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Field and Laboratory Investigation of the Effect of Cold In-Place Recycled Asphalt on Transverse Cracking

by

Katherine A. Lauter, B.Eng.

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

Master of Engineering*

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Ottawa, Ontario, Canada

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Transverse Cracking**

submitted by

Katherine A. Lauter

in partial fulfilment of the requirements
for the degree of Master of Engineering

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Abstract

Thermal cracking is a common pavement distress in Canada and the northern parts of the United States and is responsible for increasing road maintenance and rehabilitation. Cold In-Place Recycling (CIR) is a new rehabilitation method used by the Regional Municipality of Ottawa-Carleton to decrease the rate of occurrence of transverse and reflective thermal cracks. This study involves the collection of field and laboratory data to validate the in-situ performance of the CIR, and to characterize the mechanical properties of the material.

In this thesis, a review of the research carried out on the CIR process is presented. This review identifies a need for material characterization. A research plan is developed which outlines a field data collection and laboratory testing program. Field data was collected using a pavement management system and a Dynaflect Deflectometer. The laboratory tests carried out on the material were conventional test methods including bulk relative density, indirect and direct tensile strength, and moisture susceptibility tests.

The results of the field data collection and laboratory testing program are used to develop a mechanism to explain the ability of the CIR rehabilitated roads to slow down the phenomenon of transverse cracking.

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CHAPTER 1

INTRODUCTION

1.1 General

Canada began its lateral growth in 1867 as Confederation united the provinces of Nova Scotia, New Brunswick, Quebec, and Ontario. It has since matured into a single country which covers 10 million square kilometres of land and freshwater [RTAC, 1990]. Transportation for early settlers was generally by water or footpath and eventually evolved to a larger network of paths for horses and carriages. As early as 1870, when Sir John A. MacDonald risked his political reputation to establish the Canadian Pacific transcontinental railway, the vital importance of transportation to this country has been realized [Palmer, 1988]. This fact was reaffirmed in 1948 when the Federal and Provincial governments embarked on a collective project to build the Trans Canada Highway, which was to be the world's longest paved highway [Department of Public Works, 1972]. By 1994, Canada's road and highway network had expanded to include 901 903 2-lane km of roads, of which 317 920 2-lane km were paved [TAC, 1995]. These lengths are forever increasing to accommodate the travel of the nation's widely dispersed population.

Roads and highways are vital for the economic development of Canada, and therefore constitute a large part of government expenditures. The continued increase in traffic volume has contributed to the deteriorating conditions observed on all paved roads. In 1989, the cost of road maintenance in Canada totalled \$8.4 billion [RTAC, 1990]. One of the major problems of the Canadian road and highway network which exhausts a large sum of the maintenance budget is pavement distress.

1.2 Pavement Distress

Examples of pavement distress that can be seen on the surface of pavement are rutting, thermal cracking, low-temperature cracking, potholes, ravelling, and shoving. Thermal and low-temperature cracking of asphalt pavements are among the most common stress mode, leading to road failures in Canada and in other cold regions. Low-temperature cracks occur when thermal stresses exceed the tensile strength of the asphalt layer, or due to the fatigue caused by daily temperature cycles [Easa et al., 1996]. Throughout this thesis, the term 'thermal cracking' will be used to refer to both thermal and low-temperature cracking.

Thermal cracks develop in the top surface of the asphalt layer and propagate downward, partially or entirely, through the layer thickness. Cracks that develop at the bottom of the asphalt layer are typically the result of high load-induced tensile strains, or a crack or discontinuity that exists at the top of the underlying layer. Those resulting from existing cracks are called reflection cracks and commonly occur on sections of roadways that have been rehabilitated by

placement of asphalt overlays. The propagation of reflection cracks can be accelerated by freezing shrinkage of the underlayer. Cracks will allow water to penetrate the pavement structure, causing larger cracks and structural damage when subjected to freeze-thaw cycles. The consequence is often premature and costly rehabilitation.

1.3 Thermal Crack Reduction

Thermal cracks in asphalt concrete occur due to a reduction in temperature. Micro cracks occur at the top of a pavement layer as the decreasing temperature induces stresses which exceed the tensile strength of the material. Once the crack has been initiated, consistent temperature cycles cause it to propagate through the full depth of the asphalt layer [Khodadadi-Darban, 1993]. There are three main conditions contributing to the initiation of low temperature cracks. These are thermal stresses in the pavement layer in excess of the tensile strength, differential frost heave, and freezing or shrinkage in the underlayers [Haas and Joseph, 1987].

There are several factors which have been recognized to influence thermal cracking. These factors include climate, pavement age, construction processes, asphalt binder properties, asphalt material properties, and structural design properties [Seddik, 1995].

Thermal cracking can be minimized by better understanding these factors. It is essential to understand the mechanical properties of the material components of asphalt to minimize cracking. The thermal behaviour of an asphalt mix is controlled by the mechanical properties of the asphalt cement or binder. In order to prevent or reduce thermal cracking, the binder must not crack under cold temperatures. Generally, a hard asphalt binder is more prone to low

temperature cracking than a soft asphalt binder[McGennis, 1995]. Consequently, penetration and viscosity tests are very important measures in the specification of asphalt cements. Recently, however, the Strategic Highway Research Program (SHRP) in the United States has improved the method of classification by replacing it with performance based specifications. These new specifications are based on tests which assess the performance of the binder in the range of end use temperatures of the asphalt layer. A performance grade asphalt PG 58-34 would have mix properties adequate at a maximum pavement temperature of 58°C, and a minimum pavement temperature of -34 °C.

After specifying the mix design, paving can begin. During the paving process, it is critical to ensure that adequate compaction is achieved. An asphalt layer with too many air voids will be prone to distress. This allows the binder to be excessively oxidized, leaving it susceptible to low temperature damage and therefore thermal cracking.

When paving and compaction are completed, certain preventative measures can be taken to avoid pavement distress. Primarily, load restrictions are placed on roads that are subjected to extreme freeze thaw cycles during these critical months.

1.4 Road and Highway Maintenance

Once pavement distress has occurred, the roadway must eventually be repaired. Maintenance alternatives are largely dependent upon the following factors [MTO, 1990]:

- 1) rate of pavement deterioration,

- 2) types of pavement distress,
- 3) causes of pavement distress,
- 4) previous maintenance actions, and
- 5) practicality and cost-effectiveness.

The first three factors can be determined through pavement evaluations. These evaluations include continuous field observations and measurements of the pavement distress, roughness, profile, texture, and structure. There are different methods of collecting and recording these observations and measurements. They yield quantitative results often expressed in terms of Pavement Quality Index (PQI), Structural Adequacy Index (SAI), Surface Distress Index (SDI), and Ride Comfort Index (RCI) [Lee,1997].

The fourth factor, previous maintenance actions, can be used to determine the impact of specific solutions on different pavement types to eliminate or highlight various rehabilitation alternatives.

The final factor is subjective and depends upon the availability of material resources, location of maintenance, the number of roads that require maintenance, and the existing maintenance budget.

Maintenance is normally divided into three separate levels which indicate the severity of the distress. Minor safety and operational concerns can be repaired quickly and efficiently, often by patching or crack sealing, and are referred to as routine maintenance. Major maintenance activities can include the routing and sealing of cracks, hot-mix patching, and surface sealing. These repairs are necessary to preserve the roadway and increase its service life [MTO, 1990].

The third and most critical level of maintenance is rehabilitation which includes two general alternatives; resurfacing, or partial to full depth removal with resurfacing. The latter alternative and the materials used for rehabilitation will be the focus of this thesis.

1.5 Maintenance in the RMOC

The Regional Municipality of Ottawa-Carleton (RMOC) typifies the cold and variable Canadian climate. Consequently, thermal cracking is a significant pavement distress in this region.

The Regional Government of Ottawa-Carleton is responsible for the maintenance and rehabilitation of a road and street network approximately 3000 lane-km in length. The task of efficiently managing a system of this size is complex and, consequently, the RMOC has implemented a Pavement Management System (PMS) using software developed by the Pavement Management System Group from Cambridge, Ontario. The PMS includes a database which consists of all "road inventory data and pavement evaluation and performance data" [Lee, 1997]. The evaluation and performance data can identify a list of roads that require rehabilitation. Further cost considerations are evaluated before a final list is compiled.

The RMOC employs several rehabilitation strategies. The more common schemes include partial depth removal of the existing asphalt and placement of a hot mix overlay, microsurfacing, hot mix overlay, and full depth removal of existing asphalt with placement of new hot mix layers. The RMOC has observed that rehabilitation strategies carried out on roads that exhibit a high frequency of thermal cracks do not eliminate the problem. Typically, these

rehabilitation methods are susceptible to reflection cracks originating at the underlying thermal cracks in the aged pavement layers. Since 1989, however, the Region has employed a somewhat unconventional strategy for the rehabilitation of roadways with extensive thermal cracking. The new strategy is termed Cold In-Place Recycling (CIR).

1.6 Cold In-Place Recycling (CIR)

Cold In-Place Recycling is a continuous in-place process that involves partially milling the existing deteriorated pavement, crushing the milled Reclaimed Asphalt Pavement (RAP) to a suitable size in a portable crusher screen plant, treating the crushed material with an emulsion, and placing the new material on the roadway with conventional paving and compaction equipment [Miller, 1995 and Bradbury et al., 1991]. The CIR layer typically has an open texture, a poor surface durability, and an unknown moisture susceptibility. The layer must then be overlaid or surface treated.

Visual observations supported by the PMS data suggest that the CIR material with a Performance Graded overlay is a viable and even preferred method of rehabilitation for roads with significant distress related to thermal cracking. Hot mix asphalt (HMA) tends to exhibit mix characteristics, densities, and mechanical behaviour superior to the CIR material when tested in the laboratory. However, the CIR material appears to equal and often improve on the performance of the HMA in the field. The RMOC has also found that the annual cost for the CIR rehabilitation work, including a 40 mm hot mix overlay, is only 55% of the cost of their standard hot mix overlay procedure [Croteau et al., 1997]. Consequently, it is important to

characterize the material produced by the CIR in order to explain the RMOC's observations for further optimization of field rehabilitation materials and processes.

1.7 Scope and Objectives

In this introductory chapter, the importance of a well maintained road and highway system within Canada and the extreme costs associated with this maintenance have been emphasized. A major pavement distress, caused by freeze-thaw cycles and lengthy cold winters, characteristic of the Canadian climate, is the thermal cracking. The maintenance costs for roads and highways could be lowered if the frequency of thermal cracks was reduced. The RMOC exhibits the typical Canadian climate and has been chosen as a study region. The RMOC has implemented CIR as a rehabilitation strategy to minimize thermal cracking on their network of roads.

The overall goal of this research was to establish the physical and mechanical properties of the CIR asphalt to determine its viability as an alternative for flexible pavement rehabilitation.

The more specific objectives of this research can be listed as follows:

- To collect field data to verify the RMOC observations.
- To collect field data and samples from roads during the CIR construction process.
- To quantify and characterize the CIR material. This will be done through indirect tensile strength testing of laboratory and field compacted core samples, direct tensile strength tests of laboratory compacted and field cut slab samples, and moisture susceptibility tests of field core samples.

- To use the results of field data and laboratory testing to identify the conditions under which the CIR mixes give better field performance than conventional mixes. In addition, these results can be used to explain the potential long term performance of the CIR pavement.
- To identify why field data do not support the results of conventional laboratory tests which indicate that the HMA is superior to the CIR asphalt.
- To recognize the advantages and disadvantages of CIR use.

1.8 Organization of the Thesis

A literature review was carried out of the existing research and past uses and performance of the CIR material. The review is presented in Chapter 2, including a detailed explanation of the CIR process, the technology and materials utilized, and detailed background information about the phenomenon of thermal cracking.

Chapter 3 outlines the research plan that will provide reliability to the process of CIR, and will attempt to characterize the material. The field and laboratory tests are detailed, and the parameters and variables involved in both processes are outlined.

The field data collection and analysis are presented in Chapter 4. This includes Pavement Management System (PMS) data which details transverse cracking, and Dynaflect Deflectometer data which yields maximum deflection and radius of curvature values. The collected data is used to characterize the field performance of the Cold In-Place Recycled material.

The results of the laboratory tests outlined in the third chapter are presented in Chapter 5.

The results are presented in tables and figures and are used to characterize the mechanical properties of the CIR material.

Chapter 6, the final chapter, summarizes the main objectives of the research, develops mechanisms to explain the CIR performance, summarizes the field and laboratory results, and provides recommendations for future research of Cold In-Place Recycling as a rehabilitation alternative.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

History has witnessed many technological advances in the field of pavement engineering. There were many changes in paving materials and methods to combat distress problems as the material became better understood through laboratory and field studies. There were also other external factors responsible for the changes and variations in paving technology, including economic and environmental factors.

The current generation places a great emphasis on the concepts of reducing, reusing, and recycling. Since the asphalt industry uses natural resources of aggregate and bitumen, these concepts have become increasingly important. Aggregate was substituted with reclaimed asphalt pavement (RAP), and both hot and cold recycling procedures were implemented. These procedures, along with their advantages and disadvantages, will be discussed later in this chapter.

However, a common concern within the industry is whether or not the quality and long term performance of pavement are being compromised by reusing and recycling the materials.

Specific recycling materials were observed to improve some of the mechanical properties of the conventional mixes. Within the RMOC, observations have been made which support the use of the CIR as a material for road rehabilitation based on its ability to reduce reflection cracking on overlaid road sections.

2.2 Thermal and Reflective Cracking

Low temperature cycles are not necessarily the main cause of reflection cracking, however they definitely serve as a catalyst [Abd El Halim, 1985]. Reflective cracks are a result of pre-existing cracks at the top of the asphalt layer underneath the asphalt overlay. Extreme temperature variations, such as those that occur in freeze-thaw cycles, cause stresses within and between the pavement layers. Pavement failure will first occur at the existing cracks which cause a discontinuity in the material structure. For this reason, the mechanisms governing thermal cracks must be reviewed in order to understand reflective cracking.

Cold temperature, which is a characteristic of Canada, is a major factor contributing to transverse cracking. Observations over the past thirty years have shown that pavements subjected to cold temperature exhibit a high rate of transverse cracking [Shalaby, 1997]. Consequently, thermal cracks appear most often on a roadway as transverse cracks. The literal definition of transverse cracks as stated by the Roads and Transportation Association of Canada [RTAC, 1987] is "cracks which follow a course approximately at right angles to the pavement

centre line." Full transverse cracks tend to be regularly spaced along the length of the road. This uniform cracking pattern suggests that uniform loading conditions, similar to those created by a temperature cycle, cause this cracking [Shields, 1964].

2.3 Factors Affecting Thermal Cracking

In order to minimize the effects of thermal cracking, the material, environmental, and structural factors which contribute to it must be understood [Vinson et al., 1989].

2.3.1 Material Factors

Aggregates and binders are the two materials used to make asphalt concrete. Knowledge of the properties of the individual materials, and the properties of the mixture of both materials is critical to predict the performance of an asphalt concrete mix.

The binder, or asphalt cement, will crack under cold temperatures more readily if it is stiff rather than soft. A high penetration index (PI) indicates a low crack tendency [Fabb, 1974].

Aggregate quality is important in crack resistance. Aggregates with low absorption, high abrasion resistance, and high tensile strength have a greater resistance to cracking [Shalaby, 1997].

The air void content of the mix is generally controlled by the compaction process, the aggregate gradation, and the asphalt cement content. It influences many pavement distress modes, but does not have a major role in cracking due to low temperatures.

2.3.2 Environmental Factors

A pavement system, like any system, has loads placed upon it by the surrounding environment. These loads can influence the system in beneficial or harmful ways and must be considered in the planning process.

Traffic is a major load placed on the pavement structure and has the ability to increase the frequency and rate of propagation of thermal cracks over time [Deme, 1987]. It does not cause thermal cracks, but does accelerate the growth of existing cracks.

Although the load placed on the pavement structure by low temperatures is not nearly as severe as traffic loads, low temperatures create the stresses that cause thermal cracks. Cold temperatures cause high stresses and stiffen the material, making it prone to thermal cracking. The temperature difference throughout the layers will also create tensile stresses. A difference in temperature or gradient between the top and bottom of a pavement layer can cause stresses. The gradient is highest during the transition seasons of spring and fall.

Another external factor affecting the pavement is moisture. Pavement layers subjected to freeze and thaw cycles will experience damage quickly and will exhibit accelerated crack propagation in the presence of moisture [Shalaby, 1997].

The final environmental factor affecting thermal cracks is the age of the pavement. Crack frequency tends to increase with age due to an increase in stiffness of the mix with time [Haas et al., 1972].

2.3.3 Structural Factors

The structural factors that affect thermal cracking are inherent to the design and construction of the roadway.

Pavement cracking can occur during the construction process. Conventional steel rollers can induce hairline cracks on the pavement surface which will eventually propagate through the layer thickness [Halim et al, 1994]. These cracks are caused by damage in the asphalt mix from deformation under the compaction load which results in a decrease of tensile strength, and due to a difference between the stiffness of a conventional roller and the stiffness of the asphalt layer [Khodadadi-Darban, 1993]. Adaptations to conventional compaction equipment and methods will help to decrease cracks that occur in the pavement layer at the time of construction.

Thickness of the pavement layer also can be a factor. Observations of thermal cracking in pavements of different thicknesses show that thicker pavements exhibit less cracking. The thicker pavement provides improved insulating properties which help to lower the temperature gradients [Haas, Hajek, 1972].

In addition to thickness, width is another important dimension in pavement design. A typical road that is 8 m wide will have transverse cracking at 30 m spacings. An airport runway, conversely, that is 16-32 m wide will have a crack spacing of approximately 45 m [Shalaby, 1997]. Therefore, designing wider roads will decrease the frequency of transverse cracking.

The majority of paved roads are designed with a minimum of two pavement layers; a base course and a surface course. Rehabilitation will often add other pavement layers called overlays. A strong and uniform bond between the asphalt layers will help to reduce thermal cracking.

This bond can be achieved by using a tack coat.

Finally, the development of new cracks is somewhat dependent on the cracks that already exist. A transverse pavement crack will create a shorter “slab” that will require a much lower temperature to cause failure than was needed for the initial crack. With this same principle in mind, an existing crack that is sealed with a stiff material will tend to cause high tensile stresses. New cracks may open in the pavement to relieve this stress.

2.4 Prevention of Thermal Cracking

Unfortunately, there is no asphalt mix recipe that guarantees the elimination of thermal cracks. Inevitably, thermal cracks will be present to some extent as long as low temperature conditions exist. Minimizing the known material, environmental, and structural causal factors is perhaps the most effective method of dealing with this distress problem.

Pavement designers and engineers have some control over the structural and material factors, but they have very little control over the environmental factors. Consequently, most research efforts are focused on the structural and material properties. Changing and advancement of technology has allowed the greatest variability to the material factors. Conventional and newer non-conventional paving materials and methods will be discussed relative to their benefits and drawbacks.

2.5 Conventional Paving Methods and Materials

Hot mix asphalt (HMA) is a combination of aggregate uniformly coated with asphalt cement. The asphalt cement must be heated to allow sufficient fluidity to coat the aggregates, and aggregates are heated to remove moisture [Asphalt Institute, 1989]. Asphalt is classified by the gradation of the aggregate and the grade of the asphalt cement. There is a considerable amount of documented information about the laboratory and field characteristics of this material subjected to all types of variables and conditions.

The HMA is prepared at an asphalt mixing facility. Typically the operations at a batch facility include cold aggregate storage, aggregate drying or heating, screening, measuring and mixing the aggregate and asphalt cement, and discharging the mix into trucks or storage areas [Asphalt Institute, 1989]. The mix is brought to the paving site in trucks which deposit the material into pavers. The material must be between 85°C and 150°C in order to be fluid enough to allow proper compaction. The paver spreads the material over the roadway and steel-wheeled, vibratory, or rubber tired rollers are used to compact the mix to the required density. The freshly paved road can support traffic once it has been compacted and has cooled.

This is, and has been, the most common paving process in the world. Unfortunately, as visual observations can easily verify, this process is unable to provide a roadway that can control transverse thermal cracking.

2.6 Recycled Paving Methods and Materials

Conventional paving processes, used on existing paved roadways for resurfacing or reconstruction, can generate a significant amount of waste. If the old asphalt pavement is removed, it must be transported to another site for disposal. This old asphalt will either be stockpiled or landfilled. It is difficult to justify filling precious landfill space with a material that can be potentially reused. The recycling process conserves energy and resources. Most of these processes use little or no virgin aggregate, and a small amount of asphalt cement in comparison to conventional processes. Recycling can eliminate the problem of disposing the old asphalt. Using less virgin aggregate and more old asphalt will save resources, reduce material transportation needs, and require a lower total energy consumption [Emery, 1993].

There are currently six recognised methods of asphalt recycling: cold planing, full depth reclamation, hot mix plant recycling, hot in-place recycling, cold mix plant recycling, and cold in-place recycling.

2.6.1 Cold Planing

This process removes the asphalt pavement to a desired depth and width using a milling machine. The removed material can be stockpiled and used in conventional asphalt paving procedures, or mixed with aggregate and used as a granular base material. This process allows immediate use of the road by traffic. The planed surfacing can be undertaken at a later time.

2.6.2 Full Depth Reclamation

The entire thickness of the existing asphalt pavement and a predetermined portion of the underlying granular materials are uniformly pulverised. An additive may also be blended into this mix. This new material is compacted and represents a stabilised granular base which may increase the load carrying capacity of the pavement. This process has been observed to reduce the potential for reflective cracking in the new overlay material.

2.6.3 Hot Mix Plant Recycling

This is the most common asphalt recycling procedure. The process occurs at the hot mix plant and involves the combination of removed pavement with new aggregate and an asphalt cement. The hot mix is then hauled to a work site for use. Some minor modifications to the existing hot mix plant equipment may be required. Typically, this material is used as a binder layer, or for low volume surface courses.

