = 1.034$

Figure 5.26: Typical Plot of Shear Load versus Elapsed Time, Direct Shear Test

MPa, the shear stress of 1.241 MPa is lower than that resulted at 45°C. Reviewing the shear strength parameters of Mix7, at 25°C the cohesion coefficient is 0.843 MPa and ϕ is 40.5°. As the temperature increases to 45°C, the cohesion coefficient decreases to 0.360 MPa while the value of ϕ remains approximately constant (40.9°). From these values and the fact that asphalt binders are temperature-dependent, one can expect that as the temperature increases, the cohesion will decrease and there is no reason for ϕ to be changed significantly. As a result, it is expected that at 35°C, the value of C will be between 0.843 MPa and 0.360 MPa. At the same time it is anticipated that ϕ will range around 40° to 41°. The investigation of results presented in Table (5.15) suggests that the recorded shear stress value of 1.241 MPa

Table 5.15: Direct Shear Test Results for Field Mixes

Mix	Temp. (°C)	Sample No.	σ_n (MPa)	τ (MPa)	C (MPa)	ϕ (°)
7	25	40	0.689	1.418	0.843	40.5
		35	1.034	1.751		
		67	1.379	2.007		
	35	41	0.689	0.976	0.218	46.5
		42	1.034	1.241		
		12	1.379	1.702		
	45	48	0.689	0.949	0.360	40.9
		65	1.034	1.270		
		32	1.379	1.546		
8	25	19	0.689	1.272	0.401	48.6
		30	1.034	1.398		
		16	1.379	2.055		
9	25	19	0.689	1.358	0.810	36.8
		42	1.034	1.520		
		32	1.379	1.873		

is not an accurate one. Figures (D.6) to (D.10), Appendix D, illustrate the determination of shear strength parameters for all performed tests.

Similar suggestion can be considered for shear stress of Mix8 at σ_n of 1.379 MPa. The ϕ value of Mix8 is determined to be 48.6° . As the aggregate gradation of Mix8 is similar to that of Mix7, it is expected that ϕ will be very close to 40.5° (ϕ of Mix7). This may lead to a conclusion that the τ value of 2.055 MPa is too high. As a general conclusion the direct shear test is not as accurate as the triaxial test. In addition it yields lower shear properties compared to the results of triaxial test.

Figure (5.27) illustrates the variation of τ with temperature at different σ_n levels for Mix7. Generally, it is found that τ decreases as the temperature increases. Figure (5.28) shows a failed sample after direct shear test.

5.7. Other Mechanical Properties

This section presents the indirect tensile strength (ITS) and moisture sensitivity test results. For each laboratory mix, two sets, each of three samples, were tested for their ITS as described in section (4.2.7). One set was tested directly however the other set was conditioned in water before being tested. For field mixes, the number of samples in each set was five. Mean ITS values were calculated for each set and the tensile strength ratio (TSR) was calculated for each mix according to Equation (4.7). In addition, for limestone mixes (all mixes except Mix8), five core specimens extracted from slabs were tested for their ITS.

The results are summarized and introduced in Table (5.16). The effect of asphalt content (AC) on ITS can be evaluated by comparing the test results of Mix2, Mix3, and

Figure 5.27: Effect of Temperature on Shear Stress Evaluated by Direct Shear Test, Mix7

Figure 5.28: Failed Sample after Direct Shear Test

Mix4. Clearly, Mix2 (of optimum AC of 5.0 %) yields the highest ITS (dry and conditioned) and ITS for extracted cores. The effect of aggregate gradation is investigated by reviewing the results of Mix2, Mix5, and Mix6 (all have the same AC of 5.0 %). Using coarser aggregates in Mix5 results in higher ITS for extracted cores compared to Mix2. The percentage of increase is 16.2 %. However Mix6 (the coarsest laboratory mix) yields lower ITS than those of Mix2 and Mix5. The percentages of decrease are 16.7 % and 28.3 %, respectively.

In other words, using coarser aggregates (in Mix5) could improve ITS for airfield mixes. However too coarse aggregate (in Mix6) might cause ITS to decrease due to insufficient AC. Although a similar behaviour is noticed for TSR, ITS values for Marshall compacted cores show a different performance. Although Mix2 is built of finer aggregates than those of Mix5 and Mix6, it yields higher ITS for both dry and conditioned samples.

For field mixes, ITS test results ranks the mixes in the same order obtained by shear tests. Mix7 yields the highest ITS followed by Mix9 and then Mix8. The order of Mix8 and Mix9 is changed for TSR values due to the nature of coarser aggregates used in the base mix, Mix9. Unexpectedly, for the cores extracted from slabs, Mix7 yields a lower ITS compared to Mix9.

Figure (5.29) shows the relationship between surface friction and ITS of cores extracted from slabs for all limestone mixes. Higher ITS could be obtained through higher quality of mechanical and physical properties of the asphalt mixes. These properties could be obtained through better compaction, higher density, proper AC, and good quality

Table 5.16: Indirect Tensile Strength and Moisture Sensitivity Test Results

Mix	ITS (MPa)			TSR (%)
	extracted	dry	conditioned	
1	0.808	0.947	0.903	95.4
2	0.680	0.988	0.951	96.3
3	0.559	0.805	0.459	57.0
4	0.568	0.727	0.718	98.7
5	0.790	0.814	0.845	103.7
6	0.567	0.618	0.605	97.8
7	0.655	1.225	1.266	103.3
8	N.A	0.979	0.934	95.4
9	0.958	1.067	0.927	86.9

Figure 5.29: Relationship between Surface Friction and Indirect Tensile Strength

aggregate. The test results presented in this chapter showed that there is good correlation between most of these characteristics and the surface friction. Therefore, it is not surprising to obtain a relationship as the one shown in Figure (5.29). Tables (D.5) to (D.13), Appendix D, present the detailed test results for all mixes under study.

5.8. Summary

This chapter presented the test results and main findings of the experimental investigation. The results supported the use of coarser aggregates to improve the friction properties of HMAC for airfield pavements. Furthermore, the results of the shear strength and other mechanical properties suggested that the coarser aggregate could improve the desired properties and at the same time maintain higher friction values. On the other hand, using the optimum AC was found to be very important to improve frictional and mechanical properties of airfield pavements. Based on the mean values of the measured friction, it was found that using dolomitic sandstone aggregates could not improve either the frictional characteristics or mechanical properties of airfield HMAC pavements. Statistical analysis is required to examine the significance of test results. The data presented in this chapter are used to develop some mathematical models as introduced in Chapter (6).

CHAPTER 6

6. ANALYTICAL INVESTIGATION

6.1. Introduction

This chapter includes two main parts. The first part deals with the statistical analysis of the test results introduced in Chapter (5). Series of two-tailed t-tests are performed to check the null hypothesis that two sample means are equal. The t-tests are performed assuming that each sample is drawn from a normal population. Before performing the t-test, sample variances are calculated and compared. Two population variances are assumed to be equal if the ratio of the two sample variances is found to be less than or equal to 3.0 (Beaver and Beaver 1999). Results of t-tests are obtained in terms of test-statistic (t-statistic) and the observed significance level (p-value), which is the smallest value of the significance level (α) at which the null hypothesis can be rejected. In this thesis, α was chosen to be 0.05. The null hypothesis was rejected if t-statistic was greater than t-critical or if p-value was less than α . More details about t-tests may be found in Beaver and Beaver (1990). Besides t-tests, some paired difference t-tests are performed to study the effects of two different treatments on the same experimental unit or two identical experimental units (Beaver and Beaver 1999).

The second part of this chapter is concerned with the development of three prediction models based on statistical principles. The first model relates surface friction to the basic mix properties. The second model aims at correlating field friction to the laboratory friction value. Finally, the last model concerns with the relationship between shear properties

evaluated using three different shear tests. The regression analysis is performed assuming that; the random errors have normal distribution with mean equal to 0.0, any two random errors are independent, and the random errors have equal variances. The statistical Package for Social Sciences (SPSS) software was used in this part of the study to calibrate the linear regression models (Tabachnick and Fidell, 2001).

6.2. Significance of Test Results

6.2.1. Friction of Loose Aggregate Surface

The statistical significance of friction test results to study the characteristics of surface friction is presented in Table (6.1), which includes the results of a series of t-tests. The table supports the observation stated earlier in section (4.4) and the behaviour shown in Figure (4.23) where all differences are significant except when comparing the friction of the two coarsest aggregate surfaces.

6.2.2. Transversely versus Longitudinally Measured Friction

A series of t-tests has been performed to figure out if the differences between friction values in TRD and friction values in LOD are significant or not. Tables (6.2) and (6.3) present the test results in terms of t-statistic and p-values. The difference is considered significant if the two-tailed p-value is less than the significance level (α) of 0.05.

First, the phenomenon of higher friction in TRD compared to LOD is checked individually for all limestone mixes. Table (6.2) shows that the differences between initial friction values measured in the two directions are significant as indicated by p-values, which ranges between 0.024 and 9.6×10^{-7} . Then for all limestone mixes, a paired difference t-test

Table 6.1: Summary of t-Test Results for Friction of Loose Aggregate Slabs

Variable 1	Variable 2	Equal variances	t-Statistic	t-Critical	p-Value
Sand (70.0)	Glass (18.4)	yes	114.120	2.101	3.4E-27
Agg _{0.425} (86.9)	Sand (70.0)	no	22.927	2.179	2.8E-11
Agg _{2.00} (108.3)	Agg _{0.425} (86.9)	yes	21.766	2.101	2.2E-14
HL4 (103.6)	Agg _{2.00} (108.3)	no	2.475	2.179	0.029
Agg _{4.75} (114.3)	HL4 (103.6)	yes	4.484	2.101	0.000
Agg _{12.7} (113.5)	Agg _{4.75} (114.3)	yes	0.289	2.101	0.776

has been conducted and the test results confirm that the differences between initial friction values measured in TRD and those measured in LOD are significant. The p-value is found to be 7.0×10^{-5} .

Second, the differences have been checked for the field mixes over a time period of one year, test results are shown in Table (6.3). For Mix7, the friction measured in TRD is significantly higher than those measured in the LOD for the first friction measurement performed in the laboratory. Then the differences are insignificant for the second and third laboratory measurements. The data presented in Table (5.5) have been used to conduct a paired difference t-test to investigate if friction in TRD is significantly higher than that in LOD over time or not. The test results show that the differences between friction values in

Table 6.2: Summary of t-Test Results to Check Differences between Friction Values Measured Transversely and Longitudinally, Limestone Mixes

Variable 1	Variable 2	Equal variances	t-Statistic	t-Critical	p-Value
Mix1-TRD (76.3)	Mix1-LOD (72.7)	yes	4.432	2.056	0.000
Mix2-TRD (79.9)	Mix2-LOD (76.3)	yes	3.617	2.074	0.002
Mix3-TRD (78.1)	Mix3-LOD (73.1)	yes	5.318	2.056	1.5E-5
Mix4-TRD (75.0)	Mix4-LOD (73.1)	no	2.407	2.064	0.024
Mix5-TRD (83.1)	Mix5-LOD (77.5)	yes	6.369	2.056	9.6E-7
Mix6-TRD (83.8)	Mix6-LOD (79.7)	yes	4.316	2.056	0.000
Mix7-TRD (88.4)	Mix7-LOD (82.8)	no	3.540	2.262	0.006
Mix9-TRD (103.4)	Mix9-LOD (96.6)	no	2.720	2.262	0.024
Limestone mixes-TRD	Limestone mixes-LOD	paired	8.339	2.365	7.0E-5

both directions become insignificant after a short period of time (almost one year) as indicated by the p-value of 0.254. Similar performance is noticed for Mix8, where the initial friction in TRD is significantly higher than that in LOD (p-value = 0.016). However, the differences between friction values measured in the two directions become insignificant based on the results of second and third friction measurements (p-values of 0.076 and 0.692, respectively). Similar to Mix7, the data for Mix8, presented in Table (5.6), were used to check the variation of friction with the direction of compaction. A paired difference t-test

Table 6.3: Summary of t-Test Results to Check Differences between Friction Values Measured Transversely and Longitudinally, Field Mixes

Variable 1	Variable 2	Equal variances	t-Statistic	t-Critical	p-Value
Mix7-TRD (0.4 month) (82.3)	Mix7-LOD (0.4 month) (78.7)	yes	3.992	2.037	0.000
Mix7-TRD (2.8 months) (79.4)	Mix7-LOD (2.8 months) (77.6)	yes	1.850	2.037	0.074
Mix7-TRD (3.5 months) (74.2)	Mix7-LOD (3.5 months) (73.7)	yes	0.523	2.037	0.605
Mix7-TRD (over 1 year)	Mix7-LOD (over 1 year)	paired	1.229	2.306	0.254
Mix8-TRD (0.2 month) (84.5)	Mix8-LOD (0.2 month) (83.0)	yes	2.543	2.032	0.016
Mix8-TRD (2.5 months) (78.4)	Mix8-LOD (2.5 months) (76.6)	yes	1.830	2.032	0.076
Mix8-TRD (3.2 months) (74.9)	Mix8-LOD (3.2 months) (74.6)	yes	0.400	2.032	0.692
Mix8-TRD (over 1 year)	Mix8-LOD (over 1 year)	paired	1.053	2.306	0.323
“F”-TRD (over 1 year)	“F”-LOD (over 1 year)	paired	2.972	2.365	0.021
“B”-TRD (over 1 year)	“B”-LOD (over 1 year)	paired	0.435	2.365	0.677

has been performed and the results confirm that after a short period of time the friction in TRD is no longer significantly higher than that in LOD (p-value = 0.323).

Tables (5.9) and (5.10) introduced the variation in friction measured over one year period of time in both directions for Taxiways “F” and “B”, OIA. Utilizing these data, two paired difference t-tests were performed. The results show that the changes in friction values measured longitudinally and transversely are insignificant for Taxiway “B” (p-value = 0.677). However, for Taxiway “F” the difference between friction values measured in TRD and LOD is found to be significant based on the data measured over one year (p-value = 0.021). Reviewing the data shown in tables (5.9) and (5.10), the friction of Taxiway “F” decreases in one year from 68.0 to 64.7. While for Taxiway “B” it decreases from 71.6 to 61.5. Considering these variations and the two paired difference t-test results, one can conclude that more time is needed for Taxiway “F” such that differences between friction values measured in TRD and those measured in LOD become insignificant.

6.2.3. Variation of Friction with AC and Aggregate Gradation

For all limestone mixes, a series of t-tests were performed to study the variation of measured friction with the changes in asphalt content and aggregate gradations. Test results are presented in Table (6.4). The table shows that using the optimum asphalt content (in Mix2) yields significantly higher friction value compared to using lower asphalt content value (in Mix3) or higher asphalt content (in Mix4). The p-values are 0.000 and 0.001 respectively. Friction values of Mix3 and Mix4 are not examined as both mixes result in the same mean friction value.

**Table 6.4: Summary of t-Test Results for Friction Differences of Limestone Mixes,
LOD**

Variable 1	Variable 2	Equal variances	t-Statistic	t-Critical	p-Value
Mix2 (76.3)	Mix3 (73.1)	yes	3.924	2.032	0.000
Mix2 (76.3)	Mix4 (73.1)	yes	3.636	2.032	0.001
Mix5 (77.5)	Mix2 (76.3)	yes	1.638	2.032	0.111
Mix6 (79.7)	Mix2 (76.3)	yes	4.387	2.032	0.000
Mix6 (79.7)	Mix5 (77.5)	yes	3.060	2.024	0.004
Mix2 (76.3)	Mix1 (72.7)	yes	4.778	2.032	3.3E-5
Mix7 (82.8)	Mix1 (72.7)	yes	14.745	2.024	2.6E-17
Mix9 (96.6)	Mix2 (76.3)	yes	19.192	2.032	8.1E-20
Mix9 (96.6)	Mix7 (82.8)	yes	14.500	2.024	4.5E-17

The second group in the table shows that for the same asphalt content of 5.0 %, using a coarser aggregate results in better friction properties. The increase in friction values are significant at $\alpha = 0.4$ % or less. The only violation is that using the coarser aggregate in Mix5 yields slightly higher friction value compared to Mix2 (p-value of 0.111). In addition, the third group introduces the comparison between mixes of proper mix design (optimum asphalt content). The results of this group emphasize the role of using coarser aggregates along with the optimum asphalt content to enhance the friction of airfield pavement

surfaces. Using coarser aggregate in Mix9 with the optimum asphalt content of 5.0 % yields significantly higher friction compared to Mix7 and Mix2 (p-values of 4.5×10^{-17} and 8.1×10^{-20} , respectively). In turn, Mix7 built of coarser aggregate and optimum AC of 6.0 % results in higher friction compared to Mix1. The increase is significant as indicated by the resulted p-value of 2.6×10^{-17} . Same trend is noticed for Mix2 and Mix1, where Mix2 enhances the surface friction significantly as indicated by p-value of 3.3×10^{-5} .

6.2.4. Effect of Aggregate Type on Frictional Properties of Airfield Pavements

This section presents the t-test results to investigate the effect of aggregate type on the measured friction both in laboratory and in the field. Table (6.5) presents the statistical analysis to compare between friction of Mix7 built of limestone aggregates and Mix8 prepared utilizing dolomitic sandstone aggregates. For the first friction measurement in the laboratory, Mix8 results in significantly higher friction as shown by the p-value of 4.4×10^{-6} . However, the second laboratory friction measurement demonstrates that the order is reversed and Mix7 yields higher mean friction value. The difference is insignificant at $\alpha = 5.0 \%$, (p-value = 0.317).

To compare between the two mixes over time, three paired difference t-tests have been performed to check if Mix8 built of dolomitic sandstone aggregates provides any benefit to improve the frictional characteristics of airfield surfaces or not. The differences in friction values between Mix8 and Mix7 were tested in LOD and TRD in addition to the overall friction values. Utilizing the data in Tables (5.5) and (5.6), t-test results confirm that the two mixes offer the same frictional properties as indicated by the resulted p-values of 0.250, 0.316, and 0.269, respectively.

Table 6.5: Summary of t-Test Results for Comparison between Mix7 and Mix8 for their Frictional Properties

Property	Variable 1	Variable 2	Equal variances	t-Statistic	t-Critical	p-value
Laboratory friction	Mix8-LOD 1 st (83.0)	Mix7-LOD 1 st (78.7)	yes	5.519	2.037	4.4E-6
	Mix7-LOD 2 nd (77.6)	Mix8-LOD 2 nd (76.6)	yes	1.013	2.037	0.319
	Mix8-LOD	Mix7-LOD	paired	1.239	2.306	0.250
	Mix8-TRD	Mix7-TRD	paired	1.070	2.306	0.316
Field friction	“B”-LOD 1 st (71.6)	“F”-LOD 1 st (68.0)	yes	5.056	2.179	0.000
	“F”-LOD 2 nd (82.3)	“B”-LOD 2 nd (79.5)	yes	2.679	1.782	0.020
	“F”-LOD	“B”-LOD	paired	2.27	2.365	0.057
	“F”-TRD	“B”-TRD	paired	2.651	2.365	0.033
Laboratory TD	Mix8 1 st (0.206)	Mix7 1 st (0.174)	yes	3.541	4.303	0.071
	Mix7 2 nd (0.250)	Mix8 2 nd (0.234)	yes	1.859	2.306	0.100
	Mix8	Mix7	paired	0.902	2.571	0.409
Field TD	“B” 1 st (0.342)	“F” 1 st (0.198)	no	4.216	2.447	0.006
	“B” 2 nd (0.381)	“F” 2 nd (0.270)	no	4.178	2.571	0.009
	“B” 3 rd (0.381)	“F” 3 rd (0.301)	no	5.226	12.706	0.120
	“B”	“F”	paired	0.949	2.777	0.396

The work was continued to analyze the field measurements. The results of the performed t-tests on the field friction values are also listed in Table (6.5). Taxiway “B” (Mix8) results in higher initial friction compared to Taxiway “F” (Mix7). The p-value of 0.0 demonstrates that the difference in friction values is significant. Similar to the friction test performed in laboratory, the second set of field measurements show that taxiway “F” give a better friction value compared to Taxiway “B”. The difference is significant as indicated by the p-value of 0.020. Again, three paired difference t-tests were performed to compare between the field friction of the two taxiway surfaces measured in LOD and TRD in addition to the overall friction. The tests were performed utilizing the data shown in Tables (5.9) and (5.10). T-test results confirm that, over one year period of time, Taxiway “F” results in better surface friction compared to Taxiway “B”. The p-values of 0.057, 0.033, and 0.036 show that the differences in friction values are significant at $\alpha = 5.0 \%$, except for first case it is significant at $\alpha = 6.0 \%$.

Table (6.5) shows also the significance of sand patch test results in both laboratory and field. The table shows that, Mix8 provides insignificantly higher texture depth compared to Mix7 (p-value = 0.071). However the order is changed after the second sand patch test where Mix7 yields higher texture depth. The difference is insignificant as indicated by the p-value of 0.100. The paired difference t-test results show that, for one-year measurements, the differences in texture depth of Mix7 and Mix8 are insignificant (p-value = 0.409). For the actual taxiway surfaces, Taxiway “B” (Mix8) shows a significant increase in the first and second texture depth measurements compared to Taxiway “F” (Mix7). The p-values are 0.006 and 0.009 respectively. However, the third texture depth measurement shows that the

difference becomes insignificant as illustrated by the p-value of 0.120. For all field texture measurements, the paired difference t-test results confirm that the advantage of Taxiway “B” to yield better texture depth is lost over a short period of time (almost one year) as shown by the p-value of 0.396.

6.2.5. Variation of Shear Properties with AC and Aggregate Gradation

A series of t-tests were performed to study the effect of asphalt content and aggregate gradation on the shear strength of airfield mixes. The test results are shown in Table (6.6). The test results presented in the table show a similar performance to that presented in section (6.2.3).

Comparing Mix2, Mix3, and Mix4 (all have the same gradation but different asphalt content ratios), Mix2 (of optimum asphalt content of 5.0 %) yields higher mean shear strength value as shown in Table (5.12). However the t-test results show that the difference between mean shear strength values of Mix2 and Mix4 is insignificant (p-value = 0.834).

Compared to Mix3 of lower asphalt content of 4.5 %, the mean shear strength value of Mix2 is significantly higher as indicated by the p-value of 0.031. In addition, higher asphalt content of 6.0 % (Mix4) causes the mean shear strength value to increase significantly compared to Mix3 as shown by the p-value of 0.002. Test results of Mix2, Mix3 and Mix4 emphasize the role of proper asphalt content to obtain better or at least adequate shear strength values.

The effect of aggregate gradation on shear properties is investigated by comparing the mean shear strength values of Mix2, Mix5 and Mix6 (all have the same asphalt content of 5.0 %). The t-test results show that using coarser aggregate (as of Mix5) yields a higher

Table 6.6: Summary of t-Test Results for Shear Strength of Limestone Mixes

Variable 1	Variable 2	Equal variances	t-Statistic	t-Critical	p-Value
Mix2 (0.880)	Mix3 (0.468)	no	2.974	2.571	0.031
Mix2 (0.880)	Mix4 (0.851)	no	0.224	2.777	0.834
Mix4 (0.851)	Mix3 (0.468)	yes	5.019	2.447	0.002
Mix5 (0.969)	Mix2 (0.880)	no	0.718	3.182	0.525
Mix2 (0.880)	Mix6 (0.642)	no	1.784	2.777	0.149
Mix5 (0.969)	Mix6 (0.642)	no	5.851	2.777	0.004
Mix2 (0.880)	Mix1 (0.821)	yes	0.375	2.447	0.721
Mix7 (1.285)	Mix1 (0.821)	yes	3.143	2.365	0.016

mean shear strength value compared to Mix2. However the p-value of 0.525 shows that the difference is insignificant. In addition, too coarse aggregate (as of Mix6) causes the mean shear strength value to decrease significantly compared to that of Mix5 as indicated by the p-value of 0.004. However, the decrease is insignificant compared to Mix2 (p-value = 0.149).

