Modification and Verification of the InSiSST Facility for Evaluation of Asphalt Concrete Shear Properties

Submitted by

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This thesis is humbly dedicated to:

The pure soul of my beloved father,

Dr. Heshmatollah Rohani

Bestowed with God's Grace

My dear mother,

Dr. Mahindokht Khorramianpour

My treasured husband,

Peyman Misaghi

All Baha'i students, around the universe, who are
deprived of higher educations

and

To all who wholeheartedly devoted their lives, time and energy
to advance the education level of the Iranian Baha'i youth

Abstract

The quality control of newly constructed asphalt layers and their long-term performance are becoming two important issues in today's paving industry. Rutting is among the main distress types, considered as significant indicator of poor long-term performance. Shear properties of asphalt mixes are known to be among the most important factors in resisting rutting. In this regard, the InSiSST facility was developed at Carleton University to measure the in-situ shear properties. This thesis introduces the main steps leading to the present test facility. It also describes the new proposed enhancements and their effect on the overall capabilities of the machine. Laboratory and field investigations have been carried out to examine and validate the improved testing facility. It is concluded that the new enhancements have significantly improved the abilities of the upgraded facility in measuring the torque and angular displacement more accurately, controlling the test temperature and increasing the testing speed.

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Nomenclature

AC	: Asphalt Content
ALF	: Accelerated Load Facility
ANOVA	: ANalysis Of VAriance
APA	: Asphalt Pavement Analyzer
AV	: Percent Air Voids
CiSSST	: Carleton In-Situ Shear Strength Test
CoV	: Coefficient of Variation
FSCH	: Frequency Sweep at Constant Height
FST	: Field Shear Tester
GLWT	: Georgia Loaded Wheel Test
GMB	: Bulk specific gravity
HMA	: Hot Mix Asphalt
InSiSST	: In-Situ Shear Stiffness Test
LCPC	: Laboratoire Central des Ponts et Chaussees
LSAM	: Large-Stone Asphalt Mixes
LVDT	: Linear Variable Displacement Transducer
MG-type	: Multi-Grade-type
MTO	: Ministry of Transportation of Ontario
NCAT	: National Center for Asphalt Technology
NCHRP	: National Cooperative Highway Research Program
NRC	: National Research Council
OECD	: Organisation of Economic Co-operation for Development
PG	: Performance Graded
QA	: Quality Assurance

QC	: Quality Control
rpm	: revolution per minute
RSCH	: Repeated Shear at Constant Height
SHRP	: Strategic Highway Research Program
SST	: Superpave Shear Tester
Superpave	: <u>S</u> uperior <u>P</u> erforming Asphalt <u>P</u> avements
TAC	: Transportation Association of Canada
USB	: Universal Serial Bus
VFA	: Percent Voids Filled with Asphalt
VMA	: Percent Voids in Mineral Aggregate

Chapter 1. Introduction

1.1. Background

“In recent decades, pavement engineers have been challenged to use conventional methods to design cost-effective pavements that are expected to withstand increasing traffic with unconventional wheel loads and tire pressures” (Mahboub and Allen, 1990). The trend toward heavier trucks, higher tire inflation pressure, and the substantial increase in the number of load repetitions have, in turn, resulted in a significant increase in the extent and severity of surface deformation and rutting.

The fact that permanent deformation is a direct result of shear forces or stresses makes it necessary to consider shear properties when long-term performance of asphalt pavements is considered. Previous studies have shown that the shear resistance of asphalt concrete mixes is a fundamental property in determining the ability of the mix to resist lateral deformation (Abd El Halim et al. 1995). In a laboratory investigation, Abd El Halim et al. (1997) have shown that improving the shear strength of the asphalt concrete mix can substantially reduce the longitudinal depression in the wheel-paths of asphalt pavements. Therefore, shear strength is a mechanical property that has a significant influence on asphalt pavement’s durability and has motivated pavement researchers to develop laboratory shear test facilities for decades. The Strategic Highway Research Program (SHRP) has finally adopted the shear strength test in its new Superior Performing Asphalt Pavements (Superpave) design methods. The objective of the Superpave test facility is to design asphalt mixes based on mechanical properties known to govern the long-term performance of the asphalt mixes in the field.

Many improvements to the practices of asphalt pavement construction have been introduced during the last decade. All aspects of asphalt concrete mixing and construction technologies including the mix design, construction, Quality Control (QC),

and Quality Assurance (QA) were involved in these improvements. The aim of most of these improvements was to produce asphalt concrete layers that provide better performance at a lower life cycle cost. The traditional asphalt mix design methods changed to the performance-based superpave mix design method to relate the design aspects of the asphalt mix to the field performance. Construction of highways also shifted from the traditional system, where the design and specifications are completed by the agencies, to the implementation of the End-Results specifications for acceptance or rejection of a completed asphalt concrete layers.

In the implementation of the End-Results specifications, the contractor is required to be responsible for controlling the materials and construction of the asphalt concrete, while the agency determines the acceptability of the finished work. The main disadvantage of the current QC/QA practices is that they use empirical material indices as a measurement of the asphalt concrete mix quality and do not include any mechanistic material properties that can be related to the actual performance of the mix. Moreover, the QC/QA requirements are mainly based on laboratory testing results. The superpave tests can measure the mechanistic properties of the asphalt mix, however, they are performed in the laboratory rather than in the field. As a result, the problems associated with shear strength determination in the laboratory prevent these tests to be acceptable predictors of the field performance.

The quality of the laboratory prepared samples is significantly influenced by the boundary conditions imposed by the mould, compaction technique, and compaction effort. Therefore, the outcome of laboratory preparation procedure does not simulate the characteristics of mixes in the field effectively. Also, the ability of the laboratory tests in facilitating performance prediction is questionable when the test is performed on cores recovered from the field. The drilling and transportation have destructive effects on the

cored samples taken from the field. In addition, different boundary conditions would be present when the recovered cores are subject to laboratory loadings.

Moreover, because of the effects of temperature, moisture, road geometry, and traffic on pavement materials, knowledge of the in-situ material properties of pavement layers (as is in the field) is essential for evaluating the effective structural capacity of the pavement. Abdel Nabi et al. (1995) concluded that relating pavement distress to in-situ shear strength can provide a realistic criterion for Pavement Management Systems.

1.2. Problem Definition

The InSiSST facility has been designed and built at Carleton University for the purpose of evaluating the shear stiffness of the asphalt concrete mixtures in the field. The first two generations of the test facility applied the torque to the asphalt layer through forcing a steel plate bonded to the surface of the pavement to rotate about an axis normal to the surface. The testing facility has been equipped with a data acquisition system to record the instantaneous torque and the motor speed. The torque is measured directly using the torque cell. The angular displacement is calculated assuming that constant strain is applied. However, field-testing showed a number of issues that may influence the accuracy of the InSiSST and indicated that further improvement is required. These issues can be summarized as follows:

- The actual strain rate (angular displacement rate) during field-testing might not be constant; therefore, assuming a constant strain rate may significantly affect the accuracy of the field-testing results. Subsequently, the actual angular displacement needs to be directly measured during testing and should not be calculated assuming constant strain rate.
- In some cases, the epoxy used to bond the steel plate to the pavement surface failed prematurely prior to the failure in the asphalt concrete mix. Furthermore,

the epoxy which is currently used requires a 24-hour period before the test could be applied. In this case, field-testing needs to be completed over two days, because the plates are glued to the surface in the first day, while actual testing is carried out on the following day. As a result, further investigation to eliminate the use of epoxy is required to achieve a faster and more reliable field test.

- Field testing using the InSiSST is carried out with no control over the actual temperature during testing. Although this is expected for a field-testing such as the one utilizing InSiSST, the ability to control the actual temperature during testing will significantly enhance the performance of the InSiSST facility. Such an enhancement would result in the feasibility of functioning under different weather conditions as well as providing test results under a consistent field temperature.

1.3. Objectives

The objectives of the thesis can be summarized as follows:

- To modify the InSiSST facility according to the aforementioned deficiencies of the current configuration.
- To verify the performance of the modified InSiSST facility and examine the repeatability of the tests.
- To validate the results obtained from InSiSST testing and investigate the effect of different variables (plate size, temperature and plate type) on the measured shear properties.
- To investigate the correlation between mix shear properties and its long-term performance.

1.4. Scope of the study

This study focused on the improvement and verification of the InSiSST facility, validation of the new results, and laboratory testing to correlate the shear properties of asphalt concrete mixes to their long-term performance. The scope of the thesis was limited to the Hot Mix Asphalt (HMA) pavements. The shear properties of asphalt mixes were studied in the field and the effect of temperature and plate size on the in-situ measured shear properties was investigated. After verifying the applicability of the vane plates the study was narrowed down to one plate size and type. It should be noted that the effect of load rate was not considered in this research and the maximum motor speed was selected for the whole field-testing program.

1.5. Thesis Organization

In the second chapter of the thesis, a comprehensive literature review of the rutting/permanent deformation as one of the most serious distress modes in flexible pavements will be presented. The rutting mechanisms, its manifestations and evaluation methods, and finally the need for an in-situ shear tester together with the main steps leading to the development of the InSiSST at Carleton University will be explained. In Chapter Three, the proposed modifications for the InSiSST facility and the evolution of the test facility to the upgraded condition will be described in detail and the complete step-by-step testing procedure will be presented. Chapter Four mainly deals with the extensive laboratory and field testing program scheduled to meet the objectives of the study. In Chapter Five the results of the tests will be fully analyzed. Finally, the summary and the findings of the research as well as the recommendations for future work will be presented in Chapter Six.

Chapter 2. Literature Review

2.1. Introduction

Pavement performance is a function of its relative ability to serve traffic over a period of time for which it is designed. However, different failure types occur in roads during this life cycle due to the environmental factors, improper construction effort or poor design. Surface distress, defined as any indication of poor or unfavourable pavement performance or signs of impending failure can be broadly classified into the following three groups (Highway Research Board, 1970; WSDOT):

- *Fracture*, which could be in the form of cracking or spalling resulting from excessive loading, fatigue, thermal changes, moisture damage, slippage or contraction.
- *Distortion* that is in the form of deformation and can result from excessive loading, creep, densification, consolidation, swelling, or frost action. This type of distress includes rutting, corrugation and shoving.
- *Disintegration*, the form of stripping, ravelling or spalling, that can result from loss of bonding, chemical reactivity, traffic abrasion, aggregate degradation, poor consolidation/compaction or binder aging.

Among the aforementioned distresses, rutting is one of the significant distress modes in flexible pavements facing engineers and is gaining widespread attention in many parts of the world. Premature rutting of Hot Mix Asphalt (HMA) pavements has been experienced in several states in the United States in recent years (Kandhal et al. 1993). Al-Abdul Wahhab et al. (1995) mentioned that even though the roadway capacity in the Kingdom of Saudi Arabia can properly accommodate the large traffic volumes generated due to socioeconomic and industrial growth, noticeable rutting problems have been observed in the last 20 years. Similar to many cities and highway pavement

systems internationally, rutting of asphalt concrete across Canada has become a primary asphalt technology problem requiring cost-effective mitigation for both existing and new pavements (Emery, 1990).

2.2. Rutting In Flexible Pavements

2.2.1. Rutting Mechanism

Rutting is the formation of longitudinal depressions under the wheel-paths. Three types of ruts can develop in asphalt concrete pavements that can be summarized in the following three classes (Dawley et al. 1990):

- *Wear rutting*, which is due to the progressive loss of coated aggregate particles from the pavement surface and is caused by combined environmental and traffic influences. When studs wear the pavement, the material is first loosen by the stud strike and then flipped away from the pavement as the stud leaves the pavement surface (Figure 2.1a and Figure 2.2b).
- *Structural rutting*, which is due to the permanent vertical deformation of the pavement structure under repeated traffic loads. It is essentially a reflection of permanent deformation within the subgrade (Figure 2.1b).
- *Instability rutting*, which is due to lateral displacement of material within the pavement layer that occurs within the wheel-paths. Instability rutting occurs when the structural strength of the compacted pavement is inadequate to resist the stresses imposed upon the pavement, particularly by frequent repetitions of high axle loads (Figure 2.1c, Figure 2.2a, and 2.2c).

Figure 2.1 Description of Rutting Types: (a) Wear Rutting, (b) Structural Rutting, (c) Instability Rutting (Dawley et al. 1990)

The study by Dawley et al. (1990) showed that a substantial portion of the rutting resulted from mixture instability, but some portion of the rutting could have been due to structural inadequacy or the combination of both types. Trenches were excavated at eleven sites to differentiate between structural inadequacy and mixture instability rutting. The trenches revealed that essentially all of the rutting at each site was limited only to the surface layer.

Instability rutting develops in asphalt concrete surfaces through two major mechanisms: densification and plastic shear deformation. Densification of the asphalt concrete mix is the secondary compaction in the top layers due to traffic. Plastic Shear deformation is the lateral deformation of the surface layers. As a result of traffic loading, a depression is developed in the wheel-paths forcing the pavement material to heave up on the sides. Rutting caused by material densification in the wheel-path is usually minor and contributes to a small portion of the rut depth (Huber 1999). Eisenmann and Hilmer (1987) showed that if a pavement section is well compacted during construction, further densification during rutting is unlikely.

Trenching studies at the AASHO Road Test, as well as other full-scale test tracks, have indicated that shear deformation, rather than densification, is the primary rutting mechanism in asphalt concrete surface mixtures when reasonably stiff supporting layers are present (Epps et al. 1998). This statement was also confirmed through a comprehensive National Rutting Study that was completed in the United States by the National Center for Asphalt Technology (NCAT) in 1987. Moreover, Oteng-Seifah and Manke (1976) concluded that upheaval due to shear failure in the Hot Mix Asphalt layer was predominating in the absence of base and subgrade deformation.

A study commissioned for the Organization of Economic Co-operation for Development in 1988 also showed that rutting develops within the asphalt concrete layers when the stresses induced by the traffic loading are sufficient to cause shear displacement within the material (OECD, 1988). Observations have shown that in such cases rutting occurs in the top 100 mm of the asphalt-bound layers and the rut depth is proportional to the number of load applications (Al-Abdul Wahhab et al. 1995; Simpson, 1999).

Figure 2.2 (a) Rutting in a Bus Stop, (b) Wear Rutting, (c) Longitudinal Instability Rutting in the Wheel-Path

2.2.2. Manifestations of Rutting

A novel and accurate method of determining the layer at which rutting occurred was investigated by Simpson et al. (1995). They hypothesized that the area under the transverse surface profile could be used to predict the source of rutting within the pavement structure. Using data from the LTPP General Pavement Studies (GPS), Simpson et al. placed transverse surface profiles of rutted pavements in four general categories: subgrade rutting, base rutting, surface course rutting, and heave. Figure 2.3 presents the shape of the transverse surface profile associated with each category. Simpson et al. cited the elastic theory as the basis for the profile shapes. This suggests that one should be able to establish weaknesses in a pavement structure on the basis of the shape and dimensions of deformations at the pavement surface.

Figure 2.3 Transverse Surface Profiles by Rutting Failure Modes (Simpson et al. 1995)

Table 2.1 Classification Criteria for Rutting Origin (Simpson et al. 1995)

<i>Hypothesized Rutting Mode or Type</i>	<i>Transverse Profile</i>	
	Total Distortion (Total Area) (mm²)	Ratio of Distortion (Positive/Negative)
Subgrade	< -4500	< 0.4
Base	-4500 to 700	0.4 to 1.25
Surface	700 to 5000	1.25 to 3.0
Heave	> 500	> 3.0

The algebraic area between the transverse profile and the straight line connecting its end points was calculated by Simpson et al. (1995), to identify into which of the four categories the various transverse profiles fit. Area above the straight line connecting the profile end points was considered to be positive, and area below the line was considered to be negative. Therefore, sections within the subgrade category would be entirely negative and sections within the heave category would be entirely positive. Furthermore, distinguishing between the base and surface rutting, was handled using the general perception that marginally positive areas would be considered surface rutting and marginally negative areas would be considered base rutting.

The criteria used to classify transverse profiles from the US-LTPP database are shown in Table 2.1. Among the 134 sections analyzed, only six transverse profiles did not agree with the classification system; therefore, over 95 percent of the profiles were correctly categorized using the mentioned criteria.

2.2.3. Rutting in New Overlays

The use of an asphalt layer on top of an old pavement has resulted in a pavement system that needs more in-depth analysis to fully understand the main mechanisms that cause a new type of rutting within the new overlay itself. Field observations in Nova Scotia and Ontario reported the occurrence of surface rutting a few years after construction (Gervais and Abd El Halim, 1990). Premature rutting on heavily

travelled asphalt concrete overlays has also been documented to be a problem in Wisconsin (Sharma and Larson, 1990). Significant amount of this surface rutting occurs even in the newly constructed asphalt layer under moderate axle loads.

Inspection across sections taken from the distressed roads has revealed that this type of rutting does not occur at the expected locations of the top surface or the bottom of the overlay, but rather occurs within the asphalt layer. The fact that this type of rutting is limited to the new overlay would suggest that only the asphalt layer is subjected to critical compressive stresses and horizontal tensile strains.

Gervais and Abd El Halim (1990) suggested that the early occurrence of surface rutting is due to the difference in rigidity between the existing more rigid structure and the new asphalt layer which remains relatively soft for a few months after construction. When a soft layer on top of a more rigid one is subjected to the traffic loads, low compressive stresses accompanied by high tensile strains would result. The resulting stress and strain distributions can lead to the rutting observed within the new overlay.

2.2.4. Problems Caused by Rutting

Emery (1990) summarized the problems caused by rutting based on the findings of the series of the Rutting Seminars held across Canada in 1989. He classified these problems into the following categories:

1. Safety:
 - a. Vehicular
 - b. Pedestrian
2. Nuisance to pedestrians
3. Appearance
4. Cost to mitigate

Pavement rutting, when exceeding certain limits, restricts road use and can be a safety-hazard. Rutting presents a safety concern as water might accumulate in the depressed longitudinal wheel-paths and result in a potential of hydroplaning. The hydroplaning phenomenon is the build up of a thin layer of water between the pavement surface and the tires, resulting in reduced traction and a subsequent loss of steering control (Yoder and Witczak, 1975). In severely rutted highway sections, even when water is not present rutting may pose a safety hazard during lane changes and again lead to loss of steering control.

In addition, splash of the accumulated water by vehicles can bother pedestrians. Appearance of the rutted road although not regarded as a serious problem provides poor public perception and political reaction. Economically, rehabilitation of rutted roads cannot be done by localized treatment and generally requires overlaying the entire pavement section, thus costing taxpayers millions of dollars yearly.

2.3. Factors Affecting Instability Rutting

Several studies have been conducted to identify the parameters that influence susceptibility of a mix to rutting. They all agree that there is not a single factor that affects permanent deformation in the pavement. In general the factors that affect rutting rate and magnitude can be divided into two major categories, external and internal factors. External factors affecting rutting include traffic characteristics, construction effort, environmental factors and geometric design. Internal factors, on the other hand, consist of the properties of bituminous material, mineral aggregate, mix design parameters and engineering properties of the mix. The aforementioned factors will be explained in detail in the following subsections and are summarized in Table 2.2.

2.3.1. External Factors

Traffic characteristics, such as tire pressure, axle loading and axle configuration, were found to significantly affect rutting in flexible pavements (Archilla and Madanat, 2000). In recent years, several studies on the impact of tire characteristics on pavement have raised concern over the possibility of accelerated pavement deterioration, particularly rutting, caused by increasing tire pressures. The tires of the AASHO Road Test trucks of the 1950s were bias-ply construction with inflation pressures between 75 and 80 psi. The replacement of bias-ply tires with radial tires and higher inflation pressures, averaging 100 psi, resulted in a smaller size tire “footprint” on the pavement and, consequently, a concentration of weight over a smaller area. These changes hasten the wear of the flexible pavements, increasing the rate of rutting. In addition, rutting produced by two different wheel configurations for a given load, could be different due to the different stress distribution in the upper portion of the pavement layers (FHWA, 2000).