2.6.4 Hot In-Place Recycling

This process is an attempt to achieve hot recycling without the costs of hauling material between the asphalt plant and the paving site. There are several methods of performing this recycling process but the main concept is as follows:

- Pavement is removed (some processes heat the pavement surface by infrared radiation before removal).

- Removed material is heated and mixed with recycling agents and/or additional aggregates.
- The new mix is applied to the road in single or multiple layers.

This process has favourable economics and has been studied enough that documentation of the asphalt technology is readily available [Emery, 1993]. Heating the materials, however, can cause considerable gaseous emissions.

2.6.5 Cold Mix Plant Recycling

The stockpiles of Reclaimed Asphalt Pavement (RAP) at the plant are mixed with a rejuvenator or emulsion to create a new mix. The RAP can also be sized and screened to achieve a required gradation. The new mix, which is not heated, is then transported to the site to be used. This is not a very common method, however it can be done. Plants require modification to carry out this procedure.

2.6.6 Cold In-Place Recycling

Cold in-place recycling (CIR) is a continuous process which involves the cold milling of existing pavement, sizing this material with a screening and crushing unit, mixing this aggregate with an emulsion, laying the final product back onto the roadway as a base course, and compacting this material with conventional compaction equipment. The CIR is used as a rehabilitation procedure which attempts to restore the homogeneity of the pavement structure. A

hot mix overlay or surface treatment is required to achieve long term performance.

The primary ingredient in the CIR process is the emulsion. An asphalt emulsion is a homogeneous mixture of minuscule asphalt droplets that are suspended in water. Emulsions do not require heat or solvents [PRIME, 1997]. The three categories of emulsions are cationic, anionic, and non-ionic. This refers to the electric charge that surrounds the asphalt particles [McAsphalt, 1997]. Once the emulsion contacts the aggregate, the water in the emulsion begins to evaporate. The rate of evaporation is dependent upon the weather and climate conditions. This “breaking” of the emulsion produces asphalt cement. Emulsions are further classified by their rate of breaking down as rapid setting (RS), medium setting (MS), slow setting (SS), or quick setting (QS). Typically, the CIR uses the MS.

One of the main disadvantages of using the CIR is the lack of available technical data. Many agencies still treat this material like cold patch. At present there are no standard specifications for mix design or quality control. The mechanical and material properties of the CIR asphalt require a definition. It is, however, thought of as a good method of reducing thermal cracking by those who use and/or produce it. [Croteau et al., 1997].

2.7 Cold In-Place Recycling Mix Design

There are no standard mix design specifications for the CIR in Canada. Generally, the laboratory mix design is carried out in four stages [Croteau et al., 1997]: field sampling of existing asphalt, materials testing, additive selection and laboratory mix design. Regardless of the selected mix design, field adjustments are a critical part of the CIR process. On site

modifications are made based on the appearance of the CIR mat after initial compaction.

2.7.1 Field Sampling

The CIR project is divided into homogeneous sections for sampling purposes. Core samples are extracted from the pavement, or a milling machine is used to gather material. Approximately 50 kg of material is required from each test section.

2.7.2 Materials Testing

In the laboratory, a gradation is carried out on the RAP material and then an extraction is performed to separate the asphalt cement from the aggregate. The aggregate can then be tested for gradation and fine aggregate content. The asphalt cement content is quantified and the bitumen can then be characterized through some of the following tests: penetration, absolute viscosity, softening point, and asphaltene content. The asphalt cement content and aggregate gradation will help in determining whether or not the addition of virgin aggregate is required. The characteristics of the binder aid in the selection of an emulsion.

2.7.3 Additive Selection

As stated previously, the addition of virgin aggregate and asphalt cement is not always required. The only material that must be added to the mix is emulsion. The emulsion is added to soften the aged bitumen, to provide a proper aggregate coating, and to allow sufficient cohesion

and adhesion to the mix.

2.7.4 Laboratory Mix Design

The mix design used for the CIR is similar to the Marshall method of mix design for conventional asphalt. Instead of trying to optimize asphalt cement content, however, the emulsion and water content are optimized. Trial mixes are batched with varying emulsion and water contents, and the air void, density, and stability properties are tested. Emulsion content typically varies between 1.5 % and 2.0 %, while water content normally ranges from 3.5 % to 4.5 % [Croteau et al., 1997] by mass.

The water content is the key element to a successful CIR operation. Water is essential to allow the emulsion to uniformly coat the aggregate, however too much water will inhibit this procedure and will also cause problems in the compaction and curing process.

2.8 The Cold In-Place Recycling Pavement Design

The focus of this thesis is the material used in the CIR, however, a brief overview of the pavement design considerations will be discussed, recognizing the inherent importance of this design to the success of the CIR.

The pavement design is based on structural characteristics, pavement profile, existing asphalt thickness, traffic loading, and the selected wearing course [Croteau et al., 1997] .

2.8.1 Structural Design

The structural capacity of hot mix asphalt is typically denoted by a structural coefficient. CIR material, however, does not have a universally accepted coefficient. The capacity is dependent upon the following factors:

- 1) The nature of the in-place bituminous material. The quality of the recycled aggregate and the stripping properties of the mix are also important.
- 2) The emulsion type and quantity.
- 3) The curing time of mix. This property itself is dependent upon the moisture and temperature factors.

These factors should be accounted for in the structural design process to provide adequate, and not excessive pavement thickness specifications.

2.8.2 Pavement Profile

The CIR process is one that can be used to correct problems with pavement profiles. Field data gathered before rehabilitation indicate profile defects that can be fixed by modifying the design. The CIR can accommodate planing the road, adding virgin aggregate, and adding additional surface course material. The CIR can also be used to increase the lane width on roadways.

2.8.3 Asphalt Thickness

The thickness of the in-situ asphalt is a critical factor in the CIR pavement design. This will determine the maximum possible depth of milling, and consequently the thickness of the new CIR asphalt layer. It is critical to mill the asphalt pavement deep enough to mitigate the reflective cracks.

2.8.4 Traffic

Pavement must be designed to accommodate the expected traffic loads. Since, the specific structural capacity of the CIR asphalt is unknown, and its performance is doubted by many agencies, the CIR technology is often used on roads with less than 5000 Average Annual Daily Traffic (AADT). Long term pavement performance and material mechanical properties must be determined in order to design for higher volume roads.

2.8.5 Surface Course

To achieve maximum performance, the CIR asphalt must be surface treated or overlaid. The design of the pavement structure is based upon the selection of the surface course. An overlay can add structural capacity to a roadway, while a surface treatment is beneficial mainly in sealing the pavement against moisture intrusion and adding frictional properties.

2.9 Uses of Cold In-Place Recycling

As mentioned earlier, the RMOC has used the CIR on some of their roads to combat the problem of reflective cracking. They have been utilizing this technology since 1989. The CIR has also been used throughout Canada and the United States as will be outlined in the following sections.

2.9.1 The CIR in Ontario, Canada, 1990

Some US States have used the CIR successfully on low volume roads. Due to ever present environmental, economic, and material constraints, the Ministry of Transportation of Ontario (MTO) was interested in determining if the CIR could be used productively on their roads. In 1989, the MTO chose 15.6 km of Highway 15 for a demonstration rehabilitation project. This two lane highway had moderate traffic volumes and fair to poor pavement condition [Bradbury et al., 1991]. The major distress evident on this highway was extensive transverse cracking, and longitudinal wheel track cracks. In the short term, there were no significant problems during the CIR process. Based on Falling Weight Deflectometer (FWD) results, the strength characteristics of the pavement had improved with time. The immediate post construction ride measurements were better than those obtained after conventional rehabilitation methods and fewer surface defects were evident in the 50 mm thick test section as compared to the 75 and 100 mm sections. The construction costs were also 33% less than that of conventional rehabilitation. The study recommended that appropriate mix design and testing

methods be developed and the depth of the CIR treatment be optimized. It also recommended that criteria be established for the emulsion used in the CIR, a similar project be performed on a higher volume road including monitoring of the long term performance, and the methods and timing of compaction efforts be refined.

Although the project appeared to be successful, the MTO has not since used this technology on any of their roads. Reports of the long term performance of this test section were not available.

2.9.2 Indiana, USA

A CIR feasibility study was performed in Indiana in 1986. A 15.7 km rural road was selected for widening and rehabilitation. One half of the length was rehabilitated using the CIR and the other half was done with conventional resurfacing. A HMA surface overlay was placed over the entire CIR pavement length.

Field testing on the road included dynaflecting and pavement condition surveys, while laboratory testing included core bulk specific gravity, bitumen extraction, recovery and penetration, and kinematic viscosity.

The road was revisited five years later to perform dynaflecting and pavement condition surveys. The results revealed that both the conventional and the CIR sections had similar deflection values and thus were roughly structurally equivalent. The pavement condition survey, however, indicated that the section of the road rehabilitated by the CIR had performed better than the conventional resurfaced section.

The study concluded that the CIR is an alternative for rehabilitating medium-to low-volume asphalt pavements [Badaruddin et al., 1991].

2.9.3 Oregon, USA

The state of Oregon has been using the CIR techniques for asphalt concrete rehabilitation since 1984. In 1986, a joint study was carried out between the Oregon State Highway Division and the Oregon State University to develop an understanding of the performance and economics of this technology [Scholz et al., 1991].

The results of the study recommended use of the CIR on rough and ravelled, or cracked and broken pavements because the CIR improved the ride quality of rough and ravelled roads and effectively rehabilitated the structural integrity of the cracked and broken pavement. The CIR material could also be used as a base course for overlays or as a levelling course. For safety and public inconvenience reasons, it was recommended that the CIR be used on roads with Average Daily Traffic (ADT) less than 5000.

The CIR was not recommended for roads where heavy traffic volumes would cause traffic control problems or on pavements which exhibited stripping or rutting as a distress. In addition, the CIR was not recommended when cold and damp conditions were expected immediately after construction because the emulsion would have difficulty breaking and curing, or in late fall and early winter due to insufficient warm weather necessary for proper curing.

This study collected laboratory and field data over time to evaluate the field performance. Results showed that the resilient modulus, the fatigue resistance, the bulk specific gravity, and

the Marshall stability of the material all increased with time. The life expectancy of the CIR, with a 50 mm HMA overlay, was set at 10 to 15 years, and the CIR life cycle costs were the lowest for any given rehabilitation option [Scholz et al., 1991] .

2.9.4 New Mexico State

The New Mexico State Highway and Transportation Department has been using the CIR techniques for asphalt concrete rehabilitation since 1984. They paved more than 1800 lane miles of roadways with the CIR in the initial year. They spent the next several years documenting the condition of these lengths of the CIR paved roads [O'Leary et al., 1992].

Their study focused specifically on developing a cost-effective laboratory and field mix design procedure. Their research identified the benefits of using the CIR over conventional paving methods to be the reduction of material transportation costs, minimizing environmental pollution, and rehabilitation of pavement distress at minimum costs. They have documented information which describes the CIR roads as having overall excellent rideability, minimal rutting, and a reduction in reflective cracking [O'Leary et al., 1992]

2.9.5 The Regional Municipality of Ottawa-Carleton (RMOC)

The RMOC was the first road agency in Ontario to employ the CIR technology for road rehabilitation. They began to use the CIR in 1989 on many of their rural roads. The RMOC has, through visual observations supported by Pavement Management System (PMS) data, estimated

that the CIR will increase the time needed before rehabilitation from 10 years for conventional hot mix overlays to 14 years for the CIR rehabilitation [Croteau et al., 1997]. In addition, the CIR was found to have economic benefits. In the RMOC, the annual cost for the CIR rehabilitation, which includes a 75 mm lift of the CIR asphalt and a 40 mm hot mix surface course, is approximately 55% of the cost of a standard hot bituminous overlay method of 40 mm hot mix asphalt (HMA) correction course with a 40 mm HMA surface course. [Croteau et al., 1997]

2.10 Summary

This chapter has identified the pavement distress types that are relevant in Canada and the Northern United States. In addition the factors contributing to their presence have been reviewed. Alternative rehabilitation methods for dealing with these pavement distresses have been compared and discussed. Cold In-Place Recycling (CIR) as a method of rehabilitation, the focus of this thesis, has been detailed through a literature review. The conclusions arrived at by those who have studied this process are consistent with the visual observations that have been made in the RMOC. The CIR tends to reduce the frequency of thermal cracks and minimizes environmental impacts. Furthermore, it is structurally equivalent to conventional hot mix overlays (CHOL) and can be economically viable based on life cycle costs.

This thesis will focus on the CIR roads that have been rehabilitated in the RMOC since 1990 and intends to support the visual and PMS observations made by the RMOC. In addition,

the material properties of the CIR asphalt that contribute to the decrease in transverse crack frequency will be determined.

CHAPTER 3

RESEARCH PLAN

3.1 Introduction

The previous sections have identified Cold In-Place Recycling of asphalt concrete pavement as an attractive option for reducing thermal cracking. Unfortunately, there is limited data about the reliability of the CIR and the material has not been characterized for its physical and mechanical properties.

In order to help better understand the field performance of the CIR asphalt, this thesis defines the general characteristics of the CIR material, and provides analytical explanations about its behaviour when subjected to different conditions. The purpose of this chapter is to present the plan which is adopted in this research. The plan, summarized in Figure 3.1, includes the following stages:

- 1) Literature review to determine the present use and performance of Cold In-Place Recycling.

- 2) Collection of field data and observations which include the observed performance of the CIR asphalt in the RMOC since 1989, and deflection data from the CIR roads in this region. This data is analysed to verify the field performance of the CIR material.
- 3) Develop a material sampling plan for a CIR road as it is being paved. Carry out appropriate laboratory tests on the CIR material samples to relate the laboratory performance to the field performance.
- 4) Create a database of all observed performance and laboratory test results to provide information about the material characteristics and the material behaviour under a variety of conditions.

3.2 Literature Review

A literature review was carried out in Chapter 2. The literature review identified transverse thermal cracking as the main pavement distress mode in Canada. The methods of resisting thermal cracks were reviewed, and Cold In-Place Recycling of asphalt pavements was recognized as a successful method. Several cases of the CIR use throughout North America were noted and their conclusions were summarized. Historically, the CIR appeared to reduce the amount and frequency of thermal and reflective cracking. It was noted to have the structural strength of the Conventional Hot Mix Overlay (CHOL), and was generally more cost-effective than conventional rehabilitation methods.

The literature review confirmed a need for the characterization of the physical and mechanical properties of the material used in the CIR process.

Figure 3.1 : Research Plan

3.3 Data Collection and Field Observations

The RMOC has been using the CIR for road rehabilitation since 1989. Presently, there are approximately 190 lane km of the CIR roads within the Region. The RMOC is responsible for the development of new roads and the maintenance of existing roads within its jurisdiction. In order to efficiently manage the responsibility of expending public funds for road maintenance, the RMOC has implemented a pavement management system to collect road inventory and performance data [Lee, 1997]. This system collects raw data from the field and calculates performance indices for specific road sections. In addition, a Dynaflect Deflectometer is used to collect deflection data from the roadway which can be used to determine the Structural Adequacy Index (SAI) of the road. The RMOC has observed that the CIR roads tend to perform well in reality, while theoretically their performance indices do not always indicate this to be so. This thesis uses the RMOC pavement management system database and historical Dynaflect values to quantify the physical observations.

3.3.1 Pavement Management System

The RMOC began implementation of a pavement management system (PMS) in 1987 [Lee, 1997]. This system consists of a pavement management database with both project and network level pavement management analysis applications. The system is able to generate rehabilitation design strategies to address the deficiencies that are present in the road system [Lee, 1997]. The capabilities of the system are not fully realized for this research. Since the CIR rehabilitation is consistent with decreasing transverse and reflective crack production, only the

video component of the PMS data library is used to determine the frequency of transverse cracking on selected roads.

Two groups of roads are selected as study sets. The first group includes all roads that had been rehabilitated with the CIR since 1990, while the second group consists of an equivalent length of roads that had been rehabilitated with the CHOL procedure over the same time period. Tables 3.1a and 3.1b show the roads that comprise each group and detail the significant characteristics of each road. All roads chosen were in rural areas and had similar traffic conditions. Videos showing the distress on these roads in 1994/95 and 1997 are reviewed to determine the transverse crack frequency per lane kilometre for each road. These data are used to determine the frequency of transverse cracks per lane kilometre versus age for both the conventional hot mix overlay (CHOL) and the CIR roads.

3.3.2 Deflection Analysis

Deflection analysis of flexible pavement systems is a non destructive test (NDT) procedure used to gather information relevant to the design process. The RMOC uses a Dynaflect Deflectometer to carry out these tests. This is a common test apparatus for NDT testing which was developed in the U.S. in the early 1970's. The apparatus is trailer mounted and consists of two counter-rotating masses which apply a 4.448 kN peak-to-peak sinusoidal load to the pavement through two polyurethane-coated steel wheels spaced 50.8 cm apart at a frequency of 8 Hz. A static weight of 8.9 kN is also applied to the pavement through these rigid steel

wheels. The pavement deflection is highest at the application of the load, and propagates outward from the source. The deflection is measured by five geophones which are located at the centre of the two wheels and, starting at the loading point, are spaced at equal distances of 30.5 cm apart [Leduc, 1994]. The apparatus is shown in Figure 3.2 and a plan view of the geophones and loading wheels are shown in Figure 3.3.

Deflection data is collected with the Dynaflect Deflectometer on each of the roads in the two study groups. Since the RMOC typically dynaflects only half of their roads each year, deflection values from both 1996 and 1997 are used. The data from each road are analysed in terms of the average maximum deflection and the average radius of curvature. These two parameters are compared between the selected study groups to determine any change in the characteristic relative to rehabilitation method. In addition, the parameters are compared to one another within the study group to determine the existence of any relationships. These parameters are also compared to the PMS results to determine whether or not the deflection values are capable of indicating trends in visual distress.

Deflection data from the Dynaflect Deflectometer is also collected from one newly Cold In-Place Recycled road. This road was rehabilitated in 1996 and is detailed in Section 3.5. Deflection data is obtained from 1995, the year prior to rehabilitation, from 1996 immediately after the CIR, and again in 1996 following the hot mix surface overlay. The same parameters are used in the analysis. In addition, the influence of the CIR material on the maximum deflection and radius of curvature is determined.

Figure 3.2: The Dynaflect Deflectometer used by the RMOC

Figure 3.3: Plan View of Dynaflect Wheel Load and Geophones

Geophone Sensor

A: Spacing between geophones = 30.5 cm

B: Load wheel spacing = 50.8 cm

3.3.2.1 Maximum Deflection Values

The Dynaflect will produce one deflection reading from each of the five geophones at one hundred metre intervals along each test road. Of these five readings, the maximum deflection will be at the geophone which lies directly between the two load applying wheels. Maximum deflection values are direct readings, in mils, at this location. The values can be plotted against distance along the length of the road, or can be averaged for each road to compare between different road sections. The maximum deflection can provide information about the strength of the pavement and sublayers, and also about the stiffness of the material. A high maximum deflection value implies a greater chance of cracking. This value alone, however, is not sufficient to complete the analysis of the pavement layer. The shape of the deflection bowl, which includes the readings from all geophones, must be examined to better understand the material and its properties.

3.3.2.2 Radius of Curvature

The measured deflections from the Dynaflect represent a deflection bowl similar to that which would occur under a wheel load. Analysing the shape of this bowl can provide information about the strength of the pavement and underlying layers. It is important to understand the stress that is occurring in the pavement layer, since it is this stress that will contribute to failure.

The radius of curvature helps in defining the strength of a pavement layer. A surface with a smaller radius of curvature has less stiffness and is therefore in a more stressed condition than a

surface with a larger radius [Chi-Kit Chan, 1987]. Since two roads having the same maximum deflection value may have different radii of curvature and thus different stress conditions, it is imperative to use both parameters to interpret the strength conditions of the pavement.

The radius of curvature (R_c) was calculated based on the concept of finite difference approximation which describes the second derivative of a profile. The second derivative of a profile, by the central difference method, is described as:

$$\frac{d^2 y}{dx^2} = \frac{l}{c^2} (y_{i-1} - 2 y_i + y_{i+1}) \quad (3.1)$$

where: $d^2 y/dx^2$ = Second derivative at the centre point x_0

c = Distance between equally spaced nodes (geophones)

y_i = Displacements at selected nodes (geophones)

i = Node number (where 0 is the origin)

This equation, however, has a low degree of accuracy. Adding the error into this equation, the second difference would be:

$$\frac{d^2 y}{dx^2} = \frac{l}{12 c^2} (-30 y_0 + 32 y_1 - 2 y_2) \quad (3.2)$$

Subsequently, the radius of curvature is calculated as the inverse of curvature or:

$$R_c = \frac{l}{\frac{l}{12 c^2} (-30 D_0 + 32 D_{305} - 2 D_{610})} \quad (3.3)$$

In this case, c is 12 inches or 304.8 mm, and D_0 , D_{305} , and D_{610} are the deflections located at sensors 0, 305, and 610 mm away from the centre of the loading wheels.

3.4 Laboratory Testing and Analysis

Thermal cracking is a phenomenon which occurs in pavements due to stresses induced by low temperatures, and by fatigue caused by daily temperature cycles. The thermal cracking problem in conventional asphalt has been investigated in great detail in the laboratory. A controlled laboratory environment offers several advantages including the opportunity to easily observe and measure the physical and behavioural properties of the material, and it provides control over variables.

At the present time there is very little, if any, technical data relating to the physical and mechanical properties of the CIR asphalt. Furthermore, it is not evident from the available data whether this material can be used for all environmental and loading conditions or if there are certain ranges and limitations for its effective use. Clearly, one of the most important characteristics that needs to be defined is its tensile behaviour under very cold temperatures.

3.5 Material Sampling Procedure

This phase began with the selection of a CIR project that would be occurring within the RMOC in 1996. The regional roads involved in the selected project were Snake Island Road (RR6), a 12.9 km section between RR19 and RR31, and River Road (RR19), a 6.2 km section

between RR6 and the Ottawa-Carleton boundary. There was a 1.3 km section with some structural deficiencies that was to be repaved with conventional asphalt (HL3). Paving began on July 23, 1996 and was completed on September 9, 1996. There was no paving carried out between August 10 and September 3.