The combined effect of both aggregate gradation and proper asphalt content on the shear properties of airfield mixes is shown in the last two rows in the table. Using coarser aggregates in Mix2 along with its optimum asphalt content yields insignificant increase in

shear strength value compared to that of Mix1 as shown by the p-value of 0.721. However, the increase in shear strength value is significant when comparing shear strength values of Mix7 and Mix1. This significance is demonstrated by the resulted p-value of 0.016.

6.2.6. Significance of Shear Properties of Field Mixes

Table (6.7) presents the t-test results for significance of shear strength values for field mixes. The test results presented in this table provide the needed data to investigate the effect of aggregate type on shear properties of airfield mixes. In addition, the capability of the torque machine to rank airfield mixes based on their shear properties is studied utilizing these data. Based on their mean shear strength values, the field mixes are ranked as Mix7, Mix9 and finally Mix8 (built of dolomitic sandstones). For tests at room temperature, the resulted p-values of 0.004, 0.010, and 0.464 suggest that the differences between mean shear strength values are significant except when comparing shear strength of Mix8 and Mix9. At 35°C, the differences are significant except when comparing Mix7 and Mix9 (p-value = 0.205). Finally at 45°C, all differences are not significant. The results presented in Table (6.7) suggest that, at relatively higher temperatures, different asphalt concrete mixes behave similarly as they lose a large amount of their shear properties. In addition, Mix8 (built of dolomitic sandstones) does not show any benefit in terms of shear properties improvement compared to Mix7 of limestone aggregates.

6.2.7. Variation of Indirect Tensile Strength with AC and Aggregate Gradation

Table (6.8) presents the t-test results to study the effect of asphalt content and aggregate gradation on indirect tensile strength of airfield mixes. For cores extracted from slabs, using

Table 6.7: Summary of t-Test Results for Shear Strength of Field Mixes

Temp. (°)	Variable 1	Variable 2	Equal variances	t-Statistic	t-Critical	p-Value
25	Mix7 (1.285)	Mix8 (0.748)	yes	3.970	2.306	0.004
	Mix7 (1.285)	Mix9 (0.839)	yes	3.345	2.306	0.010
	Mix9 (0.839)	Mix8 (0.748)	yes	0.769	2.306	0.464
35	Mix7 (0.868)	Mix8 (0.563)	no	3.180	2.447	0.019
	Mix7 (0.868)	Mix9 (0.735)	no	1.457	2.571	0.205
	Mix9 (0.735)	Mix8 (0.563)	yes	3.477	2.306	0.008
45	Mix7 (0.510)	Mix8 (0.444)	yes	0.574	2.306	0.582
	Mix7 (0.510)	Mix9 (0.502)	yes	0.058	2.306	0.955
	Mix9 (0.502)	Mix8 (0.444)	yes	0.627	2.306	0.548

the optimum asphalt content in Mix2 increases the indirect tensile strength compared to lower and higher asphalt content ratios in Mix3 and Mix4, respectively. The p-values of 0.014 and 0.004 indicate that these increases are significant. In addition, although Mix4 of higher asphalt content (6.0 %) yields higher mean indirect tensile strength value compared to that of Mix3 (asphalt content of 4.5 %), the increase in indirect tensile strength is insignificant as indicated by the p-value of 0.793.

Table 6.8: Summary of t-Test Results for ITS of Limestone Mixes

Property	Variable 1	Variable 2	Equal variances	t-Statistic	t-Critical	p-Value
ITS for cores extracted from slabs	Mix2 (0.680)	Mix3 (0.559)	yes	3.145	2.306	0.014
	Mix2 (0.680)	Mix4 (0.568)	yes	4.018	2.306	0.004
	Mix4 (0.568)	Mix3 (0.559)	no	0.274	2.447	0.793
	Mix5 (0.790)	Mix2 (0.680)	yes	3.126	2.306	0.014
	Mix2 (0.680)	Mix6 (0.567)	yes	3.650	2.306	0.007
	Mix5 (0.790)	Mix6 (0.567)	yes	6.639	2.306	0.000
	Mix1 (0.808)	Mix2 (0.680)	yes	3.686	2.306	0.006
	Mix1 (0.808)	Mix7 (0.655)	yes	3.029	2.306	0.016
ITS for dry cores compacted using Marshall compactor	Mix2 (0.988)	Mix3 (0.805)	no	1.130	4.303	0.376
	Mix2 (0.988)	Mix4 (0.727)	no	1.525	3.182	0.225
	Mix3 (0.805)	Mix4 (0.727)	yes	0.859	2.777	0.439
	Mix2 (0.988)	Mix5 (0.814)	no	1.089	4.303	0.390
	Mix2 (0.988)	Mix6 (0.618)	no	2.284	4.303	0.150
	Mix5 (0.814)	Mix6 (0.618)	yes	2.964	2.777	0.041
	Mix2 (0.988)	Mix1 (0.947)	no	0.267	4.303	0.814
	Mix7 (1.225)	Mix1 (0.947)	no	2.825	2.777	0.048
ITS for conditioned cores compacted using Marshall compactor	Mix2 (0.951)	Mix3 (0.459)	no	2.165	4.303	0.163
	Mix2 (0.951)	Mix4 (0.718)	no	1.018	4.303	0.416
	Mix4 (0.459)	Mix3 (0.459)	no	7.013	3.182	0.006
	Mix2 (0.951)	Mix5 (0.845)	no	0.443	3.182	0.688
	Mix2 (0.951)	Mix6 (0.605)	no	1.509	4.303	0.270
	Mix5 (0.845)	Mix6 (0.605)	no	2.641	3.182	0.078
	Mix2 (0.951)	Mix1 (0.903)	no	0.209	4.303	0.854
	Mix7 (1.266)	Mix1 (0.903)	no	3.958	2.447	0.008

The effect of aggregate gradation on indirect tensile strength of airfield mixes is studied by comparing indirect tensile strength values of Mix2, Mix5, and Mix6. Using coarser aggregates in Mix5 causes indirect tensile strength to increase significantly (p-value = 0.014) compared to Mix2. However, too coarse aggregates (in Mix6) cause the indirect tensile strength values to decrease significantly compared to Mix5 (p-value = 0.0) and Mix2 (p-value = 0.007). The t-test results for mixes of proper mix design show that Mix2 of coarse aggregate and optimum asphalt content of 5.0 % results in significantly lower indirect tensile strength compared to that of Mix1. The p-value is 0.006. Same behavior is noticed by comparing indirect tensile strength values of Mix7 and Mix1 (p-value = 0.016).

For dry Marshall compacted cores, although Mix2 results in higher mean indirect tensile strength value compared to those of Mix3 and Mix4, all differences are insignificant at $\alpha = 5.0\%$. For Mix2, Mix5, and Mix6, the difference between indirect tensile strength of Mix5 and Mix2 is insignificant as indicated by the p-value of 0.390. The coarsest laboratory mix, Mix6, causes the indirect tensile strength value to decrease significantly when compared to that of Mix5 (p-value = 0.041) and insignificantly when compared to indirect tensile strength of Mix2 (p-value = 0.150). Comparing mixes of optimum asphalt content ratios, (Mix1, Mix2, and Mix7), Mix2 and Mix7 (both built of coarser aggregates compared to Mix1) yield higher mean indirect tensile strength values compared to that Mix1. The increase is significant for Mix7 (p-value = 0.048) while it is insignificant for Mix2 (p-value = 0.814).

For conditioned Marshall compacted cores, Mix2 results in insignificant increase in mean indirect tensile strength value compared to those of Mix3 and Mix4 as indicated by the

p-values of 0.163 and 0.416, respectively. On the other hand, the p-value of 0.006 shows that Mix4 of higher asphalt content causes mean indirect tensile strength value to increase significantly compared to that Mix3 of lower asphalt content. For Mix2, Mix5, and Mix6, all differences between mean indirect tensile strength values are insignificant at $\alpha = 5.0\%$. Similar to dry samples, Mix7 yields significantly higher value of indirect tensile strength compared to Mix1 (p-value of 0.008). However, the difference between indirect tensile strength values of Mix2 and Mix1 is insignificant (p-value = 0.854).

6.2.8. Significance of ITS Results of Field Mixes

Table (6.9) shows the results of t-tests performed to compare between field mixes based on their indirect tensile strength. The table shows that, for dry samples, Mix7 yields insignificantly higher indirect tensile strength compared to that of Mix8 (p-value = 0.080). The differences between indirect tensile strength values of Mix7 and Mix9, and Mix8 and Mix9 are also insignificant as indicated by the p-values of 0.206 and 0.365, respectively.

For conditioned sample, Mix7 results in significant increase in indirect tensile strength values compared to those of Mix8 and Mix9. The significance of increases is determined by the p-values of 0.014 and 0.018, respectively. Similar to dry samples, the difference between indirect tensile strength values of Mix8 and Mix9 is insignificant as indicated by p-value of 0.947. The t-test results discussed in this section demonstrate that Mix8, of dolomitic sandstones, does not add any benefit to improve indirect tensile strength of airfield mixes.

Table 6.9: Summary of t-Test Results for ITS of Field Mixes

Property	Variable 1	Variable 2	Equal variances	t-Statistic	t-Critical	p-Value
ITS for dry cores	Mix7 (1.225)	Mix8 (0.979)	yes	2.003	2.306	0.080
	Mix7 (1.225)	Mix9 (1.067)	no	1.419	2.447	0.206
	Mix9 (1.067)	Mix8 (0.979)	yes	0.960	2.306	0.365
ITS for conditioned cores	Mix7 (1.266)	Mix8 (0.934)	yes	3.143	2.306	0.014
	Mix7 (1.266)	Mix9 (0.927)	yes	2.985	2.306	0.018
	Mix8 (0.934)	Mix9 (0.927)	yes	0.069	2.306	0.947

6.3. Relating Surface Friction to Mix Properties

This section presents the development of an analytical model to predict surface friction evaluated in the laboratory to the basic asphalt mix properties. Based on the test results presented in Chapter (5), it was found that aggregate gradation explained a large amount of variability in the measured surface friction. In addition, there were good to fair correlations between the measured friction and Bulk relative density (BRD), Av and VMA. Independent variables were chosen such that they might be available in any job mix formula. They included aggregate gradation and BRD. Aggregate gradation was contributing in terms of percentage passing different sieves used to characterize aggregate size of airfield surface mixes. These sieves were introduced in Table (4.3). As Av and VMA are usually calculated as functions of BRD, the latter was chosen to represent the physical properties of asphalt mixes. Data presented in Tables (B.10) to (B.17) and Tables (C.2) to (C.9) were used to develop the model. Linear regression analysis was performed utilizing the statistical software *SPSS version 11.0*. Many transformations and combinations of the independent

variables were performed and included in the model. The model is developed using the stepwise method and the following three equations are obtained:

$$F = 117.856 - 0.662 \times (BRD \times P_{0.425}), \quad (6.1)$$

$$R^2 = 0.654$$

$$F = 126.183 - 0.609 \times (BRD \times P_{0.425}) - 2.638 P_{0.075}, \quad (6.2)$$

$$R^2 = 0.751$$

$$F = 100.995 - 0.347 \times (BRD \times P_{0.425}) - 4.807 P_{0.075} + 0.648 \times (BRD \times R_{2.00}), \quad (6.3)$$

$$R^2 = 0.917$$

where:

F = friction in LOD (BPN)

BRD = Bulk relative density (gm/cm³)

P_{0.425} = Percentage passing sieve 0.425 mm (%)

P_{0.075} = Percentage passing sieve 0.075 mm (%)

R_{2.00} = Percentage passing sieve 4.75 mm and retained on sieve 2.00 mm (%)

Equation (6.3) is chosen to predict friction of airfield surfaces from the basic mix properties as it has the highest R^2 value of 0.917, which indicates that a large amount of the variation in friction could be explained by the independent variables included in the model. In addition, aggregate gradation, in terms of percentage passing most sieve sizes used to characterize aggregate gradation for airfield mixes, according to the Canadian Specification (ASG-06, 1996), is included in that equation. The signs of all terms in Equation (6.3) are rational where there is an inverse relationship between percent passing a specific sieve and the gained surface friction. The negative sign of the second term and the positive sign of the fourth term in the proposed model suggest that the aggregate gradation is more important than the BRD. When aggregate gradation is introduced in terms of percentage passing, the sign of the product ($BRD \times P_{0.425}$) is negative to match the inverse relationship between the friction and the percentage passing. When the aggregate gradation is introduced in terms of percentage retained, the sign of the product ($BRD \times R_{2.00}$) is positive to match the direct relationship between friction and percentage retained.

ITS was evaluated in this research as an indicator for the mechanical properties of mixes under study. Figure (5.29) showed the relationship between friction values and ITS. This figure along with the developed model presented in Equation (6.3) can be used to predict the friction of airfield mixes from the basic information available in any job mix formula. Further than that, pavement engineers can utilize Figure (5.29) to evaluate mechanical properties of candidate mixes prior to construction.

6.4. Relating Field Friction to Laboratory Friction

6.4.1. Effect of Temperature on Friction of Airfield Surfaces

This section aims to develop an analytical model predicting field friction from laboratory friction values without any site visits. The laboratory friction was evaluated at the room temperature while the field friction was measured at a wide range of temperature. To relate both frictions, it was important to investigate the effect of temperature on the measured surface friction of airfield pavements. To achieve that, four small slabs of $250 \times 250 \times 75$ mm for each of Mix7 and Mix8 were tested for their friction using the BPT. The test temperatures were 35°C, room temperature (25°C), 15°C, 5°C, -10°C, -20°C and -30°C. The temperature was controlled utilizing an environmental chamber in the Civil and Environmental laboratories, Carleton University. Tables (E.1) to (E.20), Appendix E, present the friction test results at different temperatures for Mx7 and Mix8.

For temperatures below 0°C, a special fluid was used to conduct the friction test. To adjust friction values for fluid usage, the above-mentioned slabs were tested once using the fluid at temperatures of 35°C, 25°C, 15°C and 5°C. These measurements are also introduced in Tables (E.1) to (E.4) for Mix7 and Tables (E.11) to (E.14) for Mix8. At each temperature, the ratio between average friction measured using water and average friction measured utilizing the fluid was calculated. Then the correction factor for fluid adjustment (CF_f) was calculated as the average of these ratios. The CF_f for Mix7 is found to be 0.13 at all temperatures. For Mix8, CF_f is 0.13, 0.14, 0.14, and 0.14 for temperatures of 35°C, 25°C,

15°C and 5°C, respectively. The average CF_f is found to be 0.14. Then CF_f values are used to adjust friction values measured at -10°C, -20°C, and -30°C.

Figures (6.1) and (6.2) show that the friction decreases linearly as the temperature increases. For the ease of analysis and use, the linear relationships shown in Figures (6.1) and (6.2) are preferable. In spite of the fact that nonlinear relationships will produce higher R^2 values, the obtained R^2 values using the linear relationships are acceptable. These relationships could be represented by the following equations for Mix7 and Mix8, respectively;

$$F = 73.874 - 0.2624T \quad (6.4)$$

$$R^2 = 0.89$$

and,

$$F = 75.991 - 0.3208T \quad (6.5)$$

$$R^2 = 0.9373$$

The *SPSS version 11.0* software was used to develop an applicable model for both mixes.

The developed model is represented by;

$$F = 73.957 - 0.292T + 1.950M, \quad (6.6)$$

$$R^2 = 0.910$$

where:

F = friction (BPN)

T = temperature (°C)

Figure 6.1: Effect of Temperature on Frictional Properties, Mix7

Figure 6.2: Effect of Temperature on Frictional Properties, Mix8

M = dummy variable representing the mix type, (M = 0 for Mix7 and 1 for Mix8)

The data presented in Tables (E.1) to (E.20) were used to develop a correction factor for temperature adjustment. Friction values were normalized by calculating the ratio between friction at a given temperature and the corresponding friction measured at room temperature. Based on the normalized friction values, the following relationships are obtained for Mix7 and Mix8, respectively;

$$CF_t = 1.1158 - 0.0040T, \quad (6.7)$$

$$R^2 = 0.9054$$

and

$$CF_t = 1.1376 - 0.0048T, \quad (6.8)$$

$$R^2 = 0.9264$$

The relationships of Equation (6.7) and Equation (6.8) are shown in Figures (6.3) and (6.4), respectively.

For both mixes, *the SPSS version 11.0 software* was used to develop the following equation,

$$CF_t = 1.117 - 0.00439T + 0.01938M, \quad (6.9)$$

$$R^2 = 0.910$$

where:

CF_t = correction factor for temperature adjustment

T = temperature (°C)

Figure 6.3: Temperature Correction Factor, Mix7

Figure 6.4: Temperature Correction Factor, Mix8

M = dummy variable representing the mix type, (M = 0 for Mix7 and 1 for Mix8)

6.4.2. Correlation between Field Friction and Laboratory Friction

The two developed correction factors were used to adjust field friction results for both temperature and fluid effects. Tables (E.21) and (E.22), Appendix E, present the adjusted field friction values for Taxiway “F” and Taxiway “B”. These tables and Tables (5.5) and (5.6) were used to establish relationships between laboratory friction and the adjusted field friction measured approximately at the same time. Figure (6.5) shows that for both mixes, the relationships can be represented by a second-degree polynomial or S-curve. The R^2 values of 0.9232 and 0.9469 demonstrate the very good correlation between laboratory and field friction values. However, the relationships were found to be linear after a few months.

Figure (6.6) shows this behavior where laboratory friction is plotted versus the field friction after ignoring the initial measured friction values. The figure shows that, after a few months field friction can be predicted from laboratory one according to the following equations for Mix7 and Mix8, respectively;

$$F_f = -74.134 + 2.0112F_l, \quad (6.10)$$

$$R^2 = 0.8726$$

and

$$F_f = -95.223 + 2.2426F_l, \quad (6.11)$$

$$R^2 = 0.9272$$

Figure 6.5: Correlation between Field and Laboratory Friction Values

Figure 6.6: Correlation between Field and Laboratory Friction Values, Initial Measurements Excluded

where:

F_f = field friction in LOD (BPN)

F_l = laboratory friction in LOD (BPN)

Figures (6.5) and (6.6) suggest that there is a strong correlation between friction evaluated in laboratory and friction measured in the field.

6.4.3. Models Relating Field Friction to Laboratory Friction

6.4.3.1. Model (1)

The work was continued to develop a model capable of predicting field friction from the laboratory one. The *SPSS software* was used to relate field friction to laboratory friction. In this level of analysis all measured friction data after applying the CF_f were used as the dependent variable while the independent variables included time (t) and field temperature (T). The following models are developed for Taxiway “F”, Taxiway “B”, and for both of them, respectively;

$$F_f = 86.785 - 1.231t - 0.540T, \quad (6.12)$$

$$R^2 = 0.897$$

$$F_f = 87.097 - 1.725t - 0.457T, \quad (6.13)$$

$$R^2 = 0.860$$

$$F_f = 87.886 - 1.408t - 0.506T - 2.720M, \quad (6.14)$$

$$R^2 = 0.869$$

where:

F_f = field friction in LOD (BPN)

t = time (month)

T = temperature ($^{\circ}\text{C}$)

M = dummy variable representing the mix type, ($M = 0$ for Mix7 and 1 for Mix8)

The signs of all developed models are logical where both time and temperatures are found change in opposite direction to the surface friction (as t and T increases, F_f decreases). On the other hand, Equations (6.12) and (6.13) show that Taxiway “B” (built of dolomitic sandstones) is more affected by time compared to Taxiway “F” (built of limestone aggregates) as indicated by the parameters of 1.725 and 1.231, respectively.

Figures (6.7) and (6.8) show the change of laboratory friction with time. These relationships are represented as follows;

For Mix7;

$$F_l = 79.159 - 1.0088t \quad (6.15)$$

or

$$t = 78.4685 - 0.9913F_l \quad (6.16)$$

$$R^2 = 0.9258$$

and for Mix8

$$F_l = 80.706 - 1.1996t \quad (6.17)$$

Figure 6.7: Change of Laboratory Friction with Time, Mix7

Figure 6.8: Change of Laboratory Friction with Time, Mix8

or

$$t = 67.2774 - 0.8336F_l \quad (6.18)$$

$$R^2 = 0.9167$$

For both mixes, the *SPSS software* has been used to develop the following equation;

$$F_l = 79.790 - 1.103t + 0.302M \quad (6.19)$$

$$R^2 = 0.914$$

However, the variable M is not significant as its p-value is found to be 0.629. The variable has been removed and the following model is obtained

$$F_l = 79.953 - 1.105t \quad (6.20)$$

or

$$t = 72.3557 - 0.9050F_l \quad (6.21)$$

$$R^2 = 0.913$$

Equations (6.16), (6.18), and (6.21) are used to substitute t into Equations (6.12), (6.13), and (6.14), respectively and the following models are obtained;

For Mix7;

$$F_f = -9.8097 + 1.2203F_l - 0.540T \quad (6.22)$$

for Mix8;

$$F_f = -28.9565 + 1.4380F_l - 0.457T \quad (6.23)$$

and for both mixes;

$$F_f = -13.9908 + 1.2742F_l - 0.506T - 2.720M \quad (6.24)$$

6.4.3.2. Model (2)

The second model to relate field friction to laboratory one was developed using the data introduced in Tables (5.5 and 5.6) and Tables (E.21 and E.22). The independent variables are field temperature and laboratory friction. All data used to develop this model are the average values of the original measured data. Using *SPSS software*, the following relationships are obtained;

For Mix7

$$F_f = -3.018 + 1.127F_l - 0.532T \quad (6.25)$$

$$R^2 = 0.942$$

and for Mix8;

$$F_f = -21.474 + 1.335F_l - 0.526T \quad (6.26)$$

$$R^2 = 0.945$$

In addition to these models, data for both mixes are used to develop one model applicable for both mixes as follows;

$$F_f = -12.443 + 1.258F_l - 0.527T - 3.409M \quad (6.27)$$

$$R^2 = 0.942$$

The models presented in this section provide a good tool to predict frictional properties on the actual airfield pavement surfaces without any site visit. This may give an advantage especially for countries of severe winters such as Canada. It should be noted that these models are valid to estimate the friction in the first year after construction. It is expected that friction values will not change significantly after the first year. However these observations should be validated through future research work.

6.5. Relating Shear Properties Evaluated using Different Methods

Shear strength of asphalt surfaces is very important component to indicate the potential of rutting. While the shear properties of soils are well known, to date the shear properties of asphalt mixtures have not been accurately measured. The shear properties are evaluated in this thesis using three different techniques; torque machine, direct shear test facility and triaxial test facility. For most cases, they successfully ranked the airfield mixes in the same orders. As was mentioned in Chapter (5), for the direct shear test, shear stresses were determined at normal stresses of 0.6895 MPa, 1.0342 MPa, and 1.3790 MPa. To relate shear properties evaluated using different methods, shear stresses were calculated using the shear envelopes determined by the triaxial test at the above-mentioned normal stresses. Table (D.4), Appendix D, presented a summary for the data used in this phase of study.

