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STRIPPING OF ASPHALT CONCRETE SURFACES

by

EL HUSSEIN HASSAN MOHAMED

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

Department of Civil Engineering

Carleton University
Ottawa, Ontario
May 1991

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submitted by

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

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ABSTRACT

Stripping, recognized as the physical separation of asphalt cement and aggregate, has been identified as one of the major causes of pavement distress worldwide. Loss of adhesion due to stripping can cause virtually all types of failure, namely those resulting from instability, from disintegration or from cracking.

The severity and extent of stripping is primarily related to environmental conditions, materials, and inadequacy of the mix design. Based on the existing adhesion theories, quantification of stripping potential during the mix design stage has been difficult. Prediction methods based on existing knowledge failed to detect or evaluate the problem.

This thesis presents a comprehensive approach for investigating the problem of stripping. A new concept is developed and verified in the laboratory. It offers an acceptable explanation to the stripping problem and the asphalt/aggregate bond performance in general. Based on the considerable difference in the coefficients of thermal expansion of mineral aggregate and asphalt cements, the asphalt /aggregate bond is affected by differential contraction during thermal cooling. Differential contraction may also initiate internal cracks which allow water to enter the asphalt /aggregate interface, followed by stripping.

Observations made during the experimental program revealed that the separation following stripping took place in a location different than that stated in the problem definition. Based on these observations, the stripping problem is redefined in this thesis. The new definition will improve the overall understanding of the stripping phenomenon by explaining the effectiveness of the different adhesion theories. Improved understanding will help the efforts seeking a solution to the problem of stripping. This thesis developed a more reliable laboratory testing procedure to predict susceptibility of asphalt concrete mixtures to stripping.

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LIST OF SYMBOLS

Symbol	Description
AC	Asphalt Content
AMIR	Asphalt Multi-Integrated Roller
Hg	Mercury
ITS	Indirect Tensile Strength
NRC	National Research Council of Canada
α_1	Coefficient of Linear Thermal Expansion of Aggregate/°F
α_2	Coefficient of Linear Thermal Expansion of Asphalt Matrix/°F
E_2	Young's Modulus of Asphalt Matrix
E_1	Young's Modulus of Aggregate
ν_2	Poisson's Ratio for Asphalt Matrix
ν_1	Poisson's Ratio for Aggregate
ΔT	Change in Temperature T
psi	Pound Per Square Inch
SE	Sand Equivalent
Steel	Conventional Steel Roller(Static or Vibratory)
F	Degree Fahrenheit
TSR	Tensile Strength Ratio
T_{cr}	Critical Temperature
σ_t	Tangential Stress
σ_R	Radial stress
F/T	Freeze-Thaw Cycles
σ_{wa}	Interfacial Tension Between Water and Aggregate
σ_{ab}	Interfacial Tension Between Aggregate and bitumen
σ_{bw}	Interfacial Tension Between bitumen and water

CHAPTER 1

REVIEW OF STRIPPING OF ASPHALT CONCRETE

1.0. INTRODUCTION

Early failure of asphalt concrete surface layers has been attributed in many cases to stripping. Every year many miles of newly laid bituminous pavements must be replaced in Canada due to stripping. Jimenez (1) reported that stripping resulted in complete failure of asphalt pavements within two weeks after opening to traffic and failure by loss of surface fines over a period of two years. Increased maintenance and rehabilitation costs and decreased funds to preserve the pavement's serviceability, motivated many agencies to investigate the phenomenon of stripping. The Ontario Ministry of Transportation performed a survey in 1964. The results (2) revealed that stripping was evident throughout Ontario. In the U.S., the NCHRP report No. 274 (3) is a literature review on the subject indicating a similar degree of stripping damage occurrence.

Stripping is a very complex phenomenon influenced by many variables, all of which have some effect on the adhesion between aggregate and binder used. This thesis defines the problem of asphalt stripping, presents existing theories and identifies the deficiencies of the present approaches in dealing with the problem. It also investigates the effect of field compaction on stripping susceptibility and introduces differential contraction during thermal cooling as a factor capable of initiating stripping in the field.

1.1 SCOPE OF THE INVESTIGATION

The objectives of the investigation reported in this thesis are:

1. To evaluate the present approach to the stripping problem including the problem definition, adhesion theories, stripping mechanisms and methods used for prediction of stripping susceptibility of compacted asphalt concrete mixtures. The purpose of evaluation is to reveal what causes the great deal of inconsistency experienced when laboratory prediction method based on the existing knowledge are used to evaluate material resistance to stripping damage.
2. To investigate two factors that have a significant influence on stripping. These two factors are:-
 - a- differential contraction of asphalt cement and aggregate during thermal cooling and
 - b- field compaction technique used during construction
3. Based on 1 and 2 above, an attempt will be made to identify the route used by surface water to reach the asphalt/aggregate interface. Water must reach the interface before any of the stripping mechanisms may start functioning. Locating this route is of great significance to the efforts seeking a preventive measure against stripping damage.
4. To develop a more reliable laboratory testing procedure to predict susceptibility of asphalt concrete mixtures to stripping. Figure 1.1 is a flow chart summary of these objectives and the overall approach of the investigation.

1.2 ORGANIZATION OF THE THESIS

This thesis is organized to reflect the author's approach to the problem. Instead of attempting to force stripping in the laboratory and then investigate the factors that may have contributed to the problem, this study will consider external variables first and then investigate how these variables may contribute to the stripping of asphalt concrete.

Chapter 2 discusses the existing approach and identifies its deficiencies. The factors evaluated include adhesion theories, stripping mechanisms and laboratory testing procedures.

Chapter 3 presents differential thermal contraction and expansion as a factor that influences stripping resistance of asphalt concrete mixtures. A theoretical model is also derived to estimate thermally induced stresses in the asphalt matrix surrounding aggregate. The chapter also discusses how low temperature cracking may result in initiating stripping damage.

Chapter 4 discusses compaction and its influence on stripping. The chapter describes construction cracks and how they may function as a water path to water accumulation areas inside the compacted mixture.

Chapter 5 outlines an experimental program designed to verify the findings of Chapters 2, 3 and 4.

Chapter 6 presents observations made during construction, coring, curing, indirect tensile testing and after cores failure. The chapter also presents conclusions based on these observations.

The discussion of the results of the experimental program and their significance are provided in Chapter 7.

Chapter 8 summarizes the thesis, and presents the principal findings.

Chapter 9 presents areas for continuing research and recommendations for application of the research findings.

FIGURE 1.1: OVERALL APPROACH OF THE INVESTIGATION

1.3 PROBLEM DEFINITION

Over the years, a great deal of basic and applied research has been conducted to study the nature of the stripping phenomenon. Stripping is recognized as the physical separation of asphalt cement and aggregate produced by adhesion failure. Based on observations made during the experimental program, performed as part of this thesis, the author recommends changing the description of the failure location as stated in the problem definition. Chapter 6 discusses in detail the recommended change in the problem definition.

All stripping failures have been associated with presence of water. Virtually all investigators agree that stripping is caused by water displacing the asphalt from the aggregate surface (3 to 6) .

It is understood that under weathering and traffic conditions, water may cause loss of adhesion and consequently a bond failure in asphalt concrete mixtures. Bond failure will reduce the pavement strength to a certain degree depending on the amount of stripping that took place. Debonding can result in complete disintegration of asphalt concrete. A mild stage of bond loss will usually result in large decrease of tensile strength. Loss of bond also reduces pavement flexibility making it more prone to cracking.

Due to the nature of damage associated with stripping, it will only be manifested on the pavement surface in a late stage. Stripping, as observed in the field, usually starts at the bottom section of asphalt concrete layers. Simple routine survey will not detect stripping. The only definite method to detect stripping is to collect cores from the field and break them in the laboratory. Inspection of the core interior will then reveal the magnitude of the stripping damage.

The adhesion theories offered in the literature fail to explain completely the stripping phenomenon. Furthermore, the stripping mechanism associated with those theories also fails to explain what initiates stripping. Most important is the fact that none of the suggested concepts explain how water manages to reach the aggregate/ bitumen interface. The results of such deficiencies have been:

- 1- Quantification of stripping potential during material selection is inadequate.
- 2- Methods based on existing theories and used to predict stripping susceptibility fail to detect stripping potential in the field.

The author believes that the existing adhesion theories and the associated stripping mechanisms do not provide a full explanation of the problem. Therefore, more research is needed to develop a better understanding of the problem. Construction has long been ignored as an important factor that can influence stripping. A new compaction method introduced recently will be compared with the existing compaction techniques. Another factor associated with thermal cooling will be used to explain how low temperatures may influence stripping.

1.4 ECONOMIC FACTOR

The nature of the damage associated with stripping indicates the solution will not be a simple one. Overlaying areas affected by stripping will not solve the problem. Stripping will continue to cause deterioration and will soon affect the overlay. Seriously damaged sections will require complete removal of the existing asphalt concrete layer increasing the cost of construction. The need for continuous monitoring and maintenance will also mean disrupting traffic and reducing efficiency of already congested highways and city streets.

Highway agencies are now faced with the difficulty of scheduling construction and maintenance expenses. The failure of the existing prediction methods has led to the commitment of large and unexpected additional funds to effectively maintain the highway system.

The unscheduled expenses make it very difficult for highways to compete with other services required by the community. Moreover, planners find it difficult to justify the construction of large projects if the service life of such projects will be greatly reduced by the stripping problem.

The use of anti-stripping additives has been prevalent for several decades. However, their long term effects are not yet known. The use of these additives will also increase

the already high construction costs. The manner in which these additives achieve bond improvement is not fully understood. A better understanding of the problem may provide guidelines for the use of presently produced anti-stripping additives.

1.5 LITERATURE REVIEW

1.5.0 FACTORS THAT CONTRIBUTE TO STRIPPING

The difficulty involved in explaining the stripping problem is due to the fact that the adhesion of bituminous materials to aggregate surfaces is a complex phenomenon. According to previous investigations adhesion is affected by many factors related to the physicochemical properties of the binder/aggregate system. When water is added to this system the problem is even more complicated. Many of the factors that contribute to asphalt stripping have been identified (1 and 7) and the most important ones are listed below:

1- Aggregates

- surface texture
- aggregate absorption
- mineral composition
- cleanliness of fine aggregates

2- Asphalt-cement.

- surface tension
- chemical composition
- viscosity

3- The Mix

- density
- air void content
- asphalt content

4-External Factors

- climate (water and temperature)

traffic

construction techniques

Breaking field specimens, to check for stripping, revealed that stripping begins at the bottom section of the asphalt layer and slowly works its way upwards. Jimenez (1) called for differentiation between ravelling and stripping. Ravelling is the loss of cover materials of asphaltic surfaces occurring during or soon after a freezing rain. Complete disintegration or cracking of the asphalt layer that causes debonding of the asphalt from the aggregate is not a surface phenomenon as is ravelling (1).

1.5.1 ADHESION THEORIES

From the definition of stripping phenomenon the need of a better understanding of the adhesion theory is clear. In general, the bituminous mixtures used in road construction are basically a two-phase system consisting of the bituminous binder and the aggregate. The function of the bituminous binder is to form an adhesive bond between the binder and the aggregate. The intensity of the adhesion between the aggregate and the binder depends on the initial properties of both the binder and the aggregate, and on the binder - aggregate system after exposure to environmental and traffic conditions. The theories of adhesion cited in the literature are the mechanical, the chemical reaction, the molecular attraction and the interfacial energy theory. These are briefly discussed below.

The Mechanical Theory

This theory considers the mechanical interlocking of the adhesive with the surface of aggregates to be responsible for the initial adhesion. Therefore, the rougher the surface texture the better the adhesion. The rough surface of aggregate provides greater area of contact between the binder and the aggregate as well as help retards water coverage and infiltration. The presence of dust on the surface of the aggregate will prevent this type of adhesion (8).

The Chemical Reaction Theory

This theory considers adhesion as a result of a chemical reaction between asphalt and

aggregate constituents. When aggregate is wetted by the asphalt, adsorption occurs at the surface and is followed by a chemical reaction between adsorbed asphalt and aggregate. A recent study investigated the chemical functional group types in asphalt cement and their relative tendency to be absorbed on mineral aggregate surfaces (9). The theory requires a non acidic aggregate surface in order to promote the adhesion of asphalt (acidic in nature). "Active adsorption sites" on the aggregate surface are thought to be responsible for the adsorption process and their activity differs from one aggregate type to another. It is not yet known whether different surface densities of adsorption sites or "steric hindrance", which limit the accessibility to adsorption site, is the factor responsible for the different aggregate adsorption characteristics, (9).

The Theory of Interfacial Energy

According to this theory, the displacement of one liquid by another on a solid surface arises from the physio-chemical forces acting in the system. Here it is assumed that adhesion is the result of the interfacial energy relationship at the aggregate/bitumen-/air interface. The degree of spreading and wetting is a function of the surface tension, the interfacial tension and the adhesion tension of the phases involved. The adhesion tension of water/stone is known to be higher than that of bitumen/stone. This concept is widely accepted in explaining the phenomenon of adhesion and water effects (10). Considering the wide range in the physio-chemical properties of binders and the highly irregular surfaces and varied compositions of aggregates, the nature of interaction between hydrocarbons and aggregate becomes very complex. Figure 1.2 shows the interfacial forces acting at the mutual contact point M of a bitumen drop touching an aggregate surface in the presence of water. For equilibrium conditions at M (11):

$$\sigma_{wa} = \sigma_{ab} + \sigma_{bw} \cos \theta \quad 1.1$$

where,

- σ_{wa} is the interfacial tension between water and aggregate
- σ_{ab} is the interfacial tension between aggregate and bitumen
- σ_{bw} is the interfacial tension between bitumen and water

θ is the angle of adhesion.

from equation 1.1

$$\cos\theta = (\sigma_{wa} - \sigma_{ab}) / \sigma_{bw} \quad 1.2$$

$$-1 < \cos \theta < 1$$

when $\cos \theta > -1$ it is the spreading stage as shown in Figure 1.2(a).

when $\cos \theta < +1$ it is the stripping stage as shown in Figure 1.2(b).