Once the location was known, the sampling plan was outlined. This plan is summarized below and in Figure 3.4 and will be detailed in the following sections.

Sampling during the paving operation was in accordance with ASTM D979-89 (Standard Practice for Sampling Bituminous Pavement Mixtures). Samples of the CIR material were taken from the site to produce laboratory compacted samples.

All samples were taken after the emulsion was added to the material. The emulsion was a Medium Setting, Polymer Modified High Float Grade emulsion (HF-150M(P)) from McAsphalt. The contract specifications for the emulsion are given in Appendix A. A medium setting emulsion was chosen due to its ability to be mixed with cold aggregates and not to break down immediately upon contact. This allows the material to be mixed, and to maintain its workability for a short time. High float grade emulsions permit a thicker asphalt film and are good at coating the aggregates and lowering the moisture susceptibility. Their performance is further enhanced by polymer modification which also reduces the temperature susceptibility of the emulsion. These emulsions have higher stiffness during hot weather, and lower stiffness in cold temperatures. The contractor was given permission by the RMOC to substitute this emulsion with CBS-1P, a cationic base stabilization emulsion with a shorter curing time, for their own research purposes. This emulsion was used on the westbound lane of RR6 from RR31 to the hot

Figure 3.4: Sampling Plan and Laboratory Test Program

mix test section.

The sample locations were determined based on the desire to sample at various times throughout the day (the lower morning temperatures would necessitate a higher emulsion content than the afternoon), and the desire to distribute the samples over both road lanes. The sample locations for each road are detailed in Figure 3.5.

Slab samples were made by placing the sampled material into pre-made 406 mm x 406 mm wooden forms. The forms were transported to the laboratory for compaction using a WACKER vibratory plate compacter which had a 600 mm x 530 mm tamping area. This was the only available compaction device, since all rollers were too large for this sample size and would destroy the forms. The dimensions of these forms were approximately 406 mm x 406 mm and the samples were later cut to produce sample sizes of approximately 406 x 75 mm. Similar sized slab samples were also extracted from the field to compare in-situ and laboratory properties.

Core samples were made by compacting the material using both the Gyratory and Marshall compaction methods. The two methods produce standard sample sizes of 150 mm and 100 mm diameter, respectively. Both compaction methods were used to determine which was most representative of in-situ density and mechanical characteristics.

3.5.1 Marshall Compaction

Marshall compaction was performed by mechanical Marshall hammers. The device consisted of two compaction hammers with bevelled, circular tamping faces and sliding weights. The sample material was placed into 10 cm diameter sample molds. When operating, the

Figure 3.5: Sample Locations for RR6 and RR19

(NTS)

hammers would drop into the sample molds to deliver a blow. Two specimens could be compacted at one time. Compaction was performed according to MTO test method LS-261 (Method of Test for Preparation of Marshall Specimens), however, no heat was used in the process. The specified number of blows, standard for the mechanical hammer, is 75 blows per face. This was not modified for the CIR material.

3.5.2 Gyratory Compaction

The Troxler Model 4140 Gyratory Compactor was used to compact the specimens by gyration. The Gyratory apparatus is quite different from the Marshall apparatus. A digital control panel is used to operate the machine. This control panel is also used to set the number of gyrations and sample size. The sample material was placed in a 150 mm diameter steel mold that was placed on the loading table in the machine. The ram head was lowered into the mold to provide a solid base for the loading table during compaction. The Gyratory apparatus is shown in Figure 3.6. The number of gyrations was determined based on the Equivalent Single Axle Loads (ESALS). The Average Annual Daily Traffic (AADT) of these roads was 2500 and for an assumed value of 10% commercial trucks, the following calculation was used:

$$250 \frac{\text{trucks}}{\text{day}} * 1.8 \frac{\text{ESALs}}{\text{truck}} * 365 \frac{\text{days}}{\text{year}} * 20 \text{years} = 3.3 \times 10^6 \text{ ESALs} \quad (3.4)$$

This ESAL value was then used to determine the maximum number of gyrations, $N_{\max}$, which was 152 [Asphalt Institute, 1994]. The table used to determine this value is shown in Figure 3.7.

Figure 3.6: Gyrotory Apparatus

Figure 3.7: Table to Choose Design Number of Gyration

Table 5.1 Superpave Design Gyrotory Compactive Effort												
Design ESALs (millions)	Average Design High Air Temperature											
	<39°C			39 - 40°C			41 - 42°C			43 - 44°C		
	N _{ini}	N _{des}	N _{max}	N _{ini}	N _{des}	N _{max}	N _{ini}	N _{des}	N _{max}	N _{ini}	N _{des}	N _{max}
< 0.3	7	68	104	7	74	114	7	78	121	7	82	127
0.3 - 1	7	76	117	7	83	129	7	88	138	8	93	146
1 - 3	7	86	134	8	95	150	8	100	158	8	105	167
3 - 10	8	96	152	8	106	169	8	113	181	9	119	192
10 - 30	8	109	174	9	121	195	9	128	208	9	135	220
30 - 100	9	126	204	9	139	228	9	146	240	10	153	253
> 100	9	143	233	10	158	262	10	165	275	10	172	288

3.5.3 In-Situ Sampling

After the in-situ material had been compacted, cores were extracted from the designated sampling locations at different time intervals to determine the necessary curing time. These activities are detailed in Table 3.2. The results indicate that the material did not cure at a specified rate and that the rate of curing was dependent on moisture, temperature, and possibly layer thickness.

Cores could only be extracted from four sample locations before the overlay was placed. Coring was carried out using a coring machine with a 150 mm or 175 mm core barrel. Coring with a 100 mm core barrel was attempted but it was too difficult to remove a smaller core intact

from this material. The power for the coring machine was supplied by a Honda Generator. It was necessary to use water in the coring procedure to reduce the friction between the asphalt and the barrel. Tests to determine the density, indirect tensile strength (ITS), and susceptibility to stripping or moisture damage were performed on the field core specimens.

In-situ slab samples were also removed from some of the sample locations. The successful and failed attempts at slab removal are also detailed in Table 3.2. Three 200 x 400 mm slabs were dry cut from each sample location on the road using a Quick-Cut saw with a 14" diameter asphalt blade. The length of the slabs were taken in the direction of traffic. The slabs were removed from the road using metal bars and were placed into wooden forms to avoid cracking resulting from bending stresses. These slabs were approximately 200 x 400 mm and varied in thickness. Direct tensile strength laboratory tests were carried out on these samples.

Table 3.2 Field Extraction Activities

Sample	Location	Date Placed	Date of Sample Extraction	Type of Sample Extraction	Result
1	RR19, Sta 0+820 NBL	23/07/96	24/07/96	Core	Could not extract.
			30/07/96	Core	Could not extract.
1	RR19, Sta 0+820 NBL	23/07/96	07/08/96	Core	Could not extract.
			21/08/96	Core	Two cores extracted. Seemed to be fully cured
			30/08/96	Core	Tried two cores, both broke in core barrel. Cores were 150mm thick, only 125mm was cured
2	RR19, Sta 4+000, NBL	24/07/96	25/07/96	Core	Could not extract.

Sample	Location	Date Placed	Date of Sample Extraction	Type of Sample Extraction	Result
			30/07/96	Core	Could not extract.
			21/08/96	Core	Two cores extracted
3	RR19, Sta 4+900 SBL	31/07/96	21/08/96	Core	Could not extract.
			30/08/96	Core	Could not extract. Coring machine sunk into asphalt.
4	RR19, Sta 2+500 SBL	01/08/96	30/08/96	Core	Core broke in barrel. Core thickness 125 mm. Only 50 mm cured.
5	RR6, Sta 7+600 EBL	02/08/96	29/08/96	Core	Cored between wheel paths. Tried three cores, could extract 70mm of 100mm
			02/09/96	Core	Extracted 26 cores for testing. Height: 85 to 95 mm.
6	RR6, Sta 12+650 WBL	07/08/96	29/08/96	Core	Extracted 26 cores for testing. Height: 70 to 75mm.
			02/10/96	Slab	Extracted 3 slabs for testing. Height: 50-55 mm.
7	RR6, Sta 9+600 WBL	08/08/96	29/08/96	Core	Extracted 26 cores for testing. Height: 80mm.
			02/10/96	Slab	Extracted 3 slabs for testing. Height: 55mm.
8	RR6, Sta 3+500 EBL	05/09/96	25/09/96	Core	Extracted 26 cores for testing. Height: 100mm.
8	RR6, Sta 3+500 EBL	05/09/96	02/10/96	Slab	Could not extract without damaging. Too thick.
9	RR6 @ 5th line road, WBL	06/09/96	19/09/96	Core	CBS-IP emulsion. Could not extract.
9			02/10/96	Core	Could not extract. Only 60mm cured.
			02/10/96	Slab	Could not extract.
10	RR6, Sta 2+000, WBL	09/09/96	25/09/96	Core	Could not extract. Only 25mm

Sample	Location	Date Placed	Date of Sample Extraction	Type of Sample Extraction	Result
					cured both in and out of wheelpath.
			02/10/96	Core	Could not extract.
			02/10/96	Slab	Could not extract.

3.6 Laboratory Testing

The tests that were used in this research included indirect tensile strength (ITS), direct tensile strength (DTS), and moisture susceptibility. In addition, the core and cylindrical samples were measured for bulk relative density (BRD). Maximum relative density (MRD) of some samples was measured to provide an average value of the theoretical air void content. The test facility at the Civil and Environmental Engineering Laboratory of Carleton University had to accommodate direct and indirect tensile strength testing, and moisture susceptibility testing. The RMO Materials Testing Laboratory was used for all other tests.

3.6.1 Bulk Relative Density (BRD)

The equipment used to test the BRD's of the specimens were a temperature monitored water bath, a wire basket used to submerge the samples, and Mettler PE3600 and PM 30 scales with 3600g and 30kg capacities respectively.

3.6.1.1 Bulk Relative Density Test Procedure

The dimensions of all core and cylindrical specimens were recorded prior to any testing. In addition, all core specimens were tested in accordance with the MTO test method LS-262 (Method of Test for the Bulk Relative Density of Compacted Bituminous Mixtures) before any further testing was carried out. The BRD is defined as the ratio of the mass in grams of the specimen in air to the mass in grams of an equal volume of water, at a specified temperature. To find the BRD, the sample was left to dry at room temperature for 24 hours, and its dry weight was recorded as A_1 . The sample was then placed in a wire basket attached to a scale. The basket was lowered into a water bath at 23 °C and remained there until the scale reading had stabilized and the mass was recorded as B_1 . The sample was removed and the surface dried off with a towel. Its mass in air was recorded as A_2 . The BRD was then calculated by the following equation:

$$BRD = \frac{A_1}{A_2 - B_1} \quad (3.5)$$

where:

A_1	=	air dried mass of specimen (g).
A_2	=	saturated surface dry (SSD) mass of specimen (g).
B_1	=	mass of specimen in water (g).
BRD	=	Bulk Relative Density (g/cm^3).

3.6.2 Maximum Relative Density (MRD)

The main equipment used to carry out MRD testing included stainless steel beakers with 4 litre capacity, a Mettler PE3600 electronic balance, wire baskets, and a vacuum apparatus capable of maintaining a constant pressure of 30 ± 1 mm Hg.

3.6.2.1 Maximum relative density (MRD) Test Procedure

Two field cores were tested for the MRD. Testing was performed in accordance with the MTO Test Method LS 264. A small number of samples were used since the MRD values are based on material properties and not field compaction, and results do not vary within a material group. To determine the MRD, a specified amount of material was heated in a metal pan for four hours. The mixture was placed in the stainless steel beakers attached to the vacuum apparatus to remove all the air and the material was weighed at various stages of this procedure. The MRD was calculated using the following equation:

$$MRD = \frac{(D - E)}{(D - E) - (G - H)} \quad (3.6)$$

where	D	=	mass of beaker and mixture in air, g.
	E	=	mass of beaker in air, g.
	G	=	mass of beaker and mixture in water at test temperature, g.
	H	=	mass of beaker in water at test temperature, g.

The MRD values were used to calculate air void contents for the field cores, and Marshall and Gyratory samples. The formula used to calculate air voids (AV) was:

$$AV = \frac{(MRD - BRD)}{MRD} * 100 \quad (3.7)$$

3.6.3 Indirect Tensile Strength Testing (ITS)

The ITS test can be used to evaluate the material properties related to thermal cracking, and is a valid test for recycled asphalt mixtures [Kennedy, 1978]. This simple test provides information about the mechanical properties of a mix under vertical traffic loads and is often used for testing conventional HMA. The static test gives information about tensile strength. Strength is a characteristic of asphalt that is directly related to thermal cracking. Normal asphalt generally has a tensile strength between 0.6 and 1.4 MPa [Kennedy, 1978]. The parameters used in the laboratory for ITS testing on the CIR material are explained below.

3.6.3.1 Physical Variables

The physical variables imposed on the core and cylindrical samples were based on compaction mode. In-situ field cores were extracted from the road to provide a field reference for laboratory testing. These cores represented field compaction and also underwent the curing

process, associated with strength gain, on site. The core diameters ranged from 145 to 175 mm.

In the laboratory, compaction was performed by the Marshall and Gyratory compaction methods. Densities of all core and cylindrical samples were measured to compare the compaction of the laboratory samples to field samples. In addition, indirect tensile strength results were also compared between sample types.

3.6.3.2 Temperature Variables

In order to establish the effects of temperature on tensile strength, it is recommended that at least three test temperatures be used [Kennedy, 1978]. The temperatures chosen were -20°C , 0°C , and $+20^{\circ}\text{C}$. Two temperatures were chosen in the freezing range since the distress mode of interest was thermal cracking, and the temperature in Ottawa during the winter is generally within this range.

3.6.3.3 Load Rate Variables

Loading rate is a variable that is affected largely by temperature. A constant loading rate of 12.7 mm/min was used for all core samples tested at $+20^{\circ}\text{C}$. Slower loading rates, however, are generally used as the temperature becomes colder since the material behaves more elastically.

In addition, thermal cracks occur at the lower temperatures and are associated with slow loads and deformations. A constant loading rate of 2.54 mm/min was used for core samples tested at 0°C and -20°C .

3.6.3.4 Indirect Tensile Strength Apparatus

The apparatus consisted of a MTS Instron 2000 testing machine equipped with circular compression heads and linked with a data acquisition system. The test setup also included an insulated chamber in which samples could be tested under controlled temperature conditions. The temperatures were controlled by an environmental chamber with the capacity to cover a temperature range between -30 to +60°C.

3.6.3.5 Indirect Tensile Strength Test Procedure

This test involved loading a cylindrical specimen with a single compressive load acting parallel to and along the vertical diametral plane. The load was applied by a 2.54 cm wide bevelled stainless steel loading strip which allowed uniform tensile stress to develop perpendicular to the direction of the load. The loading strip also controlled the location of specimen failure. Under these conditions, the specimen failed by splitting along the vertical diametral plane [Kennedy, 1978]. The load configuration and specimen failure are shown in Figure 3.9.

The specimen loading plates were surrounded by an insulated chamber. The chamber was attached to the temperature controlled room by insulated tubing, and thus the ITS test temperatures are controlled by the main room. The MTS Instron 2000 test apparatus applied a compressive load to the specimen at a constant rate. The rate used was dependent upon temperature. The tests were performed at -20°C, 0°C, and +20°C. The rate of loading for the tests at the highest temperature was 12.7 mm / min. Since deformations associated with thermal

Figure 3.8: Load Configuration and failure mode for the ITS test

From: Kennedy, 'Characterization of Asphalt Pavement Materials Using the Indirect Tensile Test', Journal of the Association of Asphalt Paving Technologists, 1978.

cracking develop quickly under cold temperatures, a slower loading rate of 2.54 mm/min was applied to the two lower temperatures.

The data acquisition system collected time, displacement, and loading values. The ITS at failure was calculated as a function of the peak load and the cylindrical cross-sectional area. The following equation was used [Kennedy, 1978]:

$$S_T = 2 \frac{P_{ult}}{\pi t D} \quad (3.8)$$

where: S_T = maximum tensile strength, MPa.

P_{ult} = ultimate failure load, N.

t = thickness of core specimen, mm.

D = diameter of core specimen, mm.

3.6.4 Direct Tensile Strength Testing (DTS)

This test is useful for determining the tensile strength, stiffness, and limiting strain of a material. The use of a restrained test specimen to determine thermal stress related to cracking is endorsed by the Strategic Highway Research Program (SHRP) [Kanerva et al., 1993]. This test allows for simulating field conditions by applying tensile loads to slabs in the direction of compaction. Testing in the laboratory allows for different variables to be considered in this specific test.

3.6.4.1 Physical Variables

The DTS test was performed on slab specimens. The field slabs were trimmed in the laboratory to make 100 mm x 380 mm samples which varied in thickness according to that of the asphalt layer at the sample location. Laboratory compacted slab samples were made by compacting loose material, sampled at the time of construction, in wooden forms. These samples were trimmed in the laboratory to 75 mm x 400 mm with a thickness of approximately 75 mm. Tensile strength measurements of the two sample types were compared.

3.6.4.2 Temperature Variables

The tensile strength tests were performed at different temperatures to simulate field conditions and to evaluate the effect of temperature on the tensile strength. Laboratory and field compacted slabs were tested at -30°C , 0°C , and $+40^{\circ}\text{C}$. The low temperatures were used to simulate the temperatures under which thermal cracking would occur.

3.6.4.3 Load Rate Variables

A constant loading rate of 0.005 mm/s was used to test all samples at the various temperatures. This relatively slow rate was applied to enable the test to simulate the colder temperatures.

3.6.4.4 Direct Tension Test Apparatus

This test was performed on the laboratory and in-situ compacted slab samples. The apparatus consisted of [Shalaby, 1997]:

1. An environmental chamber with a temperature range of -30°C to $+40^{\circ}\text{C}$.
2. A hydraulic actuator with 100 kN capacity and 250 mm stroke.
3. A direct tensile strength testing table equipped with fixed and moving mounting plates.
4. The moving plate is connected to a load cell and a positioning system consisting of a linear variable displacement transducer (LVDT) that communicates with a data acquisition computer.

5. The moving plate is placed on a Teflon sheet to reduce friction forces, and the load cell is adjusted accordingly to eliminate the loads required to move the plate and resist friction.
6. A computer, data acquisition system, and software programs for monitoring and control of the test equipment.

3.6.4.5 Direct Tensile Strength Test Procedure

This test is performed on field extracted or laboratory prepared slab samples. The samples were individually glued to two steel plates. The plates were then bolted onto the tension table that was set up in the temperature controlled chamber. One of the plates was bolted to a moving section of the table, while the other plate was restrained. A uniform displacement rate was applied to the base of the moving plate at one end of the sample. The displacement was controlled with an LVDT while a load cell was used to measure the tensile load on the sample. Both displacement and load were recorded by the data acquisition system.

The purpose of this test was to measure the direct tensile strength of the material at different temperatures. The tests were performed at -30°C, 0°C, and +40°C with a loading rate of 0.005 mm/s . All specimens were conditioned at the desired test temperature for 24 hours. The DTS at failure was calculated as a function of the load and the dimensions of the rectangular cross section using the following stress equation:

$$\sigma_{DT} = \frac{P}{A} \quad (3.9)$$

where: σ_{DT} = maximum direct tensile stress, MPa.
 P = peak applied load at specimen failure, N.
 A = average cross-sectional area of the specimen, mm^2 .

The strain at failure was also calculated based on the portion of the sample that was subjected to load. Thus the total length of sample used in the strain equation was that portion of the sample that was not glued to the steel plates. Typically, the tested length of the sample ranged from a maximum of 75mm to a minimum of 25 mm. To determine the maximum possible strain, the value of 25 mm was used in the following strain equation:

$$\epsilon = \frac{\Delta L}{L_o} \quad (3.10)$$

where: ϵ = strain
 ΔL = change in length (Displacement at peak, L_o)
 L_o = initial length (25 mm)

3.6.5 Moisture Susceptibility Testing

The purpose of this test was to assess the effect of water on the tensile strength of the CIR material. To assess the effect of moisture on the in-situ compacted material, the field cores were the only samples tested. Each sample set contained three dry samples and three conditioned

samples. All samples were tested for ITS at room temperature.

3.6.5.1 Moisture Susceptibility Apparatus

The setup for the moisture susceptibility test included the following components:

1. A vacuum saturation apparatus which included a vacuum pump, two glass desiccators with a vacuum gauge, and desiccant. Rubber hoses with shut-off valves were used to attach the pump to the desiccators. The vacuum saturation apparatus was used with desiccant to dry samples, while the apparatus used water to saturate samples.
2. A temperature controlled water bath was used to saturate the specimens at +40 °C for 72 hours before testing.
3. The Instron was used to test the indirect tensile strength of the control and conditioned specimens.

3.6.5.2 Moisture Susceptibility Test Procedure

The purpose of this test was to assess the effect of water on the tensile strength of the CIR material. The testing for in-situ cores uses the Transport Canada Method and is given in Appendix B.

The in-situ core specimens taken from four different locations were used in the stripping tests. Six cores from each sample location were used. Three cores were used as conditioned samples, and the other three cores were the controlled samples. The conditioned samples were

vacuum saturated with water in the desiccators by applying pressure from a vacuum pump to the samples for a duration of 30 minutes. Then the vacuum pressure was taken off the samples and the cores remained submerged under ambient atmospheric pressure for another 30 minutes, after which they were taken out of the desiccators and placed in a water bath at 40 °C for 72 hours. Following the conditioning period, the three cores were tested for ITS.

The controlled samples were placed with desiccant in the vacuum saturation apparatus for 24 hours before testing for ITS to remove any moisture from the samples.

When the maximum load values were reached, the retained stability of the samples were calculated as follows [ASTM D 4867-88]:

$$TSR = \left(\frac{S_{tm}}{S_{td}} \right) 100 \quad (3.11)$$

where: TSR = tensile strength ratio, %.