Axle configuration has also an influence on the occurrence of permanent deformation. A general finding from the previous studies is that the wide-base single tires appear to cause about 1.5 times more rutting than the dual tires on flexible pavements. Another finding is that one of the wheels in a dual tire assembly is frequently overloaded due to variability in the roadway cross-section. It was also found that the average overload causes an increase in rutting similar to that caused by wide-base single and dual tire assemblies (FHWA, 2000).

Load duration is another traffic-related factor that influences rutting. Rutting accumulates faster as the load duration increases (Anani et al. 1990). This is apparent in bus stops, intersections and climbing lanes.

Field compaction must achieve specified in-situ density and air voids to ensure adequate pavement performance. In fact, compaction can come from either construction

or traffic, but better compaction in construction will reduce the rut depth caused by densification (Harvey, 1994). Kandhal et al. (1993) explained that during the early stages of a pavement life cycle rutting is directly proportional to traffic volume. However, after the initial densification, the rate of rutting decreases with increasing traffic until a condition of plastic flow occurs and the rate of rutting increases again.

In addition, field compaction must strive for consistency throughout the pavement construction. Inconsistent compaction causes localized areas containing too little or too many air voids, allowing shear flow or traffic induced densification, respectively. Therefore, quality control (QC) and quality assurance (QA) are critical to pavement performance. Kandhal et al. (1993) stated that asphalt content measured from field cores was generally lower than the value specified for the job-mix formula. Clearly, improved control over asphalt content is required for plant production since asphalt content is one of the primary factors governing rutting performance of pavements.

Environmental factors include the effect of temperature, aging and moisture damage. The ambient temperature and duration of pavement exposure to sunlight affect pavement layers. Bituminous materials are black and therefore they easily absorb external heat while exhibiting a low coefficient of thermal conductivity (OECD, 1975). Anani et al. (1990) pointed out that in Saudi Arabia, where the pavement temperature can reach to more than 60°C, rutting is significantly reduced under bridges where the pavement is shaded by the deck. Emery (1990) also mentioned that rutting is more reported in hot weather areas.

Over time and environmental conditioning, asphalt cement loses some of its flexibility and as a result, its stiffness increases. Therefore, the oxidation of asphalt cement increases the pavement resistance to permanent deformation so long as the bond between the asphalt cement and aggregate particles is maintained. Stripping, defined as the removal of cement from the mineral aggregates through moisture

infiltration, occurs from the asphalt layers due to poor drainage conditions or sealing of the asphalt surface. The loss of bond between asphalt and aggregate allows the aggregate particles to move or shift under traffic loading, promoting pavement deformation.

Finally, geometric design influences asphalt pavement distress to a great extent. Abdel Nabi (1995) demonstrated that route alignment significantly influences rutting performance. Pavement sections on hills and curves often display increased rutting due to reduced traffic speed and gravity-induced creep.

2.3.2. Internal Factors

As mentioned earlier, the internal factors affecting rutting or permanent deformation are the properties of the binder and the aggregate, mix design parameters and mix properties. Bituminous materials are petroleum-based products of oil refining or naturally occurring asphalts (TAC, 1997). Asphalt is a dark brown to black solid or semisolid cementitious material that softens and liquefies when heated. Asphalt cement receives further refinement to meet specifications for paving and related purposes. Among the properties of asphalt cement some believe that viscosity, that governs the bond and interlock between the asphalt cement and the aggregate of the mix, influences the susceptibility to rutting. The study by Zahw et al. (1996) indicated that improving the viscosity of the material would increase the mix resistance to rutting. Nievelt and Thamfeld (1988) concluded that asphalt cements with higher viscosity values produced asphalt mixes with greater rutting resistance as tested using wheel-tracking tests on samples at multiple test temperatures. On the other hand, Huber and Heiman (1987) indicated that penetration and viscosity did not demonstrate a significant effect on rutting performance. In fact, penetration and viscosity-based specifications may be adequate for climatic regions where the performance temperature range between maximum and

minimum is not great enough to exceed critical bitumen properties (Deme et al. 1995).

As mentioned before, asphalt cement properties are highly temperature-sensitive. As temperature of the asphalt cement increases, its stiffness decreases, thereby increasing the potential for permanent deformation. This high temperature susceptibility may be reduced through the addition of polymer modifiers. The study by Abd El Halim et al. (1995) revealed that the polymer-modified asphalt cement would produce mixes with higher shear and indirect tensile strengths and better resistance to rutting, compared to the other conventional asphalt cement types used in the study.

On the other hand, Multi-Grade-type (MG-type) bitumens, with broader PG span, facilitate the use of binders of higher viscosity at high temperatures without necessarily compromising pavement performance at low temperatures. An example of this is rutting experiments carried out by Valkering et al. (1990). The MG-type binder mix yielded a reduction in rut depth of over fifty percent relative to the conventional binder mix. In addition, extrapolation on the basis of reduced rutting rate showed an increase of pavement life by a factor greater than 7.

It is believed that the ability of pavements to resist rutting from traffic load is largely dependent upon aggregate characteristics, including the aggregate type and consensus properties, and to a lesser extent on bitumen properties. Sherif et al. (1997) evaluated the effect of aggregate type (crushed stone and natural gravel) and concluded that this property of the aggregate together with one of the mechanical properties of the mix can be used to design mixes with better performance and more resistance to rutting. Moreover, Zahw et al. (1995) tested mixes with three different aggregate types, two different aggregate gradations and different asphalt contents and concluded that the use of crushed stone instead of natural gravel increased the mix resistance to rutting.

The field investigation by Button et al. (1990) indicated that excessive fine aggregate (sand-size) particles and the round shape and smooth texture of the natural

(uncrushed) aggregate particles are among the chief mixture deficiencies contributing to rutting. Their laboratory investigation, moreover, indicated that the asphalt mixtures containing some natural sands plastically deform under static or dynamic loads much more readily than similarly graded mixtures containing only manufactured particles.

There are characteristics associated with the two aforementioned materials when they are mixed together, that affect permanent deformation of the pavements. Huber and Heiman (1987) concluded that asphalt content and voids filled with asphalt were the most basic parameters affecting rutting. However, Abd El Halim et al. (1995) indicated that rutting decreases with increasing asphalt content to a maximum value, after which rutting increases with increasing asphalt content. For rutting resistance, asphalt cement content should be relatively low to prevent shear flow under loading and elevated temperature. However, enough asphalt cement must be present to bind the aggregate particles in place.

The use of percent Voids in Mineral Aggregate (VMA) and/or percent Voids Filled with Asphalt (VFA) to predict pavement performance has been debated for long. Anani et al. (1990) indicated that VMA was a primary variable with regard to rutting. VMA of the unrutted sections was higher than that of the rutted sections. Carpenter and Enockson (1987) also concluded that VMA was a significant variable for rutting, however, Huber and Heiman (1987) expressed that VFA was one of the most basic parameters affecting rutting. In the study by Roque et al. (1995), however, VMA was not found to be related to shear resistance of the mix. In addition, the National Center for Asphalt Technology (NCAT) study indicated that VMA was more significant for the base course than the surface course (Brown and Cross, 1992)

A study of rutting in Canada by Huber and Heiman (1987) analyzed the condition of asphalt concrete as it was designed, after it was constructed and as it existed at the time of their study. They found that the threshold air voids content was 4 percent

minimum. The threshold value for voids in the mineral aggregate (VMA) was 13.5 percent minimum, and the voids filled with asphalt threshold value was approximately eighty percent maximum. Later, the comparison of the results from laboratory and field densities showed that in-place air void contents below 3 percent greatly increase the probability of premature rutting (Brown and Cross, 1991). This, in fact, is in conformance with the findings of both Huber and Heiman (1987) and Kandhal et al. (1993).

A study of 32 asphalt concrete overlays placed over rigid pavements, completed by Carpenter and Enockson (1987), indicated that the majority of the rutting problems could be attributed to gradation. Brown and Cooper (1984) compared mixes with dense-graded and gap-graded aggregates and realized that the gap-graded mix experienced significantly more permanent deformation. Sherif et al. (1997) found that the use of coarser gradation would result in increasing shear strength of the mix, thus reducing susceptibility to rutting. In the case of finer gradations, however, the rutting resistance would be influenced by binder characteristics (Krutz and Sebaaly, 1993).

Brown and Bassett (1990) indicated that increasing the maximum aggregate size of the mix increased the mix quality with regard to creep performance, resilient modulus and tensile strength. As mentioned by both Mahboub and Allen (1990) and Kandhal (1990), Large-Stone Asphalt Mixes (LSAMs) are becoming a popular means for reducing rutting in flexible pavements. The performance-oriented laboratory test results of their study indicated that higher levels of structural capacity and rutting resistance, as compared with conventional hot mix asphalt, can be achieved by using LSAMs, especially for heavy-duty pavements. However, it should be mentioned that if large stones float in a matrix of conventional mix it will have little effect on resisting rutting (Coree and Button, 1997).

The major deficiency in all mixture design methods is the inability to properly grade an aggregate to maximize shear resistance. Roque et al. (1995) expressed that

asphalt mixture shear resistance appears to be most strongly related to the gradation characteristics of the coarse aggregate fraction of the mixtures. Coarseness of the aggregate, and the shape (curvature) and position of the coarse aggregate gradation curve relative to maximum density line, were found to influence mixture shear resistance. Moreover the tests showed that introduction of fine aggregate did not diminish the effect of coarse aggregate gradation.

Finally, the mix resistance to permanent deformation is greatly related to the physical and mechanical properties of the mix, known as density and tensile / shear strength respectively. In this regard, Finn (1983) evaluated the performance of four mixes designed to resist rutting formation for the first twelve months after construction. It was concluded that lower densities were associated with greater increase in rutting and a greater decrease in tensile strength. Moreover, Sherif et al. (1997) showed that based on statistical analysis the indirect tensile strength affects the rutting depth much more than the density of the mix.

The study by Zahw et al. (1995), however, clarified that the adoption of optimum density as the only design criterion is not necessarily associated with the best design for the minimization of rutting. The results showed that density and indirect tensile strength can be used to predict rutting for asphalt mixes with a specific range of asphalt contents. The authors concluded that shear strength of asphalt mixes is more accurate in identifying the gradation and types of aggregate employed in the mix.

Gervais and Abd El Halim (1990) also suggested that in order to minimize the surface rutting during the early years of service life of the pavement, one has to use asphalt mixes with optimum stiffness and shear strength. This is in conformance with the recommendation of the Strategic Highway Research Program (SHRP, 1994) that the mixes be designed based only on maximizing their mechanical properties. In order to

Table 2.2 Factors Affecting Instability Rutting

External Factors		Internal Factors	
Traffic characteristics	Tire pressure	Bituminous material	Viscosity
	Axle loading		Modifiers
	Axle configuration	Mineral aggregate	Fine / Course angularity
	Load duration		Type
Construction	Field compaction	Mix design parameters	Shape and texture
	Plant production		Gradation
	Consistency		Asphalt content
Environment	Temperature	Mix properties	Air voids
	Aging		VMA / VFA
	Moisture damage		Density
Geometric design	Hill		Tensile Strength
	Curve		Shear Strength

predict rutting depth of asphalt mixes according to their mechanical properties, Zahw et al. (1996) studied two approaches, one based on shear and tensile strength and the second based on shear and tensile modulus. It was then found that both approaches predict rutting adequately.

As can be realized from the previous discussion, shear properties of asphalt concrete mixes are better indicators of the resistance to permanent deformation. In other words, the shear properties can better represent the strength of the asphalt concrete as a mixture of aggregate materials and binder. Shearing forces or stresses have to overcome both the aggregate interlock and the cohesive forces of the binder. Moreover, knowing that the rutting is observed as a shear flow, it is reasonable to expect the shear properties of the asphalt concrete mixes to better describe its rutting susceptibility.

2.4. Evaluation of Rutting Susceptibility

As mentioned in the previous sections, rutting is one of the most serious modes of distress that causes severe problems for the users of the road. Although most currently used mix design procedures are reliable in eliminating extremely poor asphalt mixes, they offer little assistance in distinguishing among mixes with high, moderate, or

low rutting resistances. Therefore, considerable effort has been spent to understand rutting causes and to measure rutting potential of HMA. To evaluate rutting resistance of asphalt mixes researchers have used either empirical/simulative or fundamental tests. In the former tests, the field conditions are simulated and the rutting performance is observed while, in the other tests a fundamental material property is evaluated and related to the rutting performance of the mix.

2.4.1. Simulative Tests (Wheel Track Rut Testers)

Accelerated rut testers have been used by several countries for quite some time. The Europeans have taken the leading role in developing accelerated pavement testing devices (i.e. simulative tests). In simulative tests, LCPC Wheel Tracker, Hamburg Wheel Tracking Device, Georgia Loaded Wheel Tester, Asphalt Pavement Analyzer, Accelerated Load Facility, etc. are being used.

The French Pavement Rutting Tester (LCPC Wheel Tracker)

The French Pavement Rutting Tester was developed at the Laboratoire Central des Ponts et Chaussées (LCPC) in France. During testing slabs are loaded by reciprocating pneumatic tires inflated to a standard pressure of 0.60 MPa. The load will be applied approximately 67 cycles per minute on two test slabs at a constant temperature between the range of 35 to 60°C (Romero and Stuart, 1998; Miller et al. 1995). It is interesting that the developers of the French Rutting Test do not believe that statistical correlation between rutting observed in the test and that observed in the field can be developed because the rut tester simulates extremely severe rutting conditions (Huber, 1999). However, LCPC reports that roads that meet the French Rut Tester specifications do not exhibit rutting in service.

A large amount of work has been completed in Colorado by Aschenbrener (1994), using the LCPC device to correlate laboratory and field rutting performance of

asphalt pavements. After reducing the severity of the test, by changing the temperature to values associated with Colorado conditions, the regression analyses yielded high correlation between field rutting and the rutting observed with the French device (R^2 of 0.87 for high traffic and 0.68 for medium traffic). Furthermore, LCPC tests were completed on specimens recovered from the Westrack experiment where a good correlation ($R^2 = 0.694$) between laboratory values and observed rutting at the test track was achieved through regression analysis (FHWA, 1998c).

Hamburg Wheel Track Tester

The Hamburg Wheel Tracking Device was developed in Germany to measure the combined effect of rutting and moisture damage. A solid steel wheel rolls across an asphalt concrete slab immersed in 40°C or 50°C hot water. A fixed load of 0.69 kN is applied on the wheels that produces an average contact stress of 0.73 MPa. Approximately, 53 passes per minute is applied on test slabs. This device was originally used to measure rutting susceptibility, however, after increasing the number of wheel passes it was found that some mixtures could deteriorate due to stripping (Romero and Stuart, 1998).

Aschenbrener (1995) completed the correlation between the field performance and the results obtained from the Hamburg Wheel Tracking Device at the Colorado Department of Transportation (DOT). Although the Hamburg stripping slope and stripping inflection point were able to distinguish between good and poor field stripping performances, the moisture conditioning system used by the device appeared too severe for rutting determination. Nevertheless, regression analysis between the Hamburg device and rutting at Westrack yielded good correlation ($R^2 = 0.756$) (FHWA, 1998c).

Georgia Loaded Wheel Tester

The Georgia Loaded Wheel Test (GLWT) developed by the Georgia Department of Transportation in the mid 1980's, is used to test rutting resistance of asphalt mixes. The GLWT measures rutting resistance by rolling a concave steel wheel across a pressurized (0.69 MPa) 29 mm diameter rubber hose placed along a test beam. The device operates at 67 passes per minute for 16000 passes (Romero and Stuart, 1998; Lai, 1986).

Later, numerous improvements were introduced to the original design of GLWT and the resulting device was renamed the Asphalt Pavement Analyzer (APA). Unlike the GLWT, the APA incorporates a water storage tank for testing specimens under water. It is capable of testing on both beam and gyratory specimens. Similar to the French and Hamburg devices, good correlation between field rutting performance and the GLWT/APA has been observed. For example, regression analysis between the APA rut depth and field rutting observed at Westrack yielded $R^2 = 0.797$ (FHWA, 1998c).

Accelerated Load Facility

The Accelerated Load Facility (ALF) is a full-scale wheel-tracking device incorporating one half of a single truck axle traveling along a 29-meter frame over a full-scale pavement test section, which is about 10 meters long. Loads between 44.5 to 100.1 KN may be applied on specimens. Unlike laboratory wheel tracking tests, the ALF applies loads in one direction only and can impose lateral distribution of the load to better simulate truck traffic loading. ALF can simulate 20 years of cumulative traffic in six months or less (Romero and Stuart, 1998).

2.4.2. Fundamental Tests

Fundamental tests, evaluate an engineering property of the asphalt concrete mix, such as the dynamic modulus or the shear stiffness that can be directly related to the

performance or can be modelled to predict the performance. These tests can be summarized as follows:

- Unconfined and confined cylindrical specimens in creep, repeated, or dynamic loading
- Cylindrical specimens in diametral creep or repeated loading
- Superpave Shear Tester (SST), Field Shear Test (FST)
- Shear modulus and strength tests

Among the aforesaid tests, Superpave Shear Tests and Field Shear Test that measure shear properties of the mix will be explained.

Superpave Shear Tester (SST)

The Superpave Shear Tester (SST) was introduced by the Strategic Highway Research Program (SHRP) to evaluate the shear properties of the asphalt concrete mixes (SHRP, 1994). SST was designed to perform a variety of performance-related tests on HMA, including characterization of the complex modulus and phase angle, determination of the bulk modulus, and evaluation of various aspects of the nonlinear/plastic behaviour typical of granular materials at high temperatures. Data gathered using the SST, along with a variety of other information, were in turn used as input to a computer program meant to provide performance predictions for a given pavement system as a function of time (NCHRP, 2002).

Among many of the tests performed with the SST two of the procedures were shown to be reasonably accurate and to relate well to various aspects of pavement performance: the Frequency Sweep at Constant Height (FSCH) test and the Repeated Shear at Constant Height (RSCH) test. The Frequency Sweep test is a technique for evaluating the complex shear modulus of HMA. The shear modulus defines the relationship between shear stress and shear strain and is an essential information in

analyzing the behaviour of a pavement system under traffic loading and during changes in temperature. The RSCH test is a repeated load test designed to characterize the resistance of an HMA mixture to permanent deformation at high temperatures. In this test a haversine shear load is applied to a cylindrical specimen, while supplying necessary axial stress to maintain a constant height. The rutting is then related to the accumulated permanent shear strain, or the number of loading cycles to achieve a fixed strain (NCHRP, 2002). The FSCH is the only Superpave mix analysis that uses dynamic loading (Wang et al. 2001). In this test, a very small, repeated, and controlled shear strain is applied to the specimen through a shear actuator. The complex shear modulus and the phase angle are evaluated from this test, which can then be related to the pavement performance.

Field Shear Tester (FST)

The SST device is too large, complex, and expensive for use in QC/QA testing. As a result, during National Cooperative Highway Research Program (NCHRP) projects 9-7 and 9-18 the Field Shear Test (FST) device was developed as a simple method of providing test data comparable to those produced by the SST but in a faster way and at a substantially lower cost. This prototype is compact, rugged, and extremely quick and easy to operate. In executing tests with the new FST, gyratory specimens are placed directly in the device with a similar geometry to that of SST and are held in place by hydraulic clamps.

Modulus measurements made with this device appeared to be well suited for QC use, except that the variability in results appeared too high. A new protocol was then developed that involves averaging four determinations for each test. Complex modulus measurements made with the device using this new protocol are sensitive to changes in mixture composition, coarse aggregate and asphalt binder content. The overall

coefficient of variation for complex shear modulus values determined with the device was found to be 8.0 percent that is quite good for modulus measurements on asphalt concrete specimens (NCHRP, 2003).