6.5.1. Relating Shear Properties Evaluated by Triaxial Test and Direct Shear Test

Figures (6.9) and (6.10) show the relationship between triaxial test results and direct shear test results. These figures illustrate that there is a linear relationship between shear properties evaluated utilizing the two techniques. The data shown in Tables (D.4) and (5.15)

Figure 6.9: Correlation between Triaxial and Direct Shear Test Results, Field Mixes

Figure 6.10: Correlation between Triaxial and Direct Shear Test Results, Mix7

were analyzed using the *SPSS software* to develop a mathematical model relating the two test results to each other. The dependent variable was chosen to be the shear stress evaluated the triaxial test (SST). Independent variables included shear stress evaluated by direct shear test (SSD), normal stress (NS), test temperature (T), and mix type. Mix type entered to the model through two dummy variables M_1 and M_2 . The first dummy variable, M_1 , took a value of 1 for Mix7 and a value of 0 for other mixes. Similarly, M_2 took a value of 1 for Mix8 and a value of 0 for other mixes. First, the analysis was performed considering all independent variables. Based on this analysis, SSD was found to be insignificant as indicated by a p-value of 0.838. As a result, the stepwise analysis is performed to develop the following model;

$$SST = 0.735 + 1.024SSD, \quad (6.28)$$

$$R^2 = 0.877$$

where:

SST = shear stress evaluated by triaxial test (MPa)

SSD = shear stress evaluated by direct shear test (MPa)

The developed model shows that triaxial test yields higher shear stress values compared to the direct shear test. In addition, the relationship between shear properties evaluated by triaxial test and those evaluated by direct shear test does not depend on either mix type or test temperature.

6.5.2. Relating Shear Properties Evaluated by Triaxial Test and Torque Machine

Data introduced in Table (D.4) and (5.13) were used to develop a model relating SST to SS evaluated by the torque machine. The dependent variable was SST while independent variables included SS, NS, T and mix type (M_1 and M_2). Linear regression analysis including all independent variables was performed using *SPSS software* and it was found that SS and M_1 were insignificant. As a result, the stepwise analysis is performed to develop the following models;

$$SST = 1.6 + 0.348SS - 0.0204T + 0.979NS + 0.081M_2, \quad (6.29)$$

$$R^2 = 0.999$$

and

$$SST = 1.384 + 0.450SS - 0.0167T + 0.979NS, \quad (6.30)$$

$$R^2 = 0.996$$

where:

SST = shear stress evaluated by triaxial test (MPa)

SS = shear strength evaluated by torque machine (MPa)

T = test temperature (°C)

NS = normal stress (MPa)

M_1 = dummy variable representing the mix type, ($M_1 = 1$ if Mix7 and 0 if not Mix7)

M_2 = dummy variable representing the mix type, ($M_2 = 1$ if Mix8 and 0 if not Mix8)

As the variable M_2 in Equation (6.29) causes R^2 value to increase by 0.003 only, Equation (6.30) is chosen to relate between shear properties evaluated by triaxial test and torque machine. Because there are very good relationships between (SST and SSD), and (SST and SS), no such relationship is attempted between SSD and SS.

The assumptions of the linear regression analysis, for all developed models in this thesis, were checked by plotting the residuals versus the fitted values. All plots showed random scatter of points about zero and they did not follow any specific pattern. Figure (F.1), Appendix F, shows an example of the residual plots. This figure was obtained by the regression analysis performed to develop the model presented in Equation (6.3).

6.6. Summary

This chapter presented the statistical analysis for the results of experimental investigation and provided some mathematical models utilizing the test results and measured data from the field. The statistical analysis suggested that coarse aggregate could improve the frictional properties of airfield mixes without compromising other mechanical properties. Caution should be paid, however, through the process of mix design to choose sufficient AC to be used with the coarser aggregate gradations. Too coarse aggregates may result in poor mechanical properties if there is lack of sufficient AC. On the other hand, surface friction was found to increase as the aggregate size increased until the friction reached its maximum value. Beyond this value, any increase in aggregate size did not result in friction improvement.

The prediction models presented in this chapter were developed to: (1) relate friction of airfield mixes to the main mix properties, (2) relate the friction measured on the actual airfield pavements to the laboratory evaluated friction, and (3) relate shear properties evaluated using different shear test facilities. Based on the statistical analysis and developed models, the conclusions and recommendations of this thesis are presented in Chapter (7).

CHAPTER 7

7. CONCLUSIONS AND RECOMMENDATIONS

7.1. Introduction

The main objectives of this thesis were defined as follows:

1. To carry out a thorough literature review to identify the main parameters governing the characteristics of surface friction of airport asphalt pavements,
2. To plan and implement a research plan based on experimental and statistical studies to quantify the main variables affecting surface friction,
3. To examine the possibility of improving the frictional characteristics as well as the mechanical properties of airfield mixes, and
4. To develop guidelines for the design of improved airfield mixes with higher quality both in terms of safety and performance.

In order to meet these objectives, comprehensive experimental and analytical investigations including field and laboratory testing programs were performed. A total number of thirty-four asphalt slabs were constructed and tested for their frictional properties during the experimental investigation phase of this study. The slabs were prepared from nine different airfield asphalt mixes and six loose aggregates. Field measurements were taken on two field taxiway surfaces at OIA. In addition to slab samples, a total of 239 core specimens were prepared in the laboratory. Of them 205, core specimens were tested for their

mechanical properties. Test results and field data were used to develop the prediction models as presented and discussed in the analytical investigation phase.

The topic of this thesis is a unique one in terms of the following features:

1. The research is a pioneer study in the field of safety of airfield pavements and the consideration of the surface texture and surface friction as the main contributing factors to safer pavements,
2. This is the first time that a large-scale research investigation on Canadian airfields is completed with the three basic research elements are covered; laboratory, field and analytical investigations.
3. This is the first attempt to investigate the shear properties of asphalt mixes using three different testing techniques. Furthermore, it is also the first attempt at relating the shear properties of airfield mixes to surface friction.

Guidelines for proper selection of safer and better airfield asphalt mixes have been established based on the results and conclusions of this thesis. The results of the laboratory and field investigations were utilized to set up the bases of these guidelines. The guidelines include the equations and models estimating the properties of the mix and the measured surface friction. The following sections summarise the main conclusions and recommendations of this thesis.

7.2. Conclusions

7.2.1. Conclusions Based on the Experimental Investigation

The observations and results of the laboratory and field investigations support the following conclusions:

- It has been shown that surface friction of airport mixes could be quantified both in the field and the laboratory utilizing simple measuring devices.
- The initial friction in TRD is significantly higher than that in LOD. This phenomenon diminishes over a short period of time.
- Using coarser aggregates can improve frictional properties of airfield pavement surfaces. In most cases, this improvement is accompanied with better shear properties and other mechanical properties. However in few cases, using coarse aggregate does not enhance the mechanical properties.
- There is a good correlation between frictional characteristics and shear properties of airfield asphalt mixes. Subsequently, one can also conclude that improving the rutting resistance may result in developing higher friction properties when considered as a design criterion.
- Use of relatively excessive coarse aggregate can enhance frictional properties but it compromises other mechanical properties for airfield pavements. The analysis indicates the presence of an optimum value after which increasing the proportion of coarse aggregates may actually reduce the desired properties.

- Improving the surface friction is not necessarily a pre-requisite for better mechanical properties. Although glass has a relatively high tensile strength, it yields too low friction value compared to loose aggregates of zero tensile strength.
- Using the optimum asphalt content has been shown to be a key factor in obtaining asphalt mixes with high surface friction. It can enhance frictional properties of airfield pavements and improve or at least preserve mechanical properties of airfield mixes.
- Using dolomitic sandstone aggregates instead of limestone aggregates does not provide long-term benefits in terms of improving frictional properties or mechanical properties. In fact the results showed that it did not enhance surface friction, and it resulted in lower mechanical properties. This conclusion emphasized the role of laboratory friction evaluation prior to construction. However, due to the fact that only one type of dolomitic sandstones was used in this investigation, the results and conclusions presented in this thesis regarding the effect of aggregate type need to be confirmed through further investigations.
- The field and laboratory data showed that both temperature and ageing have significant effects on the measured surface friction. This conclusion is very important to plan and manage future landing and take-off operations during warm and cold seasons.
- The friction measurements and the shear properties of the base asphalt mix (Mix9) were shown to be the best when compared with those of other surface mixes. This is

important since the use of base mixes could be considered as a starting point for developing better surface ones.

7.2.2. Conclusions Based on the Prediction Models

In addition to the results of the experimental investigations, the data were utilized to develop several prediction models relating frictional characteristics to physical and mechanical properties of airfield asphalt mixes. The developed statistical models in this thesis support the following conclusions:

- Aggregate gradation is found to be the most important factor influencing the friction of airfield mixes compared to the effect of their physical properties. The analytical investigation provided a statistically based model to predict surface friction from the basic mix properties available in any job mix formula in terms of aggregate gradation and bulk relative density.
- The model can be used along with a developed relationship between friction and indirect tensile strength to check the mechanical properties of the airfield mixes prior to construction.
- Friction of pavement surface is shown to be temperature dependent, it is important for comparison purposes to perform the test at the same temperature. The work presented in this thesis resulted in master curves for correction factors to adjust the measured friction for temperature variation.
- Laboratory friction and field friction evaluated using the procedures presented in this thesis were highly correlated to each other. The work completed in the analytical

investigation provides some statistical models to be used by pavement engineers to predict field friction from the laboratory one without any site visit.

- Models relating shear properties evaluated by different test techniques were developed through the analytical investigation. These models could be used to evaluate shear properties confidently using any available shear test facility.

7.3. Recommendations and Future Research Work

Based on the test results and conclusions of the research work presented in this thesis, it is recommended to:

- Review the Canadian specifications and update them to incorporate higher percentage of coarser aggregates in airfield surface courses.
- More research work needs to be performed to evaluate other asphalt mixes. Specifically, the improvement obtained from the base mix and further consideration to upgrade such a mix to be utilized for runway surfaces.
- The results presented in this thesis showed that there are limitations to achieve higher friction surfaces through the development of new asphalt mixes as discussed in Chapter (4). This conclusion should be further investigated with the main objective of studying the techniques of changing surface friction through mechanical means as the ones utilized in creating better frictional characteristics on the PCC surfaces.

For future research, the work presented in this thesis could be used to provide the basic and fundamental groundwork to build a new body of knowledge that is essential for the safe and economic use of airfield pavements. Furthermore, while the main thrust of the

thesis deals with asphalt materials, the most important application of the results of the thesis could be the proper operation and better planning of landing and take-off of airplanes with the possible elimination of the occurrence of serious type of airport accidents.

8. REFERENCES

- Abdul-Malak, M.A.U., Meyer, A.H. and Fowler, D.W., "Research Program for Predicting the Frictional Characteristics of Seal-Coat Pavement Surfaces", Transportation Research Record, No. 1217, pp. 53-64, Transportation Research Board, Washington, D.C., 1989.
- Abd El Halim, A.O., "Design, Development and Validation of an Advanced In-Situ Shear Stiffness Test Facility for Asphalt Concrete Pavements", NCHRP IDEA # 55 Final Report, For IDEA Orogram, Transportation Research Board, Washington D.C., April 2001.
- Abd El Halim, A.O., " Measurement of In-Situ Shear Strength of Asphalt Pavements", Final Report submitted to the Ministry of Transport of Ontario, Toronto. June 2001.
- Abd El Halim, A.O., Easa, S., Abdel Nabi, R. and Haas, R., "Effect of Highway Geometry on Transverse Cracking of Asphalt Pavements", ASCE Transportation Congress, San Diego, California, Vol. 1, pp. 651-663, 1995
- Abd El Halim, A.O., Zahw, M., Bahgat, A., and El Samny, M., " Influence of Types of Asphalt Binder on the Tensile and Shear Strengths of Asphalt Mixes", Proceedings, Canadian Technical Asphalt Association, Vol. 40, pp. 291-304, Charlottetown, PEI, 1995.
- Abd El Halim, A.O., Rickards, I., Haas, R.C., and Abd El Nabi, R.M., "The Development and Use of In-Situ Shear Test Device for Evaluation of Asphalt

Performance", 8th ISAP International Conference on Asphalt Pavements, Vol. II, pp.1311-1328, Seattle, Washington, D.C., 1997.

- Abd El Halim, A.O., Abdel Nabi, R. and Abd El Aleem, A., "Determination of Shear Strength of Asphalt Layers in the Field", Proceedings of the 5th International Conference on the Bearing Capacity of roads and Airfields, Vol. II, pp. 1183- 1196, Trondheim, Norway, 1998.
- Abd El Nabi, R.M, "Influence of Highway Geometry on Distress of Asphalt Concrete Pavements", Ph.D. thesis, Zagazig University, Egypt, 1995.
- Abd El Nabi, R.M., Abd El Halim, A.O., Easa, S.M., and Takeichi, K., "Verification of the Role of Road Geometry on Distress of Asphalt Pavements Using the New In-Situ Shear Test Machine", Proceedings, Canadian Technical Asphalt Association, Vol. 40, pp. 387-402, Charlottetown, PEI, 1995.
- Al-Abdul Wahhab, H.I., "A Laboratory Study of Asphalt Mix Improvement Techniques and Test Methods to Assess Rutting Potential", Road and Transport Research, Vol. 3, No. 4, pp. 78-90, 1994.
- Al-Abdul Wahhab, H.I. and Balghonaim, F., "Asphalt Pavement Temperature Related to Arid Saudi Environment", Journal of Materials in Civil Engineering, Vol. 6, No. 1, pp. 1-14, 1994.
- Al-Dhalaan, M.A., Khan, S.S., Farouki, T.A., and Farwana, T.A., "A Comprehensive Approach to the Rutting Problem in the Kingdom of Saudi Arabia", Asphalt Paving Technology, Journal of the Association of Asphalt Paving Technologists, Vol. 59, pp. 560-583, 1990.

- Andrey, J., Mills, B., and Vandermolen, J., “A Temporal Analysis of Weather-Related Collision Risk for Ottawa, Canada: 1990-1998”, Transportation Research Board, 2003 Annual Meeting, National Research Council, Washington DC, 2003.
- Antvik, G., “History of Friction Measurement-Airport Surface Friction Tester AB”, <http://www.asft.se/history.html>, 1997, accessed 05/03/2003.
- Archilla, A.R. and Madanat, S., “Development of a Pavement Rutting Model from Experimental Data”, Journal of Transportation Engineering, Vol. 126, No. 4, pp. 291-229, 2000.
- Archilla, A.R. and Madanat, S., “Statistical Model of Pavement Rutting in Asphalt Concrete Mixes”, Transportation Research Record, No. 1764, pp. 70-77, Transportation Research Board, Washington D.C., 2001.
- ASG-06, Pavement Construction: Materials and Testing, Canadian Standards and Recommended Practices, Airport Engineering, Public Works and Government Services Canada, 1996.
- ASTM, Designation: C 127-88, “Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate”. Annual Book of ASTM Standards, Section 4, Construction, Vol. 04.02, Concrete and Aggregates, pp. 78-82, 1996.
- ASTM, Designation: C 136-95a, “Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates”, Annual Book of ASTM Standards, Section 4, Construction, Vol. 04.02, Concrete and Aggregates, pp. 78-82, 1996.

- ASTM, Designation: C 702-93, “Standard Test Method for Reducing Samples of Aggregate to Testing Size”, Annual Book of ASTM Standards, Section 4, Construction, Vol. 04.02, Concrete and Aggregates, pp. 350-353, 1996.
- ASTM, Designation: D 546-94, “Standard Test Method for Sieve Analysis of Mineral Filler for Road and Paving Materials”, Annual Book of ASTM Standards, Section 4, Vol. 04.03, Road and Paving Materials; Vehicle-Pavement System, pp. 81-82, 1996.
- ASTM, Designation: D 1559-89, “Standard Test Method for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus”, Annual Book of ASTM Standards, Section 4, Vol. 04.03, Road and Paving Materials; Vehicle-Pavement System, pp. 137-142, 1996.
- ASTM, Designation: D 2041-95, “Standard Test Method for Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures”, Annual Book of ASTM Standards, Section 4, Vol. 04.03, Road and Paving Materials; Vehicle-Pavement System, pp. 181-187, 1996.
- ASTM, Designation: D 2726-90, “Standard Test Method for Bulk Specific Gravity and Density of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens”, Annual Book of ASTM Standards, Section 4, Vol. 04.03, Road and Paving Materials; Vehicle-Pavement System, pp. 244-246, 1996.
- ASTM, Designation: D 3203-94, “Standard Test Method Percent Air Voids in Compacted Dense and Open Bituminous Paving Mixtures”, Annual Book of ASTM

Standards, Section 4, Vol. 04.03, Road and Paving Materials; Vehicle-Pavement System, pp. 285-286, 1996.

- ASTM, Designation: D 4123-82, Re-approved 1995, “Standard Test Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixtures”, Annual Book of ASTM Standards, Section 4, Vol. 04.03, Road and Paving Materials; Vehicle-Pavement System, pp. 423-425, 1996.
- ASTM, Designation: D 4867-92, “Standard Test Method for Effect of Moisture on Asphalt Concrete Paving Mixtures”, Annual Book of ASTM Standards, Section 4, Vol. 04.03, Road and Paving Materials; Vehicle-Pavement System, pp. 499-504, 1996.
- ASTM, Designation: E 274-90, “Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire”, Annual Book of ASTM Standards, Section 4, Vol. 04.03, Road and Paving Materials; Vehicle-Pavement System, pp. 652-656, 1996.
- ASTM, Designation: E 303-93, “Standard Test Method for Measuring Surface Frictional Properties Using the British Pendulum Tester”, Annual Book of ASTM Standards, Section 4, Vol. 04.03, Road and Paving Materials; Vehicle-Pavement System, pp. 657-661, 1996.
- ASTM, Designation: E 670-94, “Standard Test Method for Side Force Friction on Paved Surfaces Using the Mu-Meter”, Annual Book of ASTM Standards, Section 4, Vol. 04.03, Road and Paving Materials; Vehicle-Pavement System, pp. 685-694, 1996.

- ASTM, Designation: E 965-87, Re-approved 1995, "Standard Test Method for Measuring Surface Macrottexture Depth Using a Volumetric Technique", Annual Book of ASTM Standards, Section 4, Vol. 04.03, Road and Paving Materials; Vehicle-Pavement System, pp. 707-709, 1996.
- Atkins, W.S., "Modified Mixes Used to Combat Rutting", Highways and Transportation, Vol. 43, No. 12, pp. 11-12, 1996.
- Bean, D. and Pidwerbesky, B., "Does the Polish Stone Value Test Assure Skid Resistance?", Road and Transport Research, Vol. 3, No. 2, pp. 84-92, 1994.
- Beaver, B.M. and Beaver, R.J., "Introduction to Probability & Statistics", Tenth Edition, Brooks/Cole Publishing Company, ITP Inc., Pacific Grove, CA, USA, 1999.
- Bekheet, W., "Investigation of in-Situ Shear Properties of Asphalt Concrete" Ph.D. thesis, Carleton University, Canada, 2002.
- Bekheet, W., Hassan, Y., and Abd El Halim, A.O., "Modeling in-Situ Shear Strength Testing of Asphalt Concrete Pavements Using the Finite Element Method", Canadian Journal of Civil Engineering, Vol. 28, pp. 541-544, 2001.
- Bowes, W.H., Russel, L.T., and Suter, G.T., "Mechanics of Engineering Materials", BRS Publications, pp. 299, 1990.
- Brown, S.F., "Practical Test Procedures for Mechanical Properties of Bituminous Materials", Proceedings of the Institution of Civil Engineers Transport, Vol. 111, pp. 289-297, 1995.

- Brown, S.F. and Cooper K.E., “The Mechanical Properties of Bituminous Materials for Road Bases and Base Courses”, *Asphalt Paving Technology, Proceedings of the Association of Asphalt Paving Technologists*, Vol. 53, pp. 415-437, 1984.
- Brown, S.F. and Gibb, J.M., “ Validation Experiment for Permanent Deformation Testing of Bituminous Mixtures”, *Asphalt Paving Technology, Journal of the Association of Asphalt Paving Technologists*, Vol. 65, pp. 255-290, 1996.
- Brown, S.F., and Scholz, T.V., “Permanent Deformation Characteristics of Porous Asphalt Determined in the Confined Repeated Load Axial Test”, *Highways and Transportation*, Vol. 45, No. 12, pp. 7-10, 1998.
- Browne, A.I., and Whicker, D., "An Interactive Tire-Fluid Model for Dynamic Hydroplaning", In *Frictional International of Tire and Pavement*, ASTM STP 793 (W.E. Meyer and J.D. Walter, Eds.), American Society for Testing and Materials, Philadelphia, Pa., pp. 130-150, 1993.
- Cenek P.D., Fong, S., and Donbavand, J., “New Zealand’s Experience in Applying the International Friction Index to Coarse Textured Road Surfaces” *World Road Congress, 21st*: Kuala Lumpur, 1999.
- Chelliah, T., Stephanos, P., Shah, M.G, and Smith, T., “Developing a Design Policy to Improve Surface Characteristics”, *Transportation Research Board, 2003 Annual Meeting*, National Research Council, Washington D.C., 2003.
- Cooley, L.A., Brown, E.R., and Watson D.E., “Evaluation of Open-Graded Friction Course Mixtures Containing Cellulose Fibers” In *Transportation Research Record*,

- No. 1723, Transportation Research Board, National Research Council, Washington D.C., pp. 19-25, 2000.
- Dahir, S.H., Meyer, W.E., and Hegmon, R.R., “Laboratory and Field Investigation of Bituminous Pavements and Aggregate Polishing”, Transportation Research Record, No. 584, Transportation Research Board, National Research Council, Washington, D.C., pp. 4-14, 1976.
 - Descornet, G., “A Criterion for Optimizing Surface Characteristics”, Transportation Research Record, No. 1215, Transportation Research Board, National Research Council, Washington, D.C., pp. 173-177, 1989.
 - Deshpande, V.S. and Cebon, D., “Uniaxial Experiments on Idealized Asphalt Mixes”, Journal of Materials in Civil Engineering, Vol. 12, No. 3, pp. 262-271, 2000.
 - Do, M.T., Kerzreho, J.P., Balay, J.M., and Gothie, M., “Full Scale Tests for the Assessment of Wear of Pavement Surfaces”, Transportation Research Board, 2003 Annual Meeting, National Research Council, Washington D.C., 2003.
 - Eisenmann, J., and Hilmer, A., “Influence of Wheel Load and Inflation Tire Pressure on The Rutting Effect at Theoretical Investigation”, 6th International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, 1987.
 - El-Desouky, A., “Response of Flexible Pavements under Various Wheel Loads” Master thesis, MTC, Egypt, 1998.

- Epps, J., Monismith, C.L., Seeds, S.B., Alavi, S.H., Ashmore, S.C., Leahy, R., and Mitchell, T.M., “Westrack Performance-Interim Findings”, Journal of the Association of Asphalt Paving Technologists, Proceedings of Asphalt Paving Technologists, St. Paul, Minnesota, Vol. 67, pp. 738-777, 1998.
- FAA, Federal Aviation Administration, “Open-Graded Asphalt Emulsion Mixes”, Engineering Brief # 23, January 9, 1981.
- FAA, Federal Aviation Administration, “Measurement, Construction, and Maintenance of Skid-Resistant Airport Pavement Surfaces”, Advisory Circular, U.S. Department of Transportation, 1997.
- Flintsch, G.W., de Leon, E., McGhee, K.K., and Al-Qadi, I.L., “Pavement Surface Macrottexture Measurement and Application”, Transportation Research Board, 2003 Annual Meeting, National Research Council, Washington D.C., 2003.
- Forster, S.W., "Aggregate Microtexture Profile Measurements and Related Frictional Levels", Report FHWA-RD 81-107, US Department of Transportation, 1981.
- Forster, S.W., "Pavement Microtexture and Its Relation to Skid Resistance", Transportation Research Record, No. 1215, Transportation Research Board, National Research Council, Washington, D.C., pp. 151-164, 1990.
- Fulop, I.A, Bogardi, I., Gulyas, A., Csicsely-Tarpay, M., “Use of Friction and Texture in Pavement Performance Modeling”, Journal of Transportation Engineering, Vol. 126, No. 3, May/June, 2000.