S_{tm} = tensile strength of conditioned samples, MPa.

S_{td} = tensile strength of controlled samples, MPa.

3.7 Test Procedure Summary

A summary of all tests carried out is given in Table 3.3. Laboratory and field cut samples were tested for moisture sensitivity, and direct and indirect tensile strengths over a range of temperatures. Laboratory samples were both Marshall and Gyratory compacted. Generally, for each test parameter, a minimum of six samples were tested. More laboratory samples were tested

at room temperature to identify the effects of age on material strength. Six field cores from each of the four sample locations were tested for the ITS. Another six samples from each location were used for moisture sensitivity tests. The number of samples for the DTS tests was less due to the difficulty in extracting field slabs.

Table 3.3: Summary of Laboratory Tests

Test	Temperature (°C)	Sample Type	No. of samples tested	No. of Tests per temperature
ITS	+20	Field Core	24	
		Gyratory	26	
		Marshall	24	74
	0	Field Core	24	
		Gyratory	12	
		Marshall	10	46
	-20	Field Core	24	
		Gyratory	6	
		Marshall	6	36
DTS	+40	Field Cut	2	
		Lab Compacted	4	
		Hot Mix	2	8
	0	Field Cut	2	
		Lab Compacted	8	
		Hot Mix	2	12
	-30	Field Cut	2	
		Lab Compacted	14	
		Hot Mix	3	19

Test	Temperature (°C)	Sample Type	No. of samples tested	No. of Tests per temperature
Moisture Sensitivity	--	Field Core	24	24

3.8 Summary

The main purpose of this chapter was to clearly identify the data that would be collected to assess the field performance of the CIR material, and to identify the laboratory evaluation methods that would provide the basis for characterizing the CIR material. The laboratory tests used were relevant methods for interpreting tensile strength of conventional HMA. An analytical investigation of the laboratory results under different controlled conditions will establish the relevance of these tests for CIR material and may define its material and mechanical properties. These results were used as a basis for comparing the performance of the CIR in the laboratory and the field.

CHAPTER 4

FIELD DATA

4.1 Introduction

This chapter presents the field data collected in the Regional Municipality of Ottawa-Carleton to verify that Cold In-Place Recycling (CIR) minimizes thermal cracking and is a viable rehabilitation strategy. The research plan in the previous chapter identifies the field data to be collected as surface condition survey and structural performance data. The surface condition data is collected using a pavement management system (PMS) to quantify the transverse cracks on the selected CIR and conventional hot mix asphalt overlay (CHOL) roadways. Throughout this chapter, all roads referred to as “CIR roads” can be assumed to have a hot mix surface course. Dynaflect data is collected annually for all roads by the RMOC. The Dynaflect data is collected to compare the maximum deflection and radius of curvature values for the CIR roads for comparison with the CHOL roads. This will determine if the Dynaflect can be used to evaluate

the CIR roads in the same way that it is used to determine the structural performance of the CHOL roads. Field data was collected for the CIR and CHOL sample groups presented in Section 3.3.1, Regional Road 6 (RR6) which was rehabilitated by the CIR in 1996, and a CHOL road rehabilitated in 1995. Also, the effect of the subgrade type on both pavement groups is examined.

4.2 Pavement Management System

The pavement condition data for the RMOC is collected using an automated multi-purpose data collection vehicle which uses high resolution video equipment to collect images of the road right-of-way and pavement surface [Lee et al., 1997]. This data is catalogued on video tapes which comprise the PMS video database.

For the purposes of this research, the video tapes were used to count the number of transverse cracks on roads in each group. These values were then converted to the number of transverse cracks per lane kilometre. Linear regression analysis was performed on the yearly average data points. The pavement age of the roads in each group was taken to be the number of years having passed since the last rehabilitation occurred. Videos from 1994, 1995, and 1997 were used in order to provide a comparison between the roads over a two to three year period. The database from 1994 contains only half of the Regional Roads, while the other half were surveyed in 1995. The 1997 video library contains all roads within the RMOC.

4.2.1 Cold In-Place Recycled Roads with a Hot Mix Asphalt Overlay

Group 1, as detailed in Chapter 3, contains roads that have been rehabilitated by the CIR and then overlaid with a layer of conventional hot mix asphalt. The ages of the roads in this group range from 1 to 7 years. Table 4.1 presents the results of the 1997 transverse crack survey for roads in Group 1. It details the number of transverse cracks per lane kilometre (TC/ln-km) of individual roads and gives an average value for all roads rehabilitated within the same year. Figure 4.1 presents the relationship between transverse crack frequency and pavement age. Linear regression was used to determine the relationship between the two variables. A maximum of five types of line fits were attempted including linear, power, exponential, and polynomials of the second and third degree. The best line fit was chosen by selecting the line type that generated the highest Coefficient of Determination (R^2) value. The R^2 values and equations for Figure 4.1 are detailed in Appendix C. A third degree polynomial best fit this data set.

Figure 4.1 show that the number of TC/ln-km increases with increasing age. It is evident that cracking on these roads did not begin until the second year following construction after which cracking propagated at a relatively high rate during the third and fourth year and appears to have levelled off thereafter.

Table 4.2 presents the results of the 1994/1995 transverse crack survey for the same roads. The average number of TC/ln-km for roads rehabilitated in the same year is plotted against age in Figure 4.2. For example, the roads used to create the seventh year average data for Figure 4.1 would be the same roads comprising the fourth year data set in Figure 4.2. Linear regression analysis was carried out on this data and a third degree polynomial was inserted on

this graph. The R^2 values and equations for Figure 4.2 are detailed in Appendix C. The graph shows that the number of TC/ln-km increases as the pavement ages, supporting the results found in 1997. Although this graph shows the average number of TC/ln-km in the first year to be approximately eight, the insignificant number of cracks in year two suggests that cracking does not occur until after the second year. Between the third and fourth year a levelling off is observed. This occurs one year sooner than the 1997 data shows.

Figure 4.3 compares the change in the number of TC/ln-km for the individual CIR roads across the two or three year survey time periods. As anticipated, the number of TC/ln-km increases for each individual road section between 1994/95 and 1997. In addition, this figure shows that there is a significant increase in cracking for all roads sometime before the third year. The cracking frequency remains constant between the third and fifth year, and an increase in cracking occurs again between the fifth and seventh year. This observation is consistent with the trend observed in Figures 4.1 and 4.2, however because the data in Figure 4.3 spans a 2 to 3 year period, it does not pinpoint exactly when the cracking occurs.

4.2.2 Conventional Hot Mix Asphalt Overlay Roads

Group 2, as detailed in Chapter 3, contains roads that have been rehabilitated by applying a hot mix asphalt overlay directly over the existing surface course. The ages of the roads in this group range from 2 to 7 years. Table 4.3 presents the results of the 1997 transverse crack survey for roads in Group 2. It details the average number of transverse cracks per lane kilometre

(TC/ln-km) for individual roads and calculates an average value for all roads rehabilitated within the same year. Figure 4.4 presents the relationship between transverse crack frequency and pavement age. Linear regression analysis was carried out on this data to determine the relationship between the frequency and age. The best line fit with the highest R^2 value was a third degree polynomial and the representative equation is shown in Appendix C. The graph shows a general increase in the TC/ln-km as the asphalt ages. The RMOC ceased using this rehabilitation method in favour of the CIR on rural roads in 1995, thus only two to seven year data for this group were available. Linear regression of the data shows a high crack frequency in year two and a significantly lower value in years three to five. The year two data appears high in the case of the two surveyed roads and is not representative of the road network. Cracking increases from the third to fourth year, followed by a stabilization between the fourth and fifth year. A significant increase in the number of cracks occurs by the sixth year and can be attributed to age hardening of the asphalt binder.

Table 4.4 presents the 1994/1995 transverse crack survey results for the same roads. The average number of TC/ln-km for roads that are the same age at the time of the survey is plotted against age in Figure 4.5. Linear regression analysis has produced a third degree polynomial as the line of best fit. Additional regression results are given in Appendix C. Figure 4.5 shows that the number of TC/ln-km increases with age, similar to the 1997 CHOL data and the CIR data, however at a rate defined by the above equation. This graph indicates that CHOL roads experience significant transverse cracking after one year. Between first and fourth year, cracking continues to increase, but at a decreased rate. As the roads approach the fifth year after

rehabilitation, cracking again propagates at a high rate.

Figure 4.6 compares the change in the number of TC/ln-km for individual CHOL roads across the two or three year survey time periods. Similar to the CIR data, the number of TC/ln-km increased for the individual road sections between 1994/95 and 1997. In addition, this figure shows a significant increase in cracking during the first three years, a constant crack frequency between years three and five, and an increase in cracking again between the fifth and seventh year. From the figure, a small increase in cracking for the 1992 road from the second to fifth year is given compared to the large increase in the 1991 road from year 0 to 3. This suggests that most of the initial cracks originated in the first two years.

4.2.3 Comparison Between the CIR and CHOL Rehabilitation Methods

Figures 4.7 and 4.8 are histograms showing a comparison between the number of TC/ln-km versus age for the CIR and the CHOL roads in 1997 and 1994/95 respectively. They show that at any given age, a road that was rehabilitated by a CHOL method would have a greater frequency of transverse cracking than a road rehabilitated with the CIR.

Relationships for TC/ln-km frequency with respect to age can be compared for both rehabilitation methods by looking at the differences between Figures 4.1, 4.2, 4.4, and 4.5. For the CIR roads, cracking did not begin until somewhere between the first and second year post rehabilitation. Cracking at this time was at a rate of approximately 10 cracks per year. This rate continued until the end of the fourth year. The CHOL, conversely began to crack even in the first

year after rehabilitation. In the first year, the CHOL roads experienced up to 20 transverse cracks. After the first year, however, the rate of cracking decreased to approximately 5 cracks per year for the next 3 years. Both types of rehabilitation methods produced a similar amount of transverse cracks after four years, with a difference in the rate of occurrence. For example CHOL roads had 30 cracks by the second year, while CIR roads did not reach this amount until the fourth year.

In the case of the CHOL roads, a levelling off of cracking occurred between the fourth and fifth year followed by a rapid increase between the fifth and sixth year. The CIR roads, on the other hand, experienced a levelling off between the fourth and seventh year. Since the CIR roads have only been constructed for 7 years, it is not known if and when a rapid increase in transverse cracks will again occur.

Based on the field data that has been collected to date, it is evident that the CIR and CHOL rehabilitation methods produce pavement structures that perform quite different. The structural performance during years 0 to 4 are not the same, however, yield similar results. The significant difference occurs after 5 years at which time CHOL experiences significant cracking and CIR does not. It is this performance by CIR that increases its economic viability above and beyond conventional rehabilitation methods.

4.2.4 Evaluation of Effect of Rehabilitation Method on Transverse Cracking

In order to demonstrate the better performance of the CIR rehabilitation method, two

roads were chosen and their transverse crack density was measured before and after rehabilitation. One road was repaired with a CHOL, while the other was rehabilitated with the CIR and a HMA overlay. The roads were selected from among roads that were rehabilitated in 1995, since the earliest available PMS transverse crack data was from 1994. Figure 4.9 summarizes the results of this investigation. Both roads have silty sand subgrades and exhibit identical transverse crack densities in the year before rehabilitation. Two years after rehabilitation, however, the CHOL road shows as much as 40% of the amount of cracks as in 1994, while the CIR road shows less 2% of the same amount of cracks. This suggests that compared to CHOL, the CIR asphalt is less susceptible to reflective cracking, and also may have developed fewer construction cracks produced by rollers during compaction as will be discussed later.

4.3 Dynaflect Deflectometer

As described in the previous chapter, the Dynaflect Deflectometer is used to collect data to determine the structural condition of the in-place pavement. This data is used to calculate the average maximum deflection, and the average radius of curvature. Comparing the calculated values for the CIR roads to those for CHOL roads will determine if the Dynaflect can be used to evaluate the CIR roads in the same way that it is used to characterize the CHOL roads

The collection of deflection data can be divided into three relevant and separate studies. The first study involved the collection of Dynaflect data for each road in Group 1 which contains the CIR rehabilitated roads and Group 2 which includes roads rehabilitated by the CHOL

method. The second study involved the collection of Dynaflect data from a road within the RMOC that was rehabilitated by placement of a CHOL in 1995. Data for this road was collected before rehabilitation, as well as immediately after the overlay is placed. This was to provide a reference for the CIR rehabilitation of the road in the third study. Deflections on this road were taken in 1995 prior to the CIR rehabilitation, in 1996 immediately after placement of the CIR base course, and again in 1996 after the final hot mix surface course was laid.

4.3.1 Dynaflect Results for Groups 1 and 2

Tables 4.5 and 4.6 present the average maximum deflection values and average radius of curvature for roads in Groups 1 and 2 respectively. The values represent field measurements that were carried out in the survey year of either 1996 or 1997.

In an attempt to establish performance predictions in terms of these two variables, they are plotted for each rehabilitation type on graphs against the age of the roads since rehabilitation. Figure 4.10 shows the results of plotting the average radius of curvature (R_c) for roads rehabilitated with the CIR. It indicates a general increase in the R_c with age. Therefore, the strength of the pavement may be improving with time.

Figure 4.11 presents the same variables plotted for the CHOL roads. The trend is similar to that of the CIR roads, however, at any given age, the R_c for the CHOL roads is higher. This suggests that, when both pavements experience similar loading conditions, the CHOL pavement structure is stronger. The oldest CHOL roads, however, experienced a significant drop in R_c which can be attributed to increasing stiffness and decreasing strength of the asphalt due to age

hardening. As stated in Chapter 3, the R_c cannot be analyzed by itself in order to gain a clear understanding of the field performance. The maximum deflection coupled with the R_c provide a more complete explanation.

The average maximum deflection value is also graphed against age. Figure 4.12 shows the average maximum deflection plotted for the CIR roads. The graph indicates a slight increase in maximum deflection with age and an eventual decrease in the fifth and sixth year. Figure 4.13 presents the maximum deflection data for the CHOL roads. There is an increase in the maximum deflection with age. For any given age, the maximum deflection of the CIR roads is higher than the CHOL roads. At the seventh year, however, there is a very large maximum deflection value for the CHOL roads. This suggests a decrease in the structural strength of the asphalt pavement. The strength of the CIR roads remains generally consistent with time.

To determine the existence of a relationship between R_c and maximum deflection, they are plotted against each other for both rehabilitation methods. Figure 4.14 presents the results for the CIR roads, while Figure 4.15 shows the results for the CHOL roads. Each individual road is plotted in order to eliminate the age variable in this analysis. As expected, the results indicate a general decrease in R_c with an increase in maximum deflection for both types of rehabilitation methods.

An attempt is made to relate both R_c and maximum deflection to the number of TC/ln-km for the CIR roads and the CHOL roads in Figures 4.16 to 4.19. No correlation exists between transverse cracking and either of these variables.

4.3.2 Dynaflect Results for the CHOL Road - Pre and Post Rehabilitation

A 2 km section of Regional Road 52 in the RMOC was rehabilitated by placing two 40 mm layers of conventional hot mix over the existing roadway in 1996. Dynaflect values for this section of roadway were obtained from 1993, three years prior to rehabilitation, and 1997, one year following the overlay procedure. Measurements were compared between the two years to determine the effect of a CHOL on the deflection characteristics of the road. It is possible that the station locations for each test year do not match exactly, however, the performance trends can still be observed.

Maximum deflection values are plotted by road station for both years in Figure 4.20. It shows that at any location, the maximum deflection is on average two times greater in 1993 than in 1997, showing that the addition of the overlay effectively reduces the deflection values of the pavement.

Figure 4.21 plots the R_c with respect to the road station for 1993 and 1997. The R_c increases, on average, by a factor of 1.5 after the rehabilitation is carried out. In the previous section, it is found that the R_c of roads in Group 2 increases with age until the seventh year when a large drop in R_c occurs. It is expected that the 1997 R_c values will continue to increase until such a time that the asphalt concrete has deteriorated and hardened with age, causing an increase in stiffness and decrease in R_c .

In summary, placement of a CHOL will cause a decrease in the maximum deflection values, and an increase in the R_c values.

4.3.3 Dynaflect Results for the CIR Road - Pre and Post Rehabilitation

A 12.9 km section of Regional Road 6 in the RMOC was rehabilitated by placement of a CIR base course and a 40 mm surface course of hot mix overlay. Dynaflect values for a 5 km section of this roadway were obtained from 1995, one year before rehabilitation, from 1996 immediately after the CIR base course was placed, and from 1996 immediately after the hot mix surface course was placed. Measurements are compared between the three data sets to determine the effect of the CIR and the CHOL layers on the deflection characteristics of the road. This data is then used with the results of the previous section to establish the performance of the CIR as compared to the CHOL.

Maximum deflection values are plotted by road station from 0 to 5 km for the three sets of data in Figure 4.22. It shows that at any location, the maximum deflection was lower in 1995, increases by nearly 50% after replacing the old pavement with a CIR layer, then decreases by 20% once the final hot mix surface course is placed. As presented in section 4.2.1, maximum deflection for the CIR roads in Group 1 increases minimally with age for the first four years, and then decreases in the fifth and sixth years. The changes are small and gradual. In the previous section, maximum deflection of the CHOL roads are observed to decrease with rehabilitation. The opposite is true for the CIR rehabilitation. The CIR layer increases the maximum deflection values significantly indicating, by conventional analysis, a decrease in structural adequacy. The final hot overlay surface course succeeded in decreasing these values, as was expected, however the net overall change was an increase in maximum deflection values. This suggests that the newly rehabilitated road, which is now thicker than the previously existing road, has lost

structural integrity. The performance of the CIR roads, however, based on the PMS distress data collected, is equivalent and at times superior to that of the CHOL roads. Based on this performance, the Dynaflect values do not accurately define the field properties of the CIR with HMA overlay roads and thus should not be the only method used to determine rehabilitation strategies.

Figure 4.23 plots the R_c with respect to station for the three noted data sets. The R_c increases, on average, by a factor of nearly 1.5 after the CIR layer is placed, and further increases by a factor of 2.8 after the addition of the hot mix surface course. In section 4.2.1, the R_c for roads in Group 1 show a general increase with age. It is difficult to compare the R_c values from the previous section for the CHOL road to the CIR road in this section since the 1993 R_c values for the CHOL road are approximately two times higher than the 1995 values for the CIR road. The placement of the CIR base layer causes an increase in R_c similar in magnitude to the increase caused by the CHOL process. The CIR road, however, places an additional layer of hot mix as a surface course which further raises the R_c values by another 300%. The CIR end product, thus, has a significantly higher R_c value than the CHOL roads. This suggests that the CIR with HMA overlay pavement structure is weaker than that of the CHOL.

In summary, placement of a CIR base course with a HMA surface course will cause a net increase in the maximum deflection values, and a significant increase in the R_c values.

4.4 Effect of the Subgrade on Pavement Performance

The strength and quality of the pavement subgrade material can affect the frequency of transverse cracks, as well as the deflection of the asphalt pavement. To determine the influence of the subgrade on the two rehabilitation methods, the roads in Groups 1 and 2 were re-organized according to the subgrade type.

Tables 4.7 and 4.8 show the number of TC/ln-km for roads in each respective group. The number of TC/ln-km was used to calculate the number of cracks per year by dividing the total number of cracks by the age of the pavement in the specific survey year. The tables show the number of cracks per year as calculated for both the 1997 and 1994/95 survey years. The average value for each subgrade type in the individual survey years is also calculated. Table 4.9 and Figure 4.24 give a direct comparison between the effect of subgrade type on each rehabilitation method. The average value for the number of cracks per year for each subgrade and rehabilitation type is calculated to be used in the comparison. For a sand subgrade, which is typically stable and strong, both the CIR and the CHOL have equivalent crack rates of 7.2 and 6.7, respectively. A significant difference between rehabilitation methods is observed for silty sand subgrades. The number of cracks on this weaker subgrade are 5.8 and 12.8 for the CIR and CHOL roads respectively. Finally clay, the least stable of the subgrades that are examined, gives the crack rate to be 6.8 and 10.9 for the CIR and CHOL roads. A comparison of these values suggests that on a strong and stable subgrade, like sand, the performance of the pavement is

independent of the rehabilitation type. On weaker subgrades, like clay, the performance of the CIR pavement is superior to that of the CHOL layer. In addition, Figure 4.24 shows that the use of the CIR appears to negate the effect of the subgrade type on the rate of transverse cracking.

The calculated R_c and maximum deflection values for Groups 1 and 2 are also presented by subgrade type in Tables 4.10 and 4.11. The average R_c and maximum deflection values are calculated for each subgrade type in each table and are plotted graphically in Figures 4.25 and 4.26 respectively. Figure 4.25 shows that the R_c of the CHOL pavement increases as the quality of subgrade decreases. The R_c of the CIR structure, however, decreases as the subgrade quality also decreases. This means that the CIR pavement will perform poorly over weak subgrade material, contradicting the previous results for the number of cracks per year which show the CIR roads to perform well over less stable subgrades. The deflection data presented in Figure 4.26 yields similar results. As expected, the maximum deflection for the CHOL roads increases as the subgrade strength and stability decreases. The CIR deflection values, however, show a high maximum deflection on sand, the most stable subgrade, with lower values for the weaker subgrades. Clearly, sand, the stronger subgrade, will exhibit the lower deflection value.

It appears from the data that the use of the CIR rehabilitation on strong subgrades is not effective, however, use of the CIR will be more effective when used on weaker subgrades like clay or silty clay. This observation will be used later to explain the mechanisms which enable the CIR to reduce the rate of thermal cracking .

4.5 Summary

The conclusions made from the analysis of data and observations of the CIR and CHOL roads using the distress data from the PMS to determine transverse crack frequency indicate that both types of roads exhibit an increase in the number of TC/ln-km with increasing age. The same trend exists for both years (1994/95 and 1997) for which data was collected. It is difficult to establish a relationship to predict performance based on these two values, and to conclusively determine which rehabilitation method is a better deterrent for transverse cracks. To arrive at a conclusion about the pavement performance for each rehabilitation method, samples of both types of road were surveyed one year before rehabilitation, and two years following rehabilitation. The results indicate that the amount of transverse cracks at a given age for each road is dependent on the number of TC/ln-km that exist before rehabilitation. Roads having the same amount of TC/ln-km prior to rehabilitation, exhibit significantly fewer transverse cracks if rehabilitated with the CIR technology rather than the CHOL methods.