2.4.3. Deficiencies of Current Testing Practices

All wheel tracking devices, with the exception of the ALF, incorporate a small rolling wheel across a prepared specimen or core of known dimension. Although these tests can effectively rank asphalt mixes in terms of relative rutting resistance, they have numerous characteristics that prevent them from accurately predicting rutting performance of field pavements.

The deficiencies of the laboratory wheel tracking tests are improper boundary conditions, development of different stresses that are not representative of the real life conditions and small specimen size in proportion to the applied load (Romero and Stuart, 1998). Even though the ALF addresses the problems of dimensional incompatibility due to its full-scale nature, it does not measure fundamental engineering properties that can be input into a mechanics of materials model for performance prediction. It is believed that the performance-based tests hold the most promise for modelling and predicting long-term performance of pavements since engineering properties can be directly related to performance. Furthermore, the ALF is extremely expensive and not feasible for field QC/QA.

Although the development of the SST represented an important step toward measuring asphalt shear properties, neither of the tests is ideally suitable for widespread implementation. Specimen preparation for the SST is complicated and time-consuming, requiring careful sawing of the gyratory specimen, gluing platens onto the specimen, and, in some cases, fastening transducers onto the sides of the specimen. Furthermore, the SST device itself is a large, complex, and expensive servohydraulic testing system

not suitable for routine use in state highway laboratories or hot mix plants (NCHRP, 2002). While the FST is a portable device, the diametral loading condition is not representative of field loading conditions. Moreover, Sousa et al. (1991) reported that diametral loading, from the indirect tension test, is inappropriate for permanent deformation characterization because the state of stress is non-uniform and strongly dependent on the specimen. Furthermore, the effectiveness of measurements made using the FST has not yet been evaluated in field projects.

Finally, all tests require the preparation of specimens either through laboratory compaction or coring of in-service pavements. These preparation methods either are non-representative of the mix in the field, or damage the specimen to a large degree. Therefore, the development of an in-situ test would better represent the field performance.

2.4.4. In-situ Shear Testers

An innovative approach was adopted by the researchers at Carleton University to evaluate the shear properties of the asphalt concrete mixes, both in the laboratory and in the field. Zahw (1995) used a laboratory-testing device to measure the shear properties of asphalt mixes by applying torsion on cylindrical core samples. His results, as further analyzed by Goodman (2000), showed good correlation with the rutting susceptibility of the tested asphalt concrete mixes through creep testing. Therefore, the idea of applying torsion was extended to the field as a source to introduce pure shear to the pavement by developing the CiSSST (Abd El Nabi, 1995) and later the InSiSST (Goodman, 2001).

The Carleton In-Situ Shear Strength Test (CiSSST)

The first generation of in-situ shear testers was developed at Carleton University in Ottawa, Canada and was named the Carleton In-Situ Shear Strength Test (CiSSST). As shown in Figure 2.4, the CiSSST facility consisted of a cart-like framework on small

castor wheels for positioning. Torque was applied via an electric motor coupled to a gear reducer and a right-angled gearbox. The gearbox transmitted this torque through a vertical drive shaft to a torque cell, which in turn was attached to a steel loading plate bonded to the asphalt surface with a strong epoxy. Torque was applied until the asphalt surface failed. The failure torque was measured with the torque cell connected to a data recording processor and the failure strain was determined using a protractor. The maximum torque and angle of twist would then be used to calculate shear properties such as strength and stiffness of the asphalt layer (Goodman, 2000).

Contrary to laboratory torsion test on cylindrical samples, the field-test boundary conditions were not clearly defined, therefore, a different set of constitutive equations was required to determine the shear properties. Abd El Nabi (1995) developed Equation 2.1 for calculating the mix shear strength based on the assumption that the failed surface is a frustum of a cone as shown in the Figure 2.5.

Figure 2.4 The Carleton In-Situ Shear Strength Test (CiSSST) Facility (Goodman, 2000)

Figure 2.5 Loading and Boundary Conditions of CiSSST

$$\tau = \frac{3T}{2\pi \left[\left(\frac{r_f^3 - r_o^3}{r_f - r_o} \right) h_f + r_o^3 \right]} \quad \text{Equation 2.1}$$

where, T is the maximum applied torque (N.m), τ is the in-situ shear strength (Mpa), h_f is the failure depth (cm) and r_f and r_o are the upper and lower radius of the frustum of the failed cone (cm) respectively.

The tests carried out with this device were promising as the CiSSST was able to make a distinction between different mixes based on their shear strength. It also could differentiate between the shear properties of a pavement mix in different geometric locations, i.e. curve sections versus straight sections. This indicated that the device was sensitive to changes in mix shear properties and could potentially differentiate between mixes with good or poor rutting resistance.

Even though the Carleton In-Situ Shear Strength Test (CiSSST) facility represented an important first step to measure in-situ shear strength of asphalt concrete pavements, a number of mechanical and operational deficiencies were observed during investigations completed with this facility (Goodman, 2000). Transportation of the

CiSSST was inefficient and dangerous as it was heavy and required at least four operators to manually lift it into and out of the transport vehicle during initial setup or departure. As only one or two operators were required during the actual test procedure, this scenario was clearly an inefficient use of operator time and resources.

Stability of the device was also an issue of concern for getting accurate measurements. Stabilization of the CiSSST was originally achieved by locking the castor wheels to prevent rolling, however, this method proved unsatisfactory. Therefore, a second technique was applied. In this method, six large steel stakes were fed through hollow tunnels welded vertically to the device and driven into the pavement surface using a sledgehammer. Although the new method was relatively effective, it required a significant operator effort. Also, it would damage the pavement surface and could affect the sensitive electronics within the torque cell, the most costly component of the device that was attached to the test plate before hammering.

The epoxy used for CiSSST testing required a relatively long curing period of 24 hours. This required the closure of the test site to traffic twice, once to epoxy the loading plates and once for actual testing. Closing roads to traffic at any time period increases congestion and driver stress, as well as presenting significant safety risk to highway personnel.

Although the angle of twist at failure was measured crudely with a protractor, the CiSSST device was not capable of recording instantaneous angle of twist. Therefore, a graph of applied torque versus displacement could not be obtained.

Even though the CiSSST device performed extremely well to its credit, given its basic construction and control system, its overall capacity was low as it was not able to provide sufficient torque to fail some pavements at lower temperatures. As a result, a limiting test temperature of 10°C was assigned to the CiSSST device to achieve failure.

The In-Situ Shear Stiffness Test (InSiSST)

Considering the aforementioned deficiencies of the CiSSST facility the In-Situ Shear Stiffness Test (InSiSST) facility, shown in Figure 2.6, was developed in 2000 at Carleton University. Reasonable cost, portability and safety, fewer number of operators and ease of use, minimal test time and damage to pavement surface, and correlation of results to pavement performance indicators, formed the basis for the design of the new generation of in-situ shear tester (Goodman, 2000).

Basically, the InSiSST facility follows the same concept for testing as was used with the CiSSST, while it is assembled in a more advanced system leading to a portable, stable, and rugged test facility. In this new generation the components are mounted to a small A-frame trailer to provide exceptional portability. Like the CiSSST facility, an electric motor and gearbox is utilized to apply a forced circumferential displacement to a steel plate bonded to the asphalt surface. However, the motor/gearbox combination is positioned vertically saving a significant amount of space and providing an increased capacity and reduced backlash compared to right-angled gearboxes. In addition, the

Figure 2.6 Side View of the In-Situ Shear Stiffness Test (InSiSST) Facility

overall capacity of the motor and gearbox system was increased significantly to ensure the failure of all asphalt surfaces encountered, over a reasonable temperature range. However, Bekheet (2002) noted that at lower temperatures, where the pavement had higher stiffness, and when using a bigger plate size, the torque needed to fail the pavement was sometimes higher than the capacity of the testing facility in its current configuration.

The motor/gearbox combination is mounted on a steel platform attached to a positioning system that incorporates two sets of electric worm-screw slides working in tandem allowing positioning of the platform in both the longitudinal and transverse directions. The entire positioning system, whose movement is controlled by an electronic controller, is mounted to a two-level frame, which is attached to the trailer frame via four jacks. During the transportation of the InSiSST, the jacks are retracted to hold the frame in the air to prevent damage.

During testing the frame will be lowered until it sits on the pavement thoroughly and the whole facility lifts off the ground. Stability against the rotational force applied to the test plate is achieved through frictional force between the neoprene (rubber) pad epoxied to the bottom of the test frame and the pavement surface. Much of the required weight is provided by the trailer, test frame and the test equipment.

The main drive motor is controlled through a variable speed motor controller that allows the selection of variable test speeds between very small values close to zero revolutions per minute (rpm) and the maximum of 1800 rpm. Torque is recorded with a torque cell similar to that used with the CiSSST facility but with higher capacity. A laptop computer provides control over the test procedure and data acquisition. Results are saved directly to the computer and other relevant information such as test site location, weather conditions, temperatures, etc. may also be directly entered into a database for

future analysis. All of the individual components operate under the same power requirements as provided by the central generator.

The testing facility was originally designed to provide a constant strain rate loading. In such case, the angular displacement varies linearly with time. Consequently, the angular displacement in radians for each time step can be calculated using the motor speed and the gearbox reduction ratio as shown in Equation 2.2, where A_t is the angular displacement at time t in radians, Δt is the time step for data acquisition, S is the motor speed in rpm and R is the gearbox reduction ratio. Having this calculated displacement values and the torque values recorded during the test, the torque–twist curve is obtained. Bekheet (2002) explained that the first part of the curve does not represent real asphalt mix behaviour and the recorded data might have been reflective of the time required for the motor to pick up its speed, as well as the small movements in the rubber pad. Therefore, this part is corrected by extending the linear segment of the torque–time curve and adjusting the zero point of the graph (Figure 2.7).

$$A_t = A_{t-\Delta t} + 2\pi \cdot \Delta t \frac{S}{60R}$$

Equation 2.2

Figure 2.7 Typical Output from the InSiSST facility (Bekheet, 2002)

The mechanistic properties of the asphalt concrete mix are then evaluated according to the maximum displacement (strain at the time of failure), the average shear strength as calculated using Equation 2.1, average maximum shear stress at failure calculated based on secant shear modulus, or the shear stiffness of the mix (Bekheet, 2002). Shear modulus, G , is calculated using Equation 2.3 that is derived for the case of a forced angular displacement applied to the surface of a linear elastic half space (Reissner and Sagoci, 1944). In this formula a is the radius of the loading area, T is torque, and ϕ is angular displacement.

$$G = \frac{3}{16} \frac{T}{\phi a^3} \quad \text{Equation 2.3}$$

Chapter 3. Proposed Modifications for the InSiSST Facility

3.1. Introduction

As explained in the previous chapter, the InSiSST facility is a portable, stable and rugged facility that has a potential of becoming an effective tool for QA/QC of asphalt concrete mixes. Preliminary results of research work completed by Abd El Nabi (1995) and Goodman (2000) showed that the InSiSST has the potential of being used to quantify the in-situ shear properties of asphalt concrete mixes. Moreover, Bekheet (2002) used the InSiSST facility to measure the in-situ shear stiffness of flexible pavements. Bekheet (2002) then concluded that in-situ shear stiffness provides a new approach to characterize asphalt concrete mixes and correlates well with rutting in flexible pavements; therefore, it can be used to predict the rutting potential.

However, experimental investigations have showed that the facility requires further improvements in order to become an efficient and more reliable tool for pavement engineers and practitioners. In this chapter the enhancements considered for the InSiSST will be explained as well as the testing procedure that should be followed when using the new version of the facility.

3.2. Rotary Displacement Measurement

Although the InSiSST facility was originally designed to provide a constant strain rate loading, visual observations during in-situ testing by Bekheet (2002) showed that the actual strain rate was not always constant and was dependent on the material properties as well as the pavement temperature. The higher the material resistance (or the lower the temperature), the slower the strain rate at the early stages of loading. Figure 3.1 illustrates this phenomenon by comparing the results of three asphalt mixes

with different properties. For a weak material (sample 1) the variation of angular displacement with time is almost linear, while for a relatively stronger material (samples 2 and 3) the variation is nonlinear. However, Bekheet (2002) mentioned that the total rotation angle after the failure is similar to the total rotation angle under constant strain rate, as shown by the dashed line.

In addition, the analytical investigation showed that the shear stiffness of asphalt concrete mixes is less sensitive to the variation in the strain rate when the angular displacement is applied at, or close to, a constant strain rate. However, as the strain rate becomes nonlinear, the shear stiffness becomes more sensitive to the variation in the actual angular displacement. Therefore, it was realized that the actual angular displacement during in-situ testing has to be accurately evaluated to ensure accurate shear stiffness measurements, especially at lower temperatures, where the angular displacement at failure is relatively small.

As mentioned earlier, the InSiSST facility was designed such that the rate of the angular displacement could be controlled during testing. In the current configuration, the maximum rate of angular displacement that can be applied is 0.222 rpm. Such a rate

Figure 3.1 Variation of Displacement by Time (Bekheet, 2002)

was quite fast, regarding the fact that the failure happens in a very short period of time (2-5 seconds as documented by Bekheet, 2002). Therefore, manual measurement of the angular displacement with time was not feasible. To overcome this problem, Bekheet (2002) used a digital video camera to record the entire testing process. The recorded digital video would then be downloaded to a computer and viewed frame by frame. This would allow measuring the angular displacement at the time intervals as small as 0.033 sec.

However, the data compilation from the recorded video is a fairly time-consuming process and would not be a practical approach for a large-scale testing program. Therefore, it is necessary to use an electronic device that automatically measures the angular displacement. After investigating alternative methods, a Rotary Displacement Transducer (RDT) capable of providing 360° unrestricted continuous rotation was selected (Figure 3.2). Detailed specification of the device is presented in Appendix A.

The RDT measures the displacement through a set of gears, one connected to the collar between the torque cell and the test-plate and the other connected to the RDT itself. The readings of the RDT are then corrected for the ratio of the gears' diameters. The RDT is installed on a seat with three adjustable legs so that the two gears can be levelled (Figure 3.3). The previously used collar was also modified to hold the gear used

Figure 3.2 Rotary Displacement Transducer (RDT)

Figure 3.3 The RDT Connected to the Collar with a Set of Gears

to transmit the angle of rotation to the RDT. Moreover, the collar was redesigned due to the fractures observed on the previous collars. Four holes were made on the gear mounted to the collar to provide visibility for the operator when aligning the collar with the square cavity on the test-plate.

3.3. Vane Steel Plates

As mentioned earlier, epoxy used for both CiSSST and InSiSST testing required a relatively long curing time of 24 hours leading to the closure of the test site to traffic twice within this period. Although numerous adhesive systems were tested in the previous study, most of them either required cure times that were similar to the existing one or did not provide suitable strength at all (Goodman, 2000).

Moreover, in the current approach, the required number of test-plates is equal to the number of tests in one day of testing. In this case, either a considerable number of test-plates should be manufactured that is costly, or the testing would be limited to the available number of test-plates. Furthermore, after testing is finished, test-plates should be cleaned and roughened for the next testing day. This is time-consuming and needs

significant labour work. Finally, the experimental investigations showed that 10% to 15% of the tests failed in the epoxy rather than the asphalt mix itself (Bekheet, 2002).

3.3.1. Overall Design

To address the aforesaid issue, a new approach was developed for attaching the plates to the pavement. This method is based on using modified steel plates provided with vertical blades to be driven into the pavement surface. The steel blades provide enough area of contact between the blades and the asphalt concrete and eliminate the need for the epoxy. Two types of vane plates were designed and manufactured at Carleton University Civil and Environmental Engineering Laboratory:

- Cross-vane plate (Figure 3.4a)
- Single-vane plate (Figure 3.4b)

Figure 3.4 (a) Cross-Vane Plate, (b) Single-Vane Plate

The shape of the blades was chosen based on the shape of the failure surface observed from the previous tests i.e. frustum of a cone. The height of 25 mm (1 in) was selected for blades based on the findings of Abd El Halim et al. (1997) who showed that the test would be successful if the height of the failed sample is greater than the nominal aggregate size of the mix. The thickness of the blades, 6 mm for the cross-vane and 10 mm for the single-vane, was calculated according to the maximum torque value obtained from the previous tests (those with plates glued to the pavement surface). The distributed lateral load, equivalent of the applied torque value, was considered on the blades and the thickness was calculated so that the maximum stress in the steel blade would not exceed the yield strength of the material.

Figure 3.5 Vane Plate Manufactured for the Laboratory Torsion Machine (a) Cross-Vane, (b) Single-Vane

To investigate the effect of the plate size on the shear properties of the asphalt mix measured with the InSiSST facility, three vane plates with diameters of 100, 125 and 150 mm (4, 5 and 6 in) were manufactured for each type (cross-vane and single-vane). A prototype of such plates with a diameter of 100 mm (4 in) was also made for the laboratory torsion machine (Figure 3.5). A set of tests (as will be explained later) were completed in the laboratory using these plates in order to verify the applicability of using the vane plates for asphalt concrete mixes.

3.3.2. Cutting Tool

As stated earlier, the blade(s) of the modified steel plate is (are) to be driven into the asphalt pavement. Therefore, grooves with the same size and dimensions as the blade(s) should be formed on the pavement surface. The instrument used to make such grooves is a 5" angle grinder that has the advantage of providing dry cut. The angle grinder is mounted to a system, which has been designed and manufactured at Carleton University Civil and Environmental Engineering Laboratory to provide either a cross or a single groove on the pavement. This device consists of two main parts, a stationary frame and a mobile unit (Figure 3.6). The stationary part is an 840 x 840 mm (33" x 33") box-tube frame with parallel angle sections mounted on it in both directions. The angle sections act as a rail for the mobile unit and are squared to provide perpendicular grooves. The mobile unit holds the angle grinder and moves on the angle sections through 4 wheels. The angle grinder can also be moved manually in vertical direction using a handle provided for this purpose.

(a)

(b)

Figure 3.6 (a) Stationary Frame, (b) Mobile Unit

Figure 3.7 Different Cutting Stages: (a) the Angle Grinder Just Located on the Main Frame, (b) Lowering the Angle Grinder, (c) Horizontal Movement

The main frame is equipped with four big screws in the corners that are used to adjust the whole system on the pavement. After positioning the frame on the pavement and adjusting the screws, the mobile unit would be located on the angle sections. At this moment, the angle grinder is in its highest position on the mobile unit (Figure 3.7a). In order to make a cut, the grinder should be turned on and lowered so that the blades start to cut the asphalt mix (Figure 3.7b). By moving the mobile unit forward and backward one groove will be formed on the pavement (Figure 3.7c). In case of a cross-cut, the same process will be followed for the other direction. To control the length of the cut, adjustable blockers are mounted on the main frame. These blockers would be set for each plate size to control the horizontal movement of the mobile unit in order not to pass a certain limit.

After investigating different alternatives, 125 mm (5") turbo-blades compatible with the speed of the angle grinder and suitable for dry cut were selected. When mounted on the angle grinder, a 125 mm blade would have 25 mm free length that can be driven into the pavement. This length is equal to the height of the vanes. As the blades are not thick enough to provide the cut as wide as the vane thickness, more than one blade should be mounted on the angle grinder. Specific spacers were designed to fit between the blades to provide the required cut width. For cross-vane plates with smaller

vane thickness, two blades and a spacer were used, while for the single-vane plates that are thicker, three blades and two spacers were used. The space between the blades would cause a thin layer of asphalt to remain in the groove after cutting. This layer can be easily removed using a screw driver.