Molecular Attraction Theory

According to this theory adhesion is the result of attraction between asphalt molecules and the outer molecules of aggregate. Interior molecules of a liquid are subjected to inter-molecular attraction which contributes mainly to the cohesive forces. Molecules at the surface of a wettable solid have a "free surface" and are exposed to a stronger field of forces, which is responsible for adhesion. The effective distance over which these forces act is different for polar and non polar liquids. Polar and non-polar terms are used to designate secondary forces between molecules. Polar molecules may have positive, negative, or dipole type active surface ; non-polar molecules have neutral surfaces (12). According to Mack (13), in non-polar liquid the free surface zone is probably never more than a few molecules thick. In polar liquids, to which asphalt belongs, this surface zone is usually large enough to be observed by a microscopic method. The principal effects of these forces are considerably increased viscosity and great elastic strength of the liquid near the surface of the solid. To explain this, Mack (13) suggested that the surface molecules of the aggregates orient themselves in the direction of their polarization, and this causes a similar alignment of the molecules within the liquid. This aligned layer

(a)

(b)

FIGURE 1.2 : INTERFACIAL FORCES ON ASPHALT-AGGREGATE-MOISTURE SYSTEM

AFTER REF. (11)

often extends to a distance of thousands of molecules from the surface. Mack reported x-ray diffraction patterns of such surface zones that tend to confirm this theory. This theory gives a rather logical explanation for the adhesion phenomenon. But, the existence of molecular orientation has not been fully supported by other investigators.

1.5.2 STRIPPING MECHANISMS

Modes of failure of bituminous mixtures could be adhesion failure or cohesion failure. The presence of water at the binder/aggregate interface may change the bond from proper to improper by displacing or detaching the binder from the aggregate. The expected failure will be an adhesion failure. The following hypotheses given in the literature are attempts to explain how failure may take place.

Detachment

As mentioned previously, the theory of interfacial energy is more popular than the other adhesion theories. The theory provides a rationale for explaining the detachment mechanism. The detachment concept considers the case where the bituminous binder is separated from an individual aggregate particle yet the bitumen coating remains intact. Detachment is usually attributed to replacement of the bitumen by a thin film of water, which may come from outside the binder or within the aggregate. As shown by equation 1.2 interfacial tension between water and aggregate and between aggregate and bitumen determines the stage of adhesion, namely; spreading or stripping. As explained earlier, the theory of interfacial energy emphasizes the effect of polarity of the molecules present at the surface of the two phases. Most aggregates have electrically charged surfaces. Asphalt, which is composed of high molecular weight hydro-carbons exhibits little polar activity; therefore, the bond that develops with aggregates is primarily due to relatively weak dispersion forces (14). Water molecules are highly polar and are attracted to aggregates by much stronger orientation forces.

Displacement

This concept assumes the presence of a partially coated aggregate where the water can reach the interface between the aggregate and the binder. Incomplete coating or rupture of the coating film will be explained later while discussing other stripping mechanisms. The concept of stripping by displacement is congruent with the interfacial energy approach to adhesion; that is, water will displace asphalt from an aggregate surface when a three- phase interface exists. The chemical reaction theory can be used, also, to explain stripping by displacement (15). Changes in the pH of the microscopic water accumulations at the mineral surface can alter the type of polar groups absorbed; as well as their state of ionization/dissociation, leading to the build-up of opposing, negatively charged, electrical double layers on the aggregate and asphalt surfaces. The drive to reach equilibrium attracts more water and leads to physical separation of the asphalt from the aggregate (15).

Spontaneous Emulsification

This concept and the following one results in either thinner film or in rupture of the asphalt film. Emulsification occurs when water and asphalt combine to form an inverted emulsion where asphalt represents the continuous phase and water represents the discontinuous phase (4 and 15). The rate of emulsification depends on the nature of asphalt cement and the presence of additives. This process is a reversible one because evaporation of the water from the emulsion returns the asphalt to its original condition (4).

Film Rupture

Rupture of the film of bituminous binder coating the aggregate particles can occur at the sharp corners of the aggregate, where the binder film is thinnest. Rupture may be initiated by the stress induced by the dynamic and kneading action of traffic. Once the binder is ruptured the water may reach the interface and stripping occurs by displacement.

Pore Water Pressure

This concept introduced by Lee and Nicholas (16) was simulated in the laboratory by Jimenez (1). It is thought that in higher void ratio mixes, such as base course materials, water may freely enter through the voids. Stresses induced by traffic may further compact the road mixture and trap or greatly retard the drainage of water. As a result of traffic; the water is in frequent motion and considerable pore pressure is building and destroying the bond between the aggregate and the binder resulting in stripping. This concept has not been developed further and no model is available to explain the mechanism described by Lee and Nicholas (16).

Repulsive Forces

In a concept suggested by Scott (15), the functional group types of an asphalt adsorbed on an aggregate surface consist mainly of the acid fraction of the asphalt. In the presence of water, the acid molecules are separated into two ions (e.g. carboxylic acid separates into $R-COO^-$ and H^+), causing the asphalt surface to have a negative polarity at the interface. In the presence of water, aggregates are negatively charged to varying degrees. As a result, a repulsive force develops between the negatively charged aggregate surface and the negatively charged asphalt surface at the interface, causing the separation of the asphalt from the aggregate surfaces i.e. stripping.

The intensity of repulsion developed between asphalt and aggregate depends on the surface charge of both the asphalt and the aggregate. Yoon and Tarrer (7) measured the surface charge of aggregates in water. Results indicate that aggregates which had a relatively high surface potential in water were more susceptible to stripping.

The following section discusses laboratory testing methods used to predict stripping susceptibility of field compacted mixtures. The strength of conditioned samples is usually compared with that of dry samples and the ratios indicates moisture susceptibility to stripping.

1.6 LABORATORY TESTS

The quantification of the stripping potential during material selection and mixture design phases remains difficult. Conventional specifications for asphalt mixtures do not totally evaluate the asphalt aggregate bond. A possible approach that would consider stripping with other mixture requirements could be achieved by following the conventional design method first, then evaluating the proposed mixture by conducting moisture susceptibility tests.

According to many investigators, field and laboratory data to date indicates that stripping is primarily an aggregate problem(17)(18). Aggregate mixture consists usually of several materials that are blended naturally or by the contractor to satisfy grading requirements. These individual materials and the total mixture vary in size, shape, surface texture and chemical composition. The existing approach evaluates stripping of material susceptible to moisture damage in either a mixture form (19)(20)(21) or by testing individual aggregate (22). However, there are anomalies that occur between results obtained from testing individual aggregate and from testing mixtures and they are not presently understood (21).

Research work is now centred around developing a suitable testing procedure for the measurement of stripping susceptibility of asphalt concrete and for determining anti-strip additive effectiveness. Anti-strip additives are compounds including various metal oxides, hydrated lime, metal salts of organic acids ...etc. They are used to improve the resistance of mixtures to moisture damage. At this time there is a certain controversy over the appropriate description of the possible chemical reaction between additives and asphalt cement (23). Stripping susceptibility tests usually include testing the effect of a certain anti-strip additive. Experts agree that the best test method to use is one in which the laboratory moisture-damage mechanism closely simulates what occurs in the field. The tensile type tests seem to be the most suitable ones for stripping investigations (24). The most commonly used test methods are discussed below and are given titles indicating the type of conditioning recommended prior to loading.

1.6.1 FREEZE-THAW CYCLES

Four water sensitivity tests currently in use are similar in nature, yet each method differs in the degree of exposure to water (saturation level) and the use of freeze-thaw cycles. Indirect tensile strength (ITS) is usually measured in order to compare conditioned specimens with the test results of unconditioned samples. Testing of conditioned samples is usually performed on saturated surface dry specimens (SSD). Lottman (19)(20)(25) developed three of the above mentioned tests. Samples are usually loaded after one freeze-thaw cycle. The step preceding the freeze-thaw cycles differs from one test method to another:-

- a. Method 1 requires vacuum saturation at a level of 24 inches of mercury (Hg) for 30 minutes. A level of saturation as high as 100% is expected in this method.
- b. Method 2 uses the same saturation level as method 1. Prior to testing of sample, hot water submergence at 140° F is required for a period of 24 hours.
- c. In the third method, the vacuum level is less than that of method 1 (14 inches Hg) and is followed by 30 minutes under atmospheric pressure. Level of saturation expected is between 55% and 80%.
- d. Scherocman (21) developed the fourth method which is quite similar to method 3 with one difference which is the use of multi-freeze-thaw cycles. The number of cycles depends on local weather conditions.

Copplantz and Newcomb (24) compared the four methods using a variety of field prepared mixtures obtained during construction. Test results indicated that stripping damage did not occur in specimens subjected to vacuum saturation alone. Freeze-thaw cycles were needed to properly determine sensitivity to water damage. High saturation levels resulted in increased damage to specimens, but the rank of relative water sensitivity of the mixtures was found to be the same.

1.6.2 PORE WATER PRESSURE

Jimenez (1) developed a new apparatus and procedure for measurement of stripping susceptibility where a saturated Hveem specimen was subjected to repeated pore water pressure. Water saturation took place at 122° F. Pore water pressure was used to simulate traffic loading effects on saturated pavements. The main features of the test procedure are outlined below:

- a. Saturation: specimen is placed in hot water (122° F) for 2 minutes. Then a 20 inches Hg vacuum pressure is applied for 15 minutes after which the specimen is placed in a hot water bath for 20 minutes.
- b. Cyclic stressing: loading is accomplished through a layer of water (1/2 inch above sample top). The cyclic load operates at a certain rpm in order to generate pressure within the saturated specimen ranging from 5 to 30 psi (a range comparable to that expected in saturated pavements under traffic).
- c. Strength test: after stressing and cooling, the tensile strength of the specimens is determined by the double punch test. The specimen must be tested while it is fully saturated. Jimenez used maximum load "P" in pounds, radius of punch "a" in inches, radius of specimen "b" in inches and height of specimen "H" to calculate tensile strength (psi) using the following formula to calculate the tensile stress σ_t (1);

$$\sigma_t = \frac{P}{\pi (1.2bH - a^2)} \quad 1.3$$

Jimenez performed a laboratory investigation in order to study the effect on tensile strength of the following :

1. Variable degrees of saturation that occur after the saturation process.

2. Variable stress duration

3. Relationship between sand equivalent (SE) and tensile strength as affected by asphalt content (AC)

The data obtained revealed a possible relation between degree of saturation of the different aggregate mixtures and their resistance to stripping. A degree of saturation exceeding 100% was attributed to the increase of permeable voids in the mineral aggregate from application of about 5800 repetitions of the fluctuating pore water pressure, an increase in gross volume and to satisfying aggregate absorption after asphalt film rupture.

The data also indicated that resistance to stripping was improved with increased AC and SE as expected based on prior experience (1). Immersion compression testing indicated higher retained strengths for mixtures having higher viscosity.

1.6.3 TEXAS BOILING TEST

Kennedy (22) developed a test procedure based on the currently used boiling test. The test is a rapid method to evaluate moisture susceptibility of an aggregate-asphalt mixture prior to using the mixture in the field. This method is based on the assumption that stripping is primarily an aggregate problem.

The test is performed on mixtures while in loose state which disregards the effect of compaction on moisture susceptibility. In this test, a visual observation is made to determine the extent of stripping after the mixture has been subjected to the action of boiling water for a specified period of time.

1.6.4 OTHER TESTS

Unconfined Compression (ASTM D 1075)

The immersion compression test is the only existing standard test known. The method uses a numerical index for the complete and compacted design mixture to measure the loss of cohesion resulting from the action of water. The exposed specimens are generally soaked for 24 hours in a water bath held at 77° F temperature. The index of retained

strength is obtained by dividing the strength of exposed specimens by that of control or dry specimens. Acceptable values of the retained strength vary from 55% to 75%.

Marshall Compression

The Marshall test method has been used to determine resistance to stripping in much the same manner as the immersion compression test.

The concept and test are based on characterizing the moisture damage sensitivity by long-term durability curves which express the variation of retained strength with hot immersion (60° F) for a period of 14 days.

Lottman (26) and Maupin (27) conducted fatigue tests to predict stripping susceptibility. Constant stress diametral tensile fatigue tests are lengthy and therefore the procedure is not widely used in moisture studies.

The following are deficiencies observed in the test procedures described before:

- 1- Prediction methods and their laboratory conditioning procedures are not based on any of the adhesion theories and do not simulate the stripping mechanisms described earlier.
- 2- The described prediction methods fail to predict stripping susceptibility of field compacted mixtures.
- 3- The use of laboratory prepared samples in prediction methods has been another element contributing to the failure of these methods in simulating field conditions.
- 4- Recently developed conditioning procedures do not include freezing as an element in the conditioning cycle (24). Such action may not be accurate since freezing may affect the bond between asphalt cement and aggregates.

All these factors will be evaluated in chapter 2. The large variation found in test results will also be discussed.

1.7. COMPACTION

1.7.0 GENERAL

Expenditures on modern multi-lane highways may average from one million to two million dollars per mile, but the final construction operation of rolling the asphalt surface of these highways costs only a few cents per square yard. This actually has led to the casual manner in which the rolling operation is often performed. In the motorist's mind any resulting rough riding surface can cancel out all the care and expense for high quality material and for the excellent engineering and workmanship that may have gone into every other part of the roadway structure (29). Therefore, one of the main objectives of compaction by rolling should be to provide finished pavements with nearly perfect smoothness of ride.

Based on the existing mix design methods, compaction by rolling should also be able to attain the specified density with minimum rolling effort and without shoving, cracking or checking. Density of compacted pavements is used in specifications because of its relationship with Marshall Stability. Such a relation was established by Lefebvre (30) and is shown here in Figure 1.3. The density provided by Laboratory compaction using 60 blows on each phase by the Marshall mechanical compactor is considered 100% and the specified degree of compaction obtained by rolling during construction should be as close as possible (95 to 98%) to laboratory-compacted density.

The range of 95 to 98% appears to be the maximum that can be achieved by conventional steel wheel rollers, which have been the principal compaction equipment for many decades. Completion of compaction of hot-mix asphalt binder and surface courses to 100% of laboratory-compacted density is usually left to traffic.

Another controlling factor included in the design is the percent air voids at 100% laboratory-compacted density. For dense graded asphalt concrete mixtures the percent air voids are specified to be 3% to 5%. This is expected to allow for a slight amount of additional compaction under traffic loading without flushing, bleeding and loss of stability, yet low enough to avoid rapid hardening of the asphalt binder.