- Gargett, T., “The Introduction of a Skidding-Resistance Policy in Great Britain”, In Symposium: Surface Characteristics of Roadways, ASTM, Philadelphia, pp. 30-38, 1990.
- Goodman, S.N., “Design, Development and Validation of the in-Situ Shear Stiffness Test (INSISSTTM) Facility for Asphalt Concrete Pavements”, Master thesis, Carleton University, Canada, 2000.
- Goodman, S.N., Hassan, Y., and Abd El Halim, A.O., “Shear Properties as Viable Measurement for Characterization of Permanent Deformation of Asphalt Concrete Mixtures”, Transportation Research Board, 2002 Annual Meeting, National Research Council, Washington, D.C., 2002.
- Harald, A., "Skid Resistance and Road Surface Texture", In Symposium: Surface Characteristics of Roadways, ASTM, Philadelphia, pp. 5-13, 1990.
- Hardwood, D.W., Blackburn, R.R., Jhon, A.D. and Sharp, M.C., "Evaluation of Accident Rate-Skid Number Relationships for Nationwide Sample of Highway Sections", Transportation Research Record, No. 624, Transportation Research Board, National Research Council, Washington, D.C., pp. 142-150, 1976.
- Harvey, J., Guada, I., and Long, F., “ Effects of Material Properties, Specimen Geometry, and Specimen Preparation Variables on Asphalt Concrete Tests for Rutting”, Asphalt Paving Technology, Journal of the Association of Asphalt Paving Technologists, Vol. 69, pp. 236-268, 2000.
- Harvey, J. and Popescu, L., “Accelerated Pavement Testing of Rutting Performance of Two Caltrans Overlay Strategies”, Transportation Research Record, No. 1716,

Transportation Research Board, National Research Council, Washington, D.C., pp.116-125, 2000.

- Hatherly, L.W. and Young, A.E., “The Location and Treatment of Urban Skidding Hazard Sites”, Transportation Research Record, No. 623, Transportation Research Board, National Research Council, Washington, D.C., pp. 21-28, 1976.
- Hegmon, R.R., “Tire-Pavement Interaction”, Public Roads, Vol. 51, No. 1, pp. 5-11, June 1987.
- Henry, J.J., "Tyre Wet-Pavement Traction Measurement: A State-of-The-Art Review", Proceedings of a Symposium on the Tyre-Pavement Interface, Baltimore: ASTM Special Technical Publication 929, Baltimore, pp. 3-25, 1986.
- Henry, J.J., “Evaluation of Pavement Friction Characteristics”, NCHRP Synthesis 291, Transportation Research Board, National Research Council, Washington D.C., 2000.
- Higdon, A., Ohlsen, E.H., Stiles, W.B., Weese, J.A., and Riley, W.F., “Mechanics of Materials”, Jhon Wiley & Sons Inc., New York, Fourth Edition, 1985.
- Hosking, J.R., "Relationship between Skidding Resistance and Accident Frequency: Estimates Based on Seasonal Variations", TRRL Report RR76, Department of Transport, Crowthorne, UK, 1987.
- Huber, G., “Methods to Achieve Rut-Resistant Durable Pavements”, NCHRP Synthesis 274, Transportation Research Board, National Research Council, Washington D.C., 1999.

- Instron, "Operating Instruction for the Instron Universal Testing Instruments Floor Models TTD", Standard, Low Speed, Metric, Manual 10-70-1 (A), 1967.
- Jayawickrama, P.W. and Graham, G.L., "Use of Skid Performance History as Basis for Aggregate Qualification for Seal Coats and Hot Mix Asphalt Concrete Surface Courses," Transportation Research Record, No. 1501, Transportation Research Board, National Research Council, Washington, D.C., pp. 31-38, 1995.
- Jayawickrama, P.W., Prasanna, R., and Senadheera, S.P., "Survey of State Practices to Control Skid Resistance on Hot-Mix Asphalt Concrete Pavements", Transportation Research Record, No. 1536, Transportation Research Board, National Research Council, Washington, D.C., pp. 52-58, 1996.
- Kandhal, P.S., and Mallick, R.B., "Open-Graded Friction Course: State of the Practice", Transportation Research Circular, No. E-C005, Transportation Research Board, National Research Council, Washington, D.C., December 1998.
- Kennedy, C.K., Young, A.E., and Butler, I.C., "Measurement of Skidding Resistance and Surface Texture and the Use of Results in the United Kingdom", In Symposium: Surface Characteristics of Roadways, ASTM, Philadelphia, 1990, pp. 87-102.
- Kokkalis, A.G., "Prediction of Skid Resistance from Texture Measurements", Proceedings of the Institution of Civil Engineers-Transport, Vol. 129, No. 2, pp. 85-93, 1998.
- Kummer, H.W. and Meyer, "NCHRP Synthesis of Highway Practice 37: Tentative Skid-Resistance Requirements for Main Rural Highways", Highway Research Board, National Research Council, Washington, D.C., 1967.

- Leu, M.C., and Henry, J.J., “Prediction of Skid Resistance as a Function of Speed from Pavement Texture”, Transportation Research Record, No. 666, Transportation Research Board, National Research Council, Washington D.C., pp. 7-13, 1978.
- Meyer, W.E., “Pavement Texture Significance and Measurement”, ASTM Standardization News, pp. 28-31, 1991.
- Monismith, C.L. and Tayebali, A.A., “Permanent Deformation (Rutting) Considerations in Asphalt Concrete Pavement Sections”, Asphalt Paving Technology, Proceedings of the Association of Asphalt Paving Technologists, Vol. 57, pp. 414-446, 1988.
- Moore, D.F., “Design of Texture in Standard Surfaces: Surface Texture versus Skidding; Measurements, Frictional Aspects, and Safety Features of Tire-Pavement Interactions, ASTM STP 583, American Society for Testing and Materials, Philadelphia, Pa., pp. 62-74, 1975.
- Moshansky, P.J., Commission of Inquiry into the Air Ontario Crash at Dryden, Ontario, Final Report, Ottawa, Ontario, 1992.
- NASA, National Aeronautics and Space Administration, “Runway Friction Index”, <http://www.hq.nasa.gov/office/aero/library/chicago/runway.htm>, October 2000, accessed 04/04/2003.
- Nikolaidis, A., “Rutting and Volumetric Properties of SMA Mixtures”, Proceedings of the Institution of Civil Engineers–Transport, Vol. 141, No. 3, pp. 135-141, 2000.

- Norheim, A., Sinha, N.K., and Yager, T.J., “Effects of Structure and Properties of Ice and Snow on the Friction of Aircraft Tyres on Movement Area Surfaces”, *Journal of Tribology International*, Vol. 34, Issue 9, pp. 617-623, September 2001.
- NRC, “Triaxial Compressive Strength Test with Servo Computer Control Press”, *Standard Operating Procedures*, National Resources Canada, 2001.
- Oliver, J.W.H. and Tredrea, P.F., “Relationships between Asphalt Rut Resistance and Binder Rheological Properties”, *Asphalt Paving Technology, Journal of the Association of Asphalt Paving Technologists*, Vol. 67, pp. 623-643, 1998.
- Panagouli, O.K., and Kokkalis, A.G., “Skid Resistance and Fractal Structures of Pavement Surface”, *CHAOS Solitons and Fractals*, Vol. 9, No. 3, pp. 493-505, 1998.
- PIARC World Road Association, “International PIARC Experiment to Compare and Harmonize Texture and Skid Resistance Measurements”, *Report, PIARC Technical Committee on Surface Characteristics C.1*, 1995.
- PWGSC, *Private Communications with Public Works and Government Services Canada Personnel*, (2001).
- Reissner, E. and Sagoci, H., “Forced Torsional Oscillation of an Elastic Half-Space”, *Journal of Applied Physics*, Vol. 15, pp. 652-654, 1944.
- Rogers, C., Gorman, B., and Lane, B., “Skid-Resistant Aggregates in Ontario”, *Report, Materials Engineering and Research Office, Ministry of Transportation of Ontario, Canada*, (2003)
- Roque, R., Dominguez, G., and Romero, P., “Effect of Asphalt Mixture Characteristics and Design on Frictional Resistance of Bituminous Wearing Course

Mixtures", Transportation Research Record, No. 1507, Transportation Research Board, National Research Council, Washington, D.C., pp. 39-50, 1995.

- RTAC, Roads and Transportation Association of Canada, Pavement Management Committee, "Pavement Management Guide," 1977.
- Schonfeld, R., "Photo-Interpretation of Skid Resistance", Highway Research Record, No. 311, pp. 11-25, 1970.
- Schonfeld, R., "Photo-Interpretation of Pavement Skid Resistance in Practice", Transportation Research Record, No. 523, Transportation Research Board, National Research Council, Washington, D.C., pp. 95-75, 1974.
- Shatnawi, S., "Performance of Asphalt Rubber Mixes in California", International Journal of Pavement Engineering, Vol. 2, No. 1, pp. 1-16, 2001.
- Shergold, F.A., "A Study of the Variability of Roadstones in the Relation to Sampling Procedures", Quarry Managers Journal, Vol. 47, No. 1, pp. 3-8, 1963.
- Strategic Highway Research Program, SHRP, "Summary Report on Permanent Deformation of Asphalt Concrete", SHRP-A/IR-91-104, Asphalt Research Program, University of California, Berkeley, 1991.
- Stroup-Gardiner, S.M, and Brown, E.R, "Segregation of Hot Mix Asphalt Pavements", NCHRP Report 441, Transportation Research Board, National Research Council, Washington D.C., 2000.
- Stuart, K.D. and Mogawer, W.S., "Validation of Asphalt Binder and Mixture Tests that Predict Rutting Susceptibility Using the FHWA ALF", Asphalt Paving

Technology, Journal of the Association of Asphalt Paving Technologists, Vol. 66, pp. 109-143, 1997.

- Sybilski, D., "Relationship between Absolute Viscosity of Polymer-Modified Bitumens and Rutting Resistance of Pavement", Materials and Structures, Vol. 27, No. 166, pp. 110-120, 1994.
- Tabachnick, B.G. and Fidell L.S., "Using Multivariate Statistics", Fourth Edition, Allyn & Bacon, A Pearson Education Company, Needham Heights, MA, 2001.
- Thenoux, G., Allen, W., and Bell, C.A., "Study of Aircraft Accident Related to Asphalt Runway Skid Resistance", Transportation Research Record, No. 1536, Transportation Research Board, National Research Council, Washington, D.C., pp. 59-63, 1996.
- Wambold, J.C., "Obtaining Skid Number at any Speed from Test at Single Speed", Transportation Research Record, No. 1196, Transportation Research Board, National Research Council, Washington, D.C., pp. 300-305, 1988.
- Wambold, J.C., "Road Characteristics and Skid Testing", Transportation Research Record, No. 1196, Transportation Research Board, National Research Council, Washington, D.C., pp. 294-299, 1988.
- Wambold, J.C. and Kulakowski, B.T., "Limitations of Using Skid Number in Accident Analysis and Pavement Management", Transportation Research Record, No. 1311, Transportation Research Board, National Research Council, Washington, D.C., pp. 43-51, 1991.

- Wambold, J.C., Henry, J.J., and Hegmon, R.R, "Skid Resistance of Wet Weather Accident Site", In the Tire Pavement Interface, ASTM ATP 929, M.G. Pottinger and T.J. Yager, Eds., American Society for Testing and Materials, Philadelphia, Pa., pp. 47-60, 1986.
- Watson, D.E., Moore, K.A., Williams, K., and Cooley, L.A., "Refinement of New Generation Open-Graded Friction Course Mix Design" Transportation Research Board, 2003 Annual Meeting, National Research Council, Washington D.C., 2003.
- Weissman, S., Harvey, J., and Long, F., "Asphalt Concrete Laboratory Test and Specimen Dimensions Selection Based on Mechanical Constraints", Proceedings of the 12th Engineering Mechanics Conference, ASCE, La Jolla, California, pp. 1130-1133, 1998.
- Wessex Skid Tester, "Operating Instruction", Version 4.0, Wessex Engineering Limited, 2001.
- Whitlow, R., "Basic soil mechanics", Second Edition, Longman Group, UK Limited, 1990.
- Yager, T.J., Wambold, J.C., Henry, J.J., Anderson, A., and Bastain, M., "Joint Winter Runway Program Accomplishments", the Virginia Department of Transportation and Virginia Tech Pavement Evaluation 2002 Conference, October 21-25, Roanoke, Virginia, 2002.
- Zahw, M.A, "Development of Testing Framework for Evaluation of Rutting Resistance of Asphalt Mixes", Ph.D. thesis, Carleton University, Canada, 1995.

APPENDIX A:

MIX DESIGN

Table A.1: Bulk relative density, maximum relative density, air voids and voids in mineral aggregate for Mix1, 4.5% AC

		Briquette number			
		1	2	3	Average
A	Weight of compacted specimen in air	1131.16	1135.23	1159.78	
B	Surface dry weight of specimen after immersion in water	1132.00	1135.63	1160.10	
C	Weight of compacted specimen in water	660.96	678.80	685.96	
D	Volume = B - C	471.04	456.83	474.14	
E	Bulk relative density = A / D	2.401	2.485	2.446	2.44
F	Weight of flask & mixture in air	1656.90		1791.70	
G	Weight of flask in air	651.00		647.30	
H	Weight of mixture in air = F - G	1005.90		1144.40	
I	Weight of flask & mixture in water	1185.80		1267.40	
J	Weight of flask in water	575.35		573.82	
K	Weight of mixture in water = I - J	610.45		693.58	
L	Volume = H - K	395.45		450.82	
M	Maximum relative density = H / L	2.544		2.538	
N	Air voids % = 100*(M-E)/M				3.98
O	VMA % = 100-(E (100-AC%))/G _{Agg} .				15.02

Table A.2: Bulk relative density, maximum relative density, air voids and voids in mineral aggregate for Mix1, 5% AC

		Briquette number			
		1	2	3	Average
A	Weight of compacted specimen in air	1278.37	1203.29	1197.99	
B	Surface dry weight of specimen after immersion in water	1278.96	1203.44	1198.53	
C	Weight of compacted specimen in water	767.00	725.61	719.12	
D	Volume = B - C	511.96	477.83	479.41	
E	Bulk relative density = A / D	2.497	2.518	2.499	2.50
F	Weight of flask & mixture in air	1651.2		1616.10	
G	Weight of flask in air	651.00		647.30	
H	Weight of mixture in air = F - G	1000.20		968.80	
I	Weight of flask & mixture in water	1181.50		1165.60	
J	Weight of flask in water	575.35		573.82	
K	Weight of mixture in water = I - J	606.15		591.78	
L	Volume = H - K	394.05		377.02	
M	Maximum relative density = H / L	2.538		2.570	
N	Air voids % = $100 \times (M - E) / M$				2.11
O	VMA % = $100 - (E (100 - AC\%)) / G_{Agg.}$				13.39

Table A.3: Bulk relative density, maximum relative density, air voids and voids in mineral aggregate for Mix1, 5.5% AC

		Briquette number			
		1	2	3	Average
A	Weight of compacted specimen in air	1239.97	1324.72	1277.34	
B	Surface dry weight of specimen after immersion in water	1240.18	1324.81	1277.45	
C	Weight of compacted specimen in water	746.98	799.67	770.55	
D	Volume = B - C	493.2	525.14	506.9	
E	Bulk relative density = A / D	2.514	2.523	2.52	2.52
F	Weight of flask & mixture in air		1707.00	1673.60	
G	Weight of flask in air		645.40	647.30	
H	Weight of mixture in air = F - G		1061.60	1026.30	
I	Weight of flask & mixture in water		1218.50	1198.00	
J	Weight of flask in water		574.86	573.82	
K	Weight of mixture in water = I - J		643.64	624.18	
L	Volume = H - K		417.96	402.12	
M	Maximum relative density = H / L		2.540	2.552	
N	Air voids % = 100*(M-E)/M				1.02
O	VMA % = 100-(E (100-AC%))/G _{Agg} •				13.15

Table A.4: Bulk relative density, maximum relative density, air voids and voids in mineral aggregate for Mix1, 6% AC

		Briquette number			
		1	2	3	Average
A	Weight of compacted specimen in air	1217.40	1179.62	1268.27	
B	Surface dry weight of specimen after immersion in water	1217.52	1179.78	1268.68	
C	Weight of compacted specimen in water	732.26	708.09	761.39	
D	Volume = B - C	485.26	471.69	507.29	
E	Bulk relative density = A / D	2.509	2.50	2.50	2.50
F	Weight of flask & mixture in air	1600.80		1613.10	
G	Weight of flask in air	651.00		647.30	
H	Weight of mixture in air = F - G	949.80		965.80	
I	Weight of flask & mixture in water	1147.70		1155.20	
J	Weight of flask in water	575.35		573.82	
K	Weight of mixture in water = I - J	572.35		581.38	
L	Volume = H - K	377.45		384.42	
M	Maximum relative density = H / L	2.516		2.512	
N	Air voids % = $100 \times (M - E) / M$				0.57
O	VMA % = $100 - (E (100 - AC\%)) / G_{\text{Agg.}}$				14.30

Table A.5: Bulk relative density, maximum relative density, air voids and voids in mineral aggregate for Mix1, 6.5% AC

		Briquette number			
		1	2	3	Average
A	Weight of compacted specimen in air	1194.39	1180.03	1220.99	
B	Surface dry weight of specimen after immersion in water	1194.60	1180.24	1221.15	
C	Weight of compacted specimen in water	714.81	705.41	732.37	
D	Volume = B - C	479.79	474.83	488.78	
E	Bulk relative density = A / D	2.489	2.485	2.498	2.49
F	Weight of flask & mixture in air	1677.60		1727.80	
G	Weight of flask in air	651.00		647.30	
H	Weight of mixture in air = F - G	1026.60		1080.50	
I	Weight of flask & mixture in water	1191.40		1221.30	
J	Weight of flask in water	575.35		573.82	
K	Weight of mixture in water = I - J	616.05		647.48	
L	Volume = H - K	410.55		433.02	
M	Maximum relative density = H / L	2.501		2.495	2.498
N	Air voids % = $100 \times (M - E) / M$				0.32
O	VMA % = $100 - (E (100 - AC\%)) / G_{Agg.}$				15.10

Table A.6: Bulk relative density, maximum relative density, air voids and voids in mineral aggregate for Mix2, 4.5% AC

		Briquette number			
		1	2	3	Average
A	Weight of compacted specimen in air	1247.44	1278.12	1268.13	
B	Surface dry weight of specimen after immersion in water	1247.64	1278.59	1268.83	
C	Weight of compacted specimen in water	727.97	746.25	740.02	
D	Volume = B - C	519.67	532.34	528.81	
E	Bulk relative density = A / D	2.40	2.40	2.40	2.40
F	Weight of flask & mixture in air	1621.20		1623.50	
G	Weight of flask in air	651.00		647.30	
H	Weight of mixture in air = F - G	970.20		976.20	
I	Weight of flask & mixture in water	1165.80		1161.30	
J	Weight of flask in water	575.35		573.82	
K	Weight of mixture in water = I - J	590.45		587.48	
L	Volume = H - K	379.75		388.72	
M	Maximum relative density = H / L	2.555		2.511	2.533
N	Air voids % = $100 \times (M - E) / M$				5.25
O	VMA % = $100 - (E (100 - AC\%)) / G_{Agg}$				16.11

Table A.7: Bulk relative density, maximum relative density, air voids and voids in mineral aggregate for Mix2, 5% AC

		Briquette number			
		1	2	3	Average
A	Weight of compacted specimen in air	1183.60	1234.91	1251.60	
B	Surface dry weight of specimen after immersion in water	1184.43	1235.83	1252.16	
C	Weight of compacted specimen in water	697.27	727.68	739.82	
D	Volume = B - C	487.16	508.15	512.34	
E	Bulk relative density = A / D	2.43	2.43	2.44	2.43
F	Weight of flask & mixture in air		1603.30	1618.30	
G	Weight of flask in air		645.40	647.30	
H	Weight of mixture in air = F - G		957.90	971.00	
I	Weight of flask & mixture in water		1150.10	1157.30	
J	Weight of flask in water		574.86	573.82	
K	Weight of mixture in water = I - J		575.24	583.48	
L	Volume = H - K		382.66	387.52	
M	Maximum relative density = H / L		2.503	2.506	
N	Air voids % = 100*(M-E)/M				2.97
O	VMA % = 100-(E (100-AC%))/G _{Agg.}				15.51

Table A.8: Bulk relative density, maximum relative density, air voids and voids in mineral aggregate for Mix2, 5.5% AC

		Briquette number			
		1	2	3	Average
A	Weight of compacted specimen in air	1195.57	1193.3	1286.08	
B	Surface dry weight of specimen after immersion in water	1195.97	1193.69	1286.34	
C	Weight of compacted specimen in water	704.19	708.84	757.04	
D	Volume = B - C	491.78	484.85	529.30	
E	Bulk relative density = A / D	2.431	2.461	2.428	2.44
F	Weight of flask & mixture in air	1674.10		1695.00	
G	Weight of flask in air	651.00		647.30	
H	Weight of mixture in air = F - G	1023.10		1047.70	
I	Weight of flask & mixture in water	1190.60		1200.70	
J	Weight of flask in water	575.35		573.82	
K	Weight of mixture in water = I - J	615.25		626.88	
L	Volume = H - K	407.85		420.82	
M	Maximum relative density = H / L	2.509		2.490	
N	Air voids % = 100*(M-E)/M				2.36
O	VMA % = 100-(E (100-AC%))/G _{Agg} .				15.61

Table A.9: Bulk relative density, maximum relative density, air voids and voids in mineral aggregate for Mix2, 6% AC

		Briquette number			
		1	2	3	Average
A	Weight of compacted specimen in air	1233.47	1218.83	1267.70	
B	Surface dry weight of specimen after immersion in water	1233.60	1219.02	1267.90	
C	Weight of compacted specimen in water	728.30	721.07	753.50	
D	Volume = B - C	505.30	497.95	514.40	
E	Bulk relative density = A / D	2.441	2.448	2.464	2.45
F	Weight of flask & mixture in air	1653.80	1764.50		
G	Weight of flask in air	651.00	645.40		
H	Weight of mixture in air = F - G	1002.80	1119.10		
I	Weight of flask & mixture in water	1172.00	1241.00		
J	Weight of flask in water	575.35	574.86		
K	Weight of mixture in water = I - J	596.65	666.14		
L	Volume = H - K	406.15	452.96		
M	Maximum relative density = H / L	2.469	2.471		2.470
N	Air voids % = $100 \times (M - E) / M$				0.80
O	VMA % = $100 - (E (100 - AC\%)) / G_{\text{Agg.}}$				15.71

Table A.10: Bulk relative density, maximum relative density, air voids and voids in mineral aggregate for Mix2, 6.5% AC

		Briquette number			
		1	2	3	Average
A	Weight of compacted specimen in air	1220.35	1234.10	1318.27	
B	Surface dry weight of specimen after immersion in water	1220.55	1234.27	1318.47	
C	Weight of compacted specimen in water	720.31	731.55	775.03	
D	Volume = B - C	500.24	502.72	543.44	
E	Bulk relative density = A / D	2.44	2.455	2.426	2.44
F	Weight of flask & mixture in air	1720.80	1692.50		
G	Weight of flask in air	651.00	645.40		
H	Weight of mixture in air = F - G	1069.80	1047.10		
I	Weight of flask & mixture in water	1210.50	1195.10		
J	Weight of flask in water	575.35	574.86		
K	Weight of mixture in water = I - J	635.15	620.24		
L	Volume = H - K	434.65	426.86		
M	Maximum relative density = H / L	2.461	2.453		2.457
N	Air voids % = $100 \cdot (M - E) / M$				0.70
O	VMA % = $100 - (E (100 - AC\%)) / G_{\text{Agg.}}$				16.50