3.3.3. Advantages of Vane Steel Plates

Introducing the vane as a new approach for attaching test plates to the pavement surface had a number of advantages. First, it eliminated the use of epoxy, which sometimes failed prematurely. The second and the most important advantage was the significant reduction of the total time required to complete field testing, where testing can be completed within 1 to 1.5 hours, instead of having to wait for the epoxy to harden for 24 hours. The detailed required time to complete one test with InSiSST facility is listed below:

- Cutting the grooves (10 min)
- Heating the pavement, if desired (can vary between 20 to 60 min)
- Testing process (less than 10 min)

In addition, for each plate size only one test plate is required. This test plate can be used for all the tests per testing day, eliminating the need for a lot of test plates to be manufactured or the limitation in testing program due to the lack of test plates. Moreover, there would be no need for subsequent cleaning and roughening of the test plates. The last advantage is a more conspicuous failure surface obtained from the tests with vane plates in comparison to those obtained using epoxy. In the latter case, the operator should assume the dimensions of the approximated frustum of a cone and therefore, the results would be subjective. However, in the case of the vane plate the failure surface follows the geometry of the blades with specific dimensions that leads to more accurate results.

3.4. Data Acquisition System

As explained before, Bekheet (2002) used a digital video camera to measure the actual strain rate. Although video taping the testing process provided the angular displacement variation versus time, the problem was not completely solved because the torque and the angular displacement were measured separately and had to be cross-referenced. This might result in some degrees of inaccuracy when defining the exact starting point of the torque data as compared to the exact starting point of the angular displacement values. Figure 3.8 shows a torque-time curve for a test carried out on a 125-mm (5-inch) test-plate as well as the corresponding displacement-time curve with three different starting points. Three torque-displacement curves will be derived from these three adjustments as reflected in Figure 3.9. Looking at the figure, one can realize that the shape of the curve and the maximum angular displacement at the time of failure vary significantly from one curve to another. As a result, the accuracy of the results obtained from the tests, is dependant on a precise cross-referencing of torque and angular displacement data.

Figure 3.8 The Starting Point of Angular Displacement Data in Relation with the Torque-Time Curve

Figure 3.9 Torque-Displacement Curves Based on Different Starting Points of Angular Displacement Data

The solution for this problem is the instantaneous measurement of torque and rotary displacement that is achieved by using a data acquisition system, which can read the data from both devices simultaneously.

3.4.1. Personal Daq

The data acquisition system added to the InSiSST facility is Personal Daq, which is a compact data acquisition device that makes use of the Universal Serial Bus (USB) (Figure 3.10). USB provides both high-speed communication and power to the Personal Daq. The specifications of the Personal Daq device is briefly presented in Appendix B and the detailed specifications can be found in Personal Daq User Manual (2001).

Personal Daq can directly measure multiple channels of volts, thermocouples, pulse, frequency, and digital input/output (I/O). It provides measurement durations ranging from very slow (610 milliseconds) to very fast (12.5 milliseconds) that are equal

to 1.6 and 80 samples per second respectively. In addition, Personal Daq provides effortless thermocouple measurements. The device includes built-in Cold Junction Compensation (CJC), which is automatically invoked when thermocouple measurements are selected. The Personal Daq automatically converts acquired voltage readings into compensated–linearized temperature readings. Moreover, the Personal Daq system can make thermocouple and volt measurements concurrently that would facilitate measuring temperature during testing while the torque and displacement values are being measured through the torque cell and the RDT.

Furthermore, in Personal Daq each digital I/O line can be programmed individually as input or output. Digital input lines can be read as part of the analog scan sequence and can be scanned over 100 times per second. Digital output lines can be updated at any time prior to, or during an acquisition sequence; or can be automatically updated. Digital I/O applications include monitoring and control: in the case of this study the digital I/O line is used to control the heaters as will be explained later in this chapter.

Figure 3.10 Data Acquisition Device (Personal Daq)

3.4.2. Personal Daq View

The Personal Daq device is controlled through a program called Personal Daq View (PDV). PDV is a graphic Microsoft Windows-based program that can be used for various data acquisition applications. It allows the following tasks to be performed:

- Set up Analog Input Channels for acquiring various voltages or temperature.
- Set up Frequency/Pulse Input Channels to measure various frequency-related parameters.
- Set up Digital I/O Channels for input or output.
- Save acquired data to disk.
- Transfer data to spreadsheets, data bases, and Post View.
- View real-time data values in screen columns, scrolling charts, bar graph, analog, or digital meters.

PDV is configured for the InSiSST devices namely the Torque cell, the RDT and the thermocouple. This version of the program is not capable of showing the readings of different channels, in one chart display. Therefore, during each test the variations of angular displacement and torque versus time are shown in two separate chart displays and the temperature values are viewed in screen columns. The data from the InSiSST devices are saved in text format and will be later analyzed for the shear properties of the asphalt concrete mix.

3.4.3. RDT and Torque Cell Calibration

The RDT and torque cell were first calibrated with the new data acquisition system. The RDT was calibrated using a protractor designed and manufactured for this purpose (Figure 3.11a). The protractor was mounted on the three-leg seat. Ten holes that are distributed at 5° intervals were made on one end of the protractor. A ruler was used with one end fixed to the RDT and the other end left free. A hole, with the same

Figure 3.11 Calibration of (a) the RDT, (b) the Torque Cell

diameter as those on the protractor, was made on this free end. The free end of the ruler was then fixed on each hole of the protractor using a pin and the reading of the RDT was saved. The average difference between the readings of successive holes was taken as an equivalent to the 5° rotation. Therefore, the factor to convert readings in volts to degrees was calculated and plugged into the PDV software.

The torque cell was also calibrated using a wrench with a known length (Figure 3.11b). 16-Kg weights were applied on the end of the wrench and the readings of the torque cell were saved. Knowing the equivalent torque value for each reading (from multiplication of the load by the related arm), the factor for converting torque cell readings to Nm unit was obtained. This value was also entered into the PDV software.

3.5. Pavement Temperature Control

Due to its viscoelastic nature, shear properties of asphalt concrete mixes, i.e. shear strength and stiffness, are highly dependent on temperature. Bekheet (2002) studied the effect of the variation in the temperature on the properties of the asphalt concrete mixture and showed that this variation can have a significant effect on the in-

situ measurement of the shear stiffness. Several sets of in-situ testing were completed at different temperatures. However, the study was limited to the prevailing temperature of the testing pavement. In other words, it was not possible to investigate the shear properties of the pavement material at the desired temperatures because the InSiSST (in its previous form) could test only at the pavement temperature present in the field. In addition, previous studies (e.g. Meyer, 1978) suggested that a temperature shift curve could be developed for asphalt concrete mixes to correct the results with respect to temperature. Having this master curve, the in-situ measurements can be adjusted to a standard or reference temperature.

3.5.1. Flexible Heaters

To account for the temperature variation in time of testing, two alternatives namely, an environmental chamber and flexible sheet heaters were studied and the latter was selected. Sheet heaters are normally preferred because of their lightweight, thin profile, low thermal mass and flexibility. The ability to provide an infinite array of patterns and concentrations of heat are additional characteristics. Furthermore, the fact that the flexible heaters have low thermal mass leads to the fast response in both heating and cooling down. They produce a very even, controllable heat, avoiding the temperature gradients associated with power resistors or radiant elements. They can be constructed so that only the areas that need heating get it by varying the pattern of the resistive element within the heater. It was suggested to have four separate sheets that would be spread on both asphalt and test plate and would be left in place until the pavement reaches the desired temperature. At such moment the heaters would be removed and the InSiSST frame would be lowered. After that, the collar would be connected to the test plate, the RDT will be located in place, and the test can be started.

There are different types of materials used for these heaters such as silicone rubber, Kapton, HT foil and Neoprene. Silicone rubber, which is by far the most popular, was selected because it offers a variety of thicknesses and grades, including low smoke/low toxicity, and UL-rated versions (Bohannon). Silicone Rubber Heaters are rugged, moisture and chemically resistant and are easily bonded or cemented to heat sinks or other parts to be heated. Temperatures from – 62 to 200°C in a wide range of watt densities are regularly supplied. The thin profile transfers heat quickly because the actual resistance element is so close to the heated part. Silicone Rubber Heaters can be insulated with silicone sponge rubber or fibber glass.

Figure 3.12 shows the components of the heating system. Sheet heaters (250 x 250 mm) were bonded to aluminum plates that are good heat conductors. Since the heaters are supposed to cover both the plate and the surrounding pavement, the aluminum plates were shaped so that part of it sits on the test plate and the rest covers the pavement surface. However, a gap would appear between the heaters and asphalt surface in the areas close to the test plate. To overcome this problem two aluminum heat-transfer pieces were manufactured for each plate size. These pieces would fill the gap and eliminate the presence of air as a nonconductive material. Each sheet heater is equipped with a thermocouple mounted beneath the aluminum plate to measure the temperature on the pavement surface. These thermocouples are required for controlling the heater's temperature. Finally, the heaters were insulated using fibber glass. The insulation pads were altered to fit the shape of the aluminum plates and were covered with aluminum foils. Figure 3.13 shows the heaters while they were working on the testing area. It should be noted that the power for the heaters was provided by the power generator.

Figure 3.12 Heater Components

Figure 3.13 Heaters after Installation

3.5.2. Heater Control

A heater will rise in temperature until heat losses equal heat input, if it is powered without any regulation. This may be acceptable in rare situations, but normally is to be avoided because the equilibrium temperature is highly unpredictable. In most cases it is preferred to control heater temperature to prevent overshooting and burning out the heater. Therefore, a macro was developed in Microsoft Excel environment to control the heaters. Basically, the heaters were controlled by turning them on and off periodically. In the beginning, a set-point should be defined for the heaters by the operator. For each heater, when the temperature exceeds the set-point, it would be turned off and when it goes below the set-point, it would be turned on. This process would continue until the asphalt mix temperature reaches the desired value. Figure 3.14 demonstrates the temperature variation versus time on the surface and in the depth. In this example, the heaters' temperature is set to 50°C. Looking at the figure, one can realize that the existing pavement temperature was 35°C and it took 30 minutes to heat the asphalt mix to 43°C.

Figure 3.14 Temperature Variation versus Time (One Heater)

Surface Temperature	Depth Temperature	Time	Heater (1)	Heater (2)	Heater (3)	Heater (4)	Elapsed Time (min)
44.02	40.72						
43.81	40.68	1:56:14 PM	started	started	started	started	0.00
47.51	40.66	1:56:28 PM					0.23
60.36	40.65	1:56:43 PM					0.48
73.55	40.62	1:56:57 PM					0.72
85.73	40.63	1:57:11 PM		stopped	stopped	stopped	0.95
93.75	40.64	1:57:25 PM	stopped				1.18
94.10	40.60	1:57:40 PM					1.43
91.67	40.58	1:57:54 PM					1.67
88.73	40.55	1:58:08 PM					1.90
85.97	40.53	1:58:22 PM					2.13
83.49	40.54	1:58:36 PM					2.37
81.24	40.53	1:58:50 PM				started	2.60
80.35	40.51	1:59:04 PM		started		stopped	2.83
81.92	40.51	1:59:18 PM	started	stopped	started		3.07
83.58	40.45	1:59:33 PM			stopped		3.32
87.06	40.49	1:59:47 PM	stopped				3.55
87.70	40.46	2:00:01 PM					3.78
86.09	40.47	2:00:15 PM				started	4.02

Figure 3.15 Real-time Output of the Macro (Four Heaters)

For the tests on the plates epoxied to the pavement, there is no clue when the temperature of the pavement (in a specific depth) reaches the desired value. Therefore, a formula has been developed to relate the desired temperature to different variables as will be explained later in the thesis. This formula was used in the macro to calculate the set-point automatically. In the case of using vane plates, another thermocouple was used to measure the pavement temperature just under the vane. A cover was provided for the thermocouple so that the measurements are not affected by the vane's temperature, which might be higher than the actual pavement temperature.

A real-time output of the macro is presented in Figure 3.15. In this figure, the surface temperature represents the average readings of the four thermocouples and the depth temperature is the measurements of thermocouple located under the vane. In addition, the current time and elapsed time is given in minutes for later analysis. Finally, the heaters' on and off periods are given so that the operator can check if each of the

heaters is working properly. The macro stops automatically when the pavement temperature reaches the desired temperature.

3.6. Testing Procedure

The use of the newly modified test facility requires certain steps and careful attention to detail before, during and after the testing. The following sections describe, in details, the steps and procedures to be considered when the shear properties of a given asphalt mix are required.

3.6.1. Equipment Checklist

Before leaving to test site, it should be ensured that all necessary equipment for field testing, as listed below, are packed and are in proper working order.

- Fuel and oil for the generator
- Laptop computer
- Testing plates
- Torque Cell
- RDT
- Collar
- Clamps
- Cutting tool
- Stiff brush to clean dust from asphalt surface
- Heaters, heat transfer pieces and necessary cables (if heating is desired)
- Tool Box

3.6.2. Transportation

The InSiSST is a trailer-mounted device that is towed behind a vehicle to various test-sites. Therefore, safety is an important issue that should be taken into consideration.

Towing safety guidelines recommended by trailer manufacturers and government agencies should be followed to ensure a safe journey. Sherline Products Incorporated (Sherline 1999) has produced a comprehensive guide to towing safety. Ontario Ministry of Transportation (MTO, 2000) has also developed a quick checklist for trailer safety. Prior to transportation, the jacking system of the InSiSST facility should be completely retracted such that the test frame is in its uppermost position. Furthermore, the gearbox and drive motor must be secured to the test frame using the ratcheting nylon straps. The test facility should be inspected for loose parts and covered with a protective plastic sheet.

3.6.3. Securing the Test Site

Safety rules stated in safety books such as MTO safety book or Traffic Control Guidelines Book (2000) should be followed to ensure a safe work zone. Since closing a road section or lane always involves some risk, traffic control should be carried out by trained professionals with the proper equipment. If possible, traffic control measures should be initiated and completed by local or provincial agency personnel to ensure the safest working condition.

3.6.4. Cutting the Grooves

After marking the desired testing locations on the test site, cutting can be started. The steps listed below should be followed in order:

1. Set the blades based on the type of the vane plate that would be used for testing (3 blades for single-vane plates and 2 blades for cross-vane plates).
2. Set the location of the blockers on the main frame based on the size of the test plate (Figure 3.16).
3. Place the main frame on the testing area such that the center of the mark matches the center of the frame.

Figure 3.16 Blockers' Position

4. Adjust the frame with the four screws on each corner.
5. Locate the mobile unit, which holds the angle grinder, on the angle sections of the main frame. Make sure that the angle grinder is in its uppermost position and that all the wheels are in contact with the angle sections.
6. Turn the generator on.
7. Plug and turn on the angle grinder.
8. Lower the angle grinder by rotating the handle (in front of the mobile unit) counter clockwise.*
9. Move the mobile unit forward and backward between the blockers.*
10. Follow the previous two steps in the other direction, if using a cross-vane plate.

* It is preferable if cutting is done in more than one phase. For instance, it is more appropriate if the grinder is first lowered to half the total depth and moved horizontally and then the same steps are followed for the second half of the groove depth.

3.6.5. Heating the Testing Area

After cutting the grooves, if heating is required, the following steps should be followed:

1. Clean the testing area using the brush.
2. Manoeuvre the InSiSST over the testing area (the cut).
3. Chock the tires of the trailer once in position to prevent its movement.
4. Drive the test plate into the grooves and locate the thermocouple under the vane.
5. Place the heat-transfer pieces around the test plate.
6. Install the heaters.
7. Turn the generator on.
8. Turn the computer on.
9. Connect the USB port of the data acquisition device and check if the light on the device is on and steady.
10. Plug the heaters and thermocouples based on their numbers.
11. Open the Heat-Control excel program and run the macro*.
12. Check if all the heaters are working properly.
13. Wait until the pavement's temperature reaches the desired value and the macro stops.
14. Save the file to the hard drive.

* PDV program should be closed when using Microsoft Excel otherwise no readings will be saved because all the devices are captured by the PDV.

3.6.6. InSiSST Test Procedure

Once the grooves are made in the testing area and the pavement is heated (if required) the field testing with the InSiSST facility may commence. The following steps should be completed:

1. Attach the torque cell cable to the torque cell.
2. Attach the RDT cable to the RDT.
3. Install the torque cell and connecting collar to the gearbox driveshaft.
4. Turn the generator on (if it is not already on).
5. Turn the computer on (if it is still not on).
6. Connect the motor controller RS232 port.
7. Turn the slide controller on.
8. Use the slide control pad to align the torque cell over the test plate.
9. Lower the test frame to the ground as much that the collar is not in contact with the test plate.
10. Run the PDV software, set the file name and open the column and chart displays.
11. Activate the motor controller software on the laptop and turn the torque cell until it is aligned with the square cavity on the test plate.
12. Lower the connecting collar on the torque cell and drive it into the test plate's cavity.
13. Lower the InSiSST frame completely and ensure that the weight of the trailer has been transferred to the test frame.
14. Fix the gearbox to the frame using the clamps to eliminate the movements that will affect the accuracy of the torque and displacement readings.
15. Click the play button on both displays of the PDV. The software would then begin to take readings at the predefined sampling rate. However, these readings would

not be recorded until the record button is clicked.

16. Check the temperature readings and wait until the pavement cools down to the desired value.
17. Click on the record button in PDV.
18. Select the desired motor speed. After the motor starts to rotate, the torque and displacement readings should increase until failure occurs. After failure the torque readings decrease while the displacement readings would still be increasing.
19. When failure occurs, stop the motor and stop recording in PDV. The readings would be automatically saved in the defined destination, file name and format.
20. Raise the InSiSST frame.
21. Inspect the failed asphalt surface.

Note: If heating is required, it is preferable to follow the first three steps before the heating procedure. This would reduce the time spent for setup before starting the actual test and therefore, reduce the temperature loss.

3.6.7. Leaving the Test Site

After completing all the desired tests, the tested locations should be sealed with a slurry mix to prevent moisture infiltration. The equipment should then be collected and packed for the transportation. The same procedures as explained in section 3.6.2 should be followed. The InSiSST should be reattached to the tow vehicle and transported while following the previously discussed safety recommendations.

All data, information and observations should be well documented and recorded during the entire period of the testing. Pictures should be also taken during the testing for further analysis later.

Chapter 4. Experimental Investigation

4.1. Introduction

The core objective of the research was to improve the InSiSST facility to address the shortcomings discovered through the extensive literature review and comprehensive study of previous research. After applying the modifications to the facility, as explained in the previous chapter, it was essential to examine the validity and applicability of the new modifications. As is always the case in any engineering project, it was important to check the performance of the improved components both individually and collectively. This would meet the second objective of the thesis that was verifying the performance of the modified InSiSST facility as well as the repeatability of the tests.

The third objective of the study, on the other hand, was to validate the results obtained from the InSiSST testing with the new proposed technique that would be achieved through an extensive set of field testing. Therefore, a testing program consisted of both laboratory and field tests, was formulated to address the aforesaid objectives. The laboratory testing commenced in April, 2003 while the field testing started in July, 2003 and ended in September, 2003. All the laboratory tests were carried out at Carleton University Civil and Environmental Engineering Laboratory and the field tests were completed on Carleton University campus and the National Research Council (NRC).

4.2. Laboratory Testing

The objectives of the laboratory testing were to verify the applicability of testing with the vane plates and to correlate the shear properties of asphalt concrete mixes with their long-term performance. In this regard, two laboratory tests namely pure torsion and repetitive uniaxial compression test were selected. As mentioned earlier, Zahw (1995)

measured the shear properties of asphalt mixes by applying torsion on cylindrical core samples. Further analysis on his results showed a good correlation with the rutting susceptibility (Goodman, 2000). This formed the basis for using the same laboratory device (torsion machine) in this research. The repetitive uniaxial compression (dynamic compression) test was also chosen because it can determine the susceptibility of a mix to permanent deformations related to shear flow as well as those associated with changes in the microstructure of asphalt concrete (Mohamed and Yue, 1994).

All the tests were carried out on cylindrical samples (with 150 mm diameter and 100 mm length) composed of two different asphalt concrete mixes. One mix type had coarser aggregate gradation; therefore, this mix was expected to have higher shear strength and higher resistance to rutting. Accordingly, the two asphalt concrete mixes were classified as relatively weak and strong, and were named type A and B respectively. For each mix type, twelve samples were tested: nine samples using torsion machine and three samples using dynamic compression test machine. Moreover, all tests were carried out at room temperature (25°C) so that the elastic properties of the material would be dominant.