FIGURE 1.3: MARSHALL STABILITY Vs. PERCENT LABORATORY COMPACTED DENSITY

AFTER REF. (30)

1.7.1 FACTORS INFLUENCING COMPACTION

Each of the following factors can have a marked influence on the efficiency and effectiveness of compacting asphalt concrete to the specified density by rolling operation (29):

- Viscosity-temperature characteristics of the asphalt cement.
- Temperature of the mix.
- Gradual increase in density and stability of the mix as rolling proceeds.
- Rate of cooling of the mix behind the spreader.
- Type of rolling equipment.
- Low versus high viscosity asphalt-cement.
- Aggregate gradation in the mixture.

Viscosity-Temperature Characteristics Of Asphalt Cement

Viscosity of the asphalt-cement represents a major source of resistance to the compaction of a paving mixture. Compaction of a paving mixture to the specified density is facilitated when the viscosity is low, but is made more difficult when the viscosity is high. Figure 1.4, by Lefebvre, shows how rapidly the viscosity of an asphalt cement changes with the increase or decrease in temperature. Figure 1.5 shows the effect the viscosity-temperature characteristics can have on Marshall Stability.

From the point of view of the job the roller must do in the field, Marshall Stability is also a measure of resistance to compaction. Figure 1.5 shows that resistance to compaction can increase by 10 times as the rolling temperature changes from 275° to 145° F. The low viscosity-asphalt in a mix in hot summer days has been considered as the reason for tender mixes (29). The stability of a tender mix is usually very low and not enough to carry the weight of the roller. A change to a high viscosity asphalt cement increases the Marshall Stability sufficiently to carry the weight of the roller, but also increases the resistance to compaction requiring an additional compaction effort in order to achieve the specified density.

FIGURE 1.4 : COMPARISON OF VISCOSITY-TEMPERATURE CURVES

AFTER REF. (30)

FIGURE 1.5: INFLUENCE OF HIGH AND LOW VISCOSITY ASPHALT CEMENT ON MARSHALL STABILITY

AFTER REF. (29)

Mix Temperature Effect

According to an investigation performed by Parker (31), The temperature of compaction can have an important influence on the air voids content, and therefore on the density of a paving mixture. For example, compacting a particular mixture at 150° F can result in an air voids value 4 times as large as that resulting from its compaction at 275° F. The principal initial difference between the paving mixture at 150° F and 275° F is the much higher viscosity of asphalt-cement in the mix at 150° F due to its lower temperature. This higher viscosity of the asphalt binder is the source of the greater resistance to compaction, which is responsible for the four fold increase in air voids that can take place if the mix is compacted at 150° F.

Gradual Increase Of Density and Stability

The paving mixture leaves the spreader while its density is somewhere in the vicinity of 75 to 85% of laboratory-compacted density. Both density and stability increase as compaction proceeds, Figure 1.3. This increase in density and Marshal Stability resulting from each pass increases the resistance to additional compaction by the next roller pass. The effect of rolling equipment type will be discussed in the following section.

1.7.2 ROLLING EQUIPMENT

Essentially, any rolling pattern attempts to overcome the resistances to deformation within the mix which increase exponentially with each compaction pass. The total resistance is composed of friction, interlocking, cohesive and viscous components. Three stages of rolling have been defined in the literature as follows (29):

Breakdown Rolling

At this stage, the paving mixture is leaving the spreader at a relatively high temperature. The resistance to rolling is usually at its lowest state. Field observations indicate that steel rollers (pressure 250 to 350 lb/lineal inch of roller width) usually settles into the mix so far that it causes detrimental lateral displacement of the mix. The steel roller must settle into the mix until the area of contact between the roller wheel and the

mix multiplied by the resistance of the mix is equal to the weight on the roller wheel. The above mentioned displacement is quite harmful and can result in cracking. On the other hand, if a roller has less pressure, i.e. less than the mix can support without injurious lateral deformations, and each pass of the roller is therefore less effective than it should be with respect to increasing the density of the layer being compacted. The result will be low values of density. One of the current standard solutions described for this situation is to allow the mix to cool before breakdown rolling begins. This is unusual in an engineering sense because the result will be an increase in the resistance of the mix to compaction. This situation led to comments such as that by McLeod (29) when he stated that mixes are only tender when the wrong type of rolling equipment is used to compact them. Similar observations were made when pneumatic-tire rollers replaced steel wheel rollers for breakdown rolling. Tire inflation pressure of about 50 psi caused considerable lateral displacement. However, by adjusting the inflation pressure of the tire to 40 psi, the paving mixture suffered less displacement (29). This has led to the idea of adjustable tire pressure rollers. If such a system is to be adapted the operator should be able to quickly adjust the tire pressure from 25 to 50 psi, from 50 to 70 psi and from 70 to 100- psi. The adjustments from one pressure to another, if proven practically possible, should suite the mixture requirement following the exponential increase in compaction resistance.

Intermediate Rolling

Intermediate rolling can be continued immediately after breakdown rolling. Intermediate rolling with a steel roller can also cause excessive lateral displacement or movement occurs under the roller. Such condition can lead to cracking and to a loss of bond between the hot layer being rolled and the cold layer just underneath.

Intermediate rolling usually requires higher tire pressure when pneumatic rollers are used. Therefore, a fixed tire pressure at 50 psi may be less than what is required to further densify the asphalt concrete mixture.

Final Rolling

The objective of final rolling is to correct the surface because pneumatic rollers usually leave behind visible marks on the pavement surface. Steel rollers are usually used in this phase. As in the case of intermediate rolling, the pressure used in final rolling will usually be either more than the mix can withstand without hair cracking, lateral displacement, destruction of bond between the hot layer and the layer beneath, and loss of density; or it will be less than the mix could tolerate as far as increasing pavement density is concerned, and the efficiency per pass will be quite low.

1.7.3 VIBRATORY COMPACTION

After the success of vibratory rolling in compacting cohesionless soils in the 1950's, some trials were made on bituminous concrete with but poor results (32). The relatively low frequency and high amplitude are ideal properties for compaction of cohesionless soils but unsuitable for viscoelastic bituminous concrete. Some adjustments were then made to improve the functions of vibratory compactor to suit asphalt concrete. Vibratory rollers have been expected to increase efficiency of compaction operation by attaining the specified density with less number of passes. The vibrations eliminate, to a large degree, internal friction in the material which creates improved conditions for compaction of the asphalt material (33).

Variables In Vibratory Compaction

1. Variables associated with the mix characteristics.
2. Roller weight (static load).
3. Vibratory frequency.
4. Amplitude.
5. Dynamic force
6. Roller speed.
7. Number of vibratory drums.

From the long list of variables associated with vibratory compaction, it is clear that

it involves very complex relationships. Proper consideration of these variables is crucial to obtaining adequate compaction and a smooth pavement.

A review of the density results obtained with vibratory rollers on a specific site (34) indicated three areas of improvement over conventional static steel wheel rolling:

- Higher level of density (1.4%).
- Improvement in the variation of density (from 6% to 3.5%).
- Less number of passes needed to reach specified density

Some of problems associated with the use of vibratory compaction include (34):-

- Breakdown is still a problem specially over soft mixtures.
- When compacting over poor subgrade, vibration can cause serious yield and no increase in density.
- Requires thick lifts. Thin areas result in excessive aggregate breakage and low density.
- Vibration over bridge decks is not safe specially when resurfacing over an old bridge deck.
- More maintenance is required to keep rollers in good operating condition.
- Greater skill required of the roller operator. This is because of the new variables introduced such as:
 - a. The amplitude should be adjusted according to thickness of the lift.
 - b. Coarse gradation takes large amplitude.
 - c. High amplitude and fast rolling cause rippling and hopping.
 - d. High speed reduces density.

Due to the above mentioned variables; many highway organizations now require the construction of a test strip prior to starting construction to make sure that the roller is capable of satisfactory operation.

1.7.4 DEFICIENCY OF CURRENT COMPACTION METHODS

Geller (36) stated "compaction of asphalt-pavement in the field has always been emphasized as perhaps the single most important factor for achieving satisfactory pavement service life. Yet, compaction has always been somewhat of a puzzle." This is being reflected in the lack of adequate specifications for contractors to follow during construction. High variation in density across and along paved sections is a common phenomenon. Steel roller action results in cracks usually called checks. Although such cracks are common, they are not being considered a factor in pavement performance (37). Density and air voids remain the only two factors used to control compaction.

A recently introduced new compactor will be studied in the following section and compared with the conventional compaction method. AMIR, the new compactor, will make such a comparison possible for the first time. Comparison between the two methods will be limited to their ability to produce pavements with a better resistance to stripping damage.

The influence of cracks on compacted material properties will be discussed in more detail in Chapter 4.

1.7.5 ASPHALT MULTI-INTEGRATED ROLLER (AMIR)

Halim (38) to (47) investigated both conventional and new compaction procedures and compared their products. This section will describe the new compactor AMIR (35 to 47). Halim adopted the principle of "Relative Rigidity" to analyze the pavement structure at the time of construction (35). Results of the analysis indicated that during construction, the top surface of the asphalt layer is a critical interface and not the bottom of the asphalt layer as during service conditions. Distortion in the order of rigidities exists when a new layer is placed on top of a relatively rigid layer and a rigid load applied on top of this system.

According to Selvadurai (48) the transfer of stresses among various components of a multi-phase elastic material of a multi-component elastic structure is influenced by the relative stiffness characteristics of the separate components. Selvadurai suggested that when the primary mode of load transfer between a plate and the soil is flexural interaction

between the plate and the soil, then the relative rigidity parameter takes the form:

$$RR = (E_p/E_s) (t/a)^3$$

where,

E_p = the modulus of elasticity of the plate (Rigid or Flexible)

E_s = the elastic modulus of the loaded soil.

t = thickness of the plate

a = radius of the loading plate.

Figure 1.6, after reference (48), shows the typical contact stresses that result at the plate/ soil mass interface as the relative rigidity of the system varies from the flexible to the rigid case.

Following the above concept, Halim suggested a theoretical model for a pavement structure with a new asphalt layer on top of a rigid one. In order to establish a criterion that relates distortion to relative rigidity, a coefficient indicating the distortion criterion and defined as 'the slope of deflection of the deflected interface', was suggested. For one layer, the coefficient would represent the ratio between the radius of curvature of the two interfaces of the layer in question. From the above discussion, a pavement system with newly placed asphalt concrete and under compaction with conventional steel roller, represents a distorted system.

Halim's work identified two problems with the existing practice: first, the mismatch of the rigidities of the components of the pavement system, and second, the geometry of the steel roller. It was also proven that both factors will contribute to the initiation of construction cracks. To solve these problems the new compactor AMIR was designed. AMIR overcame both the rigidity and curvature problems. The new roller consists of two steel drums with a rubber belt integrating both drums into one. In this roller, the modulus ratio between the rubber belt and the asphalt is close to unity. The integration of the two steel drums with the belt has resulted in a flat surface. The surface is parallel to the bottom interface. Thus both the stiffness and geometry requirement of the relative rigidity parameter, RR, have been met.

The result of using AMIR compactor model in the laboratory proved the validity of

the mechanistic approach used (42). No cracks developed and the resulting pavement had a smooth even surface. The potential of this new approach led to the development of AMIR prototype shown in Figure 1.7. Samples for this study were collected from overlays compacted by both AMIR prototype and conventional steel roller from three different locations. The absence of construction cracks is expected to improve pavement resistance to stripping. Improved physical as well as mechanical properties will also mean better resistance to stripping.

1.7.6 INFLUENCE OF COMPACTION ON ASPHALT STRIPPING

Since only one field compaction technique existed before AMIR, no comprehensive study has been performed on the subject of stripping in the context of compaction. A laboratory investigation, performed by the author, compared between conventional and AMIR compaction methods indicated that compaction technique influences the stripping resistance of asphalt concrete mixtures (37). Chapter 4 discusses this factor and focuses on construction cracks as a possible route for surface water to the interior of the compacted mixture. Locating the water route to the asphalt/aggregate interface is of great significance to the efforts seeking a solution to the stripping problem.

**FIGURE 1.6: INFLUENCE OF RELATIVE RIGIDITY
ON STRESS AND DEFLECTION DISTRIBUTION**

AFTER REF. (48)

FIGURE 1.7: ASPHALT MULTI-INTEGRATED ROLLER (AMIR)

1.8 LOW TEMPERATURE PERFORMANCE OF ASPHALT CONCRETE MIXTURES

1.8.0 INTRODUCTION

Until early 1960, low temperature performance was neglected and pavement design did not include any design criteria in this area. Design was concerned with deformation, skid resistance and durability at high temperature levels. The rapidly increasing stability with decreasing temperature negated the requirements for any design criteria in low temperature performance. During the 1960s and the early 1970s low-temperature pavement performance became an important research topic. The non-load associated cracking was recognized as a problem in Canada in the early 60s (49) (50). This type of cracking, although associated with low temperature, is most noticeable during the spring where the temperature ranges from 25°F to 45°F. On many occasions, transverse cracking was observed after prolonged cold temperatures followed by a sudden rise in temperature (51). Some of the hypothesis offered to explain these cracks include:

- (1) Simple thermal contraction of the surface.
- (2) Base course restraint to contraction of the surface.
- (3) Sudden warming and subsequent weakening of highly stressed surface.
- (4) Shrinkage cracking of the subgrade and subsequent reflection cracking through to the surface layer.

Mechanisms based on these hypothesis were suggested by investigators and have been extended to predict the conditions under which restrained asphalt concrete specimens will fracture. Following a review of basic low-temperature rheology, the hypothesis will be reviewed in this section because of the author's belief that cold weather can influence the problem of stripping. In the last few years investigators abandoned the use of freezing which was considered a factor in many moisture damage investigations. Many of the recent studies only used warm water soaking at 140°F to force stripping in the laboratory.