**Table A.11: Marshall stability test results for Mix1
of different asphalt content ratios**

No.	AC (%)	Volume (cm ³)	Stability (kN)	Correlation ratio	Corrected Stability (kN)	Average
1	4.5	471.04	11.09	1.14	12.64	12.46
2		456.83	10.27	1.19	12.22	
3		474.14	10.52	1.19	12.52	
4	5.0	511.96	13.17	1.00	13.17	13.52
5		477.83	12.02	1.14	13.70	
6		479.41	12.02	1.14	13.70	
7	5.5	493.20	12.00	1.09	13.08	12.79
8		525.14	12.58	0.96	12.08	
9		506.90	12.70	1.04	13.21	
10	6.0	485.26	9.40	1.09	10.24	9.75
11		471.69	8.74	1.14	9.96	
12		507.29	8.70	1.04	9.05	
13	6.5	479.79	6.94	1.14	7.91	8.63
14		474.83	7.57	1.14	8.62	
15		488.78	8.58	1.09	9.35	

**Table A.12: Marshall stability test results for Mix2
of different asphalt content ratios**

No.	AC (%)	Volume (cm³)	Stability (kN)	Correlation ratio	Corrected Stability (kN)	Average
1	4.5	519.67	8.53	1.00	8.53	8.71
2		532.34	9.35	0.96	8.98	
3		528.81	8.97	0.96	8.61	
4	5.0	487.16	11.12	1.09	12.12	12.53
5		508.15	12.17	1.00	12.17	
6		512.34	13.29	1.00	13.29	
7	5.5	491.78	14.24	1.09	15.52	13.13
8		484.85	11.12	1.09	12.12	
9		529.30	12.24	0.96	11.75	
10	6.0	505.30	9.52	1.04	9.90	9.52
11		497.95	9.00	1.04	9.36	
12		514.40	9.30	1.00	9.30	
13	6.5	500.24	6.44	1.04	6.69	7.62
14		205.72	8.76	1.04	9.11	
15		543.44	7.60	0.93	7.07	

**Table A.13: Flow values from Marshall stability test for Mix1
of different asphalt content ratios**

No.	AC (%)	Flow (1/100 in.)	Average (1/100 in.)	Average (mm)
1	4.5	11.0	11.50	2.92
2		12.5		
3		11.0		
4	5.0	9.0	9.67	2.46
5		9.0		
6		11.0		
7	5.5	14.0	14.67	3.73
8		14.0		
9		16.0		
10	6.0	17.0	16.33	4.15
11		16.0		
12		16.0		
13	6.5	19.5	19.67	5.00
14		20.5		
15		19.0		

**Table A.14: Flow values from Marshall stability test for Mix 2
of different asphalt content ratios**

No.	AC (%)	Flow (1/100 in.)	Average (1/100 in.)	Average (mm)
1	4.5	10.0	11.00	2.79
2		8.0		
3		15.0		
4	5.0	9.0	8.00	2.03
5		8.0		
6		7.0		
7	5.5	12.0	12.00	3.05
8		12.0		
9		12.0		
10	6.0	14.0	13.00	3.30
11		12.0		
12		13.0		
13	6.5	15.0	16.67	4.23
14		17.0		
15		18.0		

Table A.15: Aggregate proportions by weight (%) to obtain the desired aggregate gradations

Gradation	Proportion by weight (%)					G_{Agg}
	HL4	HL3	Sand	Dust	M. Filler	
I	5.0	34.0	18.0	41.0	2.0	2.7421
II	17.5	33.0	30.0	19.0	0.5	2.7323
III	25.0	30.0	19.0	25.0	1.0	2.7427
IV	35.0	30.0	14.0	20.0	1.0	2.7481

Table A.16: Marshall mix design for Mix7 and Mix8

Design A/C %:	6.0	A/C Type:	58-34	
Mix Properties				
		Specification		
Marshall stability (kN@60°C)	14.9	9.00 (min.)		
Flow, mm	2.5	2-4 mm		
Air voids, %	3.5	3-5		
Voids in mineral aggregate, %	15.5	15 (min.)		
BRD (gm/cm ³)	2.396			
MRD (gm/cm ³)	2.484			
Retained stability, %	92	75.0 (min.)		
Aggregate Properties				
Designation:	C-1	F-1	F-2	F-3
Description:	HL-1	Reg. dust	Sand	Min. filler
Source:	Beaver	Beaver	Redmond	Beaver
Source No.:	05-67	05-67	K03-070	05-67
Specific Gravity:	2.649	2.767	2.657	2.739
Absorption, %:	0.875	0.785	1.020	
Proportion by wt.,	40.0	19.8	39.7	0.5
Sieve size (mm)	Gradation (% passing)			Job mix formula
16.0	100	100	100	100
13.2	96.1	100	100	100
9.5	67.4	100	100	100
4.75	7.8	92.6	98.1	100
2.36	2.2	61.6	94.8	100
1.18	1.9	40.4	88.0	100
0.600	1.5	28.6	70.4	100
0.300	1.4	21.0	44.0	100
0.150	1.2	16.1	10.3	100
0.075	1.1	11.7	4.0	100

* For Mix8, HL-1 stone was replaced with dolomitic sandstone.

Table A.17: Marshall mix design for Mix9

Design A/C %:	5.0	A/C Type:	58-34		
Mix Properties					
		Specification			
Marshall stability (kN@60°C)	12.9	10.00 (min.)			
Flow, mm	2.3	2-4 mm			
Air voids, %	4.1	3-5			
Voids in mineral aggregate, %	15.5	13 (min.)			
BRD (gm/cm ³)	2.423				
MRD (gm/cm ³)	2.527				
Retained stability, %	81	75.0 (min.)			
Aggregate Properties					
Designation:	C-1	C-3	F-1	F-2	F-3
Description:	HL-6 stone	HL-3	Reg.	Sand	Min. filler
Source:	Beaver		Beaver	Redmon	Beaver
Source No.:	05-67	05-67	05-67	K03-070	05-67
Specific Gravity:	2.759		2.657	2.666	2.739
Absorption, %:	0.666		0.841	0.855	
Proportion by wt.,	37.2	8.2	13.6	41.0	0.0
Sieve size (mm)	Gradation (% passing)			Job mix formula	
26.5	100	100	100	100	100
19.0	92.5	100	100	100	97.2
16.0	64.6	100	100	100	86.8
13.2	31.6	97.4	100	100	74.3
9.5	4.4	67.7	100	100	61.8
4.75	1.4	6.6	92.4	99.5	54.4
2.36	1.3	1.7	60.8	96.2	48.3
1.18	1.2	1.4	39.7	89.2	42.5
0.600	1.2	1.3	28.3	72.6	34.2
0.300	1.2	1.2	20.9	39.2	19.5
0.150	1.1	1.2	16.1	9.6	6.6
0.075	1.1	1.0	11.8	2.8	3.2

Figure A.1: Change of BRD with asphalt content for Mix1

Figure A.2: Change of air voids with asphalt content for Mix1

Figure A.3: Change of voids in mineral aggregate with asphalt content for Mix1

Figure A.4: Change of Marshall stability with asphalt content for Mix1

Figure A.5: Change of flow with asphalt content for Mix1

Figure A.6: Change of BRD with asphalt content for Mix2

Figure A.7: Change of air voids with asphalt content for Mix2

Figure A.8: Change of voids in mineral aggregate with asphalt content for Mix2

Figure A.9: Change of Marshall stability with asphalt content for Mix2

Figure A.10: Change of flow with asphalt content for Mix2

APPENDIX B:

PHYSICAL PROPERTIES

Table B.1: Dimensions and BRD for Mix1

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	63.5	101.5	1269.06	764.78	1269.34	2.515
2	62.5	101.5	1243.77	749.36	1244.17	2.514
3	58	101.6	1149.81	689.90	1150.23	2.498
4	62.5	101.7	1231.08	742.47	1231.86	2.516
5	61	101.7	120576	724.45	1206.42	2.502
6	61	101.8	122035	739.9	1220.73	2.538
7	62	101.6	1243.30	752.92	1243.45	2.535
8	59	101.6	1170.10	706.95	1170.67	2.523
9	61	101.5	1228.55	742.43	1228.87	2.526
10	59	101.7	1170.27	707.10	1170.64	2.525
Mean						2.519
STD						0.013

Table B.2: Dimensions and BRD for Mix2

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface Dry	
1	59.0	101.5	1142.98	656.10	1143.35	2.346
2	59.5	101.5	1168.49	673.88	1168.56	2.362
3	59.5	101.5	1125.96	643.98	1126.82	2.332
4	58.5	101.5	1123.53	643.7	1123.91	2.340
5	58.2	101.7	1123.52	644.03	1123.87	2.341
6	59.0	101.7	1141.54	657.46	1142.17	2.355
7	65.0	101.5	1246.37	712.38	1246.45	2.334
8	62.5	101.5	1215.77	697.30	1215.80	2.345
9	60.0	101.5	1150.83	660.39	1151.30	2.344
10	62.5	101.6	1217.01	698.92	1217.13	2.348
Mean						2.345
STD						0.009

Table B.3: Dimensions and BRD for Mix3

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	64.5	101.6	1239.20	737.42	1269.60	2.385
2	61.5	101.7	1201.33	690.56	1201.71	2.350
3	54.5	101.8	1070.43	617.61	1070.62	2.363
4	65.5	101.6	1265.79	726.06	1266.52	2.342
5	60.0	101.6	1158.97	668.06	1159.18	2.360
6	55.5	101.5	1077.2	621.33	1077.76	2.360
7	62.0	101.8	1198.67	690.71	1199.03	2.358
8	68.5	101.6	1337.16	770.26	1337.54	2.357
9	59.0	101.6	1136.27	654.52	1136.71	2.356
10	59.5	101.5	1159.00	669.2	1159.34	2.365
Mean						2.360
STD						0.011

Table B.4: Dimensions and BRD for Mix4

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	62.2	101.5	1206.41	709.63	1207.27	2.424
2	63.6	101.6	1229.75	712.28	1229.82	2.374
3	64.8	101.5	1259	731.12	1259.05	2.385
4	63.1	101.5	1222.86	708.82	1222.94	2.379
5	62.7	101.5	1214.62	703.35	1215.76	2.370
6	61.5	101.5	1189.12	697.58	1189.75	2.416
7	63.2	101.5	1233.2	723.51	1233.63	2.417
8	61.1	101.6	1199.16	702.01	1199.26	2.412
9	64.2	101.5	1255.58	735.33	1255.59	2.413
10	64.6	101.5	1257.97	728.12	1258.11	2.374
Mean						2.397
STD						0.022

Table B.5: Dimensions and BRD for Mix5

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	66.1	101.5	1311.16	773.80	1311.28	2.439
2	62.2	101.6	1234.27	725.09	1234.67	2.422
3	62.8	101.5	1251.44	739.72	1251.61	2.445
4	60.1	101.5	1186.74	694.44	1186.94	2.410
5	64.2	101.5	1261.98	741.18	1262.52	2.421
6	60.4	101.5	1206.67	719.84	1207.00	2.477
7	62.9	101.7	1254.18	744.5	1254.28	2.460
8	63.8	101.6	1266.42	745.22	1266.55	2.429
9	60.2	101.9	1193.58	704.42	1194.08	2.438
10	63.7	101.6	1253.53	734.88	1253.92	2.415
Mean						2.436
STD						0.021

Table B.6: Dimensions and BRD for Mix6

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	59.2	101.6	1172.03	692.87	1172.41	2.444
2	62.8	101.6	1222.11	720.54	1222.51	2.435
3	65.9	101.7	1315.18	774.65	1315.53	2.432
4	66.3	101.5	1293.73	764.88	1293.8	2.446
5	59.9	101.7	1188.92	707.36	1189.18	2.468
6	59.9	101.6	1168.35	692.37	1168.76	2.453
7	62.0	101.6	1237.88	735.41	1238.2	2.462
8	64.1	101.5	1260.97	741.72	1261.17	2.428
9	62.3	101.5	1239.89	734.76	1240.08	2.454
10	68.6	101.6	1352.9	800.23	1353.09	2.447
Mean						2.447
STD						0.013

Table B.7: Dimensions and BRD for Mix7

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	59.0	101.5	1149.70	661.80	1149.76	2.356
2	58.4	101.4	1140.78	661.59	1140.95	2.380
3	59.7	101.5	1145.63	653.04	1145.75	2.325
4	57.2	101.7	1110.10	636.42	1110.29	2.343
5	62.0	101.4	1192.30	683.10	1192.6	2.340
6	61.4	101.6	1187.92	682.23	1188.19	2.348
7	65.6	101.6	1262.49	719.50	1262.74	2.324
8	59.2	101.6	1155.51	666.31	1155.69	2.361
9	61.3	101.6	1178.62	673.29	1178.85	2.331
10	61.7	101.7	1206.83	694.04	1206.95	2.353
11	58.5	101.6	1126.91	644.28	1127.10	2.334
12	62.6	101.6	1216.43	697.76	1216.65	2.344
13	51.5	101.6	1000.40	573.59	1000.57	2.343
14	52.5	101.7	1014.71	581.07	1014.81	2.339
15	59.2	101.5	1141.41	653.26	1141.62	2.337
16	59.2	101.8	1141.82	649.39	1142.05	2.318
17	61.1	101.7	1169.43	663.38	1169.70	2.310
18	59.3	101.6	1151.99	660.99	1152.12	2.346
19	54.3	101.8	1036.70	590.28	1036.99	2.321
20	59.6	101.6	1154.09	663.70	1154.32	2.352
21	59.4	101.6	1148.41	655.96	1148.59	2.331
22	61.6	101.6	1185.41	675.53	1185.66	2.324
23	55.3	101.5	1062.71	607.88	1062.99	2.335
24	56.4	101.6	1074.90	611.31	1075.15	2.317
25	53.7	101.7	1042.26	598.54	1042.46	2.348
26	56.4	101.6	1078.91	614.06	1079.42	2.318
27	59.8	101.6	1132.35	640.55	1132.92	2.300
28	63.1	101.8	1202.20	681.59	1203.53	2.303
29	60.8	101.5	1167.00	663.10	1167.31	2.315
30	62.9	101.6	1201.24	682.62	1201.88	2.313
31	62.2	101.4	1206.50	692.75	1206.73	2.347
32	63.7	101.8	1242.23	713.46	1242.35	2.349
33	60.3	101.7	1155.32	655.46	1155.74	2.309
34	64.4	101.6	1234.51	698.59	1234.74	2.303
35	61.9	101.6	1215.83	701.97	1215.87	2.366
36	62.3	101.7	1195.67	680.34	1196.01	2.319

Table B.7: Dimensions and BRD for Mix7, continued

37	61.4	101.6	1205.16	697.20	1205.30	2.372
38	60.2	101.9	1166.61	664.55	1166.91	2.322
39	65.9	101.5	1282.31	736.95	1282.43	2.351
40	61.7	101.7	1191.28	679.68	1191.61	2.327
41	64.6	101.7	1235.21	700.90	1235.60	2.310
42	63.0	101.6	1214.56	692.68	1214.69	2.327
43	59.0	101.9	1139.61	649.77	1139.84	2.325
44	56.3	101.6	1095.96	629.93	1096.14	2.351
45	65.2	101.7	1267.80	726.45	1267.91	2.341
46	65.6	101.6	1279.89	736.88	1279.99	2.357
47	64.2	101.9	1244.20	709.06	1244.47	2.324
48	67.8	101.7	1304.10	740.14	1304.26	2.312
49	60.5	101.7	1173.58	674.09	1173.81	2.348
50	59.2	101.6	1143.47	653.66	1143.69	2.333
51	58.4	101.6	1131.95	651.05	1132.14	2.353
52	60.7	101.7	1166.20	664.73	1166.37	2.325
53	59.1	101.5	1123.24	635.51	1123.58	2.301
54	60.9	101.8	1166.98	662.26	1167.27	2.311
55	64.4	101.7	1230.14	696.25	1231.08	2.300
56	62.9	101.6	1185.83	665.89	1186.75	2.277
57	60.8	101.6	1153.30	650.79	1153.91	2.292
58	58.3	101.6	1129.10	648.23	1129.20	2.348
59	65.6	101.6	1247.28	704.43	1248.00	2.295
60	62.2	101.7	1184.71	670.58	1185.47	2.301
61	63.4	101.8	1213.53	689.59	1213.65	2.316
62	60.1	101.6	1164.02	666.59	1164.14	2.340
63	67.0	101.5	1280.19	727.25	1280.36	2.315
64	63.5	101.6	1224.27	698.95	1224.39	2.330
65	63.9	101.7	1228.21	700.60	1228.41	2.327
66	63.4	101.8	1228.77	702.24	1228.90	2.333
67	65.5	101.6	1270.24	728.40	1270.29	2.344
68	64.9	101.7	1256.72	718.20	1256.83	2.333
Mean						2.330
STD						0.020

Table B.8: Dimensions and BRD for Mix8

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	61.9	101.5	1179.35	671.47	1179.69	2.321
2	59.8	101.6	1142.44	651.77	1142.76	2.327
3	55.3	101.7	1059.46	603.15	1059.64	2.321
4	60.3	101.6	1152.60	657.85	1153.10	2.327
5	60.1	101.7	1152.78	657.23	1153.11	2.325
6	59.7	101.7	1132.93	642.17	1133.30	2.307
7	57.3	101.8	1094.17	623.19	1094.46	2.322
8	58.5	101.6	1106.26	624.38	1106.68	2.294
9	61.4	101.6	1174.25	670.07	1174.60	2.327
10	62.2	101.7	1190.12	677.53	1190.33	2.321
11	60.8	101.8	1172.54	670.91	1172.70	2.337
12	57.8	101.6	1098.19	622.40	1098.63	2.306
13	62.5	101.5	1185.97	677.07	1186.36	2.329
14	61.6	101.6	1170.68	663.29	1171.00	2.306
15	63.1	101.6	1194.26	676.49	1194.72	2.304
16	64.8	101.7	1245.26	714.84	1245.59	2.346
17	64.2	101.7	1224.98	700.32	1225.65	2.332
18	59.8	101.7	1135.07	642.38	1135.42	2.302
19	64.3	101.5	1218.97	690.55	1219.55	2.304
20	63.0	101.6	1203.44	687.07	1203.70	2.329
21	61.9	101.7	1170.98	664.48	1171.38	2.310
22	61.6	101.7	1194.25	688.49	1194.41	2.361
23	60.1	101.7	1152.63	660.79	1152.89	2.342
24	60.4	101.6	1144.29	650.78	1144.64	2.317
25	60.7	101.6	1153.75	655.01	1154.01	2.312
26	63.6	101.7	1219.77	699.91	1219.90	2.346
27	64.7	101.6	1237.90	711.19	1238.30	2.348
28	63.5	101.6	1206.28	685.55	1206.55	2.315
29	64.6	101.6	1242.78	713.41	1242.99	2.347
30	65.3	101.8	1243.04	707.73	1243.44	2.320
31	63.8	101.8	1199.19	678.29	1200.41	2.297
32	61.4	101.6	1166.40	668.27	1166.85	2.339
33	62.1	101.8	1193.34	685.38	1193.65	2.348

Table B.8: Dimensions and BRD for Mix8, continued

34	62.2	101.6	1199.34	693.38	1199.50	2.370
35	61.5	101.7	1176.34	675.03	1176.56	2.346
36	63.3	101.5	1219.89	706.12	1220.06	2.374
37	58.7	101.6	1127.02	651.04	1127.17	2.367
38	60.9	101.7	1168.31	671.69	1168.52	2.352
39	60.9	101.5	1163.13	672.17	1163.38	2.368
40	64.4	101.5	1235.77	701.67	1236.11	2.312
41	62.7	101.6	1190.87	679.92	1191.19	2.329
42	60.6	101.8	1154.98	659.53	1155.17	2.330
43	64.1	101.6	1234.78	702.96	1235.08	2.320
Mean						2.329
STD						0.021

Table B.9: Dimensions and BRD for Mix9

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	55.9	101.6	1080.52	632.63	1081.06	2.410
2	60.2	101.6	1156.28	671.73	1157.12	2.382
3	61.3	101.6	1191.50	694.88	1191.95	2.397
4	59.5	101.8	1132.23	656.19	1133.56	2.372
5	62.1	101.5	1189.04	694.90	1190.15	2.401
6	58.5	101.6	1121.17	652.32	1122.33	2.385
7	57.1	101.6	1100.08	641.29	1100.55	2.395
8	61.4	101.6	1170.06	686.50	1170.88	2.416
9	61.0	101.6	1164.22	687.48	1165.44	2.436
10	61.5	101.8	1160.39	683.08	1162.40	2.421
11	60.9	101.7	1149.60	678.91	1152.34	2.428
12	61.7	101.5	1166.23	687.71	1168.66	2.425
13	61.4	101.7	1158.60	686.81	1161.53	2.441
14	57.6	101.5	1097.61	648.07	1098.20	2.438
15	62.3	101.6	1198.00	710.20	1198.71	2.452
16	62.5	101.6	1188.30	701.92	1190.80	2.431
17	59.4	101.6	1138.47	676.31	1139.26	2.459
18	58.2	101.5	1113.85	663.16	1114.38	2.469
19	66.4	101.7	1272.40	754.87	1273.96	2.451
20	65.4	101.5	1246.73	738.83	1248.06	2.448
21	57.7	101.6	1087.28	647.45	1088.63	2.464
22	61.1	101.6	1168.66	694.52	1169.22	2.462
23	61.5	101.7	1172.98	692.05	1174.69	2.430
24	62.7	101.6	1197.03	711.31	1199.13	2.454
25	63.0	101.6	1213.35	723.80	1214.33	2.474
26	60.8	101.8	1176.07	701.19	1176.78	2.473
27	62.5	101.5	1204.07	720.57	1205.14	2.485
28	62.7	101.6	1202.56	717.16	1204.23	2.469
29	65.3	101.5	1262.25	758.59	1263.09	2.502
30	64.5	101.5	1244.07	739.35	1244.80	2.461
31	58.4	101.7	1140.01	688.18	1140.46	2.521
32	65.7	101.5	1286.10	771.58	1286.36	2.498
33	62.2	101.6	1201.55	716.39	1201.96	2.475

Table B.9: Dimensions and BRD for Mix9, continued

34	60.2	101.6	1150.52	687.07	1151.43	2.478
35	66.2	101.7	1270.88	756.37	1272.37	2.463
36	65.5	101.6	1283.69	773.59	1284.55	2.512
37	60.7	101.6	1176.46	706.39	1177.12	2.499
38	58.3	101.6	1126.19	672.73	1126.57	2.481
39	63.4	101.6	1218.32	725.72	1218.94	2.470
40	62.2	101.5	1192.17	715.58	1194.20	2.491
41	59.1	101.5	1121.90	666.52	1123.18	2.457
42	67.0	101.6	1276.49	761.23	1278.61	2.467
43	59.6	101.6	1151.41	692.69	1152.93	2.502
Mean						2.452
STD						0.037

Table B.10: Dimensions and BRD for extracted cores, Mix1

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	63.5	96.2	1057.19	591.25	1057.65	2.267
2	63.5	96.2	1062.77	597.48	1063.23	2.282
3	62.0	96.2	1072.84	604.25	1073.51	2.286
4	62.5	96.2	1073.5	605.22	1074.07	2.290
5	62.5	96.2	1062.42	601.25	1062.92	2.301
Mean						2.285
STD						0.013

Table B.11: Dimensions and BRD for extracted cores, Mix2

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	63.5	96.2	1089.48	614.17	1089.93	2.290
2	63.5	96.2	1085.00	616.19	1085.47	2.312
3	62.5	96.2	1065.25	604.51	1065.84	2.309
4	62.0	96.2	1064.95	603.50	1065.63	2.304
5	62.5	96.2	1066.32	602.83	1066.77	2.298
Mean						2.303
STD						0.009