4.2.1. Repetitive Uniaxial Compression Test

The repetitive uniaxial compression test was carried out to compare the resistance of the two mixes (type A and B) to permanent deformation under the vertical repeated load. In other words, this test was selected to classify the two available asphalt mixes with respect to their long-term performance. The cylindrical samples were placed between two horizontal plates and were loaded dynamically. A square wave loading function was selected based on the research work at Nottingham University (Brown and Brunton, 1984). The square wave consisted of a constant loading period of 0.2 sec followed by a rest period of 1.8 sec as shown in Figure 4.1. Maximum axial pressure of

690 kpa, representative of the tire pressure, was applied on the specimens. Therefore, the maximum load on the 150-mm samples was 1285 kg. During the rest period the load was reduced to a nominal 20 kg, almost 1.5 percent of the maximum value, to avoid separation of the loading head (top horizontal plate) from the sample surface. The loading rate was also selected to be 0.5 Hz.

The test setup is shown in Figure 4.2. A load cell was used to measure the applied load on the specimen during testing. The deformation was also measured using two Linear Variable Displacement Transducers (LVDTs) and the average value was taken as the measured deformation at each reading step. The loading system was hydraulically operated and monitored by a custom application written in National Instruments Labwindows. The load and displacement values, which were directly displayed on the computer screen throughout the testing, were stored on the hard disk and used for further analysis later.

As stated by Lu and Wright (2000), theoretically, the deformations in one cycle of the loading/unloading may consist of elastic, plastic, viscoelastic and viscoplastic components (Figure 4.1). These deformations can be decoupled according to their time dependency and whether they are recoverable during the recovery period or not. The displacement reading in the end of each cycle (i.e. the total of plastic, viscoplastic and unrecovered viscoelastic deformations), was considered as the permanent deformation. The accumulated permanent strain was then plotted versus the corresponding number of load cycles. Plots of the permanent axial strain accumulated under cyclic loading exhibit an identifiable trend as shown in Figure 4.3. Mohamed and Yue (1993, 1994) stated that the accumulated permanent axial strain undergoes three distinct stages with increasing number of cycles. These stages are referred to as primary, secondary and tertiary.

Figure 4.1 Deformation Components versus Time in One Loading Cycle

The primary stage includes an initial relatively large deformation due to surface irregularities and densification followed by a rapid decrease in the rate of deformation due to strain-hardening. During the secondary stage, the rate of accumulation of permanent deformation remains constant because the surface is already flattened and the strain-hardening is balanced by the recovery process. The permanent deformation during this stage is mainly caused by the shear flow. Finally, in the tertiary stage, the rate of deformation accelerates until complete failure takes place. This stage is usually associated with the formation of cracks, suggesting that fatigue could be the primary cause of failure.

Mohamed and Yue (1994) showed that the rate of deformation during the secondary stage is essentially a constant (k). Their results from testing two different mixes under different stress and temperature conditions clarified that this constant can

be used to evaluate the rutting potential of different asphalt mixes. Consequently, the two asphalt mixes (used in this study) were compared according to the k values obtained from the uniaxial compression tests and categorized with respect to their rutting resistance.

Figure 4.2 Repetitive Uniaxial Compression Test Setup

Figure 4.3 Typical Relationship between Permanent Axial Strain and Number of Cycles

4.2.2. Torsion / Shear Test

As stated before, one of the objectives of the torsion / shear test was to verify the applicability of the vane plates while the other objective was to correlate the shear properties of the mix to its long-term performance as measured from the dynamic compression test. Consequently, some of the cylindrical samples were tested following the conventional method and the rest were tested using the new vane plates manufactured specifically for the torsion machine. The results obtained from each method of testing were then compared to investigate if the shear properties of the asphalt mix measured with the new testing methodology were reliable. In this regard, the shear stiffness of each asphalt mix was determined using three sets of tests as listed below:

- Three specimens were tested with the circular disks glued on both sides. The resultant shear stiffness value was assumed to be the reference.

Figure 4.4 Torsion Machine

- Three specimens were tested with one side glued to the disk and the other side connected to the cross-vane plate.
- Three specimens were tested with one side glued to the disk and the other side connected to the single-vane plate.

The apparatus used in the study was a Tinien Olsen Lo-Torg torsion machine (Figure 4.4). The machine is equipped with two jaws to fasten the specimen, a dial gauge to display the torque, a scale to measure the angle of twist and two circular disks, which would be attached to the specimen to facilitate fastening the specimen to the jaws. The capacity of the dial gauge is 1100 Nm with a sensitivity of 2 Nm. Prior to each test the specimen was attached to the plates (one at each side), either with the epoxy or through the vanes. In the case of the vane plates, first the grooves were provided on the surface of the sample using a sawing machine. The sample was held in a mould made for this purpose and was fixed to the base of the machine to ensure a precise cut that would match the thickness of the vanes that are supposed to be driven into the

Figure 4.5 Cutting the Grooves on the Surface of the Cylindrical Samples

grooves (Figure 4.5 and Figure 4.6). In the case of gluing the plates to the specimen, a two-part epoxy similar to those used for the field testing, was used to attach the sample surface to the disk. The shafts/arms of the plates were then aligned using the seat shown in Figure 4.7, and the sample was left in a vertical position for 24 hours (the curing time for the epoxy).

During testing, a constant strain rate equal to three degrees per second is applied on the specimen and the torque value and angle of rotation are displayed on the dial gauge and the scale respectively. When the sample fails, the motor is stopped by the operator and the maximum torque value can be recorded at such moment. To have the correspondent angle of twist based on the applied strain rate, the total time of testing should be manually recorded by the operator and this would affect the accuracy of the results to some extent. It should be noted that the machine itself is not capable of recording the torque values electronically. Therefore, it is not possible to obtain the torque variation versus time and consequently the torque variation versus angle of twist.

Figure 4.6 Samples with (a) Cross-Cut and (b) Single-Cut

Figure 4.7 Aligning the Arms of the Plates

This means that only the secant shear stiffness of the asphalt mix was achievable. However, when using the vane plates, the failure could happen due to some stress concentration in the areas close to the edges of the vanes that would affect the maximum torque value. Therefore, the secant shear moduli of different test methods might not be completely comparable and it is preferred to have the initial shear stiffness of the material. To obtain the initial shear stiffness of the mixes as well as having more accurate observations, the torque dial gauge was video taped. After downloading the data to a computer and recording the correspondent torque and time values manually, the torque-time as well as the torque-displacement curves were plotted.

Assuming that the asphalt mix behaves elastically at lower temperatures and during the early stages of the test, when the torque value is small, the shear modulus can be calculated using Equation 4.1.

$$G = \frac{TL}{J\theta} \quad \text{Equation 4.1}$$

where, T is the torque value, θ is the corresponding angle of twist and J is polar moment of inertia that for a circular section with diameter D , can be calculated using Equation 4.2.

$$J = \frac{\pi D^4}{32} \quad \text{Equation 4.2}$$

The aforementioned equations are valid for a cylinder with a total length of L , however, when using the vane plates the value of L cannot be clearly defined. Therefore, the decision was made to calculate the slope of the initial linear portion of the torque-displacement curve (called shear index) and consider it as a representative of shear stiffness of the asphalt mix when comparing different methods of testing.

4.3. Field Testing

The field-testing was carried out to verify the proposed modifications, validate the obtained results and investigate the effect of different variables on the measured shear properties. The field-testing program was undertaken in two different places namely Carleton campus and the National Research Council (NRC) of Canada and consisted of four stages. The first stage covered the study of the heaters' functionality as well as the asphalt concrete response to the heating process. In the second stage, the performance of the RDT was verified. In addition, the effect of plate size and temperature on the shear properties of asphalt mixes was investigated. During the third stage, the in-situ performance of the vane plates and the repeatability of the tests were assessed. Finally, in the last stage, the results obtained from InSiSST testing were validated through a comprehensive testing program.

4.3.1. Test Sites

Carleton Campus

Two different asphalt concrete pavement sections were tested on Carleton University campus. The first section, Carleton-1, is a paved area used as an entrance to the Civil and Environmental Engineering Laboratory at Carleton University. The asphalt concrete mix is a standard Ontario HL3 85/100 mix, with an average in-situ compacted density of 2.30 g/cm^3 . Carleton-1 section has been in service since 1992 and the pavement surface is intact with almost no visible surface distresses. It is not subjected to regular traffic, but has been used for loading to and from the laboratory, temporary parking and as a temporary storage area for different materials and equipment. It should be noted that the asphalt concrete mixture on this section is aged and rather stiff.

The second pavement section on campus, Carleton-2, is a test section that was constructed specifically for the testing and validation of the InSiSST facility. This section

is 3 m wide by 10 m long and was constructed using a standard Ontario HL3 85/100 mix, where the asphalt concrete layer thickness is 100 mm. The section was poorly compacted, with an average in-situ density of 2.06 g/cm^3 , to provide a rather weak asphalt concrete mixture. The purpose of having such a weak asphalt concrete mix was to have different mixtures (i.e. different shear strength) when validating the in-situ testing facility.

National Research Council (NRC)

Four Sections were tested at the National Research Council experimental road in Ottawa that is a two-lane road with 5.9 m width. The criteria for selecting the sections were having different asphalt mix types as well as different compaction methods so that the researcher can compare the results obtained from the InSiSST testing under different conditions. The first two selected sections, NRC-1 and NRC-2, were constructed using a conventional HL4 mix designed according to the Ministry of Transportation of Ontario (MTO) specifications. The only difference between these two sections was that the NRC-1 section has 3% Sludge-Derived-Oil (SDO) as a proposed anti-stripping additive. The regular HL4 and the SDO-modified mixes were laid and compacted side-by-side for a total road length of 25 metres following the conventional construction procedures. The constructed layer was laid in one lift and the average compacted depth is 65 mm. It was also reported that, based on the laboratory test results, the SDO-treated mix had higher densities than the section without additive.

Sections NRC-3 and NRC-4 were constructed using a standard Ontario HL3 85/100 mix. Section NRC-3 was compacted with the Asphalt Multi-Integrated Roller (AMIR) designed by Abd El Halim (1990) as a fundamentally new approach to asphalt compaction, while section NRC-4 was compacted using a conventional steel roller.

Table 4.1 Job Mix Formula – Gradation Percent Passing

Asphalt Mix Type	HL3		HL4	
Sieve Designation	Percentage Passing By Dry Mass			
19 mm			100	100
16 mm	100	100	98 – 100	99.4
13.2 mm	98 – 100	98.7	83 – 95	90.7
9.5 mm	75 – 90	82.7	62 – 82	73.7
4.75 mm	45 – 65	50.0	40 – 67	54.3
2.36 mm	36 – 64	45.0	27 – 66	47.8
1.18 mm	25 – 58	40.4	16 – 60	42.4
600 μm	16 – 45	34.5	8 – 47	34.9
300 μm	7 – 26	15.2	4 – 27	17.5
150 μm	3 – 10	3.7	1 – 10	6.1
75 μm	0 – 5	2.5	0 – 6	4.6
	Penetration	% AC	Penetration	% AC
	85/100	5.2	85/100	5.0

It should be noted that the AMIR roller had not been used properly during compaction. In fact, section NRC-3 was compacted with only one pass of the fully loaded compactor that is insufficient. The average compacted depth for both sections is 50 mm.

The HL3 and HL4 mix properties are presented in Table 4.1. It should be noted that the first two sections were constructed in 1993 while the other two sections were constructed in 1990. All the sections have been subjected to light traffic, i.e. the local traffic in the National Research Council area.

4.3.2. Testing Program

Stage 1: Heaters and Pavement Temperature

The heaters were designed to facilitate heating the testing area up to any required temperature. In the beginning, it was essential to understand the response of asphalt mix to the applied heat. In this regard, the following questions would be raised:

- What should be the heaters' temperature set-point?

- How long should the heaters work so that the pavement heats up to the desired temperature?
- How deep would the heat be transferred in the asphalt mix?
- Is there any relationship between the heat transfer and the asphalt mix type?
- Is there any relationship between the surface temperature and the desired temperature of the mix?

To answer these questions the actual heating process was investigated. A test plate was located on the pavement, and then both the heaters and the heat-transfer pieces were installed in place. In order to measure the heating rate in depth a hole was drilled in the asphalt pavement under the test plate. In addition, to assess the heat transfer rate in different depths, five different holes namely, 10, 15, 20, 25, and 30 mm deep, were provided. A thermocouple was placed in the hole, and the opening was filled with silica sand to create good temperature conductivity. The heaters were then turned on and controlled using the developed macro in Microsoft Excel. Sections Carleton–1 and Carleton–2 were used in this stage to consider the effect of different asphalt mixes. A number of tests were completed with various set-points and time periods, such as 15, 30, 45 and 60 min. A regression model was then developed to relate the depth temperature to the chosen variables (Equation 4.3).

$$\text{Depth Temperature} = 14.678 + 0.083 (\text{Set - Point}) + 0.345 (\text{Surface Temperature}) \\ + 1.004 \text{ Depth} + 0.158 \text{ time} - 2.337 \text{ Asphalt}$$

$$R^2=0.937$$

Equation 4.3

Where *Surface Temperature* is the original pavement temperature, all *temperatures* are in °C, *time* is in minutes, *Depth* is in centimetres and *Asphalt* is equal to 0 for a relatively strong mix and equal to 1 for a relatively weak mix.

Stage 2: RDT and the Effect of Different Variables

In this stage of the study, a number of tests were carried out to verify the angular displacement measurements using the RDT. In addition, the effect of the plate size and temperature on the measured shear properties was studied. The load rate was always selected to be the maximum value that is 0.222 rpm and three test plate sizes (100, 125, and 150 mm) were selected for testing. In total, 23 tests were completed that included five tests using 100-mm plates and nine tests for each of the other two plate sizes. Some of the tests were implemented at the prevailing temperature, while for the rest of the tests the temperature of pavement was controlled using the heaters. The equation developed in the previous stage was used to calculate the heaters' temperature set-point in order to provide the desired pavement temperature in a certain period of time. Desired temperatures between 40°C to 50°C were selected with regard to the existing pavement temperature.

As the vane plates were still not manufactured at that time, the former method was used to attach the test plates to the pavement. Section Carleton-1 was chosen for these set of tests because of the sufficient available area for testing. The plates were glued to the surface a day in advance. Since the steel plates were bonded to the surface of the asphalt mix, the pavement is considered to be a half space with infinite dimensions in the horizontal direction and the positive vertical direction. Accordingly, the Reissner-Sagoci equation was used to calculate the shear modulus of the asphalt concrete mix. The shear strength of the material was also computed using Equation 2.1.

Stage 3: Vane Plate and Repeatability

After the vane plates were manufactured, they were tested with the InSiSST facility to verify their applicability in the field. The 125-mm plate was selected for the tests because, in this stage of the study, the cutting tool was capable of providing

grooves appropriate for this plate size. In other words, the blockers were added to the cutting tool after this phase. Sections Carleton–1 and Carleton–2 were used for testing. Twelve tests were completed using cross-vane plates, five on section Carleton–1 and seven on Carleton–2. Since, a small area of section Carleton–1 was permitted for InSiSST testing with the vane plates, there was no more room available on this section for additional tests. However, this number of tests was enough to investigate the repeatability of the tests. In addition, only one test was carried out using the single-vane plates because of the time restraint and section Carleton–2 was used for this test.

Stage 4: Validating the InSiSST Test Results

Through the previous three stages, the performance of the InSiSST facility regarding each enhancement was verified. The heaters were tested separately, the RDT measurements were checked and the applicability of the vane plates was clarified. Moreover, the tests carried out in the third stage ensured that the applied modifications perform properly altogether. The last stage of the testing program was scheduled to validate the results obtained from the InSiSST facility using the vane plates.

As mentioned earlier, the shear modulus of the tested pavement was calculated using Equation 2.3, when the angular displacement was applied to a steel plate bonded to the surface of the pavement. Reissner and Sagoci (1944) showed that the application of such forced displacement to the surface of a half space would result in pure shear stresses. However, when the displacement is applied through the embedded vanes, other stresses would be also distributed in the material. Therefore, the previously used equation cannot be used to calculate the shear modulus of the asphalt concrete mix. Since there is no closed form solution to address the relationship between the shear modulus of the material and the buried laterally applied load (similar to the case of the vane plates), it was decided to correlate the new InSiSST results to the shear modulus

Figure 4.8 (a) Partial Coring and Epoxy, (b) Partial Coring and Cross-Cut

obtained from applying pure torsion on cylindrical samples. In this regard, the following three methods of testing were selected:

- A cross-cut were provided on the asphalt pavement surface using the cutting tool and then the cross-vane plate was driven into the grooves. The torque was applied on the plate using the InSiSST facility and the torque-displacement relationship was obtained.
- The asphalt concrete layer was partially cored to a specific height that was less than the total thickness of the layer. Then the test plates were glued to the surface of the pavement. The torque was applied to the surface of the asphalt concrete while the tested sample was a cylinder, rather than a half space (Figure 4.8a).
- A cross-cut was provided on the pavement surface and the asphalt concrete layer was also partially cored, similar to the previous method. The vane plate was then placed in the grooves of the cylindrical sample, whose end was fixed to the pavement (Figure 4.8b).

It should be noted that the partial coring would result in a cylindrical test sample that is not completely fixed at the bottom. The material continuity between the partially cored cylinder and the remaining thickness of the asphalt layer would cause a slight

movement at the base of the cylinder, which is not accounted for during the angular displacement measurements.

The tests were completed at NRC experimental road sections i.e., NRC-1 to NRC-4. For each testing method, six tests were carried out per section, two close to the edge, two on the middle of the lane and two close to the centerline of the road (Figure 4.9). Therefore, the total number of tests on each section was eighteen and considering the four selected sections, the total of 72 tests were completed. Since the available blade size for the coring machine was 100 mm (4"), the same size vane plate was chosen for testing and the study was limited to the cross-vane plates. The pavement was heated up to 45°C prior to testing and all the tests were performed with the maximum loading rate or motor speed. It should be noted that in some cases the pavement cooled down to a lower temperature than the desired temperature because of the unpredicted problems that happened in the field before the test could be started. Among these problems the temporary malfunctioning of the motor, generator and other electronic devices can be named.

Figure 4.9 A Schematic of the Test Locations and Methods on Each Section

Chapter 5. Test Results and Data Analysis

5.1. Introduction

As discussed in the previous chapter, the testing program was formulated to meet the two objectives of the thesis that are verification of the InSiSST facility and validation of the results obtained from testing. Consequently, the testing program was followed with the order mentioned earlier and then the recorded data were analyzed comprehensively. In the first step, the results of the tests were analyzed while focusing on each of the modifications applied on the InSiSST facility individually. In this regard, the workability of the RDT, the applicability of the vane plates and the temperature response of the pavement were investigated.

Secondly, the in-situ shear properties of the asphalt concrete mix namely shear strength and shear modulus were studied and the effect of different variables i.e. temperature and plate size, on the calculated shear properties were investigated. Finally, and most importantly, the results obtained from testing with the modified InSiSST facility were correlated to the shear modulus values.

5.2. Temperature Response of Asphalt Concrete Mixture

As mentioned earlier, the tests on the NRC sections were carried out at 45°C pavement temperature. The data recorded during the heating process were analyzed to study the temperature response of asphalt concrete mixes. Table 5.1 shows the summary of the temperature-related data for the tests completed at NRC using the InSiSST and the vane plates. In this table the existing temperature represents the pavement temperature before heating and the final temperature is the pavement temperature after heating. Since the pavement would start to cool down after removing

the heaters, during the set-up period prior to the actual test, the asphalt mix was heated to a higher value than the desired temperature (45°C).