1.8.1 BASICS OF LOW-TEMPERATURE RHEOLOGY

In many situations, a material that can creep adequately to relax stresses induced by thermal or drying shrinkage, differential settlement, etc., is desirable to prevent or minimize cracking the structure. For a typical asphalt concrete mix, Haas and Phang (52) described the typical creep flow as shown in Figure 1.8, and the viscous portion of these curves as shown in Figure 1.9. Figure 1.9, therefore, shows the amount of flow between the intersection of the linear portion of the curve with the vertical axis and the point of unloading, which occurs at 30 minutes in Figure 1.8, as a function of temperature. A critical temperature T_{cr} has been defined as the point at which viscous flow decreases and below which tensile stresses build up if temperature continues to drop.

Low-temperature asphalt rheology measurement has been obtained using asphalt recovered from asphalt concrete test specimens. Many attempts have been made to develop relationships for the prediction of asphalt concrete properties including viscosity, static modulus and dynamic modulus. Ruth, et al. (53), developed such relationships as shown by Figures 1.10 and 1.11. From Figure 1.10, the modulus of asphalt concrete increases substantially with increased asphalt viscosity. Asphalt viscosity increases due to reduction in temperature and age hardening. Many of the problems with complexity and simplification have characterized the study of the rheology of bituminous materials. Complexity and the effort to simplify it has led to a number of different approaches to the explanation of flow and viscosity of asphalt. Many of the differences observed seem to be due to differences in conditions rather than differences in test methods. One common simplification is to characterize the asphalt cement behaviour with temperature change. Asphalt behaves like ordinary liquids over a large temperature interval, within which they are Newtonian liquids. The lower limit of this temperature interval varies with chemical composition of the asphalt. Below that lower limit asphalt approximates an elastic solid material.

FIGURE 1.8: TYPICAL CREEP FLOW RESULTS FOR A MIXTURE TREATED AT DIFFERENT TEMPERATURES

AFTER REF. (52)

FIGURE 1.9: IDENTIFICATION OF CRITICAL TEMPERATURE FROM CREEP TEST RESULTS

AFTER REF. (52)

1.8.2 LOW-TEMPERATURE CRACKING

Many of the suggested mechanisms of low-temperature cracking relate the problem to the increase in the stiffness of the asphalt. Although stiffness also increases because of aging, the decrease in asphalt temperature not only increases the stiffness but also creates tensile stresses due to thermal contraction. The increased stiffness not only increases the magnitude of induced stresses, but also limits the dissipation of these stresses through plastic deformation. Under decreasing temperatures a micro-crack initiates at the surface when the thermally induced stress exceeds the tensile strength. Cracks have been observed on the exposed surfaces and at the bottom of asphalt concrete pavements. Elastic layer analysis assuming dual and single wheel loads indicates that the maximum tensile stresses and strains occur at the bottom of the asphalt layer making surface cracking unlikely to occur. Therefore, it has been recognized that thermal cracking occurs when creep of the asphalt is not sufficient to accommodate the contraction strain that results during fast cooling of the pavement.

Many attempts have been made to predict the critical condition when cracks may develop due to low-temperature conditions. The following is a summary of some of the methods suggested in the literature:

(1) Ruth, et al. (53), performed asphalt rheology measurements using asphalt recovered from asphalt concrete test specimens. Asphalt viscosity was used to develop relationships for the prediction of asphalt concrete properties such as those discussed previously and shown in Figures 1.10 and 1.11. Equations for prediction of mix parameters were then used to develop a simplified incremental thermal stress equation. The equation requires input such as pavement cooling rate, coefficient of thermal contraction, and asphalt viscosity temperature data. The equation will then compute the thermal stresses at each time-temperature increment during cooling of the pavement.

Criteria for cracking were based on incremental creep strain limits, failure stress, fracture energy and mix stiffness concepts. Values for the strains, stress, and energy at failure were developed from laboratory fracture test data and their relationships to recovered asphalt viscosity were established.

FIGURE 1.11: RELATION BETWEEN MIX AND ASPHALT VISCOSITY
AFTER REF. (53)

(2) Haas and Phang (52) considered a case where a micro crack is initiated at the surface when the thermal stresses exceed the tensile strength of the mix. Such cracks will then propagate through the full depth of the bituminous layer under more thermal cycles. Combined effects of temperature and properties of the bituminous components are used to predict cracking potentiality. The critical temperature, T_{crit} , the point at which viscous flow decreases and tensile stresses starts to build-up, as shown in Figure 1.9, has been used to determine the fracture temperature of the mix. The fracture temperature has been defined as the point at which the strain due to temperature dropping below T_{crit} intersects with the strain at failure, as shown in Figure 1.12. Haas, et al. (52), emphasized the necessity of using a slow rate of loading in the laboratory in order to simulate field conditions.

(3) In an effort to improve asphalt mix design, Monismith, et al. (54), adopted a mechanism where low-temperature response governs the selection of mix characteristics. According to Monismith, as the temperature at the pavement surface drops, there is a differential in pavement temperatures between the surface of the pavement and the interior since time is required for the 'cold' to be conducted into the system. The surface of the pavement attempts to contract, but this contraction is restrained by the lower segments. Accordingly, stresses will develop due to restraint of deformation. Initially, the stresses will be small since the stiffness of the mix is relatively low. However, as the temperature becomes colder, the tendency to deform is larger, but the lower segments of the pavement still prevent the deformation from occurring. Mix stiffness is also increased at the lower temperatures; the increased stiffness, coupled with the propensity for increased contraction eventually leads to tensile stresses at the surface which exceeds the breaking strength of the asphalt concrete and result, in turn, in surface cracking.

FIGURE 1.12: DETERMINATION OF MIX FRACTURE TEMPERATURE

AFTER REF. (52)

1.8.3 RECOMMENDED SOLUTIONS FOR LOW-TEMPERATURE CRACKING

Haas, et al. (52) listed factors influencing the problem of low-temperature cracking as shown in Table 1.1. It is known that climate effects have a very high influence on cracking but they are, of course, not subject to control, except in an interactive sense with regard to climate and bituminous properties. However, selection of softer, less temperature susceptible binders can significantly reduce cracking. This alternative may not always be available, or feasible when trade offs between their influence on other mix properties such as rutting and fatigue resistance are considered. Modification of asphalt cement to include additives such as sulphur has been investigated to reduce low-temperature stiffness but because of economic considerations, sulphur modifications see little use today (52). Reinforcement with geogrid to improve tensile strength is being investigated to establish its effectiveness for low-temperature cracking resistance. A perfect bond between asphalt surface layer and underlying granular base has been considered an advantage over poorly bonded surface with respect to low-temperature cracking. This is because the shrinkage, which has to be accommodated through relaxation is dependent on a composite coefficient of contraction, i.e., the mixture coefficient is effectively reduced to the lower coefficient of the granular base, and its high temperature.

1.8.4 THE INFLUENCE OF LOW-TEMPERATURE PERFORMANCE ON THE STRIPPING PROBLEM

Performance of flexible pavements at low-temperatures must be fully understood before improved design procedures can be developed. Based on the literature review of low-temperature cracking, no evaluation procedure indicates directly any link to the stripping problem. As mentioned earlier, investigators tend to eliminate the cold element from laboratory conditioning for stripping studies.

The author believes that low-temperatures could be an important factor in the stripping problem. The existing understanding does not give such an indication. In chapter 3 of the thesis, a new hypotheses will be introduced based on differential contraction of asphalt cement and aggregate resulting in fracture of the asphalt matrix around aggregate.

Table 1.1

**Low-temperature Cracking Factors and Relative
Influence for Identification of Feasible and Effective Alternatives**

Factors	Relative Influence on Mix Cracking Susceptibility	Remarks
A. <u>Exogenous</u>		
1. Climatic effects	v. high	
2. Pavement age	high	More years → more thermal cycles and prob. of extreme low temp.
3. Traffic effects	unknown	Possible raising of fracture temp. under high speed loading.
B. <u>Full or Partially Controllable</u>		
1. Asphalt binder properties	v. high	Temp. susceptibility and proper grade of binder are very important.
2. Mix properties		
a) stiffness var. with temp.	v. high	Mainly function of binder used.
b) coeff. of thermal contraction	high	Aggreg. type selection can be imp.
c) tensile strength of mix	low	Reinf. can increase strength
d) aggregate absorptivity	low	Selec. of less absorptive aggreg.
e) mix or binder modifiers	unknown	Modifiers can alter a number of properties
3. Pavement design		
a) subgrade soil type	med. to low	Designer may have little choice
b) base restraint conditions	med.	Increased base restraint can lower fracture temp.
c) asphalt layer thickness	med. to high	Increased thickness → less cracking.
4. Construction effects	unknown	Surface defects can be crack initiators

After Reference (52)

1.9 SUMMARY

The adhesion theories offered in the literature fail to explain completely the stripping phenomenon. The major deficiency in the suggested stripping mechanisms has been its failure to describe how water reaches the asphalt/aggregate interface. The adhesion theories fail to describe the response of the adhesive bond to temperature changes. These deficiencies are due to the fact that the previous efforts neglected the following factors:

- 1- The influence of construction induced cracks on the stripping problem
- 2- Differential contraction during thermal cooling and as a result of the large difference in the coefficients of thermal contraction/expansion of asphalt cement and aggregate and,
- 3- The effect of temperature changes on the bond performance.

These three factors, if clearly understood, will offer a better explanation to the stripping phenomenon. If these three factors are simulated properly in the laboratory then prediction of stripping susceptibility will be possible. The following chapter describes other factors to be considered for successful simulation of field conditions in the laboratory.

CHAPTER 2

EVALUATION OF PRESENT APPROACH

2.0. INTRODUCTION

Based on the problem definition, an evaluation of the adhesion theories described earlier seems necessary. An accurate description of the factors affecting adhesion between asphalt cement and aggregates is necessary if quantification of stripping potential is to be performed during material selection and prior to the mix design stage. An expanded knowledge of these factors will also be necessary for developing a prediction method to be used after the mix design stage.

2.1 EVALUATION OF ADHESION THEORIES

Published research work in the area of stripping reflects the general agreement among researchers that the existing adhesion theories do not completely explain the stripping phenomenon. The following are some typical problems encountered in trying to apply adhesion concepts to explain the occurrence of stripping:

1. Although the chemical reaction theory considers non acidic aggregate surface to promote the adhesion of asphalt to aggregate surfaces, there are areas where stripping was experienced with limestone aggregates(11). Limestone aggregates are not supposed to strip because of their basic structure. Few stripping problems were observed in some areas where granite was used but not in all cases. Granite, which is acidic in nature, is

supposed to strip and not adhere well as it did in some locations. Also, the different aggregate adsorption characteristics could not be explained as described in the theory (9). The complexity of asphalt cement chemistry causes the controversy over the nature of chemical adhesion and the nature of age hardening. More basic understanding of asphalt chemistry and its aging characteristics are needed before the chemical reaction theory can be used in the selection of compatible combination of aggregates and asphalt cements.

2. The theory of interfacial energy seems correct from the physicochemical point of view, however it is open to considerable criticism from a practical point of view. It is clear that the variables involved are difficult to determine experimentally. The theory could not be used in the material selection stage or as a prediction method for estimating stripping potential in the field. Considering the wide range in the physio-chemical properties of binders and the high irregular surfaces and varied compositions of aggregates, the nature of the interaction between hydrocarbons and aggregates becomes very complex. The variables involved are difficult to determine experimentally and therefore the theory is not practical to use in material quantification or mix evaluation for stripping susceptibility.

3. Yoon and Tarrer (7) performed a laboratory investigation to relate some measurable aggregate properties to stripping propensity of a mixture of aggregate and asphalt cement. Aggregates used were characterized in terms of their physical properties such as pore volume and surface area as well as their chemical/electrochemical surface properties. Boiling water test was used to determine aggregate stripping propensity when coated with certain asphalt cement. The amount of stripping was determined visually. The results of the study are summarized below:

- a. Surface roughness was used to estimate the degree of mechanical interlock. No strong correlation could be established between the degree of adhesion and surface roughness.
- b. Pore volume and surface area were used to scale the interfacial contact. No strong correlation between these physical properties and stripping propensity could be established.

- c. The chemical properties of the aggregate used varied significantly. In general, the aggregates that impart a high pH value to water in contact with their surface exhibited a high susceptibility to stripping. Limestone imparted a relatively high pH to contacting water. Instead of exhibiting a high susceptibility to stripping, limestone showed a high stripping resistance.

In general, chemical reaction may take place, mechanical interlock occurs, and energy relationships influence the adhesion. It appears that all of the foregoing concepts must be considered together in order to explain the behavioral aspects of binder aggregates system in asphaltic pavements, and none of these concepts can by itself provide a full or definite explanation of the adhesion phenomenon applicable to all cases. Therefore, quantification of stripping potential prior to the mix design stage remains difficult. Engineers cannot always select a combination of materials with higher stripping resistance based on the information provided by the adhesion theories. Engineers, faced with this fact, continue to design asphalt concrete mixtures satisfying other performance requirements and then proceed to evaluate the designed mixture to estimate its stripping susceptibility in the laboratory. The available prediction methods used in the evaluation have also proved to be unreliable in detecting stripping susceptibility. Reasons for failure of the existing prediction methods are discussed in Section 2.3. One major deficiency is associated with inability of the adhesion theories in describing the asphalt/aggregate bond performance with temperature changes. None of the adhesion theories consider different bond strength at different temperature levels. Temperature change is known to influence pavement durability and service behaviour. The success of the adhesion theories depends on linking the bond performance to temperature changes.

2.2 EVALUATION OF STRIPPING MECHANISMS

The six stripping mechanisms mentioned earlier explain, based on adhesion theories, the process leading to the physical separation of asphalt cement from aggregate surfaces. The deficiencies associated with the adhesion theories are inherent in the

stripping mechanisms. Also, all the adhesion theories require the presence of water at the asphalt/aggregate interface, but none of the stripping mechanisms succeeded in explaining how water manages to reach the interface in the presence of protective layers of pure asphalt cement and a mixture of asphalt cement and fine aggregate. A key element in developing a preventive measure against stripping damage is to describe how water enters the pavement and then replaces or detaches the bitumen from aggregate surfaces. Water must travel through the pavement to reach the bottom section of the asphalt concrete layer where stripping usually starts. The detachment concept assumes a film of water to replace the bitumen film. This water may come from outside the binder or within the aggregate. The displacement concept assumes a film rupture which strongly indicates a water source outside the binder. The concept of spontaneous emulsification also assumes an outside source of water.