Further analysis of both rehabilitation methods was carried out using data collected by the Dynaflect Deflectometer. The data was used to determine the average maximum deflection and average radius of curvature for both types of roads. It is established that for both rehabilitation methods, an increase in maximum deflection would result in a decrease in R_c . This is consistent with the theory that a higher deflection value indicates a decrease in structural performance, and a lower R_c value indicates increasing tensile stresses on the surface of the pavement. Both situations are not desirable for asphalt pavements and are part of determining the need for rehabilitation. Based on the data collected from samples of both types of roads prior to and

immediately following the rehabilitation process, it is evident that the CIR roads exhibit an increase in the maximum deflection values and a large increase in the R_c values. The CHOL roads, conversely, show a decrease in the maximum deflection and a relatively small increase in the R_c values. These results indicate that conventional standards for Dynaflect data cannot be used to characterize the CIR roads.

The results of including the subgrade type as a variable in this analysis indicate that the most promising situation is when the CIR material is placed over weak subgrades rather than over strong subgrades. The CHOL structure does not perform well in this same situation. In addition, the Dynaflect data, when organized by subgrade type, yields results for the CIR which contradict what is actually occurring in the field. Once again, the Dynaflect Deflectometer is shown to be unable to characterize the CIR structure.

TABLE 4.1:

Group 1- CIR with HOL Roads, Transverse Cracks per lane-km, 1997 Survey

Road	Name	From	To	Year	Subgrade	Length (lane-km)	Total Cracks	TC/ln-km	Tape	Age	TC/ln-km (average per year)
RR5	Huntley	RR10	2500m N	1990	Sand	5.53	217	39.3	1997	7	
RR5	4th Line	RR73	1500 W of Century	1990	Silty Sand	9.20	253	27.5	1997	7	31.9
Total						14.72	470				
RR12	Fallowfield	Wolfgang	RR15	1991	Clay	3.66	259	70.8	1997	6	
RR5	Rock Road	RR22	Fitzroy Village	1991	Clay	2.94	147	50.1	1997	6	
RR5	Carp Road	RR36	RR49	1991	Silty Sand	19.75	1014	51.3	1997	6	53.9
Total						26.34	1420				
RR12	Fallowfield	RR5	RR49	1992	Clay	9.18	262	28.54	1997	5	
RR13	Greenbank	RR12	RR32	1992	Clay	7.18	245	34.1	1997	5	
RR11	Moodie Drive	RR8	Cambrian Road	1992	Silt	8.46	190	22.5	1997	5	28.1
Total						24.82	697				
RR18	Corkstown	RR49	2.7 km E of RR49	1993	Sand	4.48	128	28.6	1997	4	
RR28	Bearbrook	RR30	1 km S of RR34	1993	Clay	1.60	40	25.0	1997	4	27.6
Total						6.08	168				
RR29	York Corner	Kenmore Village	RR6	1994	Clay	4.60	81	17.59	1997	3	18.0
Total						4.60	81				
RR14	Leitrim	RR25	Hwy 31	1995	Silt	4.35	52	12.0	1997	2	
RR28	Navan Road	Cumberland Townline	W/L Navan Village	1995	Silty Sand	9.18	12	1.3	1997	2	
RR5	Carp Road	RR20	RR22	1995	Clay	12.13	130	10.7	1997	2	7.6
Total						25.66	194				
RR6	Snake Island	RR31	RR19	1996		25.20	7	0.3	1997	1	0.3

TABLE 4.2:

Group 1 - CIR with HOL Roads, Transverse Cracks per lane-km, 1994/95 Survey

Road	Name	From	To	Year	Subgrade	Length (lane-km)	Total Cracks	TC/ln-km	Tape	Age	TC/ln-km (average per year)
RR5	Huntley	RR10	2500 m N	1990	Sand	6.0	99	16.5	1994	4	
RR5	4th Line	RR73	1500 W of Century	1990	Silty Sand	8.0	105	13.1	1994	4	
RR12	Fallowfield	Wolfgang	RR15	1991	Clay	3.9	199	51.4	1995	4	22.6
Total						17.9	403				
RR5	Carp Road	RR36	RR49	1991	Silty Sand	20.0	663	33.2	1994	3	
RR13	Greenbank	RR12	RR32	1992	Clay	7.4	196	26.6	1995	3	
RR11	Moodie Drive	RR8	Cambrian Road	1992	Silt	8.4	132	15.7	1995	3	
RR12	Fallowfield	RR5	RR49	1992	Clay	9.2	203	22.1	1995	3	26.5
Total						45.0	1194				
RR27	Anderson	Renaud	Blackburn	1992	Clay	2.0	2	1.0	1994	2	1.0
RR18	Corkstown	RR49	2.7 km east	1993	sand	4.4	52	11.9	1994	1	
RR28	Bearbrook	RR30	1 km S	1993	Clay	3.6	11	3.0	1994	1	7.9
Total						8.0	63				
RR29	York Corner	Kenmore Village	RR6	1994	Clay	1.7	0	0	1994	0	0

TABLE 4.3: Group 2 - Conventional Hot Overlay (CHOL) Roads, Transverse Cracks per lane-km, 1997 Survey

Road	Name	From	To	Year	Subgrade	Length (lane-km)	Total Cracks	TC/ln-km	Tape	Age	TC/ln-km (average per year)
RR12 RR5 Total	Fallowfield McBean	RR49 CNR	RR59 RR10	1990 1990	Clay/Silty Clay	3.86 2.77 6.63	273 139 412	70.8 50.1	1997 1997	7 7	62.1
RR27 RR27 Total	Anderson Anderson	8th Line RR8	417 9th Line	1991 1991	Silty Sand and clay Silty Sand	6.21 5.50 11.71	381 354 735	61.4 64.3	1997 1997	6 6	62.8
RR25 RR3 Total	Albion Road	RR8 RR2	RR24 RR6	1992 1992	Silty Sand Sand	16.26 9.21 25.47	580 173 753	35.7 18.8	1997 1997	5 5	29.6
RR12 Total	Fallowfield	RR45	RR5	1993	Sand/Silty Sand	14.03 14.03	454 454	32.4	1997	4	32.4
RR27 RR41 Total	Anderson Boundary	RR6 Marvelville	RR8(less 4 km) RR6	1994 1994	Silty Sand silt - clay	8.27 9.43 17.70	242 67 309	29.3 7.1	1997 1997	3 3	17.5
RR19 RR11 Total	River Road Moodie	RR6 RR12	Flagstation Road RR32	1995 1995	Silty Sand Clay	6.69 8.13 14.82	249 398 647	37.2 49.0	1997 1997	2 2	43.7

TABLE 4.4: Group 2 - Conventional Hot Overlay Roads (CHOL), Transverse Cracks per lane-km, 1994/95 Survey

Road	Name	From	To	Year	Rehab	Subgrade	Length (lane-km)	Total Cracks	TC/ln-km	Tape	Age	TC/ln-km (average per year)
RR12	Fallowfield	RR49	RR59	1990	OL	Clay/Silty Clay	3.9	251	64.0	1995	5	64
RR5	McBean	CNR	RR10	1990	OL		4.0	141	35.3	1994	4	35.3
RR27	Anderson	8th Line	417	1991	OL	Silty Sand and clay	8.0	364	45.4	1994	3	
RR27	Anderson	RR8	9th Line	1991	OL	Silty Sand	6.0	226	37.7	1994	3	
RR3		RR2	RR6	1992	OL	Sand	6.4	49	7.6	1995	3	31.3
Total							20.4	639				
RR25	Albion Road	RR8	RR24	1992	OL	Silty Sand	16.3	483	29.7	1994	2	
RR12	Fallowfield	RR45	RR5	1993	OL	Sand/Silty Sand	14.1	343	24.4	1995	2	27.3
Total							30.3	826				
RR27	Anderson	RR6	RR8	1994	OL	Silty Sand	16.0	4	0.3	1994	<1	
RR41	Boundary	Marvelville	RR6	1994	OL	silt - clay	10.0	10	1.0	1994	<1	0.5
Total							26.0	14				

Table 4.5: Group 1 - Maximum Deflection and Radius of Curvature based on Deflection Measurements

Road	Name	From	To	Year	Rehab	Subgrade	Length (lane-km)	TC/in-km	Maximum Deflection (mils)	Radius of Curvature (mm)	Survey Year
RR5	Huntley	RR10	2500m N	1990	CIR	Sand	5.53	39.3	1.04	1.05E+07	1996
RR5	4th Line	RR73	1500 W of Century	1990	CIR	Silty Sand	9.20	27.5	1.07	7.42E+06	1996
RR12	Fallowfield	Wolfgang	RR15	1991	CIR	Clay	3.66	70.8	1.21	6.45E+06	1997
RR5	Rock Road	RR22	Fitzroy Village	1991	CIR	Clay	2.94	50.1	1.36	6.36E+06	1996
RR5	Carp Road	RR36	RR49	1991	CIR	Silty Sand	19.75	51.3	0.71	7.01E+06	1996
RR12	Fallowfield	RR5	RR49	1992	CIR	Clay	9.18	28.54	1.60	4.21E+06	1997
RR13	Greenbank	RR12	RR32	1992	CIR	Clay	7.18	34.1	1.32	5.26E+06	1997
RR11	Moodie Drive	RR8	Cambrian Road	1992	CIR	Silt	8.46	22.5	1.18	5.50E+06	1997
RR18	Corkstown	RR49	2.7 km E of RR49	1993	CIR	Sand	4.48	28.6	2.37	2.47E+06	1995
RR28	Bearbrook	RR30	1 km S of RR34	1993	CIR	Clay			1.11	9.43E+06	1997
RR5	4th Line Road	RR13	S/L North Gower	1994	CIR	Silt and Peat	15.04	39.8			
RR29	York Corner	Kenmore Village	RR6	1994	CIR	Clay	4.87	23.6	1.30	3.24E+06	1997
RR14	Leitrim	RR25	Hwy 31	1995	CIR	Silt	4.35	12.0	0.91	6.30E+06	1997
RR28	Navan Road	Cumberland Townline	W/L Navan Village	1995	CIR	Silty Sand	9.18	1.3	0.74	8.52E+06	1997
RR5	Carp Road	RR20	RR22	1995	CIR	Clay	12.13	10.7	1.12	3.78E+06	1996

Table 4.6: Group 2 - Maximum Deflection and Radius of Curvature based on Deflection Measurements

Road	Name	From	To	Year	Rehab	Subgrade	Length (lane-km)	TC/ln-km	Maximum Deflection (mils)	Radius of Curvature (mm)	Survey Year
RR12 RR5	Fallowfield McBean	RR49 CNR	RR59 RR10	1990 1990	OL OL	Clay/Silty Clay Clay	3.86 2.77	70.8 50.1	2.00 0.91	5.80E+06 1.09E+07	1997 1996
RR27 RR27	Anderson Anderson	8th Line RR8	417 9th Line	1991 1991	OL OL	Silty Sand and clay Silty Sand	6.21 5.50	61.4 64.3	0.75 1.23	9.33E+06 5.18E+06	1996 1996
RR25 RR3	Albion Road	RR8 RR2	RR24 RR6	1992 1992	OL OL	Silty Sand Sand	16.26 9.21	35.7 18.8	0.81 1.30	1.32E+07 2.87E+06	1996 1997
RR12	Fallowfield	RR45	RR5	1993	OL	Sand/Silty Sand	14.03	32.4	0.82	5.25E+06	1997
RR27	Anderson	RR6	RR8(less 4 km)	1994	OL	Silty Sand	8.27	29.3	0.96	5.61E+06	1996
RR41	Boundary	Marvelville	RR6	1994	OL	silt - clay	9.43	7.1	0.88	7.81E+06	1996
RR19 RR11	River Road Moodie	RR6 RR12	Flagstation Road RR32	1995 1995	OL OL	Silty Sand Clay	6.69 8.13	37.2 49.0	1.82 0.85	3.39E+06 8.07E+06	1995 1997

TABLE 4.7:

Group 1- CIR with HMA Overlay Roads by Subgrade Type, Transverse Cracks per lane-km, 1997 and 1994/95 Survey

Road	Name	From	To	Subgrade	Rehab Year	TC/ln-km	Age	No. of cracks per year	TC/ln-km	Age	No. of cracks per year
RR5	Huntley	RR10	2500m N	Sand	1990	39.3	7	5.6	16.5	4	4.1
RR18	Corkstown	RR49	2.7 km E of RR49	Sand	1993	28.6	4	7.1	11.9	1	11.9
Average								6.4			8.0
RR5	4th Line	RR73	1500 W of Century	Silty Sand	1990	27.5	7	3.9	13.1	4	3.3
RR5	Carp Road	RR36	RR49	Silty Sand	1991	51.3	6	8.6	33.2	3	11.1
RR28	Navan Road	Cumberland Townline	W/L Navan Village	Silty Sand	1995	1.3	2	0.7			
Average								4.4			7.2
RR14	Leitrim	RR25	Hwy 31	Silt	1995	12.0	2	6.0			
RR11	Moodie Drive	RR8	Cambrian Road	Silt	1992	22.5	5	4.5	15.7	3	5.2
Average								5.2			8.2
RR12	Fallowfield Road	Wolfgang RR5	RR15	Clay	1991	70.8	6	11.8	51.4	4	12.9
RR12	Fallowfield	RR12	RR49	Clay	1992	28.54	5	5.7	22.1	3	7.4
RR13	Greenbank	RR12	RR32	Clay	1992	34.1	5	6.8	26.6	3	8.9
RR27	Anderson	Renaud	Blackburn	Clay	1992				1.0	2	0.5
RR28	Bearbrook	RR30	1 km S of RR34	Clay	1993	25.0	4	6.3	3.0	1	3.0
RR29	York Corner	Kenmore Village	RR6	Clay	1994	17.59	3	5.9	0	0	
RR5	Carp Road	RR20	RR22	Clay	1995	10.7	2	5.4			
Average								7.0			13.5

TABLE 4.8: Group 2 - Conventional Hot Overlay (HOL) Roads by Subgrade Type, Transverse Cracks per lane-km, 1997 and 1994/95 Survey

Road	Name	From	To	Subgrade	Rehab Year	1997 TC/in-km	Age	No. of cracks per year	1994/95 TC/in-km	Age	No. of cracks per year
RR3	Fallowfield	RR2	RR6	Sand Sand	1992	18.8	5	3.8	7.6	3	2.5
RR12		RR45	RR5		1993	32.4	4	8.1	24.4	2	12.2
Average								6.9			7.4
RR27	Anderson	8th Line	417	Silty Sand and clay Silty Sand Silty Sand Silty Sand Silty Sand	1991	61.4	6	10.2	45.4	3	15.1
RR27		RR8	9th Line		1991	64.3	6	10.7	37.7	3	12.6
RR25		Albion Road	RR24		1992	35.7	5	7.1	29.7	2	14.9
RR27		RR8	RR8 (less 4 km)		1994	29.3	3	9.8			
RR19		River Road	RR6		1995	37.2	2	18.6			
Average								11.3			14.2
RR12	Fallowfield	RR49	RR59	Clay/Silty Clay Clay silt - clay Clay	1990	70.8	7	10.1	64.0	5	12.8
RR5		McBean	RR10		1990	50.1	7	7.2	35.3	4	8.8
RR41		Boundary	RR6		1994	7.1	3	2.4			
RR11		Marvelville	RR12		1995	49.0	2	24.5			
Average								11.0			10.8

Table : 4.9 Comparison of TC/ln-km per Year Between Rehabilitation Method for Selected Subgrade Type

Rehabilitation Method	Sugrade Type	1997 Survey	1994/95 Survey	Average
CIR	Sand	6.4	8.0	7.2
CHOL		5.9	7.4	6.7
CIR	Silty Sand	4.4	7.2	5.8
CHOL		11.3	14.2	12.8
CIR	Silt	5.2	5.2	5.2
CHOL				
CIR	Clay	7.0	6.5	6.8
CHOL		11.0	10.8	10.9

Table : 4.10 Group 1 - Deflection Data, Sorted by Subgrade, for CIR with HOL roads, 1996/97 Data

Road	Name	From	To	Rehab Year	Subgrade	Age (in 1997)	Length (lane-km)	Maximum Deflection (mils)	Radius of Curvature (mm)
RR5	Huntley	RR10	2500m N of RR10	1990	Sand	7	5.53	1.04	2.67E+05
RR18	Corkstown	RR49	2.7 km E of RR49	1993	Sand	4	4.48	1.99	1.18E+05
Average								1.52	1.92E+05
RR5	4th Line	RR73	1500 W of Century Rd	1990	Silty Sand	7	9.20	1.07	1.88E+05
RR28	Navan Road	Cumberland Townline	W/L Navan Village	1995	Silty Sand	2	9.18	0.74	2.14E+05
RR5	Carp Road	RR36	RR49	1991	Silty Sand	6	19.75	0.71	1.78E+05
Average								0.84	1.93E+05
RR11	Moodie Drive	RR8	Cambrian Road	1992	Silt	5	8.46	1.18	1.40E+05
RR14	Leitrim	RR25	Hwy 31	1995	Silt	2	4.35	0.91	1.60E+05
RR5	4th Line Road	RR13	S/L North Gower	1994	Silt and Peat	3	15.04		
Average								1.04	1.50E+05
RR12	Road	Wolfgang	RR15	1991	Clay	6	3.66	1.21	1.64E+05
RR12	Fallowfield	RR5	RR49	1992	Clay	5	9.18	1.60	1.07E+05
RR13	Greenbank	RR12	RR32	1992	Clay	5	7.18	1.32	1.34E+05
RR5	Rock Road	RR22	Fitzroy Village	1991	Clay	6	2.94	1.36	1.61E+05
RR28	Bearbrook	RR30	1 km S of RR34	1993	Clay	4	3.75	1.06	2.57E+05
RR29	York Corner	Kenmore Village	RR6	1994	Clay	3	4.60	1.30	8.14E+04
RR5	Carp Road	RR20	RR22	1995	Clay	2	12.13	1.12	9.61E+04
Average								1.28	1.55E+05

Table 4.11 Group 2 - Deflection Data Sorted by Subgrade, for Conventional HOL roads, 1996/97 Data

Road	Name	From	To	Year	Subgrade	Age (in 1997)	Length (lane-km)	Maximum Deflection (mils)	Radius of Curvature (mm)
RR3	Fallowfield McBean	RR2	RR6	1992	Sand	5	9.2	1.34	7.28E+04
RR12		RR45	RR5	1993	Sand	4	14.0	0.82	1.33E+05
RR5		CNR	RR10	1990	Clay	7	2.8	0.91	2.76E+05
Average								1.02	1.61E+05
RR27	Anderson Anderson Albion Road Anderson River Road	8th Line RR8 RR8 RR6 RR6	417 9th Line RR24 RR8(less 4 km) Flagstation Road	1991	Silty Sand and clay	6	6.2	0.75	2.37E+05
RR27				1991	Silty Sand	6	5.5	1.23	1.32E+05
RR25				1992	Silty Sand	5	16.3	0.81	3.34E+05
RR27				1994	Silty Sand	3	8.3	0.96	1.42E+05
RR19				1995	Silty Sand	2	6.7	1.82	8.43E+04
Average								1.11	1.89E+05
RR12	Fallowfield Anderson Boundary Moodie	RR49	RR59	1990	Clay/Silty Clay	7	3.9	2.00	1.47E+05
RR27		RR26	Renaud	1993	Clay	4	10.4	1.07	1.89E+05
RR41		Marvelville	RR6	1994	silt - clay	3	9.4	0.88	1.98E+05
RR11		RR12	RR32	1995	Clay	2	8.1	0.85	2.05E+05
Average								1.20	1.65E+05

**FIGURE 4.1: CIR Roads, Transverse Cracks per lane - km
vs. Age, 1997 Crack Survey**

**Figure 4.2: CIR with HOL Roads, Transverse Cracks per
lane-km vs. Age, 1994/95 Crack Survey**

FIGURE 4.3: Change in Number of Transverse Cracks per Year over 2 or 3 year period for Individual CIR with HOL Rehabilitated Roads

**Figure 4.4: CHOL Roads, Transverse Cracks Per Lane - km
vs. Age, 1997 Crack Survey**

**Figure 4.5: CHOL Roads, Transverse Cracks per lane - km
vs. Age, 1994/95 Crack Survey**

FIGURE 4.6: Change in Number of Transverse Cracks per Year over 2 or 3 year period for Individual CHOL Rehabilitated Roads

Figure 4.7: Comparison of No. of TC/In-km for CIR with HOL vs. Conventional HOL Roads, 1997 Survey

Figure 4.8: Comparison of No. of TC/In-km for CIR with HOL vs. Conventional HOL, 1994/95 Survey

Figure 4.9: Compare Effect of Rehabilitation Method on Number of TC / In-km for Two Roads Rehabilitated in 1995

Figure 4.10: Average Radius of Curvature vs. Age for CIR Roads

Figure 4.11: Average Radius of Curvature vs Age for CHOL Roads

Figure 4.12: Average Maximum Deflection vs. Age for CIR Roads

Figure 4.13: Average Maximum Deflection vs. Age for CHOL Roads

Figure 4.14: Radius of Curvature vs. Maximum Deflection, CIR Roads

**Figure 4.15: Radius of Curvature vs. Maximum Deflection
CHOL Roads**

**Figure 4.16: Radius of Curvature vs. Transverse Cracks,
CIR Roads**

Figure 4.17: Radius of Curvature vs. Transverse Cracks, CHOL Roads

Figure 4.18: Transverse Cracks vs. Maximum Deflection CIR Roads

Figure 4.19: Transverse Cracks vs. Maximum Deflection, CHOL

Figure 4.20: Maximum Deflection vs. Location for RR52, CHOL- Pre(1993) and Post(1997) Rehabilitation

Figure 4.21: Radius of Curvature vs. Location for RR52, CHOL- Pre (1993) and Post (1997) Rehabilitation

Figure 4.22: Maximum Deflection vs. Location, RR6, CIR 0 to 5 km

**Figure 4.23: Radius of Curvature vs. Location, RR6, CIR
0 to 5 km**

Figure 4.24: Comparison of TC/ln-km/year Between Rehabilitation Method for Selected Subgrade Type

Figure 4.25: Effect of Subgrade Type on Radius of Curvature **Figure 4.26: Effect of Subgrade Type on Maximum Deflection**

CHAPTER 5

LABORATORY TEST RESULTS

5.1 Introduction

A laboratory testing program was developed and carried out to determine the mechanical properties of the CIR material. The laboratory characterization was necessary to provide support and explanation for the results of the field data analysis of the previous chapter, and for the visual observations of the CIR performance provided by the RMOC. This chapter presents the results of four laboratory tests performed on the CIR material. These tests are; bulk relative density (BRD), indirect tensile strength (ITS), direct tensile strength (DTS), and moisture sensitivity tests. The BRD tests were performed on all core and cylindrical samples, moisture sensitivity tests were carried out on field cores, and the ITS and DTS tests were performed on various sample types over a range of temperatures.