Table 5.1 Temperature Related Data for Testing Areas at NRC (100-mm Cross-Vane Plate, Flush)

Section	Testing area	Existing Temperature (T ₀) (°C)	Final Temperature (T ₁) (°C)	Time (Min)	Set Point (°C)	Air Temperature (°C)	Wind Speed (Km/h)
NRC-1	1	26.40	48.04	20.28	104	16.4	9.4
	2	31.08	47.80	28.60	103	16.8	13.1
	3	32.67	48.25	19.85	109	17.4	15.0
	4	28.85	48.10	28.82	106	20.0	13.2
	5	33.93	48.47	16.32	113	20.5	14.9
	6	31.98	48.52	18.17	120	20.5	13.6
NRC-2	7	35.87	48.23	20.05	113	21.9	16.7
	8	37.77	48.15	16.77	104	22.4	18.3
	9	22.37	48.76	20.77	107	18.5	9.0
	10	28.52	48.35	25.52	103	19.0	7.7
	11	35.13	49.90	16.28	114	19.7	9.1
	12	34.42	49.03	24.78	109	20.2	10.5
NRC-3	13	29.43	49.81	14.42	109	19.9	18.3
	14	35.88	49.46	10.17	108	19.6	18.1
	15	30.70	49.04	27.63	109	19.3	15.2
	16	21.10	49.09	42.67	109	16.7	19.0
	17	27.39	49.14	27.58	117	18.1	16.4
	18	29.20	49.02	33.70	110	18.3	14.1
NRC-4	19	39.42	49.66	12.82	114	24.6	9.9
	20	35.42	48.51	15.63	114	24.4	12.1
	21	23.21	48.97	52.15	115	18.2	12.2
	22	31.16	54.99	20.55	116	20.0	19.1
	23	29.48	49.32	23.15	115	21.0	20.0
	24	31.44	49.13	24.37	118	21.5	21.9

Figure 5.1 Relationship between the Temperature Increase and Time

The regression analysis showed that there is a statistically significant relationship, at the 95% confidence level, between the temperature increase and the required heating time. Figure 5.1 shows the scatter plot of the temperature increase ($T_1 - T_0$) versus time as well as the linear regression model fitted to the data. The coefficient of determination (R^2) for this model was 0.758 that shows a relatively high correlation between the dependent and independent variables. The remaining 25% in the coefficient of determination ($1 - 0.758 = 0.242$) can be ascribed to the other independent variables namely, wind speed, air temperature, heaters' set-point and the mix type that did not turn out to be significant in the model.

5.3. The Effect of Temperature and Plate Size

As stated before, a set of tests was carried out on section Carleton–1 to examine the accuracy of the angular displacement measurements obtained from the RDT. In addition, the results of these tests were further analyzed to study the effect of different variables, namely temperature and plate size, on the strain rate and the measured shear properties of the mix. Since the tests were carried out on the plates glued to the pavement surface, Equation 2.3 was used to calculate the shear modulus and Equation 2.1 was used to compute the shear strength of the asphalt mix. Moreover, considering the fact that the Reissner–Sagoci equation was derived for an elastic material, the initial shear modulus was calculated. The reason was that in the early stages of the test the torque values are smaller and therefore the asphalt mix behaves more elastically. In this regard, the average of the shear modulus calculated for the first few points of the torque–displacement curve was considered as the initial shear modulus. The summary of the test results and the shear properties are reflected in Table 5.2.

As shown in the table, for some of the tests, the maximum angle of rotation and therefore the shear modulus values are missing. This was due to the problems associated with the wiring system of the InSiSST facility that resulted in losing the angular displacement readings from the RDT in a few cases. However, the torque values measured by the torque cell were available for those tests and the shear strength of the material were calculated and used in the analysis.

Moreover, the results of the tests with the vane plates at Carleton and later at NRC were studied with respect to the strain rate, to investigate the effect of the new test plates (vane plates) as well as the effect of different mix type (with different shear strengths).

Table 5.2 Shear Properties and Temperature Values for the Tests on Glued Plates (Section Carleton–1)

Diameter (mm)	Temperature (°C)	Max Torque (N.m)	Max Displacement at Failure (Radians)	Shear Strength (MPa)	Initial Shear Modulus (Mpa)
100	33	579.2	0.02276	3.16	49.34
100	33	710.4	0.02684	3.44	48.94
100	33	751.8	0.02346	4.66	61.53
100	37	461.3	0.01990	4.28	43.97
100	55	173.2	0.02553	0.84	17.58
125	33	1053.2	0.02880	2.79	56.61
125	35	1089.4	0.01834	2.89	107.03
125	37	831.7	0.02063	1.79	90.24
125	37	1026.8	0.02442	2.72	86.58
125	39	653.8	—	1.82	—
125	45	611.5	0.02409	1.80	56.52
125	47	515.6	0.02382	0.90	23.19
125	50	382.9	0.02416	0.96	36.17
125	53	389.6	0.01850	0.97	48.18
150	36	1100.5	0.01626	1.85	102.69
150	37	1012.0	0.02388	1.36	65.06
150	42	1101.2	—	1.74	—
150	44	732.2	0.02438	1.56	59.08
150	47	771.5	0.02927	0.91	45.87
150	48	735.8	0.02409	1.18	63.04
150	48	689.2	0.01827	1.22	75.63
150	51	587.6	—	0.66	—
150	51	685.8	—	0.81	—

5.3.1. Strain Rate

As discussed earlier, the study by Bekheet (2002) revealed that the actual strain rate obtained from the InSiSST testing is significantly different from the assumed constant strain rate; therefore, the RDT was added to the facility to address this issue. The actual angular displacement rate measured using the RDT confirmed this statement. The results of the tests carried out on section Carleton–1 clarified that the actual rotary displacement at failure is significantly less than the angle of rotation at failure calculated based on the constant rate assumption, especially for larger plates. Figure 5.2 demonstrates the actual strain rate for a test on 100–mm plate as well as the

Figure 5.2 Actual Strain Rate Compared to the Constant Strain Rate

corresponding torque value and the constant displacement rate. The figure displays the corrected torque–time curve and the adjusted zero point as was previously used in the calculations, besides the actual displacement–time curve. Looking at the figure, one can recognize the difference between the actual displacement at the time of failure and the displacement calculated based on the constant rate assumption.

Bekheet (2002) also concluded that the larger the plate used during testing the smaller the actual angular displacement at failure. However, the results of the tests obtained from the RDT, showed that there is no significant difference between the angles of twist at failure when using different test plate sizes. Figure 5.3 demonstrates the variation of the strain rate with plate size at a constant pavement temperature of 37°C. In this figure, the angular displacements at failure and their corresponding time values are shown with dashed lines. More specifically, the failure rotation angles are 0.0199, 0.0213 and 0.0239 radians for 100–, 125– and 150–mm plates respectively that are quite similar with respect to the maximum displacement values.

Figure 5.3 Variation of Strain Rate with Plate Size (at 37°C)

In addition, it was found that the actual strain rate is sensitive to the pavement temperature. This fact is illustrated in Figure 5.4, where the displacement–time curves are plotted for the tests carried out at three different temperatures using 100–mm plates. The figure depicts that as the pavement temperature increases the displacement–time curve tends to be a straight line that is closer to the curve of the constant strain rate. Once more, it can be seen that there is not much difference between the angular displacement values at failure.

The scatter plot of the angular displacement at failure for all the tests carried out on section Carleton–1 is shown in Figure 5.5. Looking at the figure, one can realize that no specific relation is found between the rotary displacement and the plate size or temperature of the asphalt mix. The plate size did not affect the rotation angle at failure, as the mean displacement values for tests with 100–, 125– and 150–mm plates are 0.0237, 0.0228 and 0.0227 radians respectively.

Figure 5.4 Variation of Strain Rate with Temperature (for 100-mm Plate)

Figure 5.5 Variation of the Angular Displacement at Failure with Temperature

However, both the test plate size and the temperature of the pavement influenced the time of testing. The larger the test plate diameter or the lower the asphalt concrete temperature, the longer it takes to reach the failure point. This is expected as the failure surface would be smaller for smaller test plates; therefore, the maximum required torque would be smaller and the section fails faster. In the case of high temperature, the viscous properties of asphalt concrete mix is dominant and the shear resistance is lower; as a result, the failure takes place at a smaller torque value and in shorter time period.

The results of the tests with the vane plates were also analyzed with respect to the strain rate. Figure 5.6 and Figure 5.7 show the rotary displacement variation versus time for the tests on sections Carleton-1 and Carleton-2, as representatives of strong and weak asphalt mixes with regard to compaction, respectively. It should be noted that one of the tests on the strong asphalt, at a lower pavement temperature (35°C), was unsuccessful because the InSiSST facility lost its stability before the failure could take place. Therefore, the tests on this section were carried out at higher temperatures i.e., from 45 to 51°C.

In general, in comparison with the tests on the steel plates glued to the pavement surface, the tests with the vane plates require longer time (more than 50%) to reach the failure. This is expected as the failure surface is bigger in the latter case, due to the fact that the height of the failed section is the same as the height of the vane that is almost twice as much as the height of the failed sections when using epoxy. Since the total testing time is longer in the case of vane plates the sudden change in strain rate from low to high would not be observed as it was the case for epoxy, especially for the larger plates and at lower temperatures (Figure 5.3 and Figure 5.4).

Figure 5.6 Angular Displacement Variation with Time for Strong Asphalt Mix (125-mm Cross-Vane Plate)

The displacement-time curves for the tests on sections Carleton-1 and 2 with the vane plates, as shown in Figure 5.6 and Figure 5.7 respectively, indicate that the strain rate is still nonlinear for both asphalt mixes. It can be also seen that the deviation from linearity is higher for the strong mix compared to the weak mix, taking into

Figure 5.7 Angular Displacement Variation with Time for Weak Asphalt Mix (125-mm Cross-Vane Plate)

consideration that the pavement temperature is high for the former asphalt mix. In addition, similar to the tests with epoxy, the displacement variation with time follows the same rate as that of the constant strain rate assumption, after failure takes place.

Finally, the results confirmed the fact that the boundary conditions have significant influence on the strain rate as the displacement–time curve for the tests on partially cored samples showed an almost linear variation. An example of the rotary displacement variation with time for a partially cored sample is shown in Figure 5.8. As it can be seen in the figure, the absence of lateral confinement due to coring has resulted in the angular displacement that is almost linear with time, such that loading was at a constant strain rate. It should be mentioned again that the slight difference from a straight line could be ascribed to the fact that the cylindrical samples are not completely fixed at the base.

Figure 5.8 The Effect of Boundary Conditions on the Strain Rate

5.3.2. Temperature

The effect of pavement temperature on the measured in-situ shear modulus of asphalt mixes was investigated previously. Bekheet (2002) correlated the shear modulus (G) of asphalt mix to the temperature (T) using an exponential model with a general form of Equation 5.1, where A and B are constants. The variation of the shear properties of asphalt concrete mixes (shear strength and shear modulus) with temperature are further analyzed in this study using the results obtained from the RDT.

$$G = Ae^{-BT} \quad \text{Equation 5.1}$$

The results of the tests completed on section Carleton-1 (summarized in Table 5.2) were analyzed following the same general model. Both the shear modulus and the shear strength values were correlated to the asphalt pavement temperature at the time of testing. The results related to each plate size were analyzed separately and are reflected in Figure 5.9 and Figure 5.10. The graphs show that as the temperature decreases the maximum shear stress and the initial shear modulus increase that is expected due to the fact that the stiffness is higher at lower temperatures.

Moreover, as it can be seen in the figures, the coefficients of determination (R^2) for all the cases were relatively high, ranging from 0.656 to 0.964. However, it should be mentioned that obtaining high R^2 values can be due to the fact that the sample size was not big enough for modelling. Moreover, two of the shear modulus values related to the tests with 125-mm plate and one related to the tests with 150-mm plate, were excluded from the database as potential outliers. As a result, the coefficients presented in these models are not recommended and the model is presented only to introduce the general trend. A large-scale experimental program is required to develop temperature master curves for the in-situ shear stiffness of asphalt concrete mixes. The flexible heaters and the RDT would facilitate this task by providing the possibility to control the temperature

and measure the actual displacement respectively. However, establishing such an experimental program is beyond the scope of this thesis.

5.3.3. Plate Size

The variation of in-situ shear properties of asphalt mix with temperature was studied based on the results of the tests on section Carleton-1 and was analyzed for each test plate size, separately. These results, as shown in Figure 5.9 and Figure 5.10, were then compared to account for the effect of the plate size on the measured shear strength and shear modulus.

The results shown in Figure 5.9 suggest that for a particular mix the larger the plate size, the lower the shear strength. This result is expected since the effect of the aggregate size would be more influential when the plate size is smaller. Clearly, more shear stress would be required to fail one aggregate than failing the bond between two aggregates. Furthermore, the results shown in the same figure indicate that the difference between stress values obtained from different test plate sizes increases as the pavement temperature decreases.

Figure 5.9 Maximum Shear Stress Variation with Temperature for Different Plate Sizes

Figure 5.10 Initial Shear Modulus Variation with Temperature for Different Plate Sizes

On the other hand, at higher temperatures i.e. higher than 55°C, the in-situ shear strength of the mix appeared to be independent of plate size. Such a finding supports the explanation given regarding the effect of plate size on measured shear property under normal temperature conditions. Clearly, when the asphalt mix is subject to such higher temperatures the bond provided by the asphalt cement appears to breakdown and thus the effect of the plate size appears to have no significant influence anymore.

The second shear property, whose variation with temperature and plate size was studied, was the initial shear modulus. Bekheet (2002) investigated the effect of test plate size on the measured shear moduli that were calculated using the angular displacement values obtained from the constant strain rate assumption. However, the discussion on the actual strain rate, presented earlier in the thesis, showed that the measured shear modulus will be different if the actual displacement values are used in the calculations. Figure 5.10 displays the variation of initial shear modulus with

temperature and test plate size. The graphs show that contrary to the shear strength, the shear modulus increased as the test plate diameter increased with a greater difference observed at lower temperatures.

Similar to the maximum shear stress, the shear modulus tends to become constant under higher temperatures. Interestingly, the curves related to 125– and 150–mm plate sizes indicate that the test plate diameter might not have any effect on the measured shear modulus if it is greater than a certain value. However, it should be noted again that the trend lines presented in Figure 5.10 are based on the database that is not big enough for modelling; therefore, an extended field testing is required to clarify this conclusion that was not possible in this study because of the time restraint.

5.4. Vane Plate

The applicability of vane plates was first examined in the laboratory based on a testing program that was mentioned earlier. The specimens made of two different asphalt mixes were used for pure torsion tests both with the vane plates (the new method) and the conventional method, in which the two ends are fixed to the disk using epoxy. After having the approval from the laboratory tests, the in-situ tests were carried out using the InSiSST facility and the vane plates manufactured for the field testing, to verify the workability of the new method and the repeatability of the tests.

5.4.1. Laboratory Testing

Torsion / Shear Test

The torsion tests were carried out on the two asphalt mixes (A and B) using nine samples for each mix type. The cylindrical samples were divided into three groups per mix type, with three samples belonging to each group. The specimens of the first group were tested with the disks glued to both sides, while the samples of the other two groups

had one disk glued to one side and the vane plate connected to their other side. The results of these tests are summarized in Table 5.3.

As it can be seen from the table the results for two of the tests are missing. That was due to the failure that took place in the weld, which connected the shaft to the disk. In addition, it was found that the shafts were not strong enough for the 150-mm samples used in this study and twisted during testing and in some cases the circular shaft would even rotate inside the jaws of the machine. Therefore, it should be noted that the results of the torsion tests might not be very accurate. However, since the situation is similar for all the tests their results would be comparable for the purpose of this research.

Table 5.3 Summary of the Torsion Test Results (on samples with 150 mm diameter)

Test Method	Sample No.	Density (g/cm ³)	Shear Index (Nm)	Sample No.	Density (g/cm ³)	Shear Index (Nm)
One side single- vane	A1-1	2.487	1311.31	B1-1	2.251	2006.18
	A1-2	2.485	1430.32	B1-2	2.258	1435.27
	A1-3	2.477	1685.62	B1-3	2.262	2302.71
	Mean	2.483	1475.75		2.257	1914.72
	CoV	0.205	12.96		0.251	23.03
One side cross-vane	A2-1	2.518	1616.78	B2-1	2.254	1902.83
	A2-2	2.498	1319.62	B2-2	2.303	1649.31
	A2-3	2.483	1858.49	B2-3	2.182	2068.94
	Mean	2.500	1598.30		2.247	1873.69
	CoV	0.709	16.89		2.700	11.28
Two sides epoxied	A3-1	2.515	—	B3-1	2.248	1706.70
	A3-2	2.459	—	B3-2	2.293	1842.01
	A3-3	2.494	1872.95	B3-3	2.135	1625.44
	Mean	2.490	1872.95		2.226	1724.72
	CoV	1.142	—		3.661	6.34

The initial slope of the torque–displacement curve (called shear index), that is a representative of shear stiffness of the material, was calculated for all the tests. The results showed that the asphalt mix with coarser aggregate had higher shear index, indicating that this mix was stiffer. The only exception was the shear indices obtained from the samples tested with the disks glued to both sides. However, it should be noted that for most of those tests the failure took place in the shaft and therefore the shear indices were calculated based on the available data during the early stages of loading that might not be reliable. Considering the aforesaid problem with the shafts, one can deduce that the mean of the shear indices for the tests with the vane plates are quite similar, showing that both the single–vane and the cross–vane plates can substitute epoxy. The noteworthy fact is that, based on the observations during testing, for the tests with the vane plates the failure started from the side connected to the vane plate and not the side glued to the disk. This indicated that the failure took place due to the stress distribution on the vane side. Figure 5.11 shows the failure of the samples under torsion test for the three methods.

(a)

(b)

(c)

Figure 5.11 Failure of the Samples with (a) a Single-Vane Plate on One Side, (b) a Cross-Vane Plate on One Side, (c) Two Sides Glued

5.4.2. Field Test

The laboratory verification of the applicability of the vane plates paved the way to start the in-situ testing using such plates. The first tests were carried out on section Carleton-2 and at higher temperatures to eliminate any instability or capacity problems. The pavement surface was cut using the cutting tool and the vane plates were driven into the grooves as shown in Figure 5.13. Two tests were completed at similar pavement temperatures, one using the single-vane plate and the other using the cross-vane plate, and then, the torque-displacement curves were plotted (Figure 5.12). It is clear in the figure that the test with the single-vane plate failed in a torque value twice as much as that obtained from the test with the cross-vane plate (227.6 and 119.2 Nm respectively). The reason is that in the case of the cross-vane the torque is applied on an area divided into four sections while in the case of the single-vane the same area is divided into two sections. However, the slope of the initial linear section of the curve, previously introduced as shear index, is similar for both tests (3000 Nm). This means that either of

Figure 5.12 Output of the First Two Tests using 125-mm Vane Plates

Figure 5.13 (a) Cross-Cut on the Pavement Surface, (b) Vane Plate Driven into Grooves

Figure 5.14 Side View of the RDT as Geared to the Collar and Connected to the Vane Plate after Failure

Figure 5.15 Failure View of the Tests with (a) Cross-Vane and (b) Single-Vane Plates

The vane plates can be used to get the shear index value. Figure 5.14 displays the testing area after failure while the RDT and the torque cell are not yet removed and Figure 5.15 shows the failed sections of both tests with the single-vane and the cross-vane plate.

The next step was to examine the InSiSST facility with the vane plates on a stronger asphalt mix i.e. section Carleton-1. As mentioned before, the torque value obtained from the single-vane was twice as much as that obtained from the cross-vane. In addition, the capacity of the InSiSST facility in its current configuration is limited and the instability problem had also occurred in some cases. As a result, it was decided to narrow the testing program down to the cross-vane plates. In this regard, one test was carried out on section Carleton-1 using the same plate size (125 mm) and at the same temperature as the one completed earlier on the weaker asphalt pavement (Carleton-2). The comparison of the results of these two tests together with the test carried out on a steel plate epoxied to the pavement is presented in Figure 5.16.