Mack (13) suggested osmosis as a possible means by which water can penetrate and reach the interior of the mix. This concept assumes the diffusion of water through the asphalt membrane. For this to happen, there must be a considerable film of absorbed water or pockets of moisture in the pores which contain water. This water will then penetrate through the film by osmotic pressure. Thelen (8) dismissed this concept due to the slowness of this process compared with the much smaller time needed for stripping to occur and he suggested another mechanism. According to Thelen there is a possibility of formation of pin holes on the coated asphalt film around the aggregate shortly after mixing due to dust or other materials on the aggregate surface. These pin holes will allow water from the pores to enter the bitumen aggregate interface.

All of these concepts assume the presence of water in the pores adjacent to the interface which lead many agencies in many locations, such as Ontario, to specify air void percent as low as 2% to eliminate outside water source. Air void has been blamed as the cause of water accumulation and circulation inside the material. Hydrostatic pressure can then build up under traffic loading and cause the disbonding of surface particles. Specifying less than 2% of air voids was expected to make the pavement almost impervious to the surface water and eliminate possible hydro-static pressure. In Ontario, as reported by Haas (11), water still enters the pavement. Pavements with the lowest air voids content

suffer moisture damage disqualifying high air voids percent as a major factor promoting the ease of water movement through the pavement. Pore water pressure has been suggested as a stripping mechanism (1) (16). Stripping occurred in streets with no traffic, such as dead ends. Stripping in the laboratory occurs in the absence of pore water pressure.

The foregoing review shows that there is a clear lack of general and concrete theoretical explanation for the adhesion phenomenon. The role of water in promoting stripping and the path it uses to enter and circulate inside the pavement remains a puzzle. The theories given to explain the phenomenon are based on many parameters which are difficult to determine experimentally. Hence quantification of stripping potential remains difficult. This prompted the investigators to turn to the next available option which depends on the use of laboratory prediction methods to detect stripping susceptibility of asphalt concrete after the mix design stage.

2.3 EVALUATION OF THE EXISTING PREDICTION METHOD

The present approach uses laboratory conditioning and the test method described in Section 1.5 to predict the stripping susceptibility of asphalt concrete mixtures. Laboratory prepared samples are conditioned in the laboratory either through freeze-thaw cycles or by simply soaking samples in a warm water bath for 24 hours as described earlier. The strength ratio of conditioned and dry samples is then used to indicate the percentage of retained strength. High strength ratio indicates a high resistance to stripping and a low ratio indicates a high susceptibility to stripping damage. Investigations performed using the suggested prediction methods on selected materials from different locations, have shown that there is no generally applicable way to evaluate mixtures to determine their stripping susceptibility (11 and 56). The following are some of the factors that contribute to the failure of the current approach:

1. The present approach does not consider the actual strength of the mixture prior to conditioning and therefore the strength ratio is not an accurate measure of the probable mixture performance in the field.

2. The success of predicting stripping susceptibility of asphalt concrete pavements in the laboratory require accurate simulation of field conditions which includes preparation of samples and application of the appropriate temperature and moisture conditions. Both requirements have not been satisfied by the existing prediction methods. Specific problems associated with the existing laboratory test procedures are discussed in the following subsections.

2.3.1 USE OF LABORATORY PREPARED SAMPLES IN MOISTURE DAMAGE INVESTIGATIONS

Samples prepared in the laboratory by the Marshall hammer, kneading or static compaction methods have been used in moisture damage studies with no success (1)(5)(7). Physical properties of samples prepared in the laboratory are quite different than field cores. Other differences between the laboratory samples and field cores will be discussed in Section 2.4. Laboratory samples are usually prepared with certain density and air voids requirements. Irrespective of the specified degree of compaction, field results always show much higher air voids and less density. Laboratory samples show consistent results with respect to density and air voids, while field cores show high variation in these two properties.

Attempts were made to simulate field density and air voids in the laboratory; this included reducing the number of blows specified by Marshall mix design from 75 or 50 blows on each side (7) (21). The reduced number of blows were meant to produce 6 to 8% air voids similar to those produced in the field by the conventional compaction method. These attempts also failed to produce samples that would help predict stripping susceptibility of field compacted asphalt concrete. This alone can be considered as a factor against using laboratory samples in moisture damage studies since prediction methods continue to be unreliable.

Even if field density and air voids could be simulated in the laboratory, the present laboratory procedures still result in different aggregate distribution and particle orientation. The effect of different compaction methods will be discussed in detail in the following section.

2.3.2 HIGH VARIATION IN TEST RESULTS OF FIELD CORES

A limited number of cores, collected from the field, are usually tested in the laboratory in an attempt to correlate laboratory test results with actual field performance. Indirect tensile strength tests performed on field cores show high variation of results. This variation of results makes the intended correlation a difficult task. The following factors contribute to such high variation:

- 1- According to Halim (40), the direction of rolling while compacting the asphalt mat has a significant impact on the tensile strength as well as on the stiffness of the compacted mix. As a result, an asphalt core recovered from the field could have significantly different tensile strength values depending on the loading direction during the test relative to the rolling direction during compaction.
- 2- The presence of cracks after compaction, a phenomenon associated with the use of conventional steel rollers, can also result in the high variation reported by almost all moisture damage studies. The presence of such cracks will result in significantly different tensile strength values depending on the loading direction. The construction cracks existing before loading will act as a predetermined failure zone resulting in stress concentration at the tip of the crack and hence the lowest indirect tensile strength compared with any of the other possible loading directions.
- 3- Aggregate distribution, if not uniform, can contribute significantly to the observed variation. A high degree of mixture confinement is always required in order to establish uniform distribution of aggregate during compaction. The distribution of aggregate behind the spreader has been found to be uniform, but after the use of steel rollers to compact the asphalt mat, this degree of uniformity is usually lost. The required degree of confinement is not possible due to the limited contact arc of conventional compaction rollers. Particles under the steel roller pressure will follow the path of least resistance and will move laterally rather than

vertically resulting in large variation in densities and other mixture properties.

While variations are encountered within one study using a particular mixture within a specific location, test data from different studies show conflicting results . This leads to the lack of confidence in the various laboratory tests developed to predict stripping susceptibility. Available adhesion theories failed to explain such variations and conflicting results (5) (7) (11) (24) (58). This study will address the factors mentioned in this section in order to develop a more reliable prediction method.

2.4 LABORATORY SAMPLES VERSUS FIELD CORES

In this study, the failure of the existing prediction methods has been partially attributed to the use of laboratory prepared samples. For laboratory prepared samples to be used in prediction methods they must first resemble field compacted material. The following are three factors that make such resemblance quite difficult and make the comparison between laboratory samples and field cores a challenging task. These factors can also be used to emphasise the importance of considering compaction as an important element in any investigation dealing with the problem of stripping.

2.4.1 CONFINEMENT DURING COMPACTION

The laboratory sample is usually compacted while confined inside a steel mould which will restrict material movement to the vertical direction under the action of the hammer or a static load. In the field, the mixture is free to move horizontally and confinement is only limited to the vertical direction. In general, field confinement depends on:

- Contact area
- Stiffness of underlying layer and compactor
- Flow properties of mixture (viscosity vs temperature).

Variations in the above factors will result in different particle orientation and different particle to particle contact inside the material. This will ultimately govern mechanical

properties of the material. Laboratory samples prepared using existing laboratory compaction methods do not duplicate field conditions and if used they will reflect performance characteristics different than those of field compacted mixtures.

2.4.2 MOISTURE EXPOSURE

In moisture damage studies, the degree of exposure to water is fundamental in attempting to predict field performance of asphalt concrete mixtures. Laboratory samples, after mixing and compacting, are always coated from the outside with a bitumen film. This makes the comparison between field cores and laboratory prepared samples an inaccurate method because of relatively lower permeable surface of laboratory prepared samples. The difference could be seen from Figure 2.1, showing a laboratory prepared sample and others produced in the field by coring. Using laboratory prepared samples will underestimate the influence of construction induced cracks and high air voids associated with field compaction.

2.4.3 COMPACTION ENERGY

The methods used to apply energy during compaction also differ between laboratory and field compaction. While the field compaction uses rollers to apply pressure, the laboratory method utilizes many ways and they are all different from the field methods. The Marshall method uses a falling hammer and the Static Method uses a static pressure (3000 psi) and they are not expected to produce an action similar to that of rolling used in the field. Some specifications require laboratory samples to be compacted on both sides. This means turning the sample upside down to compact the other side. Therefore, laboratory methods do not differentiate between top and bottom sections of the asphalt concrete layer.

**FIGURE 2.1: FIELD CORES Vs. MARSHALL SAMPLE (LAB)
- EXTERIOR SURFACE CONDITION**

2.4.4 SUMMARY

Due to the above mentioned factors, laboratory prepared samples should not be used for moisture damage studies and should be limited for their original purpose, i.e. to help in the mix design procedure to produce a mix that will prevent flushing and pre-mature aging of asphalt pavement. Marshall mix design method and its components were not intended to emphasize the mixture structural strength, its fatigue characteristics and any of the other uses practised today, such as moisture damage studies.

Many project specifications for pavement construction require a trial strip to check the adequacy of the mix design formula and test the capability of contractor's equipment in producing the required end-result. This could be considered the best opportunity to produce cores for moisture damage studies and any other required material properties such as density and air void percent avoiding external factors such as traffic or weather effects. After the determination of density and air void content, conditioning of cores and testing could be performed following standards based on field produced samples.

2.5 DAMAGE ASSOCIATED WITH LOW TEMPERATURE PERFORMANCE

Rapidly increasing stability with decreasing temperature, no longer obscure damage associated with thermal cooling of asphalt concrete. Transverse cracking in cold regions has been associated with low temperature performance. Previous investigations in this subject (52) (53) (59) linked low-temperature properties of asphalt concrete to transverse cracking. Even though, asphalt cement has a coefficient of thermal contraction more than 20 times higher than aggregate coefficient low-temperature cracking mechanisms described in the literature assume that asphalt concrete contracts as a homogeneous material. The concept of differential contraction mentioned earlier, will be used here to predict asphalt concrete performance during thermal cooling. The contraction mechanism and the resulting cracks are different than those described by previous research work.

Basic low-temperature rheology describes the increase in stiffness and tensile strength with thermal cooling. Figure 2.2, after reference (57), is a typical temperature and tensile strength graphical plot. Indirect tensile strength increases as the temperature is lowered

peaking at a certain temperature and then the tensile strength starts to drop with thermal cooling. No surface cracks, similar to those described in the literature, has been observed during or after conditioning and prior to loading. No explanation to such performance has been offered and no damage has been associated with the drop in tensile strength. Thermally induced stresses due to differential contraction of asphalt and aggregate may be responsible for the drop in strength. Chapter 3 describes the differential contraction concept and how cracks will form inside the samples and reduce the strength.

2.6 RESULTS OF EVALUATION

1. Adhesion theories do not explain completely the stripping phenomenon and none of the concepts can provide a reliable description of the stripping problem.
2. The variables introduced by these theories are difficult to determine experimentally, and therefore, the quantification of stripping potential prior to the mix design stage remains difficult.
3. Prediction methods used after mix design were unreliable in detecting stripping susceptibility. Laboratory prepared samples used in these methods and conditioning procedures followed do not simulate field conditions.
- 4- Construction has been neglected as a factor influencing the stripping problem. Conventional field compaction method produces surface cracks which influences the stripping resistance of asphalt concrete.
5. The stripping mechanisms, suggested in the literature, fail to describe the mode of water access to the asphalt/aggregate interface.

6. No attempt has been made to investigate the performance of the asphalt/aggregate bond with temperature change. This ignores the well known influence of temperature change on pavement durability and service behaviour.
- 7- The large difference in the coefficients of thermal contraction of asphalt cement and aggregate implies that differential contraction of the mixture components may lead to internal cracks during thermal cooling. The existing low-temperature performance mechanisms assume a homogeneous asphalt concrete mixture.

The following three chapters address all these factors, which when clearly understood will facilitate the development of a solution to the stripping problem.

FIGURE 2.2: TYPICAL TENSILE STRENGTH-TEMPERATURE RELATIONSHIP

AFTER REF. (57)

CHAPTER 3

THE INFLUENCE OF DIFFERENTIAL THERMAL EXPANSION AND CONTRACTION ON ASPHALT STRIPPING

3.0 INTRODUCTION

Current prediction methods are inadequate for estimating stripping susceptibility of asphalt concrete. The lack of a comprehensive theoretical explanation for the adhesion phenomenon has been considered the main reason for the failure of existing prediction methods (11) (37) (56) (58). While prediction methods use warm water soaking to force stripping of asphalt concrete in the laboratory, none of the adhesion theories reflects the sensitivity of the bond to temperature changes. Warm water soaking of asphalt concrete cores results in stripping and recent investigations tend to favour this laboratory conditioning procedure to freeze-thaw cycles. This trend may be taken to imply that there is no link between stripping and low-temperature conditions. However, temperature change is known to influence pavement durability and service behaviour. Therefore, the lack of a direct link between existing adhesion theories and thermally induced stresses is a clear deficiency in the existing approach to the stripping problem.

Low-temperature performance of asphalt concrete pavements although neglected in the past, is now receiving more attention because of associated cracking problems. Haas (50), Haas and Phang (52), Ruth et al., (53), Monismith et al. (54) investigated this problem and the result is an improved understanding of the problem of low-temperature cracking and low-temperature performance in general. Almost all of these investigations assume that asphalt concrete behaves as a homogeneous system during thermal changes even though it is well known that the coefficient of thermal contraction of asphalt cement is twenty times higher than that of the aggregate (55). This part of the thesis will present

an assessment of the influence of differential thermal contraction and expansion during thermal cooling and warming of asphalt concrete pavements. The objective is to evaluate stripping susceptibility of asphalt concrete pavements based on bond performance with temperature changes and based on estimates of sustained damage as a result of thermally induced stresses. The investigation will be limited to factors that contribute to stripping such as cracks initiated during periods of pavement thermal cooling. With this understanding, a new approach will be presented to describe the initiation of stripping and its relationship to low-temperature cracking and thermally induced stresses in general. Such cracks may connect water accumulation pockets in the mixture to the asphalt matrix/aggregate interface. The cracked asphalt matrix and the possibility of water penetration to the interface is considered as an initial and very significant stage of stripping.