Table B.12: Dimensions and BRD for extracted cores, Mix3

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	63.0	96.2	1065.17	598.08	1065.58	2.278
2	63.0	96.2	1077.78	607.51	1078.18	2.290
3	62.5	96.2	1074.70	607.58	1075.20	2.298
4	63.5	96.2	1087.97	611.07	1088.36	2.279
5	63.0	96.2	1079.66	606.20	1080.06	2.278
Mean						2.285
STD						0.009

Table B.13: Dimensions and BRD for extracted cores, Mix4

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	63.7	96.2	1113.96	651.68	1114.24	2.408
2	63.3	96.2	1105.38	647.39	1105.63	2.412
3	63.2	96.2	1109.16	651.45	1109.38	2.422
4	63.1	96.2	1103.66	646.86	1103.91	2.415
5	63.5	96.2	1112.96	654.57	1113.26	2.426
Mean						2.417
STD						0.007

Table B.14: Dimensions and BRD for extracted cores, Mix5

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	62.8	96.2	1113.63	660.00	1113.89	2.454
2	63.9	96.2	1133.60	672.96	1133.78	2.460
3	63.7	96.2	1131.55	672.23	1131.90	2.462
4	63.8	96.2	1134.10	674.50	1134.30	2.467
5	64.2	96.2	1133.27	672.97	1133.58	2.460
Mean						2.460
STD						0.005

Table B.15: Dimensions and BRD for extracted cores, Mix6

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	64.0	96.2	1143.87	680.98	1144.18	2.469
2	63.1	96.2	1127.02	672.2	1127.46	2.476
3	63.8	96.2	1140.17	678.89	1140.51	2.470
4	64.0	96.2	1137	676.2	1137.41	2.465
5	63.6	96.2	1132.37	674.67	1132.76	2.472
Mean						2.470
STD						0.004

Table B.16: Dimensions and BRD for extracted cores, Mix7

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	64.3	96.2	1079.22	632.02	1084.02	2.388
2	64.3	96.2	1074.95	617.89	1077.26	2.340
3	65.9	96.2	1087.92	622.96	1091.41	2.322
4	65.3	96.2	1088.67	623.05	1091.37	2.325
5	66.1	96.2	1094.43	634.52	1098.92	2.357
Mean						2.346
STD						0.027

Table B.17: Dimensions and BRD for extracted cores, Mix9

No.	h (mm)	D (mm)	Weight (gm)			BRD (gm/cm ³)
			Air	Water	Surface dry	
1	65.5	96.2	1124.03	653.5	1124.28	2.388
2	65.7	96.2	1142.48	665.96	1142.82	2.396
3	66.5	96.2	1165.12	680.15	1165.31	2.402
4	66.6	96.2	1171.95	685.16	1172.19	2.406
5	64.6	96.2	1128.32	656.93	1128.57	2.392
Mean						2.397
STD						0.007

Table B.18: MRD for limestone mixes

Mix	Weight (gm)						MRD (gm/cm³)
	Flask in air	Flask & mixture in air	Mixture in air	Flask in water	Flask & mixture in water	Mixture in water	
1	From Table (A.1)						2.541
2	From Table (A.7)						2.505
3	From Table (A.6)						2.533
4	647.1	1807.9	1160.8	565.3	1259.2	693.9	2.486
5	645.2	1917.6	1272.4	563.7	1334.9	771.2	2.539
6	647.1	1885.5	1238.4	565.3	1314.1	748.8	2.529
7	645.7	2179.8	1534.1	564.2	1480.4	916.2	2.483
9	651.3	2184.8	1533.5	568.9	1493.5	924.6	2.518

APPENDIX C:

FRICTIONAL PROPERTIES

Table C.1: Determination of average aggregate size for loose aggregate slabs

Sieve size (mm)	HL4			Sand		
	passing %	accumulative retained %	retained %	passing %	accumulative retained %	retained %
12.70	99.7	0.3	0.3	100.0	0.0	0.0
4.75	7.8	92.2	91.9	100.0	0.0	0.0
2.00	5.7	94.3	2.1	98.7	1.3	1.3
0.425	4.1	95.9	1.6	70.1	29.9	28.6
0.180	3.4	96.6	0.7	16.5	83.5	53.6
0.075	2.3	97.7	1.1	1.6	98.4	14.9
	Average size (S)		4.454	Average size (S)		0.255

$$S = \frac{\sum R \times SS}{100}$$

S = average size (mm)

R = retained on a specific sieve (%)

SS = sieve size (mm)

Table C.2: Results of friction test for Mix1 (BPN)[illegible]

Table C.3: Results of friction test for Mix2 (BPN)

[illegible]

Table C.4: Results of friction test for Mix3 (BPN)

[illegible]

Table C.5: Results of friction test for Mix4 (BPN)

[illegible]

Table C.6: Results of friction test for Mix5 (BPN)

[illegible]

Table C.7: Results of friction test for Mix6 (BPN)

[illegible]

Table C.8: Results of friction test for Mix7 (BPN)

[illegible]

Table C.9: Results of friction test for Mix9 (BPN)

[illegible]

Table C.10: Results of friction test for field mixes, Mix7, after 2.8 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	84	82	80	78	81.0	76.83	L1	79	77	76	75	76.8	76.92
	L2	76	74	72	70	73.0		L2	81	79	77	75	78.0	
	L3	78	76	76	76	76.5		L3	79	77	75	73	76.0	
2	L1	77	75	75	75	75.5	77.00	L1	78	76	75	75	76.0	79.08
	L2	80	79	76	77	78.0		L2	80	80	80	80	80.0	
	L3	80	78	76	76	77.5		L3	83	82	80	80	81.3	
3	L1	79	77	76	76	77.0	77.00	L1	80	79	75	77	77.8	79.50
	L2	N.A.						L2	80	79	78	80	79.3	
	L3	N.A.						L3	84	81	81	80	81.5	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	85	84	84	84	84.3	82.42	L1	82	81	80	80	80.8	82.83
	L2	83	83	82	81	82.3		L2	86	85	84	84	84.8	
	L3	82	81	80	80	80.8		L3	83	82	84	83	83.0	
2	L1	76	76	75	73	75.0	75.92	L1	80	80	78	76	78.5	78.42
	L2	79	77	76	75	76.8		L2	79	78	76	76	77.3	
	L3	78	76	75	75	76.0		L3	80	80	80	78	79.5	
3	L1	77	76	76	75	76.0	76.25	L1	81	81	79	79	80.0	79.33
	L2	78	76	76	76	76.5		L2	77	77	77	77	77.0	
	L3	78	76	76	75	76.3		L3	82	81	81	80	81.0	

	LS1	LS2	Mean
LOD	76.9	78.2	77.6
TRD	78.5	80.2	79.4

Table C.11: Results of friction test for field mixes, Mix8, after 2.5 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	81	79	79	77	79.0	81.17	L1	85	83	82	81	82.8	81.67
	L2	84	82	81	80	81.8		L2	81	80	80	80	80.3	
	L3	84	84	82	81	82.8		L3	84	83	81	80	82.0	
2	L1	80	80	79	78	79.3	76.92	L1	76	75	75	74	75.0	74.92
	L2	73	72	71	71	71.8		L2	75	73	74	71	73.3	
	L3	82	80	79	78	79.8		L3	79	76	75	76	76.5	
3	L1	80	77	77	77	77.8	76.42	L1	83	81	80	80	81.0	78.58
	L2	76	76	75	75	75.5		L2	77	76	75	75	75.8	
	L3	77	77	75	75	76.0		L3	82	78	78	78	79.0	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	80	79	77	77	78.3	77.17	L1	80	80	77	77	78.5	77.83
	L2	77	75	75	75	75.5		L2	78	77	77	77	77.3	
	L3	79	78	77	77	77.8		L3	80	78	76	77	77.8	
2	L1	76	76	74	72	74.5	73.92	L1	80	78	76	77	77.8	78.58
	L2	72	72	71	71	71.5		L2	78	76	76	75	76.3	
	L3	77	76	75	75	75.8		L3	83	83	81	80	81.8	
3	L1	75	74	72	72	73.3	73.75	L1	81	81	81	81	81.0	79.00
	L2	74	72	72	71	72.3		L2	75	75	74	74	74.5	
	L3	77	76	75	75	75.8		L3	84	82	80	80	81.5	

	LS1	LS2	Mean
LOD	78.2	74.9	76.6
TRD	78.4	78.5	78.4

Table C.12: Results of friction test for field mixes, Mix7, after 3.5 months**LS1**

ES1														
Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	77	76	75	75	75.8	70.67	L1	75	72	70	69	71.5	71.17
	L2	70	69	68	66	68.3		L2	74	71	71	70	71.5	
	L3	72	69	66	65	68.0		L3	73	70	70	69	70.5	
2	L1	78	75	75	75	75.8	76.33	L1	74	73	72	71	72.5	74.33
	L2	76	76	76	75	75.8		L2	76	75	76	76	75.8	
	L3	80	78	76	76	77.5		L3	75	75	75	74	74.8	
3	L1	74	74	73	71	73.0	73.00	L1	75	73	73	72	73.3	75.75
	L2	N.A.						L2	76	75	74	74	74.8	
	L3	N.A.						L3	81	79	79	78	79.3	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	78	77	77	77	77.3	76.67	L1	78	78	77	76	77.3	78.33
	L2	78	78	76	76	77.0		L2	81	80	80	80	80.3	
	L3	78	75	75	75	75.8		L3	78	78	77	77	77.5	
2	L1	74	73	72	72	72.8	72.67	L1	74	73	73	72	73.0	73.50
	L2	72	70	71	70	70.8		L2	75	74	73	72	73.5	
	L3	75	74	75	74	74.5		L3	76	75	73	72	74.0	
3	L1	74	73	74	72	73.3	72.58	L1	71	71	70	70	70.5	72.33
	L2	73	72	72	71	72.0		L2	74	72	72	72	72.5	
	L3	73	73	72	72	72.5		L3	74	74	74	74	74.0	

	LS1	LS2	Mean
LOD	73.3	74	73.7
TRD	73.8	74.7	74.2

Table C.13: Results of friction test for field mixes, Mix8, after 3.2 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	77	77	75	75	76.0	78.08	L1	80	80	79	77	79.0	78.67
	L2	79	78	78	78	78.3		L2	80	79	79	77	78.8	
	L3	80	80	80	80	80.0		L3	80	79	77	77	78.3	
2	L1	76	75	74	74	74.8	73.33	L1	72	70	70	69	70.3	71.25
	L2	71	70	69	68	69.5		L2	71	69	69	68	69.3	
	L3	77	76	75	75	75.8		L3	76	74	74	73	74.3	
3	L1	78	76	77	75	76.5	74.83	L1	80	79	78	77	78.5	76.50
	L2	75	74	74	74	74.3		L2	75	74	75	75	74.8	
	L3	74	74	74	73	73.8		L3	77	77	76	75	76.3	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	78	76	75	75	76.0	75.67	L1	78	76	75	75	76.0	73.92
	L2	76	75	75	75	75.3		L2	76	76	75	75	75.5	
	L3	76	76	76	75	75.8		L3	72	70	70	69	70.3	
2	L1	73	72	71	70	71.5	72.67	L1	75	74	72	70	72.8	73.83
	L2	73	73	72	72	72.5		L2	75	74	73	73	73.8	
	L3	75	74	74	73	74.0		L3	77	75	74	74	75.0	
3	L1	73	71	73	72	72.3	72.83	L1	78	76	75	75	76.0	75.42
	L2	74	72	72	71	72.3		L2	75	75	74	73	74.3	
	L3	75	74	74	73	74.0		L3	78	76	75	75	76.0	

	LS1	LS2	Mean
LOD	75.4	73.7	74.6
TRD	75.5	74.4	74.9

Table C.14 Results of friction test for field mixes, Mix7, after 6.7 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	80	79	77	76	78.0	72.42	L1	76	75	75	74	75.0	71.33
	L2	73	70	69	69	70.3		L2	70	68	66	64	67.0	
	L3	71	69	69	67	69.0		L3	75	72	71	70	72.0	
2	L1	74	72	71	70	71.8	73.83	L1	74	73	73	72	73.0	75.58
	L2	75	75	75	75	75.0		L2	79	77	77	76	77.3	
	L3	75	75	74	75	74.8		L3	78	78	75	75	76.5	
3	L1	77	75	74	73	74.8	74.75	L1	73	73	69	70	71.3	72.42
	L2	N.A.						L2	73	70	69	69	70.3	
	L3	N.A.						L3	78	75	75	75	75.8	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	80	77	80	79	79.0	78.42	L1	77	77	76	76	76.5	76.83
	L2	79	79	78	78	78.5		L2	78	77	76	76	76.8	
	L3	78	78	78	77	77.8		L3	78	78	77	76	77.3	
2	L1	73	71	71	70	71.3	71.83	L1	71	73	72	70	71.5	72.08
	L2	71	70	70	70	70.3		L2	73	73	71	71	72.0	
	L3	75	74	74	73	74.0		L3	72	74	73	72	72.8	
3	L1	75	75	74	75	74.8	73.17	L1	75	74	73	73	73.8	75.33
	L2	73	74	73	72	73.0		L2	75	75	75	74	74.8	
	L3	73	72	71	71	71.8		L3	79	78	77	76	77.5	

	LS1	LS2	Mean
LOD	73.7	74.5	74.1
TRD	73.1	74.8	73.9

Table C.15: Results of friction test for field mixes, Mix8, after 6.4 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	76	75	75	75	75.3	75.08	L1	74	74	73	73	73.5	74.08
	L2	76	75	74	75	75.0		L2	76	75	75	74	75.0	
	L3	75	75	75	75	75.0		L3	75	74	73	73	73.8	
2	L1	75	74	73	72	73.5	73.92	L1	71	70	70	68	69.8	71.33
	L2	76	74	73	72	73.8		L2	73	71	70	69	70.8	
	L3	76	75	74	73	74.5		L3	75	74	73	72	73.5	
3	L1	76	75	74	73	74.5	72.75	L1	76	75	74	74	74.8	74.08
	L2	74	73	72	72	72.8		L2	75	75	72	72	73.5	
	L3	73	71	70	70	71.0		L3	75	75	73	73	74.0	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	75	74	73	72	73.5	74.00	L1	74	71	70	70	71.3	73.17
	L2	75	75	74	74	74.5		L2	76	75	74	74	74.8	
	L3	75	74	74	73	74.0		L3	74	73	74	73	73.5	
2	L1	74	74	72	71	72.8	72.08	L1	74	70	69	67	70.0	71.25
	L2	73	72	70	70	71.3		L2	74	72	70	70	71.5	
	L3	75	72	72	70	72.3		L3	75	73	71	70	72.3	
3	L1	75	73	72	70	72.5	72.75	L1	75	73	73	73	73.5	73.83
	L2	74	70	71	70	71.3		L2	75	74	72	71	73.0	
	L3	75	75	74	74	74.5		L3	76	75	75	74	75.0	

	LS1	LS2	Mean
LOD	73.9	72.9	73.4
TRD	73.2	72.8	73.0

Table C.16: Results of friction test for field mixes, Mix7, after 7.4 months**LS1**

ES1

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	75	75	74	74	74.5	69.83	L1	72	70	70	68	70.0	67.83
	L2	71	67	66	65	67.3		L2	67	65	65	65	65.5	
	L3	70	70	66	65	67.8		L3	70	70	67	65	68.0	
2	L1	72	70	70	69	70.3	70.75	L1	72	70	70	69	70.3	70.92
	L2	73	71	71	70	71.3		L2	74	73	72	71	72.5	
	L3	72	71	70	70	70.8		L3	70	70	70	70	70.0	
3	L1	72	71	70	71	71.0	71.00	L1	70	69	68	67	68.5	70.83
	L2	N.A.						L2	71	69	68	67	68.8	
	L3	N.A.						L3	76	75	75	75	75.3	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	77	76	75	76	76.0	75.92	L1	75	75	75	75	75.0	77.00
	L2	76	75	75	75	75.3		L2	80	79	78	78	78.8	
	L3	78	77	76	75	76.5		L3	78	78	77	76	77.3	
2	L1	71	71	70	69	70.3	71.42	L1	71	70	70	70	70.3	72.67
	L2	71	70	69	69	69.8		L2	74	73	72	71	72.5	
	L3	76	75	73	73	74.3		L3	76	75	75	75	75.3	
3	L1	73	71	70	70	71.0	71.83	L1	75	72	70	70	71.8	73.67
	L2	75	74	73	72	73.5		L2	74	74	71	71	72.5	
	L3	73	71	70	70	71.0		L3	79	77	76	75	76.8	

	LS1	LS2	Mean
LOD	70.5	73.1	71.8
TRD	69.9	74.4	72.2

Table C.17: Results of friction test for field mixes, Mix8, after 7.1 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	75	74	73	73	73.8	73.92	L1	74	73	72	73	73.0	72.92
	L2	75	74	74	72	73.8		L2	74	73	72	72	72.8	
	L3	75	74	74	74	74.3		L3	74	73	73	72	73.0	
2	L1	75	74	73	72	73.5	73.67	L1	70	69	67	66	68.0	70.75
	L2	74	72	71	71	72.0		L2	72	71	70	70	70.8	
	L3	77	75	75	75	75.5		L3	75	74	73	72	73.5	
3	L1	75	74	73	72	73.5	71.08	L1	75	75	73	72	73.8	72.50
	L2	73	72	71	70	71.5		L2	73	71	70	70	71.0	
	L3	70	68	68	67	68.3		L3	74	74	72	71	72.8	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	75	75	73	72	73.8	72.42	L1	75	74	73	72	73.5	71.75
	L2	73	70	70	71	71.0		L2	73	72	72	72	72.3	
	L3	74	73	72	71	72.5		L3	70	69	70	69	69.5	
2	L1	69	69	68	67	68.3	68.75	L1	73	71	70	70	68.3	70.17
	L2	70	70	67	66	68.3		L2	72	71	70	70	70.8	
	L3	70	70	70	69	69.8		L3	74	72	70	70	71.5	
3	L1	71	71	70	70	70.5	71.08	L1	74	75	73	72	73.5	72.83
	L2	72	72	70	70	71.0		L2	73	71	71	70	71.3	
	L3	73	72	71	71	71.8		L3	75	74	73	73	73.8	

	LS1	LS2	Mean
LOD	72.9	70.8	71.8
TRD	72.1	71.6	71.8

Table C.18: Results of friction test for field mixes, Mix7, after 8.3 months**LS1**

ES1														
Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	74	72	70	70	71.5	67.58	L1	70	68	66	65	67.3	68.42
	L2	67	65	64	63	64.8		L2	74	73	71	70	72.0	
	L3	69	66	66	65	66.5		L3	69	66	65	64	66.0	
2	L1	71	70	69	68	69.5	69.50	L1	70	69	67	67	68.3	70.75
	L2	72	70	70	69	70.3		L2	74	72	72	70	72.0	
	L3	70	69	69	67	68.8		L3	74	72	71	71	72.0	
3	L1	72	70	69	68	69.8	69.75	L1	74	70	69	68	70.3	71.67
	L2	N.A.						L2	75	73	71	70	72.3	
	L3	N.A.						L3	74	73	72	71	72.5	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	76	76	75	74	75.3	74.50	L1	75	75	73	73	74.0	75.08
	L2	76	74	74	73	74.3		L2	77	75	75	73	75.0	
	L3	75	75	73	73	74.0		L3	78	76	76	75	76.3	
2	L1	72	70	68	67	69.3	69.08	L1	72	70	69	68	69.8	69.92
	L2	70	68	67	66	67.8		L2	70	70	70	69	69.8	
	L3	71	71	70	69	70.3		L3	72	70	70	69	70.3	
3	L1	74	73	71	70	72.0	71.25	L1	71	70	69	67	69.3	70.17
	L2	72	71	70	70	70.8		L2	71	70	70	69	70.0	
	L3	73	71	70	70	71.0		L3	73	72	70	70	71.3	

	LS1	LS2	Mean
LOD	68.9	71.6	70.3
TRD	70.3	71.7	71.0

Table (C.19) Results of friction test for field mixes, Mix8, after 8 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	75	74	73	72	73.5	72.92	L1	76	74	72	72	73.5	72.83
	L2	74	72	72	71	72.3		L2	73	71	70	70	71.0	
	L3	75	74	72	71	73.0		L3	76	75	73	72	74.0	
2	L1	75	73	72	71	72.8	71.67	L1	70	68	67	66	67.8	69.67
	L2	71	70	70	70	70.3		L2	71	70	70	69	70.0	
	L3	73	73	72	70	72.0		L3	73	72	70	70	71.3	
3	L1	74	72	71	70	71.8	71.25	L1	74	72	70	70	71.5	71.58
	L2	72	71	70	70	70.8		L2	74	72	70	70	71.5	
	L3	73	72	70	70	71.3		L3	74	72	71	70	71.8	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	74	70	70	70	71.0	71.17	L1	74	73	72	70	72.3	71.58
	L2	72	71	70	70	70.8		L2	72	71	71	70	71.0	
	L3	74	72	70	71	71.8		L3	74	72	70	70	71.5	
2	L1	70	69	69	68	69.0	69.75	L1	72	72	70	68	70.0	70.50
	L2	70	70	70	70	70.0		L2	72	71	70	70	70.8	
	L3	71	70	70	70	70.3		L3	72	71	70	70	70.8	
3	L1	74	70	70	70	71.0	71.58	L1	74	74	73	74	73.8	72.75
	L2	74	72	70	70	71.5		L2	73	70	70	70	70.8	
	L3	74	73	71	71	72.3		L3	75	74	73	73	73.8	

	LS1	LS2	Mean
LOD	71.9	70.8	71.4
TRD	71.4	71.6	71.5

Table C.20: Results of friction test for field mixes, Mix7, after 9.3 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	70	70	70	70	70.0	66.67	L1	65	65	64	64	64.5	66.00
	L2	66	65	64	64	64.8		L2	69	69	68	67	68.3	
	L3	66	66	65	64	65.3		L3	66	66	64	65	65.3	
2	L1	68	69	68	67	68.0	68.75	L1	67	66	66	65	66.0	68.58
	L2	70	69	69	68	69.0		L2	73	71	70	70	71.0	
	L3	70	70	69	68	69.3		L3	70	69	68	68	68.8	
3	L1	70	71	69	68	69.5	69.50	L1	67	65	65	65	65.5	67.22
	L2	N.A.						L2	67	67	67	65	66.5	
	L3	N.A.						L3	72	70	70	69	69.7	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	76	75	74	74	74.8	72.17	L1	71	70	69	69	69.8	72.00
	L2	73	70	72	70	71.3		L2	75	72	71	71	72.3	
	L3	71	71	70	70	70.5		L3	75	74	73	74	74.0	
2	L1	69	68	67	65	67.3	68.17	L1	68	67	66	66	66.8	68.33
	L2	69	67	67	66	67.3		L2	70	69	68	66	68.3	
	L3	71	70	70	69	70.0		L3	70	70	70	70	70.0	
3	L1	70	71	70	71	70.5	69.67	L1	68	66	65	65	66.0	68.08
	L2	71	69	69	69	69.5		L2	68	68	68	68	68.0	
	L3	69	69	69	69	69.0		L3	73	70	69	69	70.3	