5.2 Sample Labels

All samples were labelled with respect to the sample type, location, and number. The list of sample types with their corresponding label is as follows:

- M: Marshall specimen
- G: Gyratory specimen
- C: Field extracted core
- L: Laboratory compacted slab
- S: Field cut slab
- H: Hot mix slab

The sample type is followed by a location number which is taken from the locations detailed in Chapter three, Figure 3.5. The sample number for the core, Marshall, and Gyratory specimens follows and is used to differentiate between samples of the same type obtained from the same location. For example, M4-6 indicates Marshall sample number 6 from sample location 4. Slab numbers are slightly different in that two laboratory compacted slab forms are taken from each sample location and are further cut into testable slabs. Each form is labelled with a letter, and the slabs cut from each form are numbered. For example, L4-2A denotes laboratory compacted slab number 2 from form A that is sampled from location 4. Field compacted slabs are labelled in the same manner, and both processes are diagrammed in Figure 5.1.

Figure 5.1: Slab Labelling Convention

a) Lab Compacted Slab

Form A - From Location 4

* = L4-1A

b) Field Compacted Slab

Location 7

* = S7-1A

5.3 Bulk Relative Density

The BRD of all core and cylindrical specimens was measured before any laboratory tests were carried out. The mean density, standard deviation (STD), and coefficient of variation (COV) were also calculated for all samples.

5.3.1 Marshall Specimens

The Marshall specimens were divided into sample groups according to the location from which the material was sampled. The calculated BRD's and mean density for sample location 1 are presented in Table 5.1. The tables for the remaining sample locations are included in Appendix D. Table 5.2 compares the mean densities for all sample locations and uses these values to calculate an overall mean density of 2.117 gm/cm^3 for all samples. Appendix D also contains a table which combines the samples from all locations into one sample set. This table shows a higher COV than that in Table 5.2 due to the larger sample size. The calculated COV's in both cases are greater than 1.2%. This is a relatively large value considering that the sample compaction procedure is standard and does not allow for any variables to be introduced. Since the variance cannot be accounted for in the Marshall compaction process, it must be found within the material.

5.3.2 Gyratory Samples

The Gyratory specimens were also divided into sample groups according to the location

from which the material was sampled. BRD's were calculated for the Gyratory samples which were larger in diameter than the Marshall samples. The calculated BRD's and mean density for sample location 2 are presented in Table 5.3. The tables for the remaining sample locations are included in Appendix D. The mean density for each sample group is calculated and the results are given in Table 5.4. Further, the mean density for all Gyratory samples is 2.295 gm/cm^3 . The calculated COV for the density of this material is greater than 1.2%. The Gyratory compaction process requires that all specimens be compacted to a sample height of $100 \pm 1 \text{ mm}$ [AASHTO TP4-93]. This is generally achieved by using a specified mass of the material. Using the CIR material, however, a set mass was not able to achieve a standard height. The Gyratory compactor was not able to produce samples of the CIR asphalt that were as uniform as conventional HMA samples.

5.3.3 Field Extracted Core Samples

Core samples were extracted from four sample locations along RR6. Approximately twenty-six samples were cored from each location and the 140 mm to 170 mm diameter of the samples was similar to that of the 150 mm diameter Gyratory specimen. The calculated BRD's and mean density for sample location 5 are presented in Table 5.5. The tables for the remaining sample locations are included in Appendix D. Table 5.6 compares the mean densities for all four sample locations and gives the mean density to be 2.133 gm/cm^3 . The consistency of the measured values was checked by calculating the COV. A high value, considering the precision of the electronic scales, of 2.9% is given in Table 5.6. This high variation in densities indicate

that the field compaction did not achieve uniform density. The daily variations in temperature and moisture would have affected the curing and compaction processes, and in addition, the core samples were extracted at slightly different ages.

5.3.4 Comparison

A comparison between the average densities of each sample type is shown in Table 5.7. The densities of the Marshall specimens match those of the in-situ field cores. The densities of the Gyratory samples, however are significantly higher. These results suggest that the conventional method of choosing a design number of gyrations in the Gyratory compaction process, as outlined in section 3.5.2, is not valid for the CIR material.

5.4 Maximum Relative Density

The MRD tests were carried out on a selected number of field cores and the average value was used to calculate the average percent of air voids present in each sample type. These results are also shown in Table 5.7. The Gyratory samples were the most dense and consequently had the lowest air void content. The Marshall samples and field cores had similar air void contents of over 13.5 percent. Typical laboratory design void contents for hot mix base courses range from 3% to 5 % [OPSS 1150].

5.5 Indirect Tensile Strength

Indirect tensile strength tests were performed on the Marshall and Gyratory specimens, and field cores. Tests for each sample type were performed at -20 °C, 0 °C, and +20 °C. A sample size of six specimens was used at each test temperature for each sample type. The amount of laboratory specimens made was limited to six Gyratory and six Marshall per location, therefore for testing purposes, the laboratory samples were grouped according to the type rather than the location. Conversely, six field core samples from each location were tested at each temperature.

5.5.1 Marshall Samples

Marshall samples were tested for the ITS at -20 °C, 0 °C, and +20 °C. Those tested at +20 °C were subjected to an age variable. All Marshall samples were compacted immediately after field sampling, then tested at one and two week intervals. The ITS test results indicate that strength of the material increases with time. Cores were not able to be retrieved until several weeks after material placement, so the testing of Marshall samples at an early age was abandoned. Tables 5.8 to 5.10 present the ITS values for Marshall samples tested at -20 °C, 0 °C, and +20 °C respectively. The corresponding mean ITS values are 918.7, 706.0, and 287.2 kPa as summarized in Table 5.11. The results show a decrease in strength with an increase in temperature. Tests at 0 °C give ITS results that are nearly 150 % greater than specimen strength at room temperature. As the temperature decreases to -20 °C the ITS of Marshall specimens

increases to more than 200 % that of specimens at room temperature. These results are significantly lower than those obtained from the Gyratory samples, but are similar to those obtained from field cores. The coefficient of variation for the ITS of Marshall samples at each test temperature ranges from 15% to 40 % as shown in Table 5.11. This indicates a high degree of variability between the results of the individual Marshall specimens.

5.5.2 Gyratory Samples

Gyratory samples were tested for the ITS over a range of temperatures. The Gyratory samples tested at +20 °C were subjected to an age variable similar to the Marshall samples. The ITS test results are similar to the Marshall results which suggest that strength of the material increases with time. Results for the ITS tests at -20 °C, 0 °C, and +20 °C are given in Tables 5.12 to 5.14 respectively. The corresponding mean ITS values are 2189.6, 2037.1, and 505.8 kPa and are given in Table 5.15. Clearly, the ITS decreases as the temperature increases. Tests at 0 °C give ITS results that are over four times higher than strengths at room temperature. However, the ITS of the Gyratory specimens increases slightly by 7 % as the temperature is reduced from 0 °C to -20 °C. It is important to note that the maximum capacity of the load apparatus was less than that of the Gyratory samples tested at 0 °C and -20 °C. Therefore the maximum calculated strength values at these temperatures may be less than the actual maximum tensile strengths. The coefficient of variation for the ITS of Gyratory samples at each test temperature ranges from 11% to 15 % as shown in Table 5.15. While this indicates a significant variation between sample strengths, there is a much lower degree of variability or more

consistency between the individual Gyratory specimen results than the Marshall results.

5.5.3 Field Cores

The field extracted asphalt cores were subjected to similar laboratory tests. The cores were extracted from the field as soon as the material could support the coring process. The ITS values for the field cores are given in Tables 5.16 to 5.18 for tests carried out at -20°C , 0°C , and $+20^{\circ}\text{C}$, respectively. Each table shows the individual sample ITS values, mean, standard deviation, and coefficient of variation for all samples at the specific temperature. Tables 5.19 to 5.21 show the mean of the ITS for each sample location according to the test temperature. The corresponding mean ITS values for all locations at -20°C , 0°C , and $+20^{\circ}\text{C}$ are 1263.9, 681.1, and 243.6 kPa as shown in Table 5.22. Consistent with the laboratory test samples, the field cores show an increase in strength with the decrease of temperature. Tests at 0°C give ITS results that are over 150% higher than specimen strengths at room temperature. Decreasing the test temperature further to -20°C causes the ITS of field cores specimens to increase by 450% . The ITS tests on field cores give results which are significantly lower than those obtained from Gyratory samples, and similar to those obtained from Marshall specimens. The degree of variation between the ITS results for field cores at all temperatures significantly high and is between 31% and 40 % as shown in Table 5.22.

5.5.4 Comparison

A comparison between the mean ITS values of Marshall, Gyratory, and field samples is given in Table 5.23 and Figure 5.2. Clearly, the ITS of all samples is higher at cold temperatures than that at room temperature and reaches a maximum at -20°C . The ITS of the Gyratory samples at all temperatures is much higher than the corresponding ITS of the Marshall specimens and field cores. This suggests the important role of compaction in achieving high tensile strengths. In addition, although the ITS of Marshall cores appear to duplicate the ITS of field cores at $+20^{\circ}\text{C}$ and 0°C , the mean ITS of field cores at -20°C is still almost one and a half times greater than Marshall sample strength at the same temperature. This suggests that as the temperature decreases, the Marshall samples are no longer representative of the in-situ material.

5.6 Direct Tensile Strength

Direct tensile strength tests were performed on laboratory and field compacted slab samples. Tests for each sample type were performed at -30°C , 0°C , and $+40^{\circ}\text{C}$. All tests were carried out at a loading rate of 0.3 mm/min . Many of the laboratory compacted slabs broke before they were tested. A minimum of three samples at each temperature were tested, revised from the amount given in the testing plan in Chapter 3. Similarly, field compacted slabs were quite difficult to extract intact. A minimum of two samples at each temperature were tested and some of these samples had surface cracks that may have been the result of transporting the slabs from the field to the laboratory.

5.6.1 Laboratory Compacted Slab Samples

The results of the laboratory compacted slab samples that were tested for the DTS at -30 °C, 0 °C, and +40 °C are given in Tables 5.24 to 5.26. Each table shows the mean, standard deviation, and coefficient of variation for the samples at the specific temperature. The corresponding mean DTS values at -30 °C, 0 °C, and +40 °C are 124.4, 92.1, and 34.5 kPa and are shown in Table 5.33. The results show an increase in strength with a decrease of temperature. The testing at freezing temperature gives direct tensile strengths that are 150% greater than the tensile strength at +40 °C. The strength at -30 °C is over 250% higher than at +40 °C. However, as will be shown later, these tensile strength values are very low as compared to those of samples compacted in the field.

5.6.2 Field Compacted Slab Samples

The DTS tests were also carried out on field extracted slab samples over the same temperature range. Tables 5.27 to 5.29 present the DTS values for the samples tested at -30 °C, 0 °C, and +40 °C and the corresponding mean DTS values of 804.7, 757.5, and 54.9 kPa are given in Table 5.34. The mean value at -30 °C is the value of the only sample that did not fail due to a failure of the glue. The coefficient of variation and the standard deviation were not calculated due to the small sample size. The results show a decrease in strength with an increase of temperature from +40 °C to 0 °C. The testing at freezing temperature gives a mean direct tensile strength that is almost fourteen times more than the tensile strength at +40 °C. The strength increases between 0 °C and -30 °C, however only by 6%. This small increase can be

attributed in part to the glue failure, however, the strength will eventually reach a plateau in cold temperatures due to the brittle nature of the material.

5.6.3 Hot Mix Laboratory Compacted Slab Samples

Hot mix HL3 material was sampled from a paving site and compacted with a small roller in wooden forms at the site. These slabs were tested for the DTS to compare with the results of the CIR material. The DTS test results at -30 °C, 0 °C, and +40°C are shown in Tables 5.30 to 5.32. Each table shows the mean for the samples at each specific temperature. The coefficient of variation and the standard deviation were not calculated based on the small sample size that was tested. The results show a consistent increase of the DTS with a decrease in temperature. From +40°C to 0 °C the material achieves nearly 700 % more strength. From 0 °C to -30 °C the strength of the material continues to show significant gains of almost 200%. The mean DTS values at -30 °C, 0 °C, and +40°C were 1102.1, 385.7, and 49.8 kPa respectively and are shown in Table 5.35.

5.6.4 Comparison

A comparison of the mean DTS of the laboratory CIR, field compacted CIR, and laboratory hot mix slab samples is given in Table 5.36 and Figure 5.3. Clearly, at low temperatures the effect of compaction on the CIR material is significant as shown by the mean values at 0 °C and -30 °C. On the other hand, the results at +40 °C suggest that the CIR asphalt

appears to lose most of its mechanical properties regardless of the compaction method. The hot mix slabs exhibited the same trend as the CIR material, gaining strength as the temperature decreased. The hot mix slabs were not as strong at 0 °C as the field compacted CIR slabs, however, they gained more strength than the CIR slabs as the temperature reached -30 °C. As will be discussed later, this observation may have a significant effect on why and how the CIR structure reduces the appearance of transverse cracks.

5.7 Moisture Sensitivity

Moisture sensitivity tests were performed on field cores to assess the effect of water on the tensile strength of the in-situ CIR material. Field cores were taken from four sample locations, and six cores from each sample set were tested. Of the six cores, three were conditioned samples (saturated), and the other three were controlled samples (dry). All six samples were tested for the ITS at room temperature, and the percent of retained strength of the conditioned relative to the controlled samples was calculated.

5.7.1 Results and Comparison

Tables 5.37 to 5.40 show the results of moisture sensitivity tests on field cores from sample locations 5, 6, 7, and 8 respectively. Included in these tables are the individual densities, and the ITS values. The results show that the CIR material has no appreciable strength when saturated. The percent retained strength ranges from 11 to 16 %. Table 5.41 summarizes and

compares the results from each sample location. It compares the mean strengths and the percent retained strength between each sample location. Clearly, this material will have no strength once saturated. Moisture is initially required as part of the emulsion. It evaporates and allows the emulsion to coat the aggregate. If there is initially too much moisture, total evaporation will not occur and strength will be affected. Also, the addition of moisture during curing and throughout the life of the asphalt will depreciate the tensile strength. The CIR material must be protected from moisture intrusion.

5.8 Summary

Material sampling and laboratory testing of the CIR was carried out to help characterize the physical and mechanical properties of this material. Several conclusions can be made based on the results of laboratory testing.

In-situ core sampling reveals that the CIR material does not cure at a constant rate in the field. The material gains strength over time, depending on environmental factors. In addition, testing the bulk relative density of all samples shows that the Marshall samples are more representative of the field density. The number of gyrations used in Gyratory compaction must be adjusted to better simulate field density. Conventional methods for choosing this value cannot be used for this material.

Indirect tensile strength increases for all samples as the temperature decreases. Marshall and field core samples yield similar strength results, while Gyratory samples are much stronger.

At -20°C , however, the tensile strength of the field cores is significantly higher than that of the Marshall samples, suggesting that these laboratory samples cannot characterize the CIR material at very low temperatures.

The direct tensile strength also increases as temperature decreases. Laboratory compacted slabs, however, only reach 8% of the strength of field extracted slabs at 0°C . This indicates that field compaction cannot be simulated in the laboratory. The laboratory hot mix slabs have the highest strength at -30°C , however the strength of the CIR field cut slabs at 0°C is superior. Both the ITS and DTS results indicate the extreme sensitivity of the tensile mechanical properties of the CIR material to temperature variables.

Results of moisture sensitivity tests indicate that the CIR material has no anti-stripping properties and exhibits no strength when saturated. Therefore, this material must be protected from moisture intrusion in order to retain high tensile strength.

Table 5.1 Density of Marshall Compacted Samples from Location 1

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (g/cm ³)	Mean Density	STD	COV (%)
M1-1	1.008	0.5749	1.0533	2.107			
M1-2	0.991	0.5743	1.0269	2.188			
M1-3	0.944	0.5475	0.9977	2.097			
M1-4	0.982	0.5667	1.0300	2.120			
M1-5	0.994	0.5795	1.0510	2.108			
M1-6	0.959	0.5407	0.9928	2.121	2.123	0.033	1.6

Table 5.2 Comparison of Mean Density of Marshall Specimens at each Location

Sample Location	Mean Density for Location (g/cm ³)	Mean Density for all Samples (g/cm ³)	STD	COV (%)
1	2.123			
2	2.092			
3	2.157			
4	2.117			
5	2.071			
6	2.098			
7	2.146			
9	2.116			
10	2.136	2.117	0.027	1.290

Table 5.3 Density of Gyratory Compacted Samples from Location 2

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (g/cm ³)	Mean Density	STD	COV (%)
G2-1	4.018	2.3442	4.058	2.344			
G2-2	4.210	2.4461	4.255	2.327			
G2-3	4.034	2.3432	4.076	2.328			
G2-4	4.050	2.3471	4.094	2.318			
G2-5	3.908	2.2478	3.938	2.312			
G2-6	4.030	2.3126	4.065	2.300	2.322	0.015	0.7

Table 5.4 Comparison of Mean Density of Gyratory Specimens at each Location

Sample Location	Mean Density for Location (g/cm ³)	Mean Density for all Samples (g/cm ³)	STD	COV (%)
1	2.295			
2	2.322			
3	2.264			
4	2.284			
5	2.245			
6	2.322			
7	2.326			
8	2.314			
9	2.273			
10	2.304	2.295	0.028	1.21

Table 5.5 Density of Field Compacted Samples from Location 5

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
C5-1	2.884	1.5987	2.991	2.071			
C5-2	2.919	1.6280	3.034	2.076			
C5-3	2.722	1.5116	2.825	2.072			
C5-4	2.346	1.3043	2.423	2.097			
C5-5	2.861	1.5956	2.973	2.077			
C5-6	2.856	1.5836	2.958	2.078			
C5-7	2.889	1.6032	3.004	2.062			
C5-8	2.916	1.6048	3.025	2.053			
C5-9	2.952	1.6242	3.059	2.057			
C5-10	2.897	1.5836	3.000	2.045			
C5-11	2.897	1.6146	3.029	2.048			
C5-12	2.873	1.5980	2.986	2.070			
C5-13	2.910	1.6147	3.024	2.065			
C5-14	2.861	1.5897	2.978	2.061			
C5-15	2.872	1.5918	2.998	2.042			
C5-16	2.643	1.4665	2.759	2.045			
C5-17	2.901	1.6114	3.030	2.045			
C5-18	2.876	1.6139	3.009	2.062			
C5-19	2.764	1.5446	2.885	2.062			
C5-20	2.915	1.6188	3.051	2.035			
C5-21	2.993	1.6694	3.135	2.042			
C5-22	2.978	1.6657	3.128	2.037			
C5-23	2.964	1.6398	3.093	2.040			
C5-24	2.897	1.5984	3.027	2.028			
C5-25	2.885	1.5964	3.011	2.039			
C5-26	3.011	1.6914	3.152	2.061	2.057	0.017	0.8

Table 5.6 Comparison of Mean Density of Field Cores at each Location

Sample Location	Mean Density for Location (g/cm ³)	Mean Density for all Samples (g/cm ³)	STD	COV (%)
5	2.057			
6	2.182			
7	2.186			
8	2.106	2.133	0.063	2.9

Table 5.7 Percent Air Voids in Compacted Samples

Sample Type	Mean Density for all Samples (g/cm ³)	Average MRD	%AV
Marshall	2.117	2.465	14.1
Gyratory	2.295		6.9
Field	2.133		13.5

Table 5.8 ITS of Marshall Samples at -20 ° C

Sample	Age (days)	Load Rate (mm/min)	Density (g/cm ³)	ITS (kPa)	Mean ITS	STD	COV (%)
M1-5	140	12.7	2.108	573.7			
M4-5	132	12.7	2.106	976.7			
M5-5	131	12.7	2.079	904.8			
M6-6	126	12.7	2.121	686.5			
M9-4	106	12.7	2.123	1129.1	854.2	223.6	26.2

Table 5.9 ITS of Marshall Samples at 0 ° C

Sample	Age (days)	Load Rate (mm/min)	Density (g/cm ³)	ITS (kPa)	Mean ITS	STD	COV (%)
M1-6	226	2.5	2.121	222.6			
M3-4	219		2.173	711.9			
M4-6	218		2.104	900.1			
M5-3	217		2.103	768.8			
M6-5	236		2.094	453.1			
M7-3	210		2.142	907.1			
M9-3	181		2.138	1216.6			
M9-6	206		2.115	549.6			
M10-3	203		2.143	623.9	706.0	289.8	41.0