Figure 5.16 Comparison of the Test Results with Vane Plates and Epoxy

For both asphalt pavements, the trend of the torque–displacement curve was similar to the tests with epoxy. This indicated that the InSiSST testing with the vane plate is applicable. However, as it can be seen in the figure, the test with the vane plate gave lower shear index value compared to that obtained from the test on the epoxied plate (on section Carleton–1). This happens because in case of using the vane plate larger rotation angles will take place as a result of applying the torque on a larger portion of the pavement (in terms of depth). In other words, the torque is applied on the asphalt mix through the vanes with the height of more than two times as much as the height of the failed pavement section when using the glued steel plates.

The maximum torque obtained from testing with the vane plate was quite similar to the torque value obtained in the case of epoxy. Such a result was not expected due to the fact that using different test plates (vane versus epoxy) results in different failure surfaces. Therefore, if the fully plastic failure assumption is valid different shear strength values would be obtained. Further analysis is required, based on elasticity/plasticity theory, to clarify the mechanism of failure.

5.4.3. Repeatability of the Tests

The discussion in the previous section clarified that the new approach that is using vane plates instead of epoxy, is applicable for the in–situ testing. However, it was necessary to investigate the repeatability of the tests with this new method. Therefore, four more tests on section Carleton–1 and six on section Carleton–2 were completed while using the same size cross–vane plate. The data related to one of the tests on section Carleton–1 was missing due to a recording problem with the data acquisition software and another test was unsuccessful because of the instability of the InSiSST facility during testing. The summary of the results of all the tests on these two sections is presented in Table 5.4.

Table 5.4 Results of the In-Situ Tests using 125-mm Cross-Vane Plate

Section	Test No.	Temperature (°C)	Max Torque (Nm)	Max Rotation (Radians)
Carleton-1	1	45	621.10	0.10972
	2	51	627.25	0.11079
	3	49	552.16	0.12206
Carleton-2	1	33	233.90	0.10760
	2	31	253.08	0.09858
	3	37	202.31	0.09647
	4	36	181.44	0.08222
	5	35	206.45	0.10177
	6	35	221.62	0.08356
	7	46	112.74	0.13888

The relationship between the torque and angular displacement for the tests on each of the sections at similar pavement temperatures is shown in Figure 5.17 and Figure 5.18. Table 5.5 presents the mean values and the Coefficients of Variation (CoV) for the tests on these two sections excluding test 7 carried out at a much higher temperature. Looking at the table and the figures, one can realize that the results are consistent and therefore, it can be concluded that the tests with the vane plates are repeatable. The tests carried out later at NRC, on different asphalt mixes, also confirmed this conclusion. The summary of the results related to these tests is presented in Appendix C. As mentioned earlier, the pavement temperature for the tests on these sections was set to the 45°C (controlled with the heaters). The similar CoV obtained for all the asphalt mixes in each testing method shows the consistency of the tests.

Table 5.5 Mean and CoV for the Tests on Carleton-1 and 2 using the Vane Plate

Section	Max Torque (Nm)		Shear Index (Nm)	
	Mean	CoV	Mean	CoV
Carleton-1	585.52	8.40	15712.62	13.75
Carleton-2	216.47	11.68	7334.99	19.03

Figure 5.17 Torque-Displacement Relationship for Tests on Section Carleton-1 (125-mm Cross-Vane Plate)

Figure 5.18 Torque-Displacement Relationship for Tests on Section Carleton-2 (125-mm Cross-Vane Plate)

5.5. Shear Property Measurements

After confirming the workability of the modified InSiSST facility, it was timely to validate the results obtained from the tests with the upgraded facility. The InSiSST was basically designed to measure the shear properties of the asphalt concrete mixes namely shear strength and shear modulus. These parameters were calculated using Equation 2.1 and Equation 2.3 respectively, when the torque was applied on a steel test plate bonded to the pavement surface. After replacing the gluing method with the vane approach it was required to derive these shear properties from the results obtained from the InSiSST testing.

5.5.1. Shear Strength

The InSiSST testing with the cross-vane plate resulted in a failed section that is a quartet. Figure 5.19 shows one portion of such failed section that lie between the two successive vanes. Looking at the figure, one can realize that this portion is one quarter of a frustum of a cone. As stated earlier, the failure surface obtained from testing on the glued plates was also assumed to be a frustum of a cone. However, the dimensions of this frustum of a cone were unclear and can be interpreted differently by different operators. Therefore, the calculations based on these dimensions would result in the values that were not very accurate. Conversely, when testing with the vane plate the geometry of the failure surface is clearly defined as it is the same as the vane dimensions and this will lead to more accurate results.

The maximum shear stress or shear resistance of asphalt pavement was calculated at the time of failure that is the point of the maximum torque. Unlike the case of epoxy, the stresses distributed in the pavement would be a combination of shear and other types of stresses. However, the failure took place mainly due to shear stress distributed on the failure surface. Figure 5.20 demonstrates the shear stress distribution

on the failure surface as well as the applied compression load (equivalent of the torque carried by each quarter). At the early stages of loading and at lower temperatures the asphalt mix would behave elastically; therefore, the maximum stress at each moment can be calculated based on the following assumptions:

- The stresses on the bottom surface are distributed based on the relationship given by Equation 5.2 (Figure 5.21a).
- The stress is uniformly distributed on the side (Figure 5.21b).

The stress would then be calculated using Equation 5.4.

Figure 5.19 Failed Section for a Test with the Cross-Vane Plate (3-D, Side and Top View)

Figure 5.20 Stress Distribution on the Failed Section in Case of the Vane Plate

Figure 5.21 Schematic of Stress Distribution on the (a) Bottom and (b) Side Surface for Elastic Behaviour

$$\tau_{(r)} = \tau_1 \left(\frac{r}{R_2} \right) \quad \text{Equation 5.2}$$

$$\frac{T}{4} = \int_0^{R_2} \tau_1 \left(\frac{r}{R_2} \right) r \left(\frac{\pi}{2} r dr \right) + \int_{H-h}^H \tau_1 \left(\frac{\pi}{2} \frac{zR_1}{H} \right) \left(\frac{zR_1}{H} \right) dz \quad \text{Equation 5.3}$$

$$\tau = \frac{T}{\pi \left(\frac{1}{2} R_2^3 + \frac{2}{3} h \frac{R_1^3 - R_2^3}{R_1 - R_2} \right)} \quad \text{Equation 5.4}$$

where T is torque, τ is shear stress and the other parameters are defined in Figure 5.21.

At higher temperatures and at the time of failure the asphalt mix would behave non-elastic. Therefore, assuming that the failure is fully plastic, the stress would be uniformly distributed on the whole failure surface. Then, the shear strength can be calculated by integrating the torque resulted from the distributed shear stress, similar to the previous case (Equation 5.5).

$$\tau = \frac{3T}{2\pi \left(R_2^3 + h \frac{R_1^3 - R_2^3}{R_1 - R_2} \right)} \quad \text{Equation 5.5}$$

For the tests on sections Carleton-1 and Carleton-2 the shear strength was calculated using Equation 5.5 (see Table 5.6). As expected, the maximum shear stress for asphalt mix on section Carleton-2 (the poorly compacted section) is much lower. Given the high variability expected from the asphalt concrete mix, the shear strength values have a reasonable correlation with the pavement temperature at the time of testing. The variation of shear strength with temperature for the tests on section Carleton-2 is shown in Figure 5.22. The same general model as presented earlier (Equation 5.1) was fitted to the data and a high coefficient of determination ($R^2 = 0.958$) was obtained indicating that a good correlation exists between the shear strength and temperature. Moreover, it was shown that the same trend as was observed for the tests with the epoxy is observed for the tests with the vane plate. Although the high R^2 value

Table 5.6 Shear Properties of the Asphalt Mix on Sections Carleton-1 and 2

Section	Test No.	Temperature (°C)	Shear Strength (Mpa)
Carleton-1	1	45	0.95
	2	51	0.96
	3	49	0.85
Carleton-2	1	33	0.36
	2	31	0.39
	3	37	0.31
	4	36	0.28
	5	35	0.32
	6	35	0.34
	7	46	0.17

can be partly related to the small sample size, it can also mean that the shear strength values are more accurate when testing with the vane plate.

The shear strength values were also calculated for the asphalt concrete mixes at NRC based on the fully plastic failure assumption. These values are tabulated in Table C.1 presented in Appendix C and are displayed in Figure 5.23. The results indicated that asphalt mixes used for the first two sections, NRC-1 and NRC-2, had lower shear strength compared to the other two sections. The first two sections were paved with HL4 mix while the other two sections were constructed with HL3. In addition, the last two

Figure 5.22 Shear Strength Variation with Temperature (Section Carleton-2)

Figure 5.23 The Effect of Asphalt Mix Type or Compaction Method on the Shear Strength (Tests with 100-mm Cross-Vane Plate at 45°C, NRC)

sections were constructed three years earlier and were aged more than the other sections and as a result, they were expected to be stiffer. Furthermore, asphalt mix on section NRC-1 had lower shear strength as compared to NRC-2 because the former mix had anti-stripping agent that was expected to lower the strength of the material. Finally, the section compacted with AMIR roller showed lower strength compared to the same mix compacted with the conventional steel roller due to the improper compaction effort as mentioned before (with only one pass of the roller). Therefore, the upgraded InSiSST facility was able to differentiate between the pavements constructed with different asphalt mixes and compacted with different compaction methods.

5.5.2. Shear Modulus

Shear modulus is the fundamental property of the material that is directly related to the resistance of asphalt concrete mixes to rutting or permanent deformation. When the tests were carried out on plates glued to the pavement surface, the shear modulus of

the mix was calculated based on the Reissner and Sagoci's equation. However, for a case similar to the vane plates, where an embedded load is applied to a material, no closed form solution, that can relate the torque and angular displacement to the shear modulus, is available. Hence, a testing program was scheduled to correlate the shear index obtained from the tests with the vane plate to the shear modulus of cylindrical samples of the same material.

The tests were carried out on the experimental road at NRC with two different methods of testing as mentioned before. The first method included the tests on the pavement surface with the vane plate (Surface-Vane tests) while for the second method the pavement was partially cored and the test plate was bonded to the surface (Cored-Epoxy tests). Since there was a problem associated with the torsion tests in the laboratory, the third set of tests were added to the schedule to verify the result obtained from the laboratory tests. In the third method, the pavement was also partially cored; however, the vane plate was used instead of the epoxy (Cored-Vane tests). For both of the partially cored methods, Equation 4.1 was used to calculate the shear modulus. The total length of the cores that were measured individually in the field, were used in the calculations for both methods of testing (vane and epoxy). The summary of the results of the tests obtained from these three methods is tabulated in Table 5.7 and the detailed information is presented in Appendix C.

Table 5.7 Shear Modulus / Index for the Tests at NRC

Section	Shear Modulus (Mpa)				Shear Index (Nm)	
	Cored-Epoxy		Cored-Vane		Surface-Vane	
	Mean	COV	Mean	COV	Mean	COV
NRC-1	8.10	30.39	7.00	33.43	4886.45	10.31
NRC-2	13.61	25.83	11.79	28.97	7840.90	14.72
NRC-3	17.52	26.04	10.71	31.61	9658.06	18.56
NRC-4	24.52	18.50	12.50	28.60	10934.49	13.96

The ANOVA was carried out for each test method to investigate if the mix type and compaction method affect the shear modulus / Index values. The null hypothesis was that there is no difference between the mean values of the shear modulus / index for different asphalt mix or compaction method. The results showed that there was a statistically significant difference, at the 95% confidence level, between the mean shear modulus values obtained from the tests on different sections.

The results of the ANOVA are presented in Table 5.8. It can be seen in the table that the only case in which the significance level was slightly different from the critical value, at the 95% confidence level, was the cored-vane. However, such a small deviance from the acceptable significance level (0.05) can be ignored due to the following problems. In some of the tests, especially on the last two sections, the base of the core was not completely dry because it had rained during the night before and the temperature was not high enough on the day of testing to dry it. Although the effort was made to remove the water as much as possible, after testing it was found that the base for some of the tests was still moist. Moreover, two out of six tests on section NRC-3 were unsuccessful because the top layer was thin and the failure took place in the

Table 5.8 Summary of ANOVA for Shear Stiffness of Different Sections and Test Methods

	Source of Variation	Sum of Squares	F Statistic	Sig. Level	Acceptable Sig. Level	Null Hypothesis
Cored-Vane (Shear Modulus)	Sections	94.743	3.005	.059	.05	Accepted
	Error	178.635				
	Total	273.378				
Cored-Epoxy (Shear Modulus)	Sections	855.276	19.478	.000	.05	Rejected
	Error	278.093				
	Total	1133.370				
Surface-Vave (Shear Index)	Sections	111332480.85	16.391	.000	.05	Rejected
	Error	43017295.47				
	Total	154349776.32

Figure 5.24 Relationship between the Shear Modulus Obtained from the Cored-Epoxy and the Cored-Vane tests

interface between the two layers. This resulted in having a small sample size for the third section. The results of the ANOVA verified that using either the vane plate or the epoxy, the InSiSST facility was able to differentiate between the pavements constructed with different asphalt mix and compaction methods.

The shear modulus obtained from the cored-epoxy tests and the cored-vane tests were also compared. The scatter plot in Figure 5.24 shows the shear modulus values obtained from the cored-vane tests versus those obtained from the cored-epoxy tests. A regression model was then developed to correlate the results of the two test methods ($R^2 = 0.484$). The relatively low R^2 value was not unexpected due to the aforesaid problem i.e. the wet pavement as well as the slight variation in pavement temperature for the cored-epoxy tests. Because of the cold weather and the high wind speed, the temperature of the pavement in a number of the cored-epoxy tests dropped (lower than 45°C) during the setup. However, the analysis showed that there was a statistically significant relationship between the results of the two methods indicating that the vane plates can replace the epoxy.

Figure 5.25 Relationship between the Shear Index Obtained from the Surface-Vane tests and Shear Modulus of the Cored-Epoxy tests

The main objective of the testing program at NRC was to correlate the shear index obtained from the surface-vane tests to the shear modulus of the material. Figure 5.25 shows the scatter plot of the two mentioned variables as well as the related regression model. The coefficient of determination for this model was 0.713 and the independent variable was statistically significant at the 95% confidence level. The remaining 29% ($1 - 0.713 = 0.287$) can be ascribed to pavement temperature inconsistency in cored-epoxy tests and the effect of the mix type that was not considered in this model.

5.6. Shear Properties and the Long-Term Performance

Repetitive Uniaxial Compression Test

The 150-mm specimens were tested at Carleton Civil and Environmental Engineering Laboratory in order to study their long-term performance based on their resistance to rutting or permanent deformation. Three cylindrical samples per each mix type were used for repetitive uniaxial compression testing and they were compared according to the k values. As was explained earlier, it is believed that this parameter (k) is the indicator of resistance to rutting.

The summary of the results from the dynamic compression tests as well as the physical mix properties are tabulated in Table 5.9. The results were in conformance with the results of the torsion/shear tests in that, the mix with coarser aggregate (asphalt mix type B) had higher resistance to permanent deformation. All the samples made of asphalt mix type B had lower k values meaning that for a particular load cycles, the sample made of such mix would deflect less than the sample made of the other asphalt mix.

Table 5.9 Summary of the Uniaxial Compression Tests

Asphalt Type	Sample No.	Density g/cm ³	Air Voids %	Slope (k) (E-06)	Cycle No. N_f
A	A-1	2.46	0.73	24.18	108242
	A-2	2.51	0.44	19.57	121176
	A-3	2.45	1.10	26.70	97445
	Mean	2.47		23.48	108954
B	B-1	2.30	2.95	12.79	167501
	B-2	2.30	2.96	8.65	328384
	B-3	2.31	2.96	7.35	352245
	Mean	2.30		9.60	282710

The results indicated that the stronger mix failed in higher load cycles showing that it was also more resistant to fatigue. An example of the results for each mix type is presented in Figure 5.26, which shows that the weaker mix had higher k value and failed in lower number of load cycles. The photos of these two samples after the failure, are shown in Figure 5.27a and Figure 5.27b. This laboratory testing revealed that there is a correlation between the shear properties of an asphalt concrete mix and its long-term performance. In this regard, further testing and analysis is now in process at Carleton University on both laboratory prepared asphalt mixes and the cores taken from the same sections at NRC.

Figure 5.26 Comparison between Weak and Strong Mix according to Dynamic Compression Test

(a)

(b)

Figure 5.27 (a) Mix Type A, (b) Mix Type B, after Failure under Dynamic Compression Test

Chapter 6. Conclusions and Recommendations

6.1. Summary

The first objective of this thesis was to assess and identify the major deficiencies that existed in the second generation of the in-situ shear testers (the InSiSST facility). The achievement of this objective led to the formulation of the primary objective of the thesis, which was to remedy the identified shortcomings through the implementation of a research plan dealing with the design, construction and verification of the upgrades and recommended modifications to the test facility. In addition to these two objectives the third objective was to utilize the upgraded test facility to study the effect of different variables on the in-situ shear properties of several asphalt mixes. The tested variables included effects of temperature, plate size, and plate type, on the measured shear properties. Finally, a secondary objective of the thesis was to study the relationship between the shear properties of asphalt concrete mixes and their long-term performance.

An extensive literature review was carried out to identify the main advantages and disadvantages of the existing methods and techniques dealing with the quantification and evaluation of properties of asphalt mixes and their effect on the problem of surface rutting. The previous studies showed that rutting or permanent deformation, defined as the formation of longitudinal depressions under wheel-paths, is believed to be one of the major pavement distresses that present serious safety problems. It was also found that the instability rutting caused by the shear flow is the primary source of permanent deformation. As documented in literature, researchers have spent considerable effort to measure rutting potential of HMA. They have used either of empirical, simulative or fundamental tests for that purpose. However, there

have been several deficiencies associated with each of these tests that have prevented them from accurately predicting rutting performance of asphalt mixes in the field.

In summary, these deficiencies, which are caused by the differences between laboratory and field conditions, are improper boundary conditions, different stress distributions, small specimen size in proportion to the applied load, complex and time-consuming specimen preparation, and large and expensive testing systems. As a result, the in-situ testing was found to provide better representation of the field performance and therefore the development of such a test became important for prediction of pavement long-term performance.

The following sections present and discuss, in more details, the main conclusions and findings of this thesis.

6.2. The InSiSST Modifications

Previous results of field-testing with the InSiSST showed a number of issues, which may affect the operation and accuracy of the facility, indicating that further improvements were necessary. To address the first objective of this thesis, these issues were studied profoundly and a number of modifications were suggested to enhance the facility.

The InSiSST was basically designed to apply rotary displacements, with a constant rate, on the pavement surface. However, field observations showed that the actual strain rate is significantly different from the assumed constant strain rate. It was realized that the strain rate is considerably dependent on the mix properties and the pavement temperature. To address this issue, an RDT capable of measuring the actual angular displacement electronically, was added to the facility. The facility was also equipped with a torque cell in order to measure the torque values at the selected time intervals. To obtain the correct relationship between the torque and angular

displacement it was required to record the readings of the two devices simultaneously. This problem was solved by using a data acquisition system capable of recording the measurements at the same time.

Furthermore, field-testing in the past had encountered a number of problems regarding the attachment of the test plates to the pavement surface using epoxy. A new approach was introduced, in which the vane plates were used instead of the epoxy. The grooves would be provided on the pavement surface using a cutting tool designed and manufactured at Carleton University for this purpose and then the vanes would be driven into these grooves. The new setup would considerably enhance the InSiSST testing. It would reduce the total costs of the testing process and improve its accuracy by providing the following advantages:

- The elimination of the use of epoxy, which had failed prematurely in more than 15% of the tests.
- The reduction of the time required to complete field testing, where there would be no need to wait for the epoxy to harden for 24 hours before the actual test can be started.
- The elimination of cleaning process of the test plates after each testing day that was time-consuming and needed significant labour work.
- The elimination of the need to manufacture a large number of test plates as all the tests can be carried out using only one vane plate.