	LS1	LS2	Mean
LOD	68.3	70	69.2
TRD	67.3	69.5	68.4

Table C.21: Results of friction test for field mixes, Mix8, after 9 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	73	72	70	70	71.3	70.75	L1	71	71	70	70	70.5	70.33
	L2	70	70	70	69	69.8		L2	72	70	70	70	70.5	
	L3	73	71	71	70	71.3		L3	71	70	69	70	70.0	
2	L1	73	71	70	71	71.3	69.75	L1	67	65	65	64	65.3	67.42
	L2	70	70	70	69	69.8		L2	69	67	67	66	67.3	
	L3	70	68	68	67	68.3		L3	72	70	69	68	69.8	
3	L1	72	71	69	70	70.5	69.50	L1	70	70	70	70	70.0	69.75
	L2	71	70	70	69	70.0		L2	70	70	69	69	69.5	
	L3	70	69	67	66	68.0		L3	70	70	70	69	69.8	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	73	70	69	69	70.3	70.25	L1	71	71	70	70	70.5	68.92
	L2	70	70	70	69	69.8		L2	69	67	67	68	67.8	
	L3	72	71	70	70	70.8		L3	70	67	68	69	68.5	
2	L1	67	65	65	64	65.3	67.33	L1	70	67	65	65	68.8	68.92
	L2	70	69	69	67	68.8		L2	70	69	69	68	69.0	
	L3	70	68	67	67	68.0		L3	71	70	68	67	69.0	
3	L1	71	70	70	68	69.8	70.17	L1	70	70	68	69	69.3	68.75
	L2	71	71	70	70	70.5		L2	69	67	67	67	67.5	
	L3	71	71	70	69	70.3		L3	72	70	68	68	69.5	

	LS1	LS2	Mean
LOD	70.0	69.3	69.6
TRD	69.2	68.9	69.0

Table C.22: Results of friction test for field mixes, Mix7, after 10.3 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	71	69	69	67	69.0	65.25	L1	67	65	64	63	64.8	64.25
	L2	65	63	61	60	62.3		L2	67	66	64	63	65.0	
	L3	67	65	64	62	64.5		L3	65	63	62	62	63.0	
2	L1	70	69	69	68	69.0	68.92	L1	70	69	68	66	68.3	69.25
	L2	68	66	67	66	66.8		L2	72	71	70	70	70.8	
	L3	73	71	70	70	71.0		L3	70	68	69	68	68.8	
3	L1	70	70	68	67	68.8	68.75	L1	67	65	64	62	64.5	66.33
	L2	N.A.						L2	69	67	65	65	66.5	
	L3	N.A.						L3	67	69	68	67	68.0	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	74	74	73	72	73.3	71.83	L1	72	70	70	70	70.5	71.75
	L2	71	70	70	70	70.3		L2	73	72	71	71	71.8	
	L3	74	72	71	71	72.0		L3	72	74	73	73	73.0	
2	L1	69	68	66	66	67.3	67.17	L1	66	65	64	64	64.8	66.33
	L2	67	65	64	64	65.0		L2	65	65	65	65	65.0	
	L3	71	69	69	68	69.3		L3	71	69	69	68	69.3	
3	L1	72	70	70	69	70.3	69.75	L1	71	72	67	66	69.0	70.42
	L2	74	71	70	69	71.0		L2	70	70	69	69	69.5	
	L3	70	69	66	67	68.0		L3	75	74	71	71	72.8	

	LS1	LS2	Mean
LOD	67.6	69.6	68.6
TRD	66.6	69.5	68.1

Table C.23: Results of friction test for field mixes, Mix8, after 9.9 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	71	70	69	69	69.8	69.58	L1	72	70	70	70	70.5	70.17
	L2	70	68	68	69	68.8		L2	70	70	70	70	70.0	
	L3	71	71	70	69	70.3		L3	71	70	70	69	70.0	
2	L1	70	70	70	70	70.0	69.25	L1	66	65	65	65	65.3	66.92
	L2	70	69	69	68	69.0		L2	69	67	66	67	67.3	
	L3	69	69	68	69	68.8		L3	70	68	68	67	68.3	
3	L1	72	71	71	70	71.0	68.50	L1	71	70	69	69	69.8	69.33
	L2	68	68	67	67	67.5		L2	69	69	69	69	69.0	
	L3	69	67	66	66	67.0		L3	70	70	69	68	69.3	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	72	70	71	70	70.8	69.58	L1	71	71	70	70	70.5	68.92
	L2	69	68	68	67	68.0		L2	71	70	70	70	70.3	
	L3	71	70	70	69	70.0		L3	67	66	66	65	66.0	
2	L1	66	65	65	63	64.8	67.00	L1	66	64	64	63	64.3	67.67
	L2	70	69	68	67	68.5		L2	71	70	69	68	69.5	
	L3	70	67	67	67	67.8		L3	70	70	69	68	69.3	
3	L1	70	69	68	67	68.5	69.42	L1	70	70	68	68	69.0	69.75
	L2	72	70	70	68	70.0		L2	70	71	70	70	70.3	
	L3	70	70	70	69	69.8		L3	71	70	70	69	70.0	

	LS1	LS2	Mean
LOD	69.1	68.7	68.9
TRD	68.8	68.8	68.8

Table C.24: Results of friction test for field mixes, Mix7, after 11.5 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	70	68	68	68	68.5	65.17	L1	68	66	66	66	66.5	64.83
	L2	65	63	62	61	62.8		L2	64	64	63	62	63.3	
	L3	65	64	64	64	64.3		L3	67	65	64	63	64.8	
2	L1	N.A.					N.A.	L1	N.A.					N.A.
	L2							L2						
	L3							L3						
3	L1	70	69	67	67	68.3	68.25	L1	65	64	64	64	64.3	65.75
	L2	N.A.						L2	65	64	65	66	65.0	
	L3	N.A.						L3	68	68	68	68	68.0	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	73	72	71	71	71.8	70.83	L1	73	73	72	72	72.5	72.00
	L2	71	71	70	70	70.5		L2	71	71	70	70	70.5	
	L3	71	70	70	70	70.3		L3	74	73	73	72	73.0	
2	L1	68	67	67	66	67.0	66.58	L1	69	68	67	67	67.8	67.25
	L2	65	65	64	65	64.8		L2	67	65	65	65	65.5	
	L3	69	68	67	68	68.0		L3	69	69	68	68	68.5	
3	L1	70	70	69	68	69.3	69.17	L1	70	68	67	66	67.8	69.42
	L2	71	70	70	70	70.3		L2	70	70	70	68	69.5	
	L3	70	68	67	67	68.0		L3	73	71	70	70	71.0	

	LS1	LS2	Mean
LOD	66.7	68.9	67.8
TRD	65.3	69.6	67.4

Table C.25: Results of friction test for field mixes, Mix8, after 11.2 months**LS1**

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	70	69	68	68	68.8	68.75	L1	70	70	70	70	70.0	69.58
	L2	69	69	67	67	68.0		L2	70	70	70	69	69.8	
	L3	72	70	68	68	69.5		L3	70	69	68	69	69.0	
2	L1	71	70	68	68	69.3	68.00	L1	69	67	66	65	66.8	67.00
	L2	70	67	67	66	67.5		L2	66	65	65	66	65.5	
	L3	70	66	67	66	67.3		L3	70	69	68	68	68.8	
3	L1	70	68	69	68	68.8	68.42	L1	70	70	70	70	70.0	68.58
	L2	69	68	69	68	68.5		L2	67	67	66	66	66.5	
	L3	69	68	68	67	68.0		L3	70	69	69	69	69.3	

LS2

Slab	LOD							TRD						
	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.	LOC	R 1	R 2	R 3	R 4	Rav.	Avg.
1	L1	72	70	70	70	70.5	69.17	L1	70	70	70	70	70.0	68.33
	L2	70	69	69	69	69.3		L2	70	70	70	70	70.0	
	L3	69	67	67	68	67.8		L3	65	65	65	65	65.0	
2	L1	68	67	66	67	67.0	67.00	L1	66	65	65	65	65.3	67.83
	L2	67	67	67	67	67.0		L2	70	68	68	68	68.5	
	L3	68	68	66	66	67.0		L3	70	70	70	69	69.8	
3	L1	N.A.					N.A.	L1	N.A.					N.A.
	L2							L2						
	L3							L3						

	LS1	LS2	Mean
LOD	68.4	68.1	68.2
TRD	68.4	68.1	68.2

Table C.26: Sand patch test results, Mix7

Time (month)	Slab	D (mm)					TD (mm)	Mean TD (mm)
		D1	D2	D3	D4	Avg.		
1.0	1/SL1	41.5	45.0	45.0	43.0	43.6	0.170	0.174
	2/SL2	44.0	42.50	41.0	43.0	42.63	0.178	
6.3	1/SL1	34.0	36.0	37.0	35.5	35.63	0.255	0.250
	2/SL1	34.0	36.0	37.0	36.5	35.88	0.251	
	1/SL2	35.0	36.0	37.0	35.5	35.88	0.251	
	2/SL2	35.0	38.5	39.0	39.5	38.00	0.224	
	3/SL2	34.0	34.0	36.0	35.0	34.75	0.268	
7.5	1/SL1	37.0	36.0	38.0	38.0	37.25	0.233	0.240
	2/SL1	35.5	37.0	37.0	36.0	36.38	0.244	
	1/SL2	36.5	38.0	36.0	38.0	37.13	0.235	
	2/SL2	38.5	36.5	36.0	34.5	36.38	0.244	
	3/SL2	37.5	35.5	36.5	36.0	36.38	0.244	
8.2	1/SL1	37.5	37.0	34.5	40.5	37.38	0.232	0.238
	2/SL1	36.0	36.5	38.5	36.0	36.75	0.239	
	1/SL2	39.0	39.0	40.0	38.0	39.00	0.213	
	2/SL2	34.5	36.0	35.0	36.5	35.50	0.257	
	3/SL2	36.0	33.0	37.0	37.5	35.88	0.251	
8.5	1/SL1	41.0	36.0	36.0	37.0	37.50	0.230	0.221
	2/SL1	38.5	38.5	37.0	38.0	38.00	0.224	
	1/SL2	37.0	39.0	39.0	42.0	39.25	0.210	
	2/SL2	39.0	35.0	36.5	38.5	37.25	0.233	
	3/SL2	42.0	39.0	38.5	38.0	39.38	0.209	
11.7	1/SL1	35.0	36.0	36.0	34.0	35.25	0.260	0.244
	1/SL2	35.0	37.0	37.5	35.0	36.13	0.248	
	2/SL2	36.5	38.5	37.0	38.5	37.63	0.228	
	3/SL2	36.0	36.5	36.5	37.5	36.63	0.241	

Table C.27: Sand patch test results, Mix8

Time (month)	Slab	D (mm)					TD (mm)	Mean TD (mm)
		D1	D2	D3	D4	Avg.		
0.7	2/SL1	40.5	40.0	41.0	40.0	40.38	0.198	0.206
	3/SL2	38.5	40.0	39.5	37.5	38.88	0.214	
6.0	1/SL1	34.0	35.0	37.0	35.0	35.25	0.260	0.234
	2/SL1	40.0	38.0	39.0	39.5	39.13	0.211	
	3/SL1	38.0	37.5	39.0	37.5	38.00	0.224	
	1/SL2	37.5	40.0	39.0	38.5	38.75	0.215	
	2/SL2	37.5	37.0	36.0	36.5	36.75	0.239	
	3/SL2	35.0	37.0	35.5	34.5	35.50	0.257	
7.2	1/SL1	36.0	39.0	37.0	38.0	37.50	0.230	0.225
	2/SL1	36.5	39.5	40.0	38.0	38.50	0.218	
	3/SL1	38.0	39.0	37.5	40.0	38.63	0.217	
	1/SL2	39.5	40.5	40.5	39.0	39.88	0.203	
	2/SL2	37.0	41.0	40.0	39.0	39.25	0.210	
	3/SL2	35.0	35.5	35.0	32.5	34.50	0.272	
7.9	1/SL1	33.5	38.0	35.0	36.0	35.63	0.255	0.219
	2/SL1	39.0	38.5	38.0	40.0	38.88	0.214	
	3/SL1	40.0	42.0	39.0	41.5	40.63	0.196	
	1/SL2	40.0	40.5	41.0	42.5	41.00	0.192	
	2/SL2	42.0	40.0	43.0	43.0	42.00	0.183	
	3/SL2	32.0	34.0	36.0	35.0	34.25	0.276	
8.1	1/SL1	36.0	38.0	39.0	41.0	38.50	0.218	0.212
	2/SL1	40.0	40.0	38.5	37.5	39.00	0.213	
	3/SL1	40.5	41.0	42.5	41.5	41.38	0.189	
	1/SL2	40.5	40.5	44.0	41.0	41.50	0.188	
	2/SL2	39.0	40.0	43.0	37.0	39.75	0.205	
	3/SL2	37.0	36.0	34.0	33.5	35.13	0.262	
11.4	1/SL1	34.5	37.0	34.0	37.5	35.75	0.253	0.228
	2/SL1	35.0	38.0	37.0	37.5	36.88	0.238	
	3/SL1	39.0	36.5	36.0	38.0	37.38	0.232	
	1/SL2	37.0	39.5	41.0	40.0	39.38	0.209	
	2/SL2	41.5	41.0	37.5	38.0	39.50	0.207	

Table C.28: Field friction, 2nd measurement**Taxiway "F", Mix7****Time: 3.8 months****Average temperature: 1°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	84	83	83	83	83.25	80	80	78	78	79.00
L2	85	85	85	83	84.50	84	82	83	84	83.25
L3	81	79	79	78	79.25	80	79	78	77	78.50
L4	82	83	81	81	81.75	82	81	81	80	81.00
L5	85	85	85	84	84.75	88	87	86	86	86.75
L6	85	84	84	84	84.25	86	86	86	86	86.00
L7	83	83	84	83	83.25	86	86	85	84	85.25
L8	79	80	80	80	79.75	83	85	84	84	84.00
L9	82	80	80	79	80.25	85	86	85	85	85.25
Mean	82.33					83.22				

Taxiway "B", Mix8**Time: 3.5 months****Average temperature: 3°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	80	80	79	80	79.75	79	76	77	76	77.00
L2	80	78	78	77	78.25	83	82	82	81	82.00
L3	79	79	76	77	77.75	79	78	79	77	78.25
L4	82	82	81	79	81.00	84	83	83	82	83.00
L5	81	80	80	81	80.50	79	77	78	75	77.25
Mean	79.45					79.50				

Table C.29: Field friction, 3rd measurement**Taxiway "F", Mix7****Time: 6.8 months****Average temperature: -13°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	79	79	79	78	78.75	81	79	79	78	79.25
L2	75	74	74	74	74.25	76	76	75	74	75.25
L3	75	75	75	74	74.75	75	74	72	73	73.50
L4	75	75	75	74	74.75	76	75	74	74	74.75
L5	74	74	73	74	73.75	70	70	69	69	69.50
L6	70	70	69	69	69.50	72	72	71	70	71.25
L7	78	77	76	76	76.75	78	77	76	76	76.75
L8	80	78	79	78	78.75	80	80	79	78	79.25
L9	79	79	77	76	77.75	76	77	77	77	76.75
Mean	75.44					75.14				

Taxiway "B", Mix8**Time: 6.3 months****Average temperature: -17°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	80	80	81	80	80.25	75	75	75	75	75.00
L2	73	71	73	73	72.50	70	70	70	70	70.00
L3	75	73	74	74	74.00	77	76	76	75	76.00
L4	75	75	73	73	74.00	72	73	71	71	71.75
L5	77	77	76	75	76.25	74	75	75	75	74.75
Mean	75.40					73.50				

Table C.30: Field friction, 4th measurement**Taxiway "F", Mix7****Time: 7.3 months****Average temperature: -6°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	72	70	70	70	70.50	74	74	74	72	73.50
L2	73	73	73	73	73.00	75	74	74	73	74.00
L3	73	72	71	72	72.00	73	73	72	72	72.50
L4	72	72	72	71	71.75	75	73	74	74	74.00
L5	74	73	73	74	73.50	74	74	74	72	73.50
L6	73	73	71	72	72.25	72	72	71	71	71.50
L7	77	75	76	76	76.00	75	76	75	75	75.25
L8	74	74	73	73	73.50	75	75	75	75	75.00
L9	75	75	75	76	75.25	74	74	74	73	73.75
Mean	73.08					73.67				

Taxiway "B", Mix8**Time: 7 months****Average temperature: -8°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	74	74	74	73	73.75	74	74	74	74	74.00
L2	66	65	66	65	65.50	65	65	66	65	65.25
L3	67	68	67	67	67.25	67	66	66	65	66.00
L4	71	72	71	70	71.00	68	68	68	67	67.75
L5	71	70	70	70	70.25	65	66	67	66	66.00
Mean	69.55					67.80				

Table C.31: Field friction, 5th measurement**Taxiway "F", Mix7****Time: 8.3 months****Average temperature: -5°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	71	71	70	70	70.50	74	73	73	71	72.75
L2	71	71	71	70	70.75	72	70	70	70	70.50
L3	69	69	68	68	68.50	72	72	70	70	71.00
L4	72	71	71	70	71.00	72	71	70	70	70.75
L5	72	73	71	71	71.75	73	74	73	72	73.00
L6	71	71	70	69	70.25	69	70	70	70	69.75
L7	70	70	70	70	70.00	70	70	70	70	70.00
L8	74	73	73	72	73.00	75	75	75	75	75.00
L9	69	68	68	68	68.25	70	70	69	70	69.75
Mean	70.44					71.39				

Taxiway "B", Mix8**Time: 7.9 months****Average temperature: -7°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	71	71	71	71	71.00	70	71	71	72	71.00
L2	63	63	63	62	62.75	60	60	60	60	60.00
L3	66	65	65	65	65.25	63	62	63	62	62.50
L4	68	69	68	68	68.25	64	65	65	65	64.75
L5	71	71	70	71	70.75	68	69	70	69	69.00
Mean	67.60					65.45				

Table C.32: Field friction, 6th measurement**Taxiway "F", Mix7****Time: 9.2 months****Average temperature: 5°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	70	69	70	69	69.50	71	71	72	70	71.00
L2	67	67	67	66	66.75	70	70	69	70	69.75
L3	68	68	68	68	68.00	68	70	69	68	68.75
L4	71	70	70	69	70.00	71	70	71	70	70.50
L5	71	70	70	69	70.00	72	72	70	71	71.25
L6	70	68	69	68	68.75	70	69	68	68	68.75
L7	70	69	68	69	69.00	69	69	68	69	68.75
L8	69	69	69	69	69.00	70	71	70	71	70.50
L9	68	68	69	68	68.25	69	68	68	68	68.25
Mean	68.81					69.72				

Taxiway "B", Mix8**Time: 8.8 months****Average temperature: 5°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	64	65	64	64	64.25	69	66	68	67	67.50
L2	63	62	62	62	62.25	66	66	65	65	65.50
L3	68	67	66	67	67.00	70	69	67	68	68.50
L4	66	66	65	65	65.50	69	68	67	67	67.75
L5	67	66	66	66	66.25	70	70	69	69	69.50
Mean	65.05					67.75				

Table C.33: Field friction, 7th measurement

Taxiway "F", Mix7

Time: 10.3 months

Average temperature: 14°C

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	67	67	67	67	67.00	71	71	71	71	71.00
L2	66	66	67	66	66.25	69	69	69	69	69.00
L3	66	66	65	65	65.50	67	66	67	66	66.50
L4	68	67	66	66	66.75	65	65	65	65	65.00
L5	70	69	69	68	69.00	69	70	69	68	69.00
L6	65	67	66	67	66.25	69	68	68	67	68.00
L7	68	68	66	67	67.25	68	68	67	67	67.50
L8	68	68	67	68	67.75	70	69	69	69	69.25
L9	67	68	67	67	67.25	68	68	67	66	67.25
Mean	67.00					68.06				

Taxiway "B", Mix8

Time: 9.9 months

Average temperature: 13°C

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	62	61	61	61	61.25	66	65	66	65	65.50
L2	60	60	61	60	60.25	64	64	64	62	63.50
L3	64	64	63	64	63.75	67	66	66	67	66.50
L4	66	67	67	67	66.75	67	64	66	64	65.25
L5	63	64	63	64	63.50	69	67	68	67	67.75
Mean	63.10					65.70				

Table C.34: Field friction, 8th measurement**Taxiway "F", Mix7****Time: 11.5 months****Average temperature: 18°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	66	65	65	65	65.25	67	66	66	66	66.25
L2	66	64	65	65	65.00	68	66	65	64	65.75
L3	64	64	63	61	63.00	66	66	66	67	66.25
L4	66	66	66	65	65.75	66	65	66	63	65.00
L5	68	66	66	65	66.25	68	67	64	65	66.00
L6	65	66	64	64	64.75	68	66	66	67	66.75
L7	66	67	65	64	65.50	67	67	67	65	66.50
L8	64	63	64	62	63.25	67	66	66	62	65.25
L9	66	63	63	63	63.75	68	66	67	65	66.50
Mean	64.72					66.03				

Taxiway "B", Mix8**Time: 11.2 months****Average temperature: 20°C**

LOC	LOD					TRD				
	R1	R2	R3	R4	Rav.	R1	R2	R3	R4	Rav.
L1	61	61	60	60	60.50	63	62	61	61	61.75
L2	61	59	59	58	59.25	60	60	59	59	59.50
L3	61	61	61	61	61.00	60	59	59	60	59.50
L4	65	65	63	63	64.00	65	66	64	61	64.00
L5	63	63	63	61	62.50	64	62	62	61	62.25
Mean	61.45					61.40				

Table C.35: Field texture measurements, Taxiway “F”

Time (month)	Temp. (°C)	LOC	D (mm)					TD (mm)	Mean TD (mm)
			D1	D2	D3	D4	Avg.		
1.2	31	L1	39.0	38.5	38.5	38.5	38.63	0.217	0.198
		L2	36.0	38.0	37.0	40.0	37.75	0.227	
		L3	46.0	42.0	45.0	46.0	44.75	0.161	
		L4	37.0	46.0	40.0	43.0	41.50	0.188	
3.8	1	L1	32.5	35.5	32.0	32.5	33.13	0.295	0.270
		L2	35.5	36.5	37.0	37.0	36.50	0.243	
		L3	36.0	33.5	36.0	34.5	35.00	0.264	
		L4	36.5	35.0	34.0	35.0	35.13	0.262	
		L5	34.0	33.5	33.5	34.0	33.75	0.284	
6.8	-13	L1	32.5	33.0	34.0	35.0	33.63	0.286	0.301
		L2	31.0	34.0	32.0	31.0	32.00	0.316	
10.3	14	L1	28.5	30.0	28.0	29.5	29.00	0.385	0.342
		L2	31.0	32.0	35.0	32.0	32.50	0.306	
		L3	32.5	30.0	31.5	31.0	31.25	0.331	
		L4	32.5	30.5	31.0	31.0	31.25	0.331	
		L5	28.0	31.0	31.0	30.0	30.00	0.359	
11.5	18	L1	29.0	27.0	29.0	29.0	28.50	0.398	0.302
		L2	36.0	33.0	34.0	35.0	34.50	0.272	
		L3	33.0	33.0	33.0	31.0	32.50	0.306	
		L4	36.0	37.0	37.0	36.0	36.50	0.243	
		L5	32.0	35.5	32.0	33.5	33.25	0.293	

Table C.36: Field texture measurements, Taxiway “B”