3.1 APPROACH

After assuming a simplified problem geometry, each element in the suggested system will be considered as a distinct material with its own characteristic response to thermal effects. The approach will be to present an hypothesis and then to validate the hypothesis through theoretical analysis, followed by an experimental program. The theoretical approach and the analysis of test results will distinguish between low- and high-temperature performance due to temperature changes.

3.2 LOW TEMPERATURE PERFORMANCE

Due to the considerable difference in the coefficients of thermal contraction of aggregate and the asphalt cement, it is hypothesized that the asphalt matrix will attempt to contract more than the aggregate during thermal cooling. The asphalt matrix is defined as the mixture of asphalt cement, sand and other fines. Since contraction of the asphalt matrix is limited by aggregate contraction, contact pressure will develop at the aggregate/asphalt matrix interface. It has long been known that at low-temperatures; asphalt performs basically as a brittle elastic solid. Therefore, the contact pressure produced by

differential contraction will result in radial and tangential stresses in the asphalt matrix. These stresses are not expected to relax through flow as in the case of high-temperature performance. As a result of these stresses, the asphalt matrix around the aggregate will be under tension normal to the surface. With further cooling, the tangential stresses will continue to increase and at some point will exceed the fracture strength of the matrix. Film failure is considered as an initial and very significant stage of stripping. The following sections will describe the state of stress resulting from differential contraction during thermal cooling.

3.3 ASSUMPTIONS

Differential thermal contraction and expansion have not yet apparently been investigated. A major difficulty in investigating thermal contraction arises from lack of knowledge of the fundamental rheological characteristics of the asphalt matrix at different temperature levels and age intervals. The stress calculations in the solution offered by this study make the following simplifying assumptions:

Elastic Modulus

All bitumens at sufficiently low temperature will perform basically as a brittle elastic solid with very low creep or flow potential. The solution will consider conditions below the critical temperature (T_c) as described by Phang (60) and discussed in Section 3.4. The elastic modulus E , experimentally determined at the mid point of a discrete temperature interval, will be used in the numerical application in Section 3.5.

Temperature Distribution

No temperature variation with depth is considered. Temperature gradients usually exist through the depth of the surface layer but very little such data exists on temperature variations in extreme cold weather areas. The element being investigated is small in size and, therefore, uniform temperature distribution will be assumed throughout the investigation.

Mixture Components

The solution considers two distinct materials. One component is the large aggregate particles and the other is the asphalt matrix consisting of asphalt cement, sand and other fine material. The fine material includes small size aggregate and fill material. The characteristics of the asphalt matrix is actually controlled by both constituents, i.e., asphalt cement and fine aggregate, but the solution will consider the matrix as a homogeneous material.

Problem Geometry

The stress solution will assume a spherical geometry where a large aggregate particle represents the core of the sphere and the asphalt matrix represents the larger sphere as shown in Figure 3.1. Variable thickness of the asphalt matrix will be used to account for the lack of non-uniformity of aggregate distribution in field compacted asphalt concrete mixtures. Accumulation of large aggregate particles in one location indicates thin asphalt matrix cover around the aggregate.

Coefficient of Thermal Contraction

Based on a study by Jones et al., (55), a linear relationship between expansion and temperature exists for asphalt cement and asphalt concrete for the temperature range between 60 and -34°F. Therefore, a constant coefficient of linear thermal expansion will be used within this temperature range for the asphalt matrix. A constant coefficient of linear thermal expansion will also be assumed for aggregate.

Poisson's Ratio

Temperature has an effect upon the value of Poisson's ratio of asphalt concrete. Poisson's ratio varies from about 0.25 at low temperature to 0.50 at relatively high temperature (57). A constant value of Poisson's ratio of 0.30 will be assumed for this

FIGURE 3.1 : ASPHALT AGGREGATE SYSTEM

study. Similarly a value of 0.40 will be assumed for aggregate. Accurate determination of induced stresses will require determination of poisson's ratio at different temperature intervals within the temperature range used in the investigation.

3.4 STATE OF STRESS

For the contact pressure, mentioned in Section 3.3, to develop at the aggregate/asphalt matrix interface due to differential contraction, the temperature must be below the critical temperature T_c as described by Haas and Phang (52). The critical temperature, shown in Figure 1.9, is defined as the point at which viscous flow decreases and below which tensile stresses start building up if the temperature continues to drop. The viscous portions of curves similar to those in Figure 1.8, when plotted as a function of temperature, produce Figure 1.9. An arbitrary critical temperature will be selected for the numerical application in Section 3.5. The contribution of relatively high temperatures is not expected to be high enough to seriously affect the magnitude of the calculated stresses.

Thermal cooling below the critical temperature will lead to build up of stresses since the assumed elastic behaviour is considered an acceptable approximation. According to the following solution thermal cooling to any temperature below T_c will result in the tendency of asphalt concrete components to contract. As a result of the considerable difference in the coefficients of thermal contraction of these components as approximated to the system shown in Figure 3.1, the asphalt matrix will tend to contract much more than the large aggregate particle since the matrix coefficient is much higher than that of the aggregate. Table 3.1 show typical values of the coefficients of thermal contraction for the different elements of asphalt concrete. But free contraction by the asphalt matrix is limited by aggregate contraction beyond which the asphalt matrix will apply pressure on the aggregate surface.

Table 3.1

Coefficient of Linear Thermal Expansion
For Typical Asphalt Concrete Components

Component		Coefficient of Linear Thermal Expansion
*	Gravel	$7-8 \times 10^{-6}$ per °F
**	Lime Stone	$4-5 \times 10^{-6}$ per °F
**	Sand (silica)	3×10^{-6} per °F
+	Asphalt-cement	$0.7-1.5 \times 10^{-4}$ per °F
•	Asphalt-concrete	$1.2-1.4 \times 10^{-5}$ per °F

- Reference BRE Digest 228
- + Reference (55)
- Reference (60)

The contact pressure acting on both systems results in radial and tangential stresses. Timoshenko and Goodier (61) analyzed a similar system using a solution by Lam'e. In the analysis, two pressures, P and P_o are considered as shown in Figure 3.2. These two pressures produce a radial normal stress σ_R and a tangential normal stress σ_t as shown in Figure 3.3. The pressure P_o may result from externally applied loads such as traffic. Since this study will focus on the effect of thermally induced stresses, the influence of pressure P_o due to external loads will not be included in the investigation (i.e., use $P_o = 0$). Based on these assumptions, σ_R and σ_t in the asphalt matrix are determined using the following expressions from Timoshenko and Goodier (61):

$$\sigma_R = Pa^3 \frac{(b^3 - R^3)}{R^3(a^3 - b^3)} \quad 3.1$$

and

$$\sigma_t = -Pa^3 \frac{(2R^3 + b^3)}{2R^3(a^3 - b^3)} \quad 3.2$$

σ_R and σ_t are shown in Figure 3.3 and:

- P = contact pressure at the interface caused by contraction
- a = radius of aggregate particle
- b = outside radius of the sphere
- σ_R = radial normal stress due to pressure P
- σ_t = normal stress in the tangential direction
- R = radius of the sphere

FIGURE 3.2: SPHERE UNDER INTERNAL AND EXTERNAL PRESSURE

FIGURE 3.3: BUILD UP OF RADIAL AND TANGENTIAL STRESSES DUE TO THE CONTACT PRESSURE PRODUCED BY DIFFERENTIAL CONTRACTION

From Equation 3.1 and at $R=a$

$$\sigma_R = Pa^3 \frac{(b^3 - a^3)}{a^3(a^3 - b^3)} = -P \quad 3.3$$

also for $R=b$, i.e., the outer section of the sphere

$$\sigma_R = 0 \quad 3.4$$

From Equation 3.2 and at $R=a$

$$\sigma_t = -Pa^3 \frac{(2a^3 + b^3)}{2a^3(a^3 - b^3)} = P \frac{(2a^3 + b^3)}{2(b^3 - a^3)} \quad 3.5$$

Since $b > a$ σ_t is tension,
also for $R=b$

$$\sigma_t = -Pa^3 \frac{(2b^3 + b^3)}{2b^3(a^3 - b^3)} = \frac{3Pa^3}{2(b^3 - a^3)} \quad 3.6$$

and since $b > a$ σ_t is also tension.

From Equations 3.3, 3.4, 3.5 and 3.6 the greatest tangential tension is at the aggregate/asphalt matrix interface. The radial compression is maximum at the interface.

PRESSURE LEVEL DETERMINATION

The value of the contact pressure P is required to determine the stress level of both σ_R and σ_t as indicated by Equation 3.1 and 3.2. Considering displacement of the interface, i.e., at $R=a$ due to pressure P , the following relations are established:

at $R=a$

$$\Delta R = (a\epsilon_t)_{R=a} = \frac{a}{E_2} [\sigma_t - \nu_2(\sigma_R + \sigma_t)]_{R=a} \quad 3.7$$

where

ΔR = change (decrease) in the radius R due to pressure P

ν_2 = Poisson's ratio for asphalt matrix

ϵ_t = tangential strain in the asphalt matrix

E_2 = matrix Young's modulus

Substituting σ_R and σ_t as determined by Equations 3.3 and 3.5 into Equation 3.7.

For $R=a$:

$$\Delta R = (a\epsilon_t)_{R=a} = \frac{a}{E_2} \left[P \frac{(2a^3 + b^3)}{2(b^3 - a^3)} - \nu_2 \left(-P + P \frac{(2a^3 + b^3)}{2(b^3 - a^3)} \right) \right] \quad 3.8$$

Let

$$K = \frac{(2a^3 + b^3)}{2(b^3 - a^3)} \quad 3.9$$

Then

$$\Delta R = \frac{Pa}{E_2} [K + \nu_2(1 - K)] = \frac{Pa}{E_2} [K(1 - \nu_2) + \nu_2] \quad 3.10$$

For the aggregate particle considered as a sphere with only external pressure P acting at $R=a$, the radial displacement u_R is :

$$u_R = -\frac{Pa}{E_1}(1-2\nu_1) \quad 3.11$$

where ν_1 = Poisson's ratio for aggregate

E_1 = Young's modulus of aggregate

u_R is negative indicating a reduction in the value of the radius.

From Equations 3.10 and 3.11, and considering displacement because of thermal contraction at $R=a$, the displacement at the interface of the asphalt matrix must equal that of the aggregate, i.e.

$$-\alpha_1 \Delta T a - \frac{Pa}{E_1}(1-2\nu_1) = -\alpha_2 \Delta T a + \frac{Pa}{E_2}[K(1-\nu_2)+\nu_2] \quad 3.12$$

α_1 = coefficient of linear thermal expansion of aggregate/ $^{\circ}\text{F}$

α_2 = coefficient of linear thermal expansion of asphalt matrix/ $^{\circ}\text{F}$

E_2 = Young's Modulus of asphalt matrix (within ΔT)

E_1 = Young's Modulus of aggregate (within ΔT)

ν_2 = Poisson's ratio for asphalt matrix

ν_1 = Poisson's ratio for aggregate

ΔT = change in temperature T

Rearranging Equation 3.12, pressure P can be calculated as follows:

$$P = \Delta T \left(\frac{\alpha_2 - \alpha_1}{\frac{K(1-\nu_2)+\nu_2}{E_2} + \frac{1-2\nu_1}{E_1}} \right) \quad 3.13$$

Equation 3.13 can be used to calculate σ_R and σ_t at $R=a$ by substituting the value of P in Equations 3.3 and 3.5 where:

$$\sigma_R = -P$$

and $\sigma_t = P.K.$

The pressure P varies linearly within temperature change ΔT for a constant E as indicated by Equation 3.13. The value of the factor K depends on both a and b . To determine stress in the asphalt matrix along the radial distance, R , Equations 3.1 and 3.2 can be used by substituting for R the distance from the centre of the sphere to the location where the stress is to be determined.

3.5 NUMERICAL APPLICATION

To perform a numerical application using the derived formulas, many factors must be established considering material rheology as described below.

Elastic Modulus

Table 3.2 includes values of the elastic modulus of asphalt concrete estimated using indirect tensile strength tests (62). The tests were performed at different temperatures to account for the temperature dependent elastic modulus. Therefore, in using Equation 3.13 to calculate the pressure P , different temperature intervals will be used substituting for E_2 using elastic modulus values corresponding to the correct temperature interval. A constant value for elastic modulus of rock will be used throughout the analysis. A value of $E_1 = 8.5 \times 10^6$ was reported by Stagg, Zienkiewicz (63) for granite.

Fracture Strength

The fracture strength of asphalt concrete at low temperature has been measured using indirect tensile tests on cylindrical specimen. It is important in this type of test to define the conditions of temperature and rate of loading since the fracture characteristics

of mixtures are dependent on these parameters. At temperatures below freezing, strain rate effects are not as significant as at higher temperatures. Table 3.2 shows fracture strength determined for the same mix investigated in Reference (62). The rate of loading used was 0.05 inches per minute. Determination of failure stress for longer loading times may be necessary for accurate prediction of thermal cracking. The indirect tensile strength results are for an asphalt concrete mixture. As shown in the assumptions discussed in Section 3.3, the strength values, required for comparison with induced stresses, are those of the asphalt matrix alone. Such information is not available for the analysis since investigators in the past were mainly concerned with the performance of the whole mixture. To separately investigate the performance of the matrix, many factors must be investigated to define precisely the definition of the matrix. One major concern is to specify what particle size belong to the matrix and above which the aggregate particle is considered a large aggregate particle as shown in the assumed geometry. Such a study will require a comprehensive laboratory investigation to determine the relative sizes of each component used in the derived formulas. No such attempt will be made during this study and instead results from indirect tensile strength tests performed on asphalt concrete will be used to estimate the asphalt matrix properties.