Finally, a heating system was added to the facility to allow for controlling the pavement temperature during testing. If heating is required the flexible heaters would be spread on the testing area and left there until the pavement temperature reaches the desired value. The heaters are connected to the same data acquisition system and are controlled using a macro developed in Microsoft Excel environment.

6.3. Main Findings

After implementing the proposed modifications on the InSiSST facility, the new components were tested individually and collectively to verify their workability. This would meet the second objective of this study. Tests were carried out on Carleton University campus as well as the National Research Council experimental road (July 2003 to September 2003). The testing program at NRC was mainly scheduled to address the third objective of the research that was the validation of the results obtained from the modified InSiSST facility. On the whole, 108 tests were completed in the field, 36 tests at Carleton University, and 72 tests at NRC. The main findings of the testing program can be summarized as follows:

6.3.1. The Effect of Temperature

The results of the tests at NRC showed that there is a statistically significant relationship, at the 95% confidence level, between the temperature increase in asphalt pavement and the heating time when using the vane plates. The following linear regression model was developed in this study with the coefficient of determination of 0.758. The model can be used to predict the required heating time to reach the desired pavement temperature.

$$\text{Heating Time} = 1.384 \Delta T - 0.080$$

The results of the tests carried out at Carleton University with both methods of testing (vane and epoxy) revealed that the shear properties of asphalt concrete mixes are sensitive to the pavement temperature at the time of testing. The higher the pavement temperature, the lower the shear strength and shear modulus of the mix.

Exponential regression models with the general form of $G = Ae^{-BT}$ were fitted to the data. The analysis showed that there is a statistically significant relationship, at the 95% confidence level, between the dependent and independent variables. The

coefficients of determination ranged from 0.772 to 0.964 for the shear modulus models and from 0.656 to 0.886 for the shear strength models. The relatively high R^2 values indicated that there is a good correlation between the shear properties and the pavement temperature. Development of such models would help engineers to predict the shear properties of an asphalt mix at different pavement temperatures.

6.3.2. The Effect of Test Plate Size

The results of the tests completed at Carleton University also showed that the measured shear properties of asphalt mix are dependent on the test plate size. For a particular mix, the larger the plate size, the lower the shear strength. Conversely, the larger the test plate size, the higher the measured shear modulus. The tests, on the other hand, revealed that at higher temperatures the in-situ shear properties of the mix are independent of the test plate size.

Moreover, as the test plate size became larger, the initial shear modulus of the asphalt mixes appeared to be independent of the plate size. No significant difference was observed in the measured shear modulus values obtained from testing with 125-mm and 150-mm plates. These findings suggest that the InSiSST testing should be standardized for one test plate size and some factors should be defined for converting the measured shear properties when using other test plate sizes.

6.3.3. Strain Rate

The results of the tests at Carleton with the upgraded facility, which is able to measure the actual angular displacements accurately, confirmed that the strain rate is sensitive to the mix properties and the pavement temperature. For the lower pavement temperatures or the stronger asphalt mixes the deviation from linearity was higher.

The ANOVA did not show a statistically significant difference, at the 95% confidence level, between the mean values of the maximum rotation angle resulted from

the tests on different plate size and temperature. However, both the test plate size and the temperature influenced the time of testing. The larger the test plate diameter or the lower the asphalt concrete temperature, the longer it took to reach the failure point. This can be a criterion when standardizing the InSiSST plate size.

6.3.4. Vane Plates and the Shear Properties

The laboratory tests and later the field tests on the partially cored pavement samples showed that there is a statistically significant relationship, at the 95% confidence level, between the shear modulus values obtained from the tests with the two methods (epoxy and vane). Therefore, it was concluded that the vane plate can be a reliable substitute for epoxy. The consistency of the results obtained from testing on the sections at Carleton University and NRC, including different asphalt mixes, confirmed the repeatability of the tests with the new test plate. The CoV ranged from 7.45% to 18.56% for the NRC tests and 8.4% to 19.03% for the ones completed at Carleton.

The tests with the upgraded InSiSST facility indicated that the InSiSST is capable of differentiating between the shear properties of different asphalt pavements with respect to the mix type and compaction method. The ANOVA showed that there is a statistically significant difference, at the 95% confidence level, between the mean values of the shear index/modulus related to different asphalt pavements.

The variation of shear strength with temperature followed the same trend (exponential relationship) as was observed for the tests when using epoxy. The model developed for the tests on the weak asphalt mix at Carleton University had the coefficient of determination of 0.958. This value was higher than the R^2 of the similar model developed based on the results of InSiSST testing on the glued plates. The high coefficient of determination can be ascribed to the fact that the failure surface geometry

is more definite for the vane plates, and therefore the calculated shear strength values would be more accurate.

The results of the tests on NRC sections also clarified that there is a statistically significant relationship, at the 95% confidence level, between the shear index obtained from the InSiSST testing with the vane plate and the shear modulus of cylindrical samples of the same mix. The following regression model was developed and the relatively high coefficient of determination ($R^2 = 0.713$) showed that there is a good correlation between the two variables.

$$\text{Shear Modulus} = 0.002 (\text{Shear Index}) - 1.574$$

6.3.5. Shear Properties and the Long-Term Performance

To address the last objective of the research 24 tests were carried out in the laboratory, including six torsion tests and eighteen dynamic compression tests. The results of these tests indicated that the shear properties of asphalt concrete mixes are correlated to their long-term performance. Higher shear index values were obtained for the asphalt mix that showed higher resistance to rutting through the dynamic compression testing. Therefore, the InSiSST facility can be used to predict the long-term performance of the flexible pavements in the field. The higher the in-situ shear properties of an asphalt concrete mix, the higher the rutting resistance.

6.4. Recommended Test Procedure

One of the main conclusions of this study was the development of a “protocol” or “testing procedure” for the use of the modified test facility in the field. This protocol was developed based on the experience gained during the actual conduct of the experimental investigation during the summer and fall of 2003. The main steps of the procedure or protocol are described in section 3.6 and are briefly described below:

Before leaving to test site, it should be ensured that all necessary equipment for field testing are packed and are in proper working order. Safety rules should be followed when transporting the InSiSST facility and in the test site. After marking the testing locations, the grooves should be provided on the pavement surface using the cutting tool. The InSiSST should be manoeuvred over the testing location and the test plate should be driven into the grooves. The heaters should be used at such moment, if heating is required. After the pavement reaches the desired temperature the heaters should be removed, the frame should be lowered and the torque cell should be aligned and connected to the test plate. Then, the RDT should be located beside the torque cell and the testing can be started while the data would be recorded using the data acquisition system.

6.5. Recommendations for Future Work

As was mentioned earlier, the main objective of this thesis was to deal with main shortcomings of the testing facility and verify the efficiency of the implemented modifications. While these objectives have been met successfully, further research work remains to be performed so that the full potential of the new facility could be realized. The following is recommended as future research work in this area:

- As explained earlier, the analysis of the field-testing results revealed that there is a statistically significant relationship, at 95% confidence level, between the shear index and the shear modulus of cylindrical samples. However, the regression model presented in this study is related to only one pavement temperature. Therefore, a large-scale testing program is required to consider the effect of pavement temperature and asphalt mix type when correlating the shear index to the shear modulus.

- The asphalt concrete material is not a linear elastic material, although it has been traditionally modelled as so in the literature. The properties of the asphalt concrete mix can range from being linear elastic, at very low temperatures and high strain rate, to viscoelastoplastic material at higher temperatures and lower strain rates. In this study the shear strength of the asphalt mix was calculated based on the fully plastic failure assumption. However, a profound finite element analysis is required to investigate the actual shear stress distribution in different conditions namely different temperatures and strain rates. Finite element analysis would also provide the opportunity to calculate the shear strain values and therefore to draw the stress–strain curve and obtain the shear modulus directly.
- The analysis in this study showed the general relationship between the in–situ shear properties (shear strength and shear modulus) and the temperature and plate size. However, the sample size in this research was relatively small and basically included the results obtained from InSiSST testing on the glued plates and not the vane plate. The only case, in which the effect of temperature on shear strength was investigated for the tests with the vane plate, was on section Carleton–2 where the results were in conformance with the previous results. Therefore, further analysis is required to investigate the effect of different vane plate sizes and pavement temperatures on the measured shear properties.
- Because of the time restraint and capacity/stability problems with the InSiSST facility the testing program in this thesis did not include tests with the single–vane plates. Consequently, a testing program should be formulated aiming at studying the shear properties of asphalt concrete mixes obtained from field–testing using the single–vane plates. In addition, the study can be expanded to investigate the effect of the vane plate type (single–vane versus cross–vane) on the measured shear properties.

- The temperature response of the asphalt pavement was briefly studied in this research and it was found that there is a statistically significant relationship between the temperature increase and the heating time. However, a full-scale testing program should be scheduled to study the exact temperature response of asphalt mixes. Such a study will assist to optimize the heating period prior to actual InSiSST testing.
- The development of the heating system provided the opportunity to develop master curves to relate the shear stiffness of asphalt mixes to the pavement temperature. A large-scale testing program composed of testing on pavements with different asphalt mixes and at a broad range of temperatures should be followed for this aim.

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Appendix A Detailed Specification of the RDT

Special features

- Standardized output current over 90/345°
- Output current selection 0 - 20 mA or 4 - 20 mA
- Electronic system integrated in the case
- Very good absolute linearity $\pm 0.1\%$ (345°) or $\pm 0.3\%$ (90°)
- Very high repeatability 0.007°
- Unrestricted continuous rotation
- Protection class IP 65

Description

Case	Aluminum anodized, shaft passage sealed
Shaft	Stainless steel
Bearings	Stainless steel ball bearings
Resistance element	Conductive plastic
Wiper assembly	Precious metal multi-finger wiper
Electrical connections	5-pin connector (see "Accessories")
Reverse voltage protection	By means of internal diode

Mechanical Data

Mechanical travel	360, continuous	°
Permitted shaft loading (axial and radial) static or dynamic force	45	N
Starting torque	< 1.5	Ncm
Maximum operational speed	2,000	rpm
Weight	200	g

Electrical Data

Operating voltage	24 ± 6	V DC
Electrical range	90 or 345	°
Max. current consumption	35	mA

Load impedance	0-500	Ω
Absolute linearity: at 345° at 90°	± 0.1 ± 0.3	% %
Repeatability	0.007	°
Temperature coefficient of the zero point of the range	≤ 10 ≤ 15	ppm/K ppm/K
Insulation resistance (500 VDC, 1 bar, 2 s)	≥ 10	M Ω
Dielectric strength (50 Hz, 2 s, 1 bar, 500 VAC)	≤ 100	μA
Current range fluctuations relative to operating voltage alteration	≤ 10	ppm/V

Environmental Data

Temperature range	-25 to +70	°C
Vibration	5-2000 Amax = 0.75 amax = 20	Hz mm g
Shock	50 11	g ms
Life	100 million	movem.
Protection class	IP 65 (DIN 400 50 / ICE 529)

Connection Diagram

Dimensions

(a)

(b)

(c)

(d)

Note: When the shaft marking is pointing towards the slot on the rim of the housing, the wiper is located in an electronic center position.

Appendix B Personal Daq System Specification

Channel Capacities for Various Personal Daq Setups

Personal Daq System	Volts Inputs	Thermocouple Inputs	Digital I/O	Freq/Pulse Inputs
Personal Daq/55	5 DE, or 10 SE	5 DE	8	2

DE = Differential Mode, SE = Single-Ended Mode

Speed vs. Resolution ¹			
Speed Designation	Measurement Duration (per channel)	Maximum Sample Rate ² (Samples/sec)	Resolution ² (Bits RMS) (-4 V to +4 V range)
Very Slow, 50 / 60 Hz rejection	610 ms	1.6 / sec	22
Slow, 50 Hz rejection	370 ms	2.7 / sec	22
Slow, 60 Hz rejection	310 ms	3.2 / sec	22
Medium, 50 Hz rejection	130 ms	7.7 / sec	21
Medium, 60 Hz rejection	110 ms	9.2 / sec	21
Medium	40 ms	25 / sec	19
Fast	20 ms	48 / sec	17
Very Fast	12.5 ms	80 / sec	15
Notes:	<ol style="list-style-type: none"> 1. Each channel can have independent measurement duration and resolution. 2. The sample rates and resolutions shown were obtained with a 10-channel scan and with continuous self-calibration disabled. 3. Duration does not include the use of CJC measurements. 		

Channel Connection Layouts

The following figures show channel connection layouts for Personal Daq/55.

Appendix C Results of the Tests Completed at NRC

Table C.1 Summary of the Results for Tests on the 100-mm Cross-Vane Plate on the Pavement Surface

Test No.		Temperature (°C)	Max Torque (Nm)	Max Displacement (Radians)	Shear Index (Nm)	Shear Stress (Mpa)
NRC-1 Steel Roller HL4+agent	Edge	1-1	317.83	0.0771	4857.82	0.94
		1-2	369.37	0.0809	4671.49	1.09
	Across	1-3	345.78	0.0831	5564.42	1.02
		1-4	258.48	0.0670	4169.23	0.76
	Center	1-5	290.71	0.0625	5349.82	0.86
		1-6	275.48	0.0619	4705.94	0.81
		Mean CoV	309.61 13.77	0.0721 13.11	4886.45 10.31	0.91 13.77
NRC-2 Steel Roller HL4	Edge	2-1	329.48	0.0579	6091.81	0.97
		2-2	370.55	0.0541	8177.58	1.09
	Across	2-3	437.74	0.0526	9083.06	1.29
		2-4	441.84	0.0528	9021.50	1.30
	Center	2-5	339.80	0.0702	7199.71	1.00
		2-6	336.76	0.0459	7471.72	0.99
		Mean CoV	376.03 13.66	0.0556 14.64	7840.90 14.72	1.11 13.66
NRC-3 Amir Roller HL3	Edge	3-1	408.65	0.0695	8187.85	1.20
		3-2	491.03	0.0505	12524.84	1.45
	Across	3-3	452.66	0.0509	9217.52	1.33
		3-4	428.52	0.0560	7963.87	1.26
	Center	3-5	428.78	0.0396	11111.70	1.26
		3-6	402.89	0.0559	8942.60	1.19
		Mean CoV	435.42 7.45	0.0537 18.14	9658.06 18.56	1.28 7.45
NRC-4 Steel Roller HL3	Edge	4-1	504.61	0.0578	13480.41	1.49
		4-2	461.24	0.0625	11122.56	1.36
	Across	4-3	417.14	0.0543	10421.89	1.23
		4-4	535.93	—	—	1.58
	Center	4-5	543.99	0.0680	9633.10	1.60
		4-6	534.10	0.0711	10014.48	1.57
		Mean CoV	499.50 10.12	0.0627 11.08	10934.49 13.96	1.47 10.12

Table C.2 Summary of the Results for Partially Cored Samples Tested with the 100-mm Cross-Vane Plate

Test No.		L (mm)	Temperature (°C)	Max Torque (Nm)	Max Displacement (Radians)	Shear Index (Nm)	Shear Modulus (Mpa)
NRC-1 Steel Roller HL4+agent	Edge	50	—	—	—	—	—
	1-1	50	44	50.45	0.0612	1590.8	7.51
	1-2	50	44	59.10	0.0400	2261.7	10.33
	Across	47	44	50.16	0.0442	1618.3	7.59
	1-4	49	45	41.06	0.0602	986.9	4.26
	Center	45	45	47.63	0.0938	1157.4	5.34
NRC-2 Steel Roller HL4	1-6	48	45	—	—	—	—
	Mean		44.60	49.68	0.06	1523.03	7.00
	CoV		1.23	13.05	35.35	32.51	33.43
	Edge	46	45	83.94	0.0561	2125.4	9.23
	2-2	43	45	131.74	0.0449	3398.5	13.97
	Across	46	45	124.57	0.0488	3947.9	17.60
NRC-3 Amir Roller HL3	2-4	50	45	74.83	0.0545	2122.4	10.01
	Center	47	45	91.90	0.0459	2525.7	11.29
	2-6	46	45	83.75	0.0583	1970.4	8.64
	Mean		45.00	98.45	0.05	2681.72	11.79
	CoV		0.00	24.11	10.93	30.15	28.97
	Edge	—	—	—	—	—	—
NRC-4 Steel Roller HL3	3-2	46	45	90.95	0.0378	3513.4	15.45
	Across	38	45	84.82	0.0618	2527.3	9.15
	3-4	—	—	—	—	—	—
	Center	45	45	83.76	0.0481	2494.2	10.63
	3-6	49	45	81.11	0.0648	1636.4	7.63
	Mean		45.00	85.16	0.05	2542.81	10.71
NRC-4 Steel Roller HL3	CoV		0.00	4.89	23.59	30.18	31.61
	Edge	40	45	129.92	0.0512	3618.4	13.43
	4-2	38	46	129.41	0.0767	2355.8	8.56
	Across	39	46	79.66	0.0653	3276.4	11.96
	4-4	40	45	99.83	0.0451	3271.0	12.24
	Center	40	45	88.44	0.0419	2674.7	9.94
	4-6	39	45	117.11	0.0393	5156.2	18.87
	Mean		45.33	107.39	0.05	3392.09	12.50
	CoV		1.14	19.85	27.69	28.82	28.60

Table C.3 Summary of the Results for Partially Cored Samples Tested on the 100-mm Glued Plates

Test No.		L (mm)	Temp (°C)	Max Torque (Nm)	Max Displacement (Radians)	Shear Index (Nm)	Shear Modulus (Mpa)
NRC-1 Steel Roller HL4+agent	Edge	35	41	113.78	0.0411	3558.4	11.72
	1-2	31	41	127.18	0.0486	3479.7	10.42
	Across	32	44	71.81	0.0456	1990.6	6.09
	1-4	32	44	73.20	0.0367	2638.9	8.00
	Center	34	44	68.55	0.0561	1767.6	5.73
	1-6	33	44	86.57	0.0556	2137.3	6.63
	Mean CoV		43.00 3.60	90.18 27.31	0.05 16.39	2595.40 29.71	8.10 30.39
NRC-2 Steel Roller HL4	Edge	31	44	118.15	0.0444	3567.2	10.53
	2-2	32	43	129.87	0.0398	4995.5	15.25
	Across	36	45	140.88	0.0495	3867.6	13.55
	2-4	33	43	129.99	0.0403	4417.6	13.79
	Center	36	43	122.12	0.0623	2753.7	9.32
	2-6	38	42	183.25	0.0473	5071.9	19.19
	Mean CoV		43.33 2.38	137.37 17.32	0.05 17.57	4112.28 21.74	13.61 25.83
NRC-3 Amir Roller HL3	Edge	30	45	202.80	0.0537	5424.5	15.56
	3-2	35	45	119.92	0.0556	7233.5	24.21
	Across	33	45	128.87	0.0474	3925.4	11.88
	3-4	—	—	—	—	—	—
	Center	30	40	183.05	0.0482	5910.6	16.86
	3-6	32	45	176.60	0.0496	6231.4	19.06
	Mean CoV		44.00 5.08	162.25 22.20	0.05 7.03	5745.09 21.13	17.52 26.04
NRC-4 Steel Roller HL3	Edge	38	42	205.67	0.0493	7479.5	26.67
	4-2	38	42	194.32	0.0320	8064.5	29.66
	Across	40	42	138.87	0.0482	4374.0	16.47
	4-4	40	42	180.53	0.0455	6760.0	25.85
	Center	38	44	163.42	0.0424	6326.2	22.64
	4-6	40	42	194.35	0.0528	6836.1	25.81
	Mean CoV		42.33 1.93	179.53 13.72	0.05 16.20	6640.05 19.07	24.52 18.50