Time (month)	Temp. (°C)	LOC	D (mm)					TD (mm)	Mean TD (mm)
			D1	D2	D3	D4	Avg.		
0.6	32	L1	31.0	31.0	30.8	30.5	30.81	0.341	0.342
		L2	32.5	32.0	31.7	31.5	31.93	0.317	
		L3	25.0	28.0	26.5	26.5	26.50	0.461	
		L4	32.0	34.5	32.5	32.7	32.93	0.298	
		L5	33.0	33.5	31.0	35.0	33.13	0.295	
3.5	3	L1	30.5	27.0	30.0	28.0	28.88	0.388	0.381
		L2	32.0	29.5	34.0	30.5	31.50	0.326	
		L3	32.5	30.0	31.0	32.0	31.38	0.329	
		L4	29.5	28.5	28.0	27.5	28.38	0.402	
		L5	28.5	25.5	27.0	25.0	26.50	0.461	
6.3	-17	L1	29.0	30.0	27.0	30.0	29.00	0.385	0.381
		L2	30.0	29.0	27.0	31.0	29.25	0.378	
9.9	13	L1	34.0	34.0	33.0	32.5	33.38	0.290	0.271
		L2	36.0	35.0	35.0	34.0	35.00	0.264	
		L3	34.5	34.5	34.5	34.5	34.50	0.272	
		L4	39.0	34.0	39.0	34.0	36.50	0.243	
		L5	33.5	33.0	35.0	33.5	33.75	0.284	
11.2	20	L1	41.0	42.0	38.0	40.0	40.25	0.200	0.247
		L2	36.0	33.0	33.0	32.0	33.50	0.288	
		L3	37.0	33.0	34.0	37.5	35.38	0.258	
		L4	34.0	36.0	37.0	39.0	36.50	0.243	
		L5	38.0	34.0	36.5	36.0	36.13	0.248	

APPENDIX D:

SHEAR PROPERTIES

Table D.1: Shear test results using the Torque Machine, Mix7

No.	Temp. (°C)	h (mm)	D (mm)	T (N.m)	SS (MPa)	Mean
27	25	59.8	101.6	348	1.6892	1.2846
29		60.8	101.5	220	1.0711	
38		60.2	101.9	250	1.2029	
21		59.4	101.6	250	1.2135	
15		59.2	101.5	256	1.2463	
53	35	59.1	101.5	204	0.9932	0.8679
24		56.4	101.6	192	0.9320	
22		61.6	101.6	190	0.9223	
9		61.3	101.6	108	0.5242	
14		52.5	101.7	200	0.9680	
33	45	60.3	101.7	50	0.2420	0.5095
16		59.2	101.8	114	0.5501	
52		60.7	101.7	128	0.6195	
50		59.2	101.6	158	0.7670	
62		60.1	101.6	76	0.3689	

Table D.2: Shear test results using the Torque Machine, Mix8

No.	Temp. (°C)	h (mm)	D (mm)	T (N.m)	SS (MPa)	Mean
14	25	61.6	101.6	126	0.6116	0.7481
24		60.4	101.6	130	0.6310	
2		59.8	101.6	136	0.6602	
32		61.4	101.6	220	1.0679	
22		61.6	101.7	159	0.7695	
12	35	57.8	101.6	142	0.6893	0.5631
1		61.9	101.5	96	0.4674	
4		60.3	101.6	118	0.5728	
23		60.1	101.7	100	0.4840	
37		58.7	101.6	124	0.6019	
6	45	59.7	101.7	68	0.3291	0.4442
3		55.3	101.7	60	0.2904	
9		61.4	101.6	94	0.4563	
35		61.5	101.7	138	0.6679	
39		60.9	101.5	98	0.4771	

Table D.3: Shear test results using the Torque Machine, Mix9

No.	Temp. (°C)	h (mm)	D (mm)	T (N.m)	SS (MPa)	Mean
6	25	58.5	101.6	184	0.8932	0.8385
10		61.5	101.8	170	0.8204	
37		60.7	101.6	174	0.8446	
22		61.1	101.6	116	0.5631	
14		57.6	101.5	220	1.0711	
7	35	57.1	101.6	142	0.6893	0.7349
12		61.7	101.5	142	0.6913	
13		61.4	101.7	142	0.6873	
21		57.7	101.6	168	0.8155	
43		59.6	101.6	163	0.7912	
3	45	61.3	101.6	68	0.3301	0.5029
11		60.9	101.7	76	0.3678	
41		59.1	101.5	125	0.6086	
26		60.8	101.8	138	0.6659	
31		58.4	101.7	112	0.5421	

Table D.4: Calculated SS based on shear stress parameters from triaxial test

Mix	Temperature (°C)	σ_n (MPa)	τ (MPa)
7	25	0.689	2.2038
		1.034	2.5485
		1.379	2.8932
	35	0.689	1.8723
		1.034	2.2052
		1.379	2.5381
	45	0.689	1.5264
		1.034	1.8711
		1.379	2.2158
8	25	0.689	1.9496
		1.034	2.2825
		1.379	2.6154
9	25	0.689	2.0619
		1.034	2.3948
		1.379	2.7277

Table D.5: Indirect tensile strength and moisture sensitivity test results, Mix1

No.	Type	Set	h (mm)	D (mm)	Load (kN)	ITS (MPa)	Mean	TSR (%)
4	M	dry	62.5	101.7	9.25	0.9243	0.9466	95.42
8			59.0	101.6	8.99	0.9519		
10			59.0	101.7	9.11	0.9637		
1		cond.	63.5	101.5	9.24	0.9098	0.9033	
2			62.5	101.5	8.29	0.8296		
9			61.0	101.5	9.47	0.9705		
1	E	dry	63.5	96.2	7.40	0.7693	0.8079	N.A
2			63.5	96.2	7.11	0.7387		
3			62.0	96.2	7.92	0.8428		
4			62.5	96.2	7.62	0.8045		
5			62.5	96.2	8.37	0.8841		

M: Specimens compacted using the Marshall compactor.

E: Specimens extracted from slab.

Table D.6: Indirect tensile strength and moisture sensitivity test results, Mix2

No.	Type	Set	h (mm)	D (mm)	Load (kN)	ITS (MPa)	Mean	TSR (%)
1	M	dry	59.0	101.5	10.88	1.1538	0.9879	96.31
8			62.5	101.5	6.81	0.6810		
9			60.0	101.5	10.83	1.1288		
4		cond.	58.5	101.5	10.79	1.1538	0.9514	
5			58.2	101.7	11.21	1.2020		
10			62.5	101.6	4.99	0.4985		
1	E	dry	63.5	96.2	5.81	0.6037	0.6800	N.A
2			63.5	96.2	6.83	0.7099		
3			62.5	96.2	6.59	0.6962		
4			62.0	96.2	6.15	0.6541		
5			62.5	96.2	6.97	0.7359		

M: Specimens compacted using the Marshall compactor.

E: Specimens extracted from slab.

Table D.7: Indirect tensile strength and moisture sensitivity test results, Mix3

No.	Type	Set	h (mm)	D (mm)	Load (kN)	ITS (MPa)	Mean	TSR (%)
5	M	dry	60.0	101.6	8.43	0.8779	0.8050	57.04
6			55.5	101.5	6.28	0.7075		
7			62.0	101.8	8.25	0.8296		
3		cond.	54.5	101.8	4.04	0.4625	0.4591	
8			68.5	101.6	4.75	0.4330		
9			59.0	101.6	4.55	0.4820		
1	E	dry	63.0	96.2	5.12	0.5358	0.5590	N.A
2			63.0	96.2	6.42	0.6720		
3			62.5	96.2	4.65	0.4914		
4			63.5	96.2	5.45	0.5666		
5			63.0	96.2	5.05	0.5289		

M: Specimens compacted using the Marshall compactor.

E: Specimens extracted from slab.

Table D.8: Indirect tensile strength and moisture sensitivity test results, Mix4

No.	Type	Set	h (mm)	D (mm)	Load (kN)	ITS (MPa)	Mean	TSR (%)
2	M	dry	63.6	101.6	5.94	0.5835	0.7271	98.72
6			61.5	101.5	7.48	0.7604		
9			64.2	101.5	8.60	0.8374		
3		cond.	64.8	101.5	7.73	0.7465	0.7178	
4			63.1	101.5	7.64	0.7568		
8			61.1	101.6	6.36	0.6500		
1	E	dry	63.7	96.2	5.34	0.5536	0.5684	N.A
2			63.3	96.2	5.05	0.5260		
3			63.2	96.2	5.50	0.5743		
4			63.1	96.2	5.43	0.5683		
5			63.5	96.2	5.96	0.6196		

M: Specimens compacted using the Marshall compactor.

E: Specimens extracted from slab.

Table D.9: Indirect tensile strength and moisture sensitivity test results, Mix5

No.	Type	Set	h (mm)	D (mm)	Load (kN)	ITS (MPa)	Mean	TSR (%)
1	M	dry	66.1	101.5	8.62	0.8156	0.8143	103.71
3			62.8	101.5	8.91	0.8871		
5			64.2	101.5	7.60	0.7402		
2		cond.	62.2	101.6	7.29	0.7324	0.8445	
8			63.8	101.6	8.11	0.7938		
9			60.2	101.9	9.73	1.0072		
1	E	dry	62.8	96.2	7.73	0.8123	0.7904	N.A
2			63.9	96.2	6.93	0.7152		
3			63.7	96.2	7.31	0.7581		
4			63.8	96.2	7.65	0.7922		
5			64.2	96.2	8.50	0.8742		

M: Specimens compacted using the Marshall compactor.

E: Specimens extracted from slab.

Table D.10: Indirect tensile strength and moisture sensitivity test results, Mix6

No.	Type	Set	h (mm)	D (mm)	Load (kN)	ITS (MPa)	Mean	TSR (%)
2	M	dry	62.8	101.6	5.72	0.5693	0.6184	97.81
6			59.9	101.6	5.43	0.5660		
9			62.3	101.5	7.17	0.7198		
1		cond.	59.2	101.6	6.35	0.6705	0.6048	
4			66.3	101.5	6.33	0.5974		
10			68.6	101.6	6.00	0.5466		
1	E	dry	64.0	96.2	5.87	0.6054	0.5667	N.A
2			63.1	96.2	5.41	0.5658		
3			63.8	96.2	5.51	0.5700		
4			64.0	96.2	4.76	0.4906		
5			63.6	96.2	5.80	0.6015		

M: Specimens compacted using the Marshall compactor.

E: Specimens extracted from slab.

Table D.11 Indirect tensile strength and moisture sensitivity test results, Mix7

No.	Type	Set	h (mm)	D (mm)	Load (kN)	ITS (MPa)	Mean	TSR (%)
57	M	dry	60.8	101.6	14.61	1.5005	1.2246	96.77
17			61.1	101.7	12.62	1.2907		
26			56.4	101.6	9.87	1.0946		
3			59.7	101.5	8.87	0.9301		
11			58.5	101.6	12.23	1.3069		
60		cond.	62.2	101.7	13.54	1.3600	1.2655	
54			60.9	101.8	13.66	1.3987		
19			54.3	101.8	8.91	1.0251		
43			59.0	101.9	10.55	1.1143		
23			55.3	101.5	12.64	1.4295		
1	E	dry	64.3	96.2	5.34	0.5480	0.6548	N.A.
2			64.3	96.2	5.61	0.5760		
3			65.9	96.2	7.08	0.7099		
4			65.3	96.2	6.48	0.6548		
5			66.1	96.2	7.87	0.7855		

M: Specimens compacted using the Marshall compactor.

E: Specimens extracted from slab.

Table D.12: Indirect tensile strength and moisture sensitivity test results, Mix8

No.	Type	Set	h (mm)	D (mm)	Load (kN)	ITS (MPa)	Mean	TSR (%)
8	M	dry	58.5	101.6	7.93	0.8465	0.9790	95.37
21			61.9	101.7	8.70	0.8776		
7			57.3	101.8	8.07	0.8795		
42			60.6	101.8	12.00	1.2362		
33			62.1	101.8	10.51	1.0552		
18		cond.	59.8	101.7	9.87	1.0310	0.9337	
25			60.7	101.6	7.01	0.7218		
5			60.1	101.7	8.16	0.8475		
11			60.8	101.8	9.51	0.9762		
38			60.9	101.7	10.65	1.0920		

M: Specimens compacted using the Marshall compactor.

Table D.13: Indirect tensile strength and moisture sensitivity test results, Mix9

No.	Type	Set	h (mm)	D (mm)	Load (kN)	ITS (MPa)	Mean	TSR (%)
4	M	dry	59.5	101.8	9.09	0.9527	1.0666	86.87
1			55.9	101.6	8.61	0.9631		
23			61.5	101.7	12.16	1.2363		
17			59.4	101.6	10.75	1.1310		
34			60.2	101.6	10.11	1.0501		
2		cond.	60.2	101.6	7.42	0.7712	0.9266	
8			61.4	101.6	8.05	0.8189		
18			58.2	101.5	11.36	1.2221		
9			61.0	101.6	8.88	0.9104		
38			58.3	101.6	8.49	0.9103		
1	E	dry	65.5	96.2	9.84	0.9914	0.9575	N.A.
2			65.7	96.2	8.91	0.8949		
3			66.5	96.2	10.02	0.9946		
4			66.6	96.2	9.34	0.9252		
5			64.6	96.2	9.60	0.9813		

M: Specimens compacted using the Marshall compactor.

E: Specimens extracted from slab.

Figure D.1: Determination of shear parameters by triaxial test, Mix7, room temperature

Figure D.2: Determination of shear parameters by triaxial test, Mix7, 35°C

Figure D.3: Determination of shear parameters by triaxial test, Mix7, 45°C

Figure D.4: Determination of shear parameters by triaxial test, Mix8, room temperature

Figure D.5: Determination of shear parameters by triaxial test, Mix9, room temperature

Figure D.6: Determination of shear parameters by direct shear test, Mix7, room temperature

Figure D.7: Determination of shear parameters by direct shear test, Mix7, 35°C

Figure D.8: Determination of shear parameters by direct shear test, Mix7, 45°C

Figure D.9: Determination of shear parameters by direct shear test, Mix8, room temperature

Figure D.10: Determination of shear parameters by direct shear test, Mix9, room temperature

APPENDIX E:

ADJUSTED FRICTIONAL PROPERTIES

Table E.1: Friction values (BPN), Mix7 at 35°C

Slab	Direction	R1	R2	R3	R4
1	LOD	62	62	61	61
	TRD	63	62	61	61
2	LOD	64	63	64	63
	TRD	64	64	64	63
3	LOD	61	61	61	61
	TRD	65	65	65	65
4	LOD	64	64	63	64
	TRD	65	65	65	65
	TRD (Fluid)	59	58	57	57

Table E.2: Friction values (BPN), Mix7 at room temperature

Slab	Direction	R1	R2	R3	R4
1	LOD	66	65	64	63
	TRD	65	65	65	64
2	LOD	67	66	66	66
	TRD	67	66	66	66
3	LOD	65	65	65	65
	TRD	68	68	68	68
4	LOD	68	66	66	66
	TRD	69	69	69	69
	TRD (Fluid)	61	61	61	61

Table E.3: Friction values (BPN), Mix7 at 15°C

Slab	Direction	R1	R2	R3	R4
1	LOD	71	71	70	70
	TRD	70	70	70	71
2	LOD	73	72	71	71
	TRD	71	71	70	70
3	LOD	72	71	71	70
	TRD	73	73	73	72
4	LOD	71	70	70	70
	TRD	75	74	75	73
	TRD (Fluid)	66	66	66	65

Table E.4: Friction values (BPN), Mix7 at 5°C

Slab	Direction	R1	R2	R3	R4
1	LOD	74	74	73	74
	TRD	72	73	72	73
2	LOD	76	75	75	75
	TRD	75	75	75	75
3	LOD	75	75	75	74
	TRD	75	76	75	75
4	LOD	75	75	75	75
	TRD	75	75	75	75
	TRD (Fluid)	67	66	66	66

Table E.5: Friction values (BPN), Mix7 at -10°C using fluid

Slab	Direction	R1	R2	R3	R4
1	LOD	69	69	68	69
	TRD	68	69	69	69
2	LOD	69	68	69	68
	TRD	70	70	70	70
3	LOD	67	67	66	66
	TRD	70	70	70	70
4	LOD	70	68	67	68
	TRD	70	70	69	69

Table E.6: Corrected friction values (BPN), Mix7 at -10°C

Slab	Direction	R1	R2	R3	R4
1	LOD	77.97	77.97	76.84	77.97
	TRD	76.84	77.97	77.97	77.97
2	LOD	77.97	76.84	77.97	76.84
	TRD	79.10	79.10	79.10	79.10
3	LOD	75.71	75.71	74.58	74.58
	TRD	79.10	79.10	79.10	79.10
4	LOD	79.10	76.84	75.71	76.84
	TRD	79.10	79.10	77.97	77.97

Table E.7: Friction values (BPN), Mix7 at -20°C using fluid

Slab	Direction	R1	R2	R3	R4
1	LOD	69	69	69	69
	TRD	69	69	69	69
2	LOD	70	70	70	70
	TRD	70	70	70	70
3	LOD	69	69	70	69
	TRD	70	70	70	71
4	LOD	70	70	71	70
	TRD	71	73	72	71

Table E.8: Corrected friction values (BPN), Mix7 at -20°C

Slab	Direction	R1	R2	R3	R4
1	LOD	77.97	77.97	77.97	77.97
	TRD	77.97	77.97	77.97	77.97
2	LOD	79.10	79.10	79.10	79.10
	TRD	79.10	79.10	79.10	79.10
3	LOD	77.97	77.97	79.10	77.97
	TRD	79.10	79.10	77.97	80.23
4	LOD	79.10	79.10	80.23	79.10
	TRD	80.23	82.49	81.36	80.23

Table E.9: Friction values (BPN), Mix7 at -30°C using fluid

Slab	Direction	R1	R2	R3	R4
1	LOD	69	69	70	70
	TRD	69	69	69	69
2	LOD	70	70	70	70
	TRD	70	70	70	70
3	LOD	69	69	70	69
	TRD	72	71	71	71
4	LOD	75	72	71	71
	TRD	74	73	74	73

Table E.10: Corrected friction values (BPN), Mix7 at -30°C

Slab	Direction	R1	R2	R3	R4
1	LOD	77.97	77.97	79.10	79.10
	TRD	77.97	77.97	77.97	77.97
2	LOD	79.10	79.10	79.10	79.10
	TRD	79.10	79.10	79.10	79.10
3	LOD	77.97	77.97	79.10	77.97
	TRD	81.36	80.23	80.23	80.23
4	LOD	84.75	81.36	80.23	80.23
	TRD	83.62	82.49	83.62	82.49

Table E.11: Friction values (BPN), Mix8 at 35°C

Slab	Direction	R1	R2	R3	R4
1	LOD	62	62	61	62
	TRD	64	65	64	65
2	LOD	65	63	63	63
	TRD	64	64	64	64
3	LOD	64	64	63	64
	TRD	64	63	63	64
4	LOD	65	65	64	64
	TRD	64	64	64	63
	TRD (Fluid)	57	56	56	56

Table E.12: Friction values (BPN), Mix8 at room temperature

Slab	Direction	R1	R2	R3	R4
1	LOD	67	66	66	66
	TRD	70	70	69	69
2	LOD	67	66	67	66
	TRD	68	67	66	66
3	LOD	67	66	66	66
	TRD	67	67	66	66
4	LOD	67	66	66	66
	TRD	68	66	66	66
	TRD (Fluid)	59	59	58	58

Table E.13: Friction values (BPN), Mix8 at 15°C

Slab	Direction	R1	R2	R3	R4
1	LOD	74	73	73	72
	TRD	74	74	73	73
2	LOD	72	72	71	71
	TRD	73	72	72	71
3	LOD	74	73	73	74
	TRD	74	73	72	72
4	LOD	74	73	72	72
	TRD	76	75	75	74
	TRD (Fluid)	66	66	66	66

Table E.14: Friction values (BPN), Mix8 at 5°C

Slab	Direction	R1	R2	R3	R4
1	LOD	75	76	75	75
	TRD	77	76	76	75
2	LOD	74	75	75	76
	TRD	76	75	75	75
3	LOD	76	76	76	76
	TRD	75	74	75	75
4	LOD	76	76	77	76
	TRD	78	78	78	77
	TRD (Fluid)	69	68	68	68

Table E.15: Friction values (BPN), Mix8 at -10°C using fluid

Slab	Direction	R1	R2	R3	R4
1	LOD	69	68	70	70
	TRD	71	71	70	70
2	LOD	69	69	70	70
	TRD	70	70	70	70
3	LOD	70	70	70	70
	TRD	70	70	70	70
4	LOD	71	70	70	70
	TRD	73	72	71	72

Table E.16: Corrected friction values (BPN), Mix8 at -10°C

Slab	Direction	R1	R2	R3	R4
1	LOD	78.52	77.38	79.66	79.66
	TRD	80.80	80.80	79.66	79.66
2	LOD	78.52	78.52	79.66	79.66
	TRD	79.66	79.66	79.66	79.66
3	LOD	79.66	79.66	79.66	79.66
	TRD	79.66	79.66	79.66	79.66
4	LOD	80.80	79.66	79.66	79.66
	TRD	83.07	81.94	80.80	81.94

Table E.17: Friction values (BPN), Mix8 at -20°C using fluid

Slab	Direction	R1	R2	R3	R4
1	LOD	68	69	69	70
	TRD	71	72	72	72
2	LOD	70	70	71	71
	TRD	72	72	72	72
3	LOD	71	71	71	71
	TRD	71	72	72	72
4	LOD	73	72	72	72
	TRD	75	75	75	75

Table E.18: Corrected friction values (BPN), Mix8 at -20°C

Slab	Direction	R1	R2	R3	R4
1	LOD	77.38	78.52	78.52	79.66
	TRD	80.80	81.94	81.94	81.94
2	LOD	79.66	79.66	80.80	80.80
	TRD	81.94	81.94	81.94	81.94
3	LOD	80.80	80.80	80.80	80.80
	TRD	80.80	81.94	81.94	81.94
4	LOD	83.07	81.94	81.94	81.94
	TRD	85.35	85.35	85.35	85.35

Table E.19: Friction values (BPN), Mix8 at -30° C using fluid

Slab	Direction	R1	R2	R3	R4
1	LOD	72	72	71	70
	TRD	75	76	75	75
2	LOD	76	76	75	74
	TRD	76	75	75	75
3	LOD	76	75	74	73
	TRD	78	75	75	74
4	LOD	73	74	72	72
	TRD	78	78	76	75

Table E.20: Corrected friction values (BPN), Mix8 at -30°C

Slab	Direction	R1	R2	R3	R4
1	LOD	81.94	81.94	80.80	79.66
	TRD	85.35	86.49	85.35	85.35
2	LOD	86.49	86.49	85.35	84.21
	TRD	86.49	85.35	85.35	85.35
3	LOD	86.49	85.35	84.21	83.07
	TRD	88.76	85.35	85.35	84.21
4	LOD	83.07	84.21	81.94	81.94
	TRD	88.76	88.76	86.49	85.35

Table E.21: Adjusted friction values (BPN), Taxiway “F”, OIA

Time (month)	Temp. (°C)	Measured	Corrected for fluid	CF _t	Corrected for temperature
1.2	31	68.0	68.0	0.99	68.9
3.8	1	82.3	82.3	1.11	74.1
6.8	-13	75.4	85.2	1.17	73.0
7.3	-6	73.1	82.6	1.14	72.5
8.3	-5	70.4	79.6	1.14	70.1
9.2	5	68.8	68.8	1.10	62.8
10.3	14	67.0	67.0	1.06	63.2
11.5	18	64.7	64.7	1.04	62.0

Table E.22: Adjusted friction values (BPN), Taxiway “B”, OIA

Time (days)	Temp. (°C)	Measured	Corrected for fluid	CF_t	Corrected for temperature
0.6	32	71.6	71.6	0.98	72.8
3.5	3	79.5	79.5	1.12	70.7
6.3	-17	75.4	85.7	1.22	70.3
7.0	-8	69.6	79.0	1.18	67.2
7.9	-7	67.6	76.8	1.17	65.6
8.8	5	65.1	65.1	1.11	58.4
9.9	13	63.1	63.1	1.08	58.7
11.2	20	61.5	61.5	1.04	59.0

APPENDIX F:

REGRESSION ANALYSIS

Scatterplot

Dependent Variable: F

F

Figure F.1: Example of residual scatterplots, Equation (6.3)