3.5.1 PARAMETRIC ANALYSIS

Based on the assumptions stated above and on experimentally determined material properties shown in Table 3.2, Equation 3.13 can be used to determine the pressure developed at the aggregate/asphalt matrix interface for different values of asphalt matrix thickness within a temperature range between 50°F and -30°F. Typical values of pressure calculated using Equation 3.13 are given in Table 3.3 for a single asphalt matrix thickness. Table 3.3 also shows the tangential stress at the interface due to the contact pressure for the same asphalt matrix thickness. The solution is for a constant aggregate diameter of 0.5 inches. The poisson's ratio of aggregate used $\nu_1 = 0.4$ and for the asphalt matrix $\nu_2 = 0.3$. Values of ν_1 and ν_2 are assumed constant within the specified temperature range (i.e., from 50°F to -30°F). A constant value is assumed for the matrix coefficient of thermal contraction and used in calculating the thermally induced stresses

($\alpha_2 = 4 \times 10^{-5}/^{\circ}\text{F}$) throughout the investigation. The assumed value for the coefficient is much less than the asphalt cement coefficient but still higher than the asphalt concrete coefficient as shown in Table 3.1. The value of the coefficient of thermal contraction assumed for Limestone is $\alpha_1 = 4 \times 10^{-6}/^{\circ}\text{F}$.

Different asphalt matrix thicknesses exist in different pavements depending on the asphalt content, amount and type of fill material and the compaction method used. Aggregate distribution within a pavement section also differs due to compaction method in both horizontal and vertical directions. Table 3.4 gives the tangential stress values for a constant aggregate size ($a = 0.5''$) and for different asphalt matrix thicknesses.

To show the variation in calculated tangential stress with radial distance R - a , an aggregate diameter $a = 0.5$ inches and asphalt thickness $b = 0.75$ inches are assumed. Table 3.5 gives the calculated, σ_t values with respect to radial distance R starting at the interface, i.e., at $R=a$.

Table 3.2
Experimentally Determined Elastic
Modulus and Indirect Tensile Strength

Test Temperature °F	Indirect Tensile Strength (psi)	*Elastic Modulus (psi) x 10 ⁵
50	180	1.3
40	260	1.6
30	320	1.9
20	390	2.5
10	458	3.1
0	563	4.1
-10	524	5.0
-20	487	5.2

• After Reference (62)

NOTE: Elastic modulus measured for a 10°F range. Temperature shown is the middle temperature.

Table 3.3
Calculated Contact Pressure and the Resulting
Tangential Stress at the Interface

Temperature °F	Calculated Cumulative Values	
	Contact Pressure P (psi)	Tangential Stress (psi)
50	43	48
40	95	108
30	158	178
20	240	271
10	341	386
0	475	537
-10	638	722
-20	807	914

NOTE: a = 0.5"
 b = 0.75"
 E_1 = 8.4×10^6
 α_1 = 4×10^{-6}
 α_2 = 4×10^{-5}

Table 3.4
Calculated Tangential Stress for Different
Asphalt Matrix Thickness

Temperature °F	Cumulative Tangential Stress (psi)				
	Film Thickness (b-a) (inches)				
	0.25	0.20	0.15	0.10	0.05
50	48	51	53	57	61
40	108	113	119	127	137
30	178	187	198	211	227
20	271	284	301	321	345
10	386	405	428	457	492
0	537	564	597	637	686
-10	722	758	801	856	922
-20	914	959	1015	1083	1168

* Aggregate diameter $a = 0.5"$

Table 3.5

Variation of Tangential Stress with Radial Distance (R-a)

Radial Distance (R-a) (inches)	Tangential Stress (psi)
0.00	914
0.05	771
0.10	671
0.15	601
0.20	549
0.25	510

NOTE: a = 0.5"
b = 0.75"

3.5.2 DATA ANALYSIS

High values of tangential tensile stresses are calculated due to differential contraction within the specified thermal cooling range. Thus, based on the calculated stresses, cracking of the asphalt matrix due to thermal cooling can be anticipated. Tangential stresses developed at the interface within each 10°F can accumulate as assumed before and Figure 3.4 shows the increase in tangential stress with continuous cooling. For the specified conditions, the calculated stresses may reach a maximum of 840 psi at -20°F. This value is much higher than reported fracture strength of most the asphalt concrete mixtures. Therefore, failure is expected to occur as indicated by strength values in Table 3.3. Figure 3.5 shows tangential stress values in the asphalt matrix at the interface for different asphalt matrix thicknesses. On the same figure the experimentally determined failure stress from Table 3.3 is also shown.

From Figure 3.5, internal cracks similar to those shown in figure 3.6 are expected to develop in the compacted mixture when these stresses exceed the fracture strength of the asphalt matrix. For different asphalt matrix thicknesses the failure will occur at different temperature levels. To determine where cracks will start to develop, tangential stress values calculated using Equation 3.2 and presented in Table 3.4 are plotted against radial distance starting at the interface. Figure 3.7 shows the stress level with respect to radial distance $R-a$ (aggregate radius $a=0.5$ inch). The high stress level at the interface indicates that cracks will start at the interface and propagate away from the aggregate surface.

The following are some remarks concerning the accuracy of Equation 3.13 in estimating thermally induced stresses.

- (1) The temperature where thermally induced stresses start to accumulate is not a sensitive factor since temperature intervals on the relatively warm side do not add much to the accumulating stresses. If the asphalt cement flow continued to cause stress relaxation below the assumed 50°F, accumulation of stresses by thermal cooling at temperatures much below 50°F, e.g. below 20°F, will exceed the fracture strength of the mixture under investigation. The accumulating

tangential stress from 20°F to -20°F, for the conditions given in Figure 3.4, can exceed the material strength shown in Table 3.3.

- (2) The accuracy of the calculated stresses depends on the measured material properties. Equation 3.13 reflects the need for accurate measurements of the coefficient of contraction and Young's modulus of the asphalt matrix. Very little information is available about these two properties because in the past investigators were concerned with the asphalt concrete mixture which include large aggregate sizes. This study will not attempt to perform such measurements because properties depend on the asphalt matrix components which differ from one mixture to another. Further development of this concept will require investigating the asphalt matrix properties and the response of the matrix to thermal cooling.
- (3) Aggregate size and asphalt matrix thickness are two important factors when Equation 3.13 used for estimating thermal stresses in the asphalt matrix. These two factors vary within one mixture depending on the mixture design and the compaction method used. Based on the proportions of each size in the mixture, estimates of the overall internal damage can be estimated.
- (4) The intersection of the fracture strength curve with the developed tangential stress is not an indication of when cracks will develop. The drop in the measured strength of tested samples is a reflection of internally induced damage due to differential contraction. As discussed in 2 above, measurement of the actual failure strength of the asphalt matrix with different thicknesses will be needed to determine when cracks will develop during thermal cooling.

The concept of differential contraction in asphalt concrete can be applied to explain the influence of temperature changes and moisture conditions as described in the following two sections.

FIGURE 3.4: CALCULATED THERMALLY INDUCED TANGENTIAL STRESS IN THE ASPHALT MATRIX CLOSE TO THE INTERFACE

FIGURE 3.5: CALCULATED THERMALLY INDUCED STRESSES AT THE AGGREGATE/ASPHALT MATRIX INTERFACE DUE TO DIFFERENTIAL CONTRACTION FOR DIFFERENT ASPHALT MATRIX THICKNESSES

FIGURE 3.6 : CRACK INITIATED BY THERMALLY INDUCED STRESS FOLLOWING THERMAL COOLING

2

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS
STANDARD REFERENCE MATERIAL 1010a
(ANSI and ISO TEST CHART No. 2)

FIGURE 3.7: VARIATION OF CALCULATED TANGENTIAL STRESSES WITH RADIAL DISTANCE(R-a)

3.6 CRACKING DUE TO DIFFERENTIAL CONTRACTION

In the past, transverse surface cracking has been associated with pavement surface contraction at low temperatures. The manner in which low-temperature cracking is initiated by differential contraction is quite different from the previously suggested mechanisms. While existing concepts consider asphalt concrete mixture as a homogeneous material, the differential contraction concept considers two distinct materials with a large difference in their respective coefficients of thermal expansion and contraction. As Table 3.1 indicates, the coefficient of thermal contraction of asphalt cement is much higher than the aggregate coefficient depending on the aggregate nature and asphalt cement type. Such a large difference will result in high tangential tensile stress on the asphalt matrix because of thermal cooling. If the tensile stresses exceeded the material fracture strength, then hair line cracks will be initiated close to the interface. Crack location, length and orientation will differ from one mixture to another depending on aggregate size and texture, amount and size of fine material, asphalt content and type, aggregate particle orientation and location in the vertical direction, etc. Depending on the location of large aggregate particles, the initiated crack may or may not show immediately at the surface. In many recently constructed pavements large aggregates usually occupy the bottom section of the pavement as a result of vibratory rolling while finer aggregate tends to migrate to the surface. In such a condition, cracks may not show initially at the surface. Propagation of cracks to the pavement surface will depend primarily on the state of stress after crack initiation.

3.7 STRIPPING

To satisfy the study objective in seeking a better understanding of the stripping problem, it is necessary to locate the water route to the asphalt/aggregate interface. The following sections will describe how differential contraction can explain this factor.

3.7.1 CRACKS AS A WATER ROUTE TO THE INTERFACE

As explained earlier, the mechanism of water flow to the asphalt/aggregate interface has been a major deficiency in the current approach to the problem and in understanding how stripping is initiated. Water accumulates in pockets (air voids) which are distributed throughout the compacted mixture (Figure 3.8-a). Aggregate particles are protected from water by a thin bituminous film and the asphalt matrix cover. The bituminous film coating is usually formed during the mixing stage. Aggregates are known to adsorb some of the asphalt cement components while a thin film remains on the surface. The asphalt matrix will then occupy the space around the aggregate and usually adhere to the surface of the coated aggregate. The air voids are part of the asphalt matrix where water can accumulate. The asphalt matrix and the thin bituminous cover, when exposed to tangential tensile stresses exceeding their breaking strength, will crack in the manner shown in Figure 3.8-b. These cracks will then become a path for water when ice in those pockets melt after the cold season. Although not all water pockets are connected directly to surface water, in the presence of construction cracks, a large number of these pockets can attract more water, which should be sufficient for the stripping process to continue with time.

3.7.2 INFLUENCE OF THERMALLY INDUCED STRESSES ON BOND PERFORMANCE

It has been understood that tensile strength is a function of both adhesion and cohesion of asphalt concrete, but the increase in strength with thermal cooling has not been associated with an increase in the asphalt matrix/ aggregate bond strength. Based on the state of stress discussed earlier, one can describe the influence of differential contraction due to thermal cooling on the performance of the asphalt matrix/aggregate bond. The radial compressive stress will actually resist externally applied tensile stresses from pulling the asphalt matrix away from the aggregate surface. Contraction can only increase the strength of the asphalt matrix/aggregate bond prior to cracking that takes place if tangential tensile stress exceeds the fracture strength of the asphalt matrix. The

FIGURE 3.8: INFLUENCE OF CRACKS CAUSED BY THERMALLY INDUCED STRESSES ON THE STRIPPING PROBLEM

increase in the bond strength can therefore take place at mild low-temperatures. At relatively higher temperatures, the radial compressive stress is low and therefore, the bond will have less resistance to applied loads pulling the asphalt matrix from the aggregate surface and therefore indicates lower resistance to stripping since separation occurs at lower loads.

The last two sections presented two elements associated with stripping and resulting from differential thermal contraction. It must be emphasized that the influence of cracks initiated during low temperatures, on the stripping problem can only be detected after thawing when the water managed to flow into the interface. Relatively little time will then be needed for the detrimental effects of stripping to reflect in the lower resistance to stresses from traffic loads. Lower bond strength during warmer seasons must be considered in conjunction with the loss in stability due to temperature increase. These two factors together can weaken the asphalt concrete surface similar to the thaw-weakening effect as described in the literature. The improved asphalt matrix/aggregate bond due to contraction will no longer exist when the temperature increases and the asphalt matrix expands.

3.8 STRIPPING SUSCEPTIBILITY DURING WARM TEMPERATURES

High-temperature performance of asphalt concrete cannot be discussed in isolation of the low-temperature stage when stripping is being investigated. First the cracks initiated during low-temperatures will function as a water path to the interface during high-temperatures. Second, the contact pressure responsible for improving the bond at low-temperatures will now be released. Therefore, high-temperature performance can influence the bond in two ways:

- (1) At relatively low temperatures levels in the high-temperature range, i.e., between 50 and 70°F, the asphalt concrete will be expanding. Therefore, the existing contact pressure, developed during low temperature period, will now be relaxed. Also within this temperature range, asphalt concrete can also flow as described by asphalt rheology. Therefore, the contribution of the contact pressure is usually

lost. This means that the mixture is now more susceptible to stripping if water finds an access to the interface.

- (2) If cracks are initiated, particularly when the tangential stress resulting from differential contraction exceeds film tensile strength, then water can use these cracks to freely flow into the interface from water pockets as described earlier.

Above relatively low temperature levels i.e., above 70°F, the factors contributing to the stripping problem are more complex. A structural change in the bitumen nature is expected as described by Benson's experiment (64). Benson observed that the very thin film formed from asphalt cement at elevated temperatures is optically homogeneous. The same film, formed from a dilute solution in benzene shows clusters of asphaltenes after evaporation of the benzene. The solubility of asphaltenes in benzene is not very high at room temperature. Therefore, they attract each other more strongly, pile up in clusters and the thin film of the surrounding material cannot bring these clusters into solution. This reaction depends upon several factors such as oxygen supply, rate of oxidation, carbon hydrogen ratio of the asphaltenes formed because their solubility decreases with increase of this ratio. The reaction also depends on the chemical composition of the surrounding medium because their solvent power decreases with decreasing amount of aromatics (64). The exposure to high temperatures, such as soaking in a warm water bath at 140°F, is expected to have a similar effect as described by Benson. Oxidation takes place in the thin asphalt film during warm water soaking (58). Oil spots on the water surface have been observed by the author each time cores are placed in a warm water bath. This observation suggests a possible structural change in asphalt cement resulting in the separation of the oily medium from other components.

Whatever the reaction mechanism may be, it will lead, sooner or later, to segregation of asphalt into two phases, asphaltenes and an oily medium. The clusters of asphaltenes being loosely bound together, do not act as a binder. The oily medium, in view of its relatively low viscosity, will rather act as a lubricant. Finally the result of formation of asphaltene clusters will be the loss of the adhesive bond with aggregate which will be followed by stripping and a greatly reduced asphalt concrete strength.