Table 5.10 ITS of Marshall Samples at +20 ° C

Sample	Age (days)	Load Rate (mm/min)	Density (g/cm ³)	ITS (kPa)	Mean ITS	STD	COV (%)
M9-2	94	12.7	2.116	251.6			
M9-5	94		2.108	255.9			
M10-1	91		2.141	340.7			
M10-2	91		2.132	300.6	287.2	42.0	14.6

Table 5.11 Comparison Between ITS of Marshall Samples at Various Temperatures

Sample Type	Temperature (°C)	Load Rate (mm/min)	Mean ITS (kPa)	STD	COV (%)
Marshall	20	12.7	287.2	42.0	14.6
	0	2.5	706.0	289.8	41.0
	-20	12.7	854.2	223.6	26.2

Table 5.12 ITS of Gyratory Samples at -20 ° C

Sample	Age (days)	Load Rate (mm/min)	Density (g/cm ³)	ITS (kPa)	Mean ITS (kPa)	STD	COV (%)
G1-5	149	2.5	2.311	1971.9	2189.6	323.8	14.8
G2-6*	148		2.300	2447.3			
G4-4	140		2.291	1643.9			
G5-3*	139		2.272	2245.9			
G6-5*	125		2.321	2349.6			
G9-4*	104		2.284	2479.1			

* Maximum permissible machine load reached before failure of sample

Table 5.13 ITS of Gyratory Cores at 0 ° C

Sample	Age (days)	Load Rate (mm/min)	Density (g/cm ³)	ITS (kPa)	Mean ITS	STD	COV (%)
G2-5*	225	2.5	2.312	2202.4	2037.1	244.2	12.0
G3-5	219		2.194	1851.3			
G4-5	218		2.280	1506.7			
G5-6*	217		2.260	2069.4			
G7-6*	210		2.318	2349.7			
G8-3*	182		2.308	2175.3			
G9-3*	181		2.291	2155.3			
G9-5	181		2.214	2030.6			
G10-3	178		2.305	1993.0			

* Maximum permissible machine load reached before failure of sample

Table 5.14 ITS of Gyratory Cores at +20 ° C

Sample	Age (days)	Load Rate (mm/min)	Density (g/cm ³)	ITS (kPa)	Mean ITS	STD	COV (%)
G8-1	91	12.7	2.314	433.7			
G8-2	91		2.320	482.6			
G9-1	94		2.291	572.4			
G9-2	94		2.286	573.7			
G10-1	91		2.305	483.3			
G10-2	91		2.301	488.7	505.8	55.8	11.0

Table 5.15 Comparison Between ITS of Gyratory Samples at Various Temperatures

Sample Type	Temperature (°C)	Load Rate (mm/min)	Mean ITS (kPa)	STD	COV (%)
Gyratory	20	12.7	505.8	55.8	11.0
	0	2.5	2037.1	244.2	12.0
	-20	2.5	2189.6	323.8	14.8

Table 5.16a ITS of Field Cores at -20 ° C, Load Rate of 12.7 mm/min

Sample	Age (days)	Load Rate (mm/min)	Density (g/cm ³)	ITS (kPa)	Mean ITS (kPa)	STD	COV (%)
C5-2	130	12.7	2.076	1127.9			
C5-12	130	12.7	2.070	1229.9			
C5-20	130	12.7	2.035	1317.8			
C6-5	125	12.7	2.127	1693.4			
C6-9	125	12.7	2.115	1420.9			
C6-23	125	12.7	2.179	1877.2			
C7-2	124	12.7	2.144	1096.8			
C7-15	124	12.7	2.236	2157.0			
C7-17	124	12.7	2.222	2230.3			
C8-6	97	12.7	2.116	1315.3			
C8-23	97	12.7	2.116	1099.7			
C8-25	97	12.7	2.089	1020.4	1465.5	423.0	28.9

Table 5.16b ITS of Field Cores at -20 ° C, Load Rate of 2.5 mm/min

Sample	Age (days)	Load Rate (mm/min)	Density (g/cm ³)	ITS (kPa)	Mean ITS (kPa)	STD	COV (%)
C5-6	138	2.5	2.078	868.4			
C5-13	138	2.5	2.065	736.1			
C5-15	138	2.5	2.042	717.1			
C6-6	133	2.5	2.110	1354.0			
C6-7	133	2.5	2.113	1418.8			
C6-12	133	2.5	2.127	1356.3			
C7-6	132	2.5	2.147	1660.1			
C7-24	132	2.5	2.216	2080.3			
C8-4	104	2.5	2.099	1303.6			
C8-13	104	2.5	2.107	1370.5			
C8-27	104	2.5	2.017	1038.2	1263.9	407.7	32.3

Table 5.17 ITS of Field Cores at 0 ° C

Sample	Age (days)	Load Rate (mm/min)	Density (g/cm ³)	ITS (kPa)	Mean ITS (kPa)	STD	COV (%)
C5-5	241	2.5	2.077	471.7	681.1	272.9	40.1
C5-7	241		2.062	542.6			
C5-19	241		2.062	464.0			
C5-21	241		2.042	528.3			
C5-22	241		2.037	520.8			
C5-25	241		2.039	418.6			
C6-3	236	2.5	2.130	600.5	681.1	272.9	40.1
C6-13	236		2.104	572.3			
C6-14	236		2.128	882.8			
C6-24	236		2.128	963.3			
C6-25	236		2.128	1311.2			
C6-26	236		2.128	1189.1			
C7-1	235	2.5	2.146	640.94	681.1	272.9	40.1
C7-16	235		2.231	1039			
C7-19	235		2.225	830.75			
C7-20	235		2.231	921.01			
C7-22	235		2.215	985.08			
C8-1	202	2.5	2.105	400.7	681.1	272.9	40.1
C8-5	202		2.112	504.6			
C8-9	202		2.115	546.6			
C8-17	202		2.108	541.8			
C8-19	202		2.103	356.2			
C8-21	202		2.114	433.9			

Table 5.18 ITS of Field Cores at +20 °C

Sample	Age (days)	Load Rate (mm/min)	Density (g/cm ³)	ITS (kPa)	Mean ITS (kPa)	STD	COV (%)
C5-1	125	12.7	2.071	187.7			
C5-8	240		2.053	220.9			
C5-9	240		2.057	217.7			
C5-11	125		2.048	189.1			
C5-18	125		2.062	174.4			
C5-23	240		2.040	172.8			
C6-8	120	12.7	2.121	257.62			
C6-11	120		2.115	222.85			
C6-19	120		2.171	251.54			
C6-20	231		2.128	394.0			
C6-21	231		2.128	370.4			
C6-22	231		2.128	393.8			
C7-5	119	12.7	2.135	271.7			
C7-7	119		2.141	264.8			
C7-9	216		2.152	269.5			
C7-10	216		2.151	282.4			
C7-12	119		2.144	261.9			
C7-23	216		2.237	421.2			
C8-2	91	12.7	2.115	158.8			
C8-3	214		2.098	174.1			
C8-8	214		2.123	167.5			
C8-12	91		2.125	169.4			
C8-18	214		2.114	182.7			
C8-22	91		2.122	170.2	243.6	79.9	32.8

Table 5.19 ITS of Field Cores at -20 ° C

Sample Location	Mean ITS for each Location (kPa)	Mean ITS for all Locations (kPa)	STD	COV (%)
5	999.5			
6	1520.1			
7	1844.9			
8	1191.3	1263.9	419.1	30.6

Table 5.20 ITS of Field Cores at 0 ° C

Sample Location	Mean ITS for each Location (kPa)	Mean ITS for all Locations (kPa)	STD	COV (%)
5	491.0			
6	919.9			
7	883.4			
8	464.0	681.1	272.9	40.1

Table 5.21 ITS of Field Cores at +20 ° C

Sample Location	Mean ITS for each Location (kPa)	Mean ITS for all Locations (kPa)	STD	COV (%)
5	193.8			
6	315.0			
7	295.3			
8	170.5	243.6	79.9	32.8

Table 5.22 Comparison of ITS of Field Cores at Various Temperatures

Sample Type	Temperature (°C)	Load Rate (mm/min)	Mean ITS (kPa)	STD	COV (%)
Field Core	20	12.7	243.6	79.9	32.8
	0	2.5	681.1	272.9	40.1
	-20	2.5	1263.9	407.7	32.3
	-20	12.7	1465.5	423.0	28.9

Sample Type	Load Rate (mm/min)	Temperature (°C)	Mean ITS (kPa)
Marshall	12.7	20	287.2
Gyratory	12.7		505.8
Field Core	12.7		243.6
Marshall	2.5	0	706.0
Gyratory	2.5		2037.1
Field Core	2.5		681.1
Marshall	12.7	-20	918.7
Gyratory	2.5		2189.6
Field Core	2.5		1263.9
Field Core	12.7		1465.5

Table 5.24 DTS of Laboratory Compacted slabs at -30 °C

Sample	Peak Load (kN)	Displacement at Peak Load (mm)	Max. Strain % (at peak load)	DTS (kPa)	Mean DTS (kPa)	STD	COV (%)
10A-2	0.5369	0.1984	0.78	87.6			
10A-1	0.7333	0.1116	0.44	129.7			
1B-1	0.5508	0.1488	0.59	92.4			
1B-2	0.8358	0.1488	0.59	137.5			
1B-3	0.6134	0.2729	1.07	103.4			
3A-1	1.1121	0.1488	0.59	195.7			
3A-2	0.6881	0.1488	0.59	124.7	124.4	36.8	29.5

Maximum Strain based on 1" free length, load rate is 0.3 mm/min

Table 5.25 DTS results of laboratory compacted slabs at 0 °C

Sample	Peak Load (kN)	Displacement at Peak Load (mm)	Strain % (at peak load)	DTS (kPa)	Mean DTS	STD	COV (%)
2B-1	0.2659	0.1488	0.59	51.5			
2B-2	0.4622	0.1984	0.78	88.8			
2B-3	1.0391	0.1364	0.54	194.8			
3B-1	0.3892	0.2108	0.83	66.4			
6B-2	0.3406	0.2108	0.83	58.8	92.1	59.1	64.2

* Three sample results removed from this set due to crack in sample prior to testing

Table 5.26 DTS results of laboratory compacted slabs at +40 °C

Sample	Peak Load (kN)	Displacement at Peak Load (mm)	Strain % (at peak load)	DTS (kPa)	Mean DTS
4B-2	0.1251	0.2853	1.12	23.9	
4B-3	0.1321	0.1116	0.44	20.9	
6B-3	0.3058	1.0914	4.30	58.8	34.5

Table 5.27 DTS results of field cut slabs at -30 °C

Sample	Peak Load (kN)	Displacement at Peak Load (mm)	Strain % (at peak load)	DTS (kPa)	Mean DTS
S7-2A*	0.4240	1.0666	4.20	79.8	
S6-3A*	0.4240	1.0666	4.20	82.8	
S6-1B*	1.8905	0.1116	0.44	407.9	
S7-3A	4.0278	0.3101	1.22	804.7	804.7

* Glue failed before sample did
Sample S7-3A reached this load with a surface crack

Table 5.28 DTS results of field cut slabs at 0 °C

Sample	Peak Load (kN)	Displacement at Peak Load (mm)	Strain % (at peak load)	DTS (kPa)	Mean DTS
S6-1A	5.023	0.409	1.61	1074.6	
S7-1A*	2.221	0.434	1.71	440.4	757.5

*small surface crack

Table 5.29 DTS results of field cut slabs at +40 °C

Sample	Peak Load (kN)	Displacement at Peak Load (mm)	Strain % (at peak load)	DTS (kPa)	Mean DTS
S7-2B	0.2589	0.4341	1.71	50.9	
S6-3B	0.3058	0.7441	2.93	58.9	54.9

Table 5.30 DTS results of Hot mix field compacted slabs at -30 °C

Sample	Peak Load (kN)	Displacement at Peak Load (mm)	Strain % (at peak load)	DTS (kPa)	Mean DTS
2-HL3	6.568	0.434	1.71	1122.1	1102.1
3-HL3	6.097	0.508	2.00	1044.2	
4-HL3	6.846	0.744	2.93	1140.1	

Table 5.31 DTS results of Hot mix field compacted slabs at 0 °C

Sample	Peak Load (kN)	Displacement at Peak Load (mm)	Strain % (at peak load)	DTS (kPa)	Mean DTS
HLB5	2.613	0.620	2.44	442.7	385.7
HLB6	1.880	0.484	1.90	328.7	

Table 5.32 DTS results of Hot mix field compacted slabs at +40 °C

Sample	Peak Load (kN)	Displacement at Peak Load (mm)	Strain % (at peak load)	DTS (kPa)	Mean DTS
HLC12	0.316	1.228	4.83	60.0	49.8
HLB7	0.221	1.005	3.96	39.6	

Table 5.33 Comparison of Mean DTS of Lab compacted samples at different temperatures

Temperature (°C)	Load Rate (mm/min)	Mean DTS (kPa)
-30	0.3	124.4
0	0.3	92.1
40	0.3	34.5

Table 5.34 Comparison of Mean DTS of Field cut samples at different temperatures

Temperature (°C)	Load Rate (mm/min)	Mean DTS (kPa)
-30	0.3	804.7
0	0.3	757.5
40	0.3	54.9

Table 5.35 Comparison of Mean DTS of HMA Laboratory compacted samples at different temperatures

Temperature (°C)	Load Rate (mm/min)	Mean DTS (kPa)
-30	0.3	1102.1
0	0.3	385.7
40	0.3	49.8

Table 5.36

Comparison of Mean DTS of Sample types
at different temperatures

Sample Type	Temperature (°C)	Mean DTS (kPa)
Laboratory Compacted	-30	124.4
CIR	0	92.1
	40	34.5
Field Cut	-30	804.7
CIR	0	757.5
	40	54.9
Laboratory Compacted	-30	1102.1
HMA	0	385.7
	40	49.8

Table 5.37 Moisture Sensitivity of Samples from Core Location 5

Sample	Condition	Load rate (mm/min)	Density (g/cm ³)	Indirect Tensile Strength (kPa)	Mean Tensile Strength (kPa)	Retained Strength, % (sat./dry)
C5-8	Dry	12.7	2.053	220.88	203.8	
C5-9			2.057	217.72		
C5-23			2.040	172.80		
C5-3	Saturated	12.7	2.072	35.70	32.8	16.1
C5-17			2.045	32.65		
C5-26			2.061	30.13		

Table 5.38 Moisture Sensitivity of Samples from Core Location 6

Sample	Condition	Load rate (mm/min)	Density (g/cm ³)	Indirect Tensile Strength (kPa)	Mean Tensile Strength (kPa)	Retained Strength, % (sat./dry)
C6-20	Dry	12.7	2.128	394.0	386.1	
C6-21			2.128	370.4		
C6-22			2.128	393.8		
C6-1	Saturated	12.7	2.100	49.2	44.3	11.5
C6-2			2.115	37.6		
C6-4			2.127	46.0		

Table 5.39 Moisture Sensitivity of Samples from Core Location 7

Sample	Condition	Load rate (mm/min)	Density (g/cm ³)	Indirect Tensile Strength (kPa)	Mean Tensile Strength (kPa)	Retained Strength, % (sat./dry)
C7-9	Dry	12.7	2.152	269.5	324.4	
C7-10			2.151	282.4		
C7-23			2.237	421.2		
C7-14	Saturated	12.7	2.218	35.2	36.9	11.4
C7-18			2.212	35.1		
C7-25			2.221	40.4		

Table 5.40 Moisture Sensitivity of Samples from Core Location 8

Sample	Condition	Load rate (mm/min)	Density (g/cm ³)	Indirect Tensile Strength (kPa)	Mean Tensile Strength (kPa)	Retained Strength, % (sat./dry)
C8-3	Dry	12.7	2.098	174.1	174.8	
C8-8			2.123	167.5		
C8-18			2.114	182.7		
C8-15	Saturated	12.7	2.114	19.4	17.0	9.7
C8-19			2.103	16.2		
C8-20			2.094	15.4		

Table 5.41 Summary of Moisture Sensitivity Test Results

Sample	Load rate (mm/min)	Density (g/cm ³)	Mean Tensile Strength (Dry) kPa	Mean Tensile Strength (Sat.) kPa	Retained Strength, % (sat./dry)	Mean (%)	Standard Deviation	COV (%)
C5	12.7	2.055	203.8	32.8	16.1			
C6	12.7	2.121	386.1	44.3	11.5			
C7	12.7	2.199	324.4	36.9	11.4			
C8	12.7	2.108	174.8	17.0	9.7	12.2	2.7	22.6

Figure 5.2 Comparison of Mean ITS of Sample Types at Various Temperatures

Figure 5.3 : Comparison of DTS of Various Sample Types Over Range of Temperatures

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

The objectives of this research were to provide field data to support visual observations made by the Regional Municipality of Ottawa-Carleton which indicated that Cold In-Place Recycling produced a material that reduced the frequency of transverse cracking, and to carry out laboratory testing program to determine the mechanical properties of the material which may explain the field observations.

Field data about transverse cracking were collected for a number of selected CIR with HMA overlay and CHOL sites. Time series plots of the transverse crack data were completed for both rehabilitation methods. Dynaflect Deflectometer data were also collected for the same sites. From this data, the radius of curvature and maximum deflection values were computed and

analyzed. The effect of subgrade variability on rehabilitation type was also determined using the PMS and Dynaflect data.

Laboratory testing was carried out on the CIR material that was placed in 1996. The testing included the bulk relative density, direct and indirect tensile strength, and stripping tests. Variables such as compaction method and test temperature were considered. The results were used to determine the effectiveness of conventional test methods in characterizing the CIR material.

6.2 Results of Field Investigation

The field data collected with the PMS and Dynaflect Deflectometer indicate that the CIR material effectively reduces the rate of thermal and reflective cracking. This supports the visual observations made by the RMOC.

The PMS data shows that over a seven year life span, both road types act quite differently. While transverse cracks appear on the CHOL roads in the first year after rehabilitation, cracking on the CIR with HMA overlay roads did not appear until after the first or second year. The number of cracks for both road types is similar at the end of the fourth year, however the CHOL roads show a rapid increase in transverse cracks between years five and seven. Crack propagation rate on the CIR roads, conversely, remains stable over this same time period. An examination of the pre and post rehabilitation road condition of one road of each rehabilitation type shows that the CIR material is able to slow down the rate of reflective cracking, while the CHOL asphalt cannot.

The main results of the Dynaflect testing show that the deflection values for the CIR roads are not indicative of actual performance. The Dynaflect data for the CIR pavement shows that the road is in worse shape immediately after rehabilitation than it was before. As this is not true, the Dynaflect should not be used to monitor the CIR rehabilitated roads in the same manner as it is used for the CHOL roads.

Finally, observations and analysis of field data concerning the subgrade material show that the CIR and CHOL roads perform the same on strong subgrades, however, the CIR structure is superior to the CHOL structure when placed over a weak subgrade. The CIR structure appears to negate the effect of the subgrade type on the rate of cracking.

6.3 Laboratory Results

Laboratory tests were carried out on material that was sampled from a Cold In-Place Recycling project. Extraction of field samples showed that the material required a minimum of three weeks to cure. The BRD, ITS, DTS, and moisture sensitivity tests were carried out on the laboratory samples.

The results of the BRD tests show that the Marshall samples simulate the field density. The gyratory samples, however, were much denser than the field cores showing that the use of conventional methods to choose the number of gyrations for the CIR material do not produce a representative sample.

Testing the samples for the ITS show an increase of strength with a decrease of temperature for all specimen types. While the ITS of the Marshall samples were the same as

those of field cores at +20 °C and 0 °C, the strength of the field cores was higher than that of the Marshall samples at -20 °C. This suggests that the Marshall specimens do not represent in-situ conditions at very low temperatures.

The DTS results highlighted the sensitivity of the CIR material to temperature. For all samples, the strength increases with a decrease in the temperature. The results, however, show that the CIR material has very little strength at high temperatures. In addition, the DTS of the CIR material at the low temperature is less than the measured DTS of the hot mix material. In addition, it is shown that CIR slabs cannot be reproduced in the laboratory.

The moisture sensitivity testing was carried out on field cores. The conclusive results show that the material is extremely sensitive to water and will exhibit no strength when saturated.

6.4 Recommended Mechanism for Crack Retardation by the CIR Structure

The laboratory and field performance results were used to characterize the CIR material and explain its ability to reduce the frequency of thermal cracking. A proposed mechanism to explain the behaviour of the CIR pavements is discussed in the following sections.

6.4.1 Removal of Cracks

The aged asphalt surface before rehabilitation is given in Figure 6.1 which shows the existing reflective and surface transverse cracks. The first stage of the CIR process involves milling the existing asphalt to a depth of approximately 50 to 70 mm. This effectively removes

all cracks that exist in that thickness. The only cracks that remain are those which originate at the bottom of the pavement layer and have reflected upward. The new CIR base layer is placed over the old and cracked existing asphalt. The construction steps which follow, including compaction and application of the surface course, are critical in the CIR process.

6.4.2 Asphalt Compaction

The compaction mechanisms must be discussed before applying them to the CIR procedure. Halim [1985, 1986] shows that during compaction, the top layer will crack due to the influence of the relative rigidity of the underlying layers. Furthermore, Halim shows that as the stiffness of the layer immediately under the layer being compacted increases, the number of construction induced cracks increases.

Applying this concept to the CIR process, it has been shown that the CIR material is most successful on weak subgrades. This suggests that as the CIR material is being compacted on top of the clay subgrade, very few construction cracks are induced. After compaction of this layer, a hot asphalt overlay is placed at approximately 130 °C. The temperature sensitive CIR layer has very little strength at high temperatures. Thus, compaction of the HMA layer occurs over a relatively weaker layer, again causing few construction induced cracks. Figures 6.2 and 6.3 show the effect of this mechanism on the final product of the CIR and the CHOL processes, respectively. The performance of each rehabilitation type is discussed in the next section.