3.9 CONSTRUCTION INDUCED FAULTS

Construction induced faults discussed here are similar to those described by Scott (15). These faults are the result of incomplete coating of aggregate by asphalt cement and/or asphalt matrix during mixing or compacting stages. Such faults can therefore be considered as locations within compacted asphalt concrete where water pockets are directly connected to the asphalt/aggregate interface. In such location, the exposure to water for a relatively short period will result in stripping as described in Section 1.4.2. A typical fault is shown in Figure 3.9-a representing a vertical section in a newly constructed pavement.

Stripping due to water flow through construction induced faults must be considered prior to evaluation of stripping damage initiated by other elements such as internal cracks caused by exposure to low temperatures and loss of adhesion due to warm water soaking. These faults may result in internal damage during thermal cooling if water flowing through faults is frozen followed by volume increase and applying pressure on the fault walls.

3.10 FREEZE-THAW CYCLES

The action by freeze-thaw cycles is expected to be within the outlined expansion and contraction actions. The asphalt matrix in compacted mixtures exposed to freeze-thaw cycles will tend to contract during freezing and expand during thawing. According to the differential contraction and expansion concept freeze-thaw cycles are capable of initiating stripping. Cracks initiated by differential contraction during the freezing stage will be used by water to flow into the interface during thawing and to develop stripping zones. Thawing at temperatures above service temperature will also cause damage due to bond weakening caused by expansion and loss of adhesion caused by structural change in asphalt cement. Therefore, repeating a freeze-thaw cycle will cause further damage and its effect is expected to be more destructive than exposure to a single action of either expansion or contraction. It is interesting to observe that after using freeze-thaw cycles for many years to evaluate stripping potential many investigators are now using only

warm water soaking at 140°F to force stripping in the laboratory. Even the use of warm water soaking was not based on any of the adhesion theories provided in the literature.

It is difficult to determine to what extent each element of the cycle contributes to the accumulated damage. Investigating freeze-thaw cyclic action is necessary if such exposure condition is expected in the field. Cyclic temperature effects may occur diurnally, i.e., a large temperature drop in the night and an increased temperature during the day. Within such a cycle both expansion and contraction are categorized as short term exposure conditions. The actual performance of a compacted asphalt concrete mixture after freeze-thaw cycles must be investigated in the laboratory to quantify the influence of each part of the cycle. The experimental program investigating stripping must include freeze-thaw cycles using more than one freezing temperature and many thawing temperatures.

3.11 SUMMARY

In an attempt to deal with the deficiencies existing in the current approach to the problem of stripping, a new approach has been introduced to describe the asphalt/aggregate bond performance during temperature changes and how stripping can be initiated during low temperature periods. The theoretical approach is used to describe how thermal stresses develop and how it can influence stripping.

- (1) Low-temperatures influence stripping in many ways as described by the differential contraction concept:
 - (a) High tensile stresses may develop exceeding the fracture strength of the asphalt matrix. Cracks initiated will function as a path for water to reach the interface.
 - (b) The radial stress will improve the asphalt/aggregate bond by holding the asphalt matrix to the aggregate surface, resulting in an increase in the resistance to tensile stresses during low temperatures, if the tangential stress component does not exceed the film fracture strength.

- (2) At relatively higher temperatures, the state of stress will change and flow will also become a factor. Resistance to expansion due to pressure inside the air voids also becomes a factor which complicates the behaviour of mixtures at elevated temperatures (55). In general:
- (a) differential expansion will result in a decrease in the contact pressure and flow will also result in relaxation of stresses in the asphalt matrix. Therefore, bond improvement gained by the contact pressure will diminish and the asphalt will separate easily from aggregate under the applied tensile stresses, i.e. reduce the stripping resistance of the mixture.
 - (b) When ice melts during a warm period water will flow freely to the interface using cracks developed during low-temperature intervals and stripping will start and progress depending on the water supply.
 - (c) A structural change in asphalt-cement is expected to follow oxidation at elevated temperatures (i.e., 140°F) in the presence of water. The result will be loss of adhesion followed by disintegration of the thin bituminous film leading to the physical separation of asphalt matrix from aggregate surfaces.
- (3) Differential contraction and expansion can be used to explain other phenomena such as low-temperature cracking and thaw-weakening. Thaw-weakening is readily explained by the state of stress before and after a thermal cycle and the effect of stripping. To explain low-temperature cracking, propagation of the cracks initiated by differential contraction, will require further investigation which is beyond this study.
- (4) Freeze-thaw effect is also explained by the concept of differential thermal contraction/expansion. Damage that occurs during a cold period results in further damage by stripping during the following warm period. Stripping damage will accumulate after each cycle.

CHAPTER FOUR

INFLUENCE OF COMPACTION ON ASPHALT STRIPPING

4.0. INTRODUCTION

Considering the factors that may contribute to physical separation of the asphalt matrix from aggregate surfaces because of stripping, construction has been the least investigated factor. No comprehensive study has yet been performed on the subject of asphalt stripping in the context of compaction. In fact, only one field compaction method existed until Halim (43) recently introduced AMIR offering a good opportunity for such comparison. Stripping has been observed to occur in the field days or weeks after construction is completed (1). This observation suggests that stripping can be initiated during or immediately after construction. Hence, compaction is considered as one of the main parameters investigated by this thesis.

Lottman, as reported by NCHRP report No. 274 (3), compared two laboratory methods of compaction used in producing samples for asphalt concrete moisture damage studies. He concluded that methods of compaction did not influence results. NCHRP report No. 274 (3) discussed laboratory compaction methods and referred to the conclusion of the Lottman study. Since Lottman used a Marshall hammer and a kneading compactor, the NCHRP conducted its own investigation using the Marshall hammer and the static load procedure from ASTM Method D 1074. Experimental data revealed that a sample compacted by static load, when compared with Marshall compacted specimen, had a higher tensile strength. Wet indirect tensile strength was lower for static load and hence

lower tensile strength ratio (TSR). The use of another mixture also revealed a difference between TSR of the samples prepared by the two compaction methods. In spite of the differences mentioned above, the report considered both compaction methods as being capable of producing samples required for moisture damage studies. It might be noted that neither of the two methods simulate conventional, field roller compaction.

Researchers and pavement investigators did not have the opportunity of comparing between two uniquely different field compaction methods. Such a comparison could have helped in identifying the effect of compaction on the stripping resistance of compacted asphalt concrete. With the introduction of the new Asphalt Multi-Integrated Roller (AMIR), such a comparison is now possible. The elements investigated include:

- 1- Construction cracks
- 2- Compaction energy
- 3- Aggregate distribution
- 4- Particle orientation
- 5- Physical properties
- 6- Mechanical properties

4.1 CONSTRUCTION CRACKS

All the stripping mechanisms discussed earlier are attempts to explain the stripping phenomenon. They are centred around the concept of water displacing the asphalt cement from the aggregate surface. But, for water to reach the asphalt/aggregate interface and displace the asphalt cement, it must first reach the lower section of the pavement where virtually all initial stages of stripping were observed in the field (1)(3)(11). Locating the path water uses to circulate in the asphalt layer is a key element if a preventive measure is to be found. Chapter 3 discusses how water reaches the asphalt/aggregate interface from accumulation pockets inside the pavement. This study suggests that water pockets,

usually referred to as air voids, are supplied by surface water through surface cracks. A continuous supply of water is necessary for initiation and progress of stripping with time.

Not all pores in compacted asphalt concrete mixtures are connected. Preliminary laboratory experiments, performed as part of this study, showed that soaking asphalt concrete cores in water resulted in less than 50% degree of saturation. Cores with high density show even much lower degrees of saturation when soaked for a week under room temperature. The NCHRP report No. 274 (3) came to the same conclusion. Vacuum pressure has been used to increase the degree of saturation. A vacuum pressure level of 26 inches of mercury has been used to increase the saturation degree up to 80%. The actual degree of saturation reached under a certain vacuum pressure level depends on the specific mixture under investigation. Similar or higher vacuum pressure levels have been found to increase the degree of saturation beyond 100% indicating a volume change. Jimenz (1) explanation suggests a possible damage inside the samples leading to saturation levels exceeding 100% under relatively high vacuum pressure levels. Therefore, it is possible to assume that not all pores in compacted asphalt concrete are connected and for surface water to reach most of the pavement sections, another access must be investigated. Construction cracks, described in Chapter One, are discussed here as the other access responsible for water circulation in locations otherwise isolated from surface water.

Construction techniques have been mentioned as a possible factor capable of influencing stripping resistance of a mixture but no attempt has been made to investigate how construction may affect the problem. As mentioned earlier, this study suggests that surface cracks can be a factor contributing to the problem of asphalt stripping. Surface cracks that exist at an early stage in the pavement life, i.e. during and after construction, can constitute a path for surface water to penetrate inside the asphalt concrete layer where any of the suggested stripping mechanisms may start functioning. In the presence of such cracks, surface water is the source of water supply necessary for stripping to start and continue during the life of the pavement. Excessive cracking implies fast deterioration due to stripping.

Considering the presence of construction cracks as discussed earlier, Halim used a

new analytical approach to prove that such cracks will be created every time conventional (roller based) compaction methods are used to compact asphalt concrete. Laboratory investigations using models of the conventional and the new compactor proved the outcome of the theoretical approach (39).

Construction cracks are a well known phenomenon and are usually referred to as checking. Checking occurs after the first or second pass of the conventional compaction equipment over the mat. Scherocman (65) described checking as the problem of, "short transverse cracks, usually 1 to 4 inches in length and 1 to 3 inches apart, which occur in the asphalt concrete mat or layer. These surface cracks, or checks are not visible when the paver places the material". According to Scherocman's experience, these checks do not extend completely through the pavement surface layers but they are normally 1/4 to 3/8 inch in depth. Scherocman suggested that excessive deflection of pavement structure under the compaction equipment is due to the weakness of the layer beneath the new layer. Another possible reason suggested by Scherocman is a possible deficiency in the asphalt concrete mix design including too much asphalt cement, too much moisture, nonuniform sand gradation or too little fine size sand. According to Halim (40), those cracks described by Scherocman will always be present after compaction is completed using the existing compaction equipment. Conditions more severe than those described may occur. Surface cracks have been seen even in the most controlled field experiments including field tests used for this study.

Since only one compaction method existed before, mix design remained the only possible variable to adjust. However, this alternative may not always be available, or feasible. With the presence of a new compaction technique, another option is now available for finding a solution to the problem. Halim's explanation of the problem offers a sound analytical approach to the problem of surface cracking and other parameters associated with compaction. In the past, the absence of another compaction option led to the prescription of cures that influenced other material properties. When faced with such cracks in the first or second pass, operators will usually be asked to stop rolling giving the mixture a chance to cool and for the viscosity of the asphalt cement in the mix

to increase. However, this will stiffen the mix and will also increase the resistance to compaction resulting in unfavourable density and void levels. The increased stiffness will result in less cracking which supports Halim's concept involving rigidities of the different components of the pavement system. With increased rigidity in the new layer, the mismatch of rigidities between the existing surface and the new layer is reduced to a lower level reducing cracking, at the same time the density decreases and the voids increase in comparison to those found when the compaction is carried out at specified temperature and low asphalt cement viscosity.

The concern of this study is to examine the role that the cracks created by compaction play in the phenomenon of asphalt stripping from aggregate surfaces. In the past when field tests were carried out to evaluate the effect of construction on a given problem related to pavements, the finished asphalt pavement always had the same cracking characteristic. Since construction cracks could not be eliminated with current equipment, their influence was common across most of the investigated projects and as a result masked the actual influence of such cracks. As discussed earlier, the presence of cracks also resulted in poor correlation between test results of laboratory prepared samples and cores recovered from the field since such cracks were not always present in laboratory prepared samples.

In Chapter 2, which evaluated the present approach to the problem, the different concepts describing the water source and its path to the bottom section of the asphalt layer were discussed. Osmotic pressure was dismissed as a cause because of its slowness. The contribution of air voids was considered minimum since stripping occurred even after the pavement was made impervious by reducing air voids in the compacted mix. Cracks, on the other hand, can provide water with an access to the pavement interior. This is further discussed in the following subsections.

4.1.1 CRACKS, A PATH FOR SURFACE WATER INTO PAVEMENTS

Surface cracks constitute an easy access for water to penetrate the pavement. Actually surface cracks allow water to penetrate what is considered a first line of defence, that is the fine portion of the mixture which migrates to the surface during construction due to segregation. Segregation of fine material due to vibratory compaction is a well known phenomenon and has been observed in field trials used by this study. Figure 4.1 shows the effect of vibration on aggregate distribution in a core from NRC trial no. 1. This phenomenon is further discussed in the following section. The existence of cracks through what is considered an insulation layer defeats its purpose and makes the asphalt layer more prone to the effects of stripping by allowing the accumulation of large aggregate particles at the bottom section of the asphalt layer where stripping has been known to start. Virtually all reported field observations indicated that stripping starts at the bottom section of the asphalt concrete layer. This could be explained by the fact that the bottom section of the pavement surface layer is where large aggregate are located as a result of using vibratory rollers for compaction.

Investigators have noticed that continuous rolling with steel-wheel roller can cause a significant amount of cracking (66). This is the reason the steel wheel roller usually makes 3 to 4 passes and leaves the pavement; the pneumatic roller follows to continue the rolling until specified density is reached. It has been claimed that the pneumatic roller closes cracks created by the steel roller. Unlike the steel roller, the pneumatic roller does not create the mismatch of the rigidities between components of the pavement system and the compactor, but the geometry factor is still the same and therefore cracks are not expected to disappear as claimed. The author observed cracks on three field trials used by this study, even after more than 20 passes by the pneumatic roller following the removal of the steel roller. These cracks are quite visible immediately after construction, but dust or any other foreign material may conceal them later while they still constitute a path for the penetration of surface water. Other problems associated with these cracks that directly or indirectly affect the resistance of a pavement material to stripping include oxidative aging and stress concentration.

FIGURE 4.1: SEGREGATION OF AGGREGATE FOLLOWING COMPACTION BY THE VIBRATORY STEEL ROLLER

4.1.2 OXIDATIVE AGING

An increase in the oxidation rate can result on account of increased air circulation inside the pavement. Oxidation of asphalt is generally considered a major factor contributing to the hardening and embrittlement of asphalt pavement. Excessive hardening of asphalt cement is undesirable because it often leads to problems associated with pavement embrittlement and cracking. Mixtures with higher air voids are known to age more rapidly