Figure 6.1: Existing Cracked Asphalt

50 mm
CIR

Existing
Asphalt
Layer

(a) Mill existing asphalt and place CIR layer

40 mm
HMA

50 mm
CIR

Existing
Asphalt
Layer

(b) Place hot mix layer as surface course

Figure: 6.2 Mechanism for Crack Retardation by CIR

(a) Place hot mix overlay without milling existing layer

Figure 6.3: Inability of CHOL to stop reflection cracks

6.4.3 Reducing Crack Propagation

Examining Figures 6.2 and 6.3, it is evident that reflection cracking will be slowed down by the new 50 mm to 70 mm crack free layer. The number of construction induced transverse cracks are less for the CIR roads than the CHOL roads due to compaction of the HMA material over the less rigid CIR base rather than the old existing pavement layer. Therefore, the CIR roads will have less reflection cracks than the CHOL roads for a longer time, and begin their life cycle with fewer construction cracks. The effect of time on the CHOL road is given in Figure 6.3c.

The ITS and DTS results for the CIR material show that it is not exceptionally strong at low temperatures, and is very weak at high temperatures. The resistance of the CIR to thermal cracking, thus, cannot be explained in its tensile strength, rather in its lack of tensile strength. The stress in the HMA overlay induced by thermal loading normally causes transverse cracking to occur with time. Due to the lack of strength of the CIR material at high temperatures, it appears that the material will be forgiving to the expanding HMA overlay as the temperature increases from cold to warm. This will allow the material to move more freely under temperature cycles, resulting in less cracking.

It should be mentioned that a thorough cost analysis of the CIR versus the CHOL procedure was not carried out. Lee et al. [1997] outlined the cost savings for the RMOC when using the CIR. The conclusions of their analysis show the CIR with a HMA overlay process to be 25% more cost effective than the CHOL rehabilitation.

6.5 Recommendations

The presented research showed that Cold In-Place Recycling is an effective rehabilitation method for slowing down the rate of thermal cracking. The CIR process does not offer a solution to the thermal cracking problem, however by slowing down the propagation, significant life cycle cost savings can be realized.

Based on the field and laboratory work that was carried out, the following recommendations are made:

- Due to the moisture sensitivity of the material, the CIR layer must be sealed or overlaid.
- The Dynaflect Deflectometer should not be used to characterize the CIR material. The deflection values do not have the same meaning as they do for the HMA and therefore must be correlated to the specific material.
- The conventional method of choosing the number of gyrations for the Gyratory samples is not valid for the CIR material. Further work should be carried out to modify this process.. In addition, Marshall samples are representative of field cores in terms of density, however they do not represent the in-situ indirect tensile strengths at the -20 °C temperature. Therefore, it is important to modify the Gyratory process to produce samples of the correct density to see if the Gyratory samples can classify the CIR material at low temperatures.
- There are different types of modified CIR material being used in the industry. These different mixes should be tested in similar ways to determine the relationship to the conventional CIR material.
- The results show that the CIR appears to negate the effect of subgrade type on the rate of

transverse cracking. Further work should be carried out to verify this performance.

- An analytical model based on finite element techniques should be developed to investigate and verify the proposed analytical mechanism presented in this chapter.
- Carry out laboratory fatigue tests on the CIR material to obtain long term performance laboratory data.

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APPENDIX A

COLD IN-PLACE RECYCLED MIX

1. SCOPE

The work consists of cold in-place reclaiming of the existing asphalt pavement to an average depth of 75 mm, sizing, mixing with binder and water (if required). The work also consists of spreading and compacting the recycled cold mixed bituminous mixture, in accordance with this Special Provision, to the lines, grades, thicknesses and cross sections as determined by the Contract Administrator.

2. MATERIALS

2.1 Reclaimed Asphalt Pavement

The reclaimed asphalt pavement material shall be 100 percent passing the 37.5 mm sieve.

2.2 Binder

2.2.1 Specification

Polymer-modified high float emulsified asphalt HF-150M(P) meeting the OPSS 1103 requirements listed in Table VIII, Polymer (P) Modified Emulsified Asphalts shall be used.

2.3 Hot Mix Placement

Any Hot Mix HL4 used as required by Section 5 and Section 6 shall confirm to OPSS 1149.

3. DESIGN AND SUBMISSION REQUIREMENTS

Prior to commencing the work, the Contractor shall obtain representative samples of the material which will be produced during the reclaiming operation, to carry out the necessary laboratory testing to establish the design binder rate as a percentage by mass of reclaimed asphalt pavement to be used during production. A minimum of 1.0 percent binder by mass of reclaimed asphalt pavement shall be used.

APPENDIX B

Method of Test For Measuring In-Service Stripping of Asphalt Cores in The Laboratory

The twelve (12) cores obtained from each of the twenty-six (26) pavements shall be tested by following the methodology outlined in this Appendix. A summary of the type and number of tests to be performed on each 12 core set is given in Table A1.

Test Procedures

1.0 Preparation of Pavement Cores

- a) Obtain the 12 - 100 mm asphalt cores per pavement section under investigation.
- b) In consultation with Transport Canada, determine the critical construction layer for testing. Remove the core section representing the critical layer, using a masonry saw.
- c) Clean core section surfaces with brush and distilled water.

2.0 Determination of Bulk Densities

- a) Desiccate all core sections to constant dry weight.
- b) Determine bulk densities for all 12 cores in accordance with ASTM D2726.
- c) Divide the 12 cores into 3 groups of 4 cores each such that each group will have approximately the same average density.

3.0 Vacuum Saturation and Determination of Permeable Voids

Vacuum saturation at room temperature (22.7°C) is considered to be a means of watersaturating pavement cores to assess moisture damage.

- a) Desiccate all core sections to constant dry weight.
- b) Randomly select one (1) group of 4 core sections for vacuum saturation.

- c) Cores are placed in spacers in thick walled jars. The jars are connected in series to accommodate the simultaneous saturation of up to six cores. Jars are filled with distilled water to a height 2.54 cm above the specimens and cores and 45.0 cm below the top line of the jars. A wetted doughnut shaped flexible urethane gasket is placed on each jar line and an aluminum vacuum plate cover is placed on the gasket. Vacuum hoses from a vacuum pump are connected to the plate cover fittings, providing access to the inside of the jars.
- d) Vacuum saturation is achieved by the application of 66 cm of mercury vacuum to the cores submerged in the distilled water at 22.7°C for a duration of 30 minutes. During this time, the jar surfaces are gently agitated.
- e) After 30 minutes, the vacuum is removed and the inside of the jars is allowed to reach ambient atmospheric pressure. Cores are kept submerged in the jars under atmospheric pressure for a minimum of 30 minutes. A total of 60 minutes must therefore be allowed for the vacuum saturation procedure.
- f) Calculate permeable voids for each of the 4 cores using the relationship:

$$\text{Permeable Void \%} = \frac{\frac{(W_L^1 - W)}{L} + \frac{(W_a^1 - W)}{a} \times 100}{\frac{W_L^1}{a} - \frac{W}{L}}$$

where,

W_a = Weight in air of desiccated (dry) core section

W_L = Submerged weight of initially desiccated dry core section immersed in distilled water at 22.7°C for 3 minutes

W_a^1 = Weight in air of core section quickly blotted by towel (surface dried) after obtaining weight W_L

W_L^1 = Submerged weight of vacuum saturated core section in distilled water at 22.7°C

- g) At this point there are to be 4 dry core sections and 4 core sections which have undergone vacuum saturation.

4.0 Procedure for Determining Tensile Strength

- a) Cores are tested at 22.7°C, both dry and vacuum saturated.
- b) A flat loading block is placed on the core fitting through the tensile displacement device opening. The core is placed under the loading head of a compressive testing machine, set at the vertical deformation rate of 51 mm per min. at a 22.7°C test temperature. The maximum compressive load of the core is recorded. (For dry cores $2000 < P_{max} < 3000$ lb. and for saturated cores $10 < P_{max} < 40$ lb.) The load is then decreased rapidly to zero; the core is removed, and the procedure is repeated for the remaining cores.
- c) Each core after testing is measured for flattening (corresponding to maximum load condition) to the nearest 2.5 mm. A rub with a flat piece of chalk on top and bottom of the core makes a visual trace for easy measurement of flattening.
- d) After recording maximum load and measuring flattening, cores are further deformed at the prescribed vertical deformation rate until a vertical tensile crack opens. The cores or specimens are then separated in half and their interfaces examined for visual moisture damage, such as stripping.

- e) The tensile strength is calculated from the indirect tensile test results using the following equation:

$$S_t = \frac{S_{10}}{10,000} \cdot \frac{P_{max}}{L} \quad \dots(1)$$

where,

$$S_{10} = 1591 + 437a - 1889a^2 + 2854a^3 - 2474a^4 + 885a^5$$

S_t = tensile strength in psi

S_{10} = maximum tensile stress, in psi, produced in a 4 inch diameter solid cylinder by a load P of 10,000 lb./thickness of 1 inch

a = amount of core flattening, in inches, under P_{max} (0 to 1 inch)

P_{max} = maximum compressive load on core in lb.

L = thickness of core in inches.

(Note: For convenience, a graph of " S_{10} " vs " a " could be made using the equation given to enable the selection of S_{10} on a routine basis).

APPENDIX C

Appendix C: Results of Linear Regression Analysis

Figure	Linetype Fit	R ²	Equation
4.1: CIR with HOL, 1997	Linear	0.8000	$y = 6.6972x - 2.5188$
	Polynomial (2nd degree)	0.8083	$y = -0.3421x^2 + 9.092x - 4.9136$
	Polynomial (3rd degree)	0.8843	$y = -0.5488x^3 + 5.4202x^2 - 5.9997x + 0.8487$
4.2: CIR with HOL, 1994/95	Linear	0.6732	$y = 6.378x - 1.162$
	Polynomial (2nd degree)	0.6821	$y = 0.62x^2 + 3.898x + 0.078$
	Polynomial (3rd degree)	0.7184	$y = -1.2342x^3 + 8.025x^2 - 6.7158x + 1.559$
4.4: CHOL, 1997	Linear	0.4304	$y = 6.4391x + 12.37$
	Polynomial (2nd degree)	0.7156	$y = 3.5888x^2 - 25.86x + 74.576$
	Polynomial (3rd degree)	0.8662	$y = -1.9799x^3 + 30.318x^2 - 136.14x + 210$
	Power	0.2331	$y = 18.266x^{0.5093}$
	Exponential	0.3596	$y = 18.547e^{0.1574x}$
4.5: CHOL, 1994/95	Linear	0.9099	$y = 11.217x + 0.27$
	Polynomial (2nd degree)	0.9202	$y = 0.7548x^2 + 7.5042x + 2.514$
	Polynomial (3rd degree)	0.9957	$y = 1.9822x^3 - 14.326x^2 + 34.704x + 0.4296$
	Exponential	0.7964	$y = 1.314e^{0.892x}$

APPENDIX D

Density of Marshall compacted samples from sample location 2

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
M2-1	0.945	0.5492	1.0018	2.088	2.092	0.012	0.6
M2-2	0.941	0.5467	0.9972	2.089			
M2-3	0.970	0.5559	1.0217	2.082			
M2-4	0.926	0.5254	0.9644	2.109			

Density of Marshall compacted samples from sample location 3

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
M3-1	1.005	0.5800	1.0513	2.132	2.157	0.019	0.9
M3-2	0.993	0.5721	1.0373	2.134			
M3-3	0.943	0.5301	0.9631	2.178			
M3-4	1.005	0.5642	1.0267	2.173			
M3-5	0.934	0.5193	0.9503	2.167			
M3-6	0.980	0.5484	1.0022	2.159			

Density of Marshall compacted samples from sample location 4

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
M4-3	0.986	0.5451	0.9850	2.241	2.117	0.093	4.4
M4-4	0.940	0.5693	1.0353	2.017			
M4-5	1.038	0.5893	1.0823	2.106			
M4-6	1.046	0.5909	1.0878	2.104			

Density of Marshall compacted samples from sample location 5

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
M5-1	0.971	0.5460	1.0154	2.069			
M5-2	0.940	0.5238	0.9786	2.066			
M5-3	0.938	0.5230	0.9690	2.103			
M5-4	0.987	0.5440	1.0244	2.055			
M5-5	0.876	0.4857	0.9072	2.079			
M5-6	0.890	0.4871	0.9211	2.051	2.071	0.019	0.9

Density of Marshall compacted samples from sample location 6

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
M6-1	0.955	0.5364	0.9877	2.117			
M6-2	0.940	0.5266	0.9694	2.122			
M6-3	0.969	0.5299	0.9952	2.083			
M6-4	1.006	0.5507	1.0408	2.053			
M6-5	0.952	0.5258	0.9806	2.094			
M6-6	0.965	0.5346	0.9895	2.121	2.098	0.027	1.3

Density of Marshall compacted samples from sample location 7

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
M7-1	0.997	0.5556	1.0181	2.156			
M7-2	1.003	0.5653	1.0272	2.172			
M7-3	0.978	0.5336	0.9899	2.142			
M7-4	0.995	0.5524	1.0197	2.129			
M7-5	0.956	0.5279	0.9765	2.132			
M7-6	1.006	0.5547	1.0245	2.142	2.146	0.016	0.8

Density of Marshall compacted samples from sample location 9

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
M9-1	0.974	0.5393	1.0039	2.096			
M9-2	1.009	0.5508	1.0275	2.116			
M9-3	0.958	0.5248	0.9730	2.138			
M9-4	1.053	0.5743	1.0702	2.123			
M9-5	1.023	0.5632	1.0483	2.108			
M9-6	1.043	0.5771	1.0702	2.115	2.116	0.014	0.7

Density of Marshall compacted samples from sample location 10

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
M10-1	1.048	0.5730	1.0623	2.141			
M10-2	1.023	0.5605	1.0403	2.132			
M10-3	0.998	0.5426	1.0084	2.143			
M10-4	1.053	0.5669	1.0623	2.125			
M10-5	1.036	0.5649	1.0499	2.136			
M10-6	1.061	0.5831	1.0786	2.141	2.136	0.007	0.3

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
G1-1	4.180	2.4091	4.247	2.274			
G1-2	3.935	2.2887	4.022	2.270			
G1-3	4.062	2.3647	4.123	2.310			
G1-4	4.045	2.3572	4.116	2.300			
G1-5	4.065	2.3861	4.145	2.311			
G1-6	4.065	2.3829	4.145	2.307	2.295	0.018	0.8

Density of Gyratory compacted samples from sample location 3

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
G3-1	3.986	2.3370	4.042	2.338			
G3-2	3.997	2.3357	4.046	2.337			
G3-3	3.964	2.2920	4.000	2.321			
G3-4	3.972	2.2814	3.997	2.315			
G3-5	3.976	2.1974	4.010	2.194			
G3-6	3.969	2.0869	3.997	2.078	2.264	0.106	4.7

Density of Gyratory compacted samples from sample location 4

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
G4-1	4.027	2.3415	4.106	2.282			
G4-2	4.011	2.3338	4.077	2.301			
G4-3	4.006	2.3122	4.069	2.280			
G4-4	4.004	2.3121	4.060	2.291			
G4-5	4.011	2.2888	4.048	2.280			
G4-6	4.002	2.2810	4.045	2.269	2.284	0.011	0.5

Density of Gyratory compacted samples from sample location 5

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
G5-1	4.056	2.3225	4.109	2.270			
G5-2	4.059	2.3194	4.101	2.278			
G5-3	4.033	2.2808	4.056	2.272			
G5-4	4.026	2.1500	4.053	2.116			
G5-5	4.037	2.2825	4.059	2.272			
G5-6	4.038	2.2766	4.063	2.260	2.245	0.064	2.8

Density of Gyratory compacted samples from sample location 6

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
G6-1	4.048	2.3293	4.064	2.334			
G6-2	4.056	2.3431	4.076	2.341			
G6-3	4.038	2.3136	4.062	2.310			
G6-4	4.042	2.3188	4.065	2.315			
G6-5	4.026	2.3062	4.041	2.321			
G6-6	4.042	2.3163	4.064	2.313	2.322	0.012	0.5

Density of Gyratory compacted samples from sample location 7

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
G7-1	4.025	2.3200	4.039	2.341			
G7-2	4.035	2.3199	4.048	2.335			
G7-3	4.018	2.3002	4.031	2.321			
G7-4	4.021	2.3002	4.030	2.325			
G7-5	4.029	2.3067	4.045	2.318			
G7-6	4.022	2.3000	4.035	2.318	2.326	0.010	0.4

Density of Gyratory compacted samples from sample location 8

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
G8-1	4.041	2.3129	4.059	2.314			
G8-2	4.042	2.3195	4.062	2.320			
G8-3	4.022	2.2937	4.036	2.308	2.314	0.006	0.2

Density of Gyratory compacted samples from sample location 9

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
G9-1	4.059	2.3165	4.088	2.291			
G9-2	4.052	2.3108	4.083	2.286			
G9-3	4.034	2.2934	4.054	2.291			
G9-4	4.049	2.3019	4.075	2.284			
G9-5	4.032	2.2497	4.071	2.214			
G9-6	4.030	2.2933	4.069	2.270	2.273	0.030	1.3

Density of Gyratory compacted samples from sample location 10

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
G10-1	3.976	2.2660	3.991	2.305			
G10-2	3.975	2.2612	3.989	2.301			
G10-3	3.984	2.2716	4.000	2.305			
G10-4	3.982	2.2665	3.995	2.304			
G10-5	3.986	2.2699	3.998	2.307	2.304	0.002	0.1

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
C6-1	2.423	1.3432	2.497	2.100			
C6-2	2.376	1.3265	2.450	2.115			
C6-3	2.604	1.4537	2.676	2.130			
C6-4	2.424	1.3465	2.486	2.127			
C6-5	2.412	1.3319	2.466	2.127			
C6-6	2.492	1.3877	2.569	2.110			
C6-7	2.401	1.3355	2.472	2.113			
C6-8	2.405	1.3403	2.474	2.121			
C6-9	2.393	1.3257	2.457	2.115			
C6-10	2.910	1.5648	2.870	2.230			
C6-11	2.411	1.3330	2.473	2.115			
C6-12	2.416	1.3419	2.478	2.127			
C6-13	2.449	1.3501	2.514	2.104			
C6-14	2.418	1.3419	2.478	2.128			
C6-15	2.418	1.3419	2.478	2.128			
C6-16	3.568	1.8362	3.301	2.436			
C6-17	3.411	1.8201	3.266	2.359			
C6-18	3.381	1.8221	3.284	2.313			
C6-19	2.523	1.4050	2.567	2.171			
C6-20	2.418	1.3419	2.478	2.128			
C6-21	2.416	1.3419	2.478	2.127			
C6-22	3.596	1.9768	3.516	2.336			
C6-23	2.443	1.3606	2.482	2.179			
C6-24	2.418	1.3419	2.478	2.128			
C6-25	3.533	2.0100	3.556	2.285			
C6-26	3.698	2.0070	3.564	2.375	2.182	0.101	4.6

Density of Field compacted samples from sample location 7

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
C7-1	2.727	1.5183	2.789	2.146			
C7-2	2.754	1.5387	2.823	2.144			
C7-3	2.776	1.5455	2.839	2.146			
C7-4	2.790	1.5461	2.847	2.145			
C7-5	2.762	1.5246	2.818	2.135			
C7-6	2.762	1.5237	2.810	2.147			
C7-7	2.715	1.5068	2.775	2.141			
C7-8	2.792	1.5530	2.850	2.153			
C7-9	2.786	1.5534	2.848	2.152			
C7-10	2.832	1.5837	2.900	2.151			
C7-11	2.778	1.5434	2.832	2.156			
C7-12	2.911	1.6212	2.979	2.144			
C7-13	2.850	1.5806	2.906	2.150			
C7-14	3.108	1.7447	3.146	2.218			
C7-15	3.153	1.7630	3.173	2.236			
C7-16	3.164	1.7679	3.186	2.231			
C7-17	3.167	1.7704	3.196	2.222			
C7-18	3.178	1.7752	3.212	2.212			
C7-19	3.069	1.7245	3.104	2.225			
C7-20	3.126	1.7518	3.153	2.231			
C7-21	3.099	1.7269	3.116	2.231			
C7-22	3.023	1.6852	3.050	2.215			
C7-23	3.003	1.6807	3.023	2.237			
C7-24	2.912	1.6198	2.934	2.216			
C7-25	2.997	1.6698	3.019	2.221			
C7-26	3.055	1.7008	3.068	2.234	2.186	0.041	1.9

Density of Field compacted samples from sample location 8

Sample Label	Mass Dry (kg)	Mass in Water (kg)	Mass SSD (kg)	Density (kg/m ³)	Mean Density	STD	COV (%)
C8-1	4.655	2.6258	4.837	2.105			
C8-2	4.607	2.5713	4.750	2.115			
C8-3	4.615	2.5792	4.779	2.098			
C8-4	4.912	2.7747	5.115	2.099			
C8-5	4.893	2.7556	5.072	2.112			
C8-6	4.841	2.7210	5.009	2.116			
C8-7	4.092	2.3002	4.239	2.111			
C8-8	4.572	2.5739	4.727	2.123			
C8-9	4.797	2.6956	4.964	2.115			
C8-10	4.680	2.6226	4.841	2.110			
C8-11	Destroyed on Removal						
C8-12	4.753	2.6698	4.907	2.125			
C8-13	4.735	2.6586	4.906	2.107			
C8-14	4.623	2.6016	4.799	2.104			
C8-15	4.491	2.5338	4.658	2.114			
C8-16	4.626	2.6076	4.806	2.104			
C8-17	4.553	2.5655	4.725	2.108			
C8-18	4.452	2.5029	4.609	2.114			
C8-19	4.681	2.6429	4.869	2.103			
C8-20	4.759	2.6598	4.933	2.094			
C8-21	4.682	2.6338	4.849	2.114			
C8-22	4.503	2.5168	4.639	2.122			
C8-23	4.483	2.5148	4.633	2.116			
C8-24	4.144	2.3378	4.305	2.107			
C8-25	4.503	2.5292	4.685	2.089			
C8-26	4.179	2.3503	4.321	2.121			
C8-27	4.430	2.6058	4.802	2.017	2.106	0.020	1.0

IMAGE EVALUATION TEST TARGET (QA-3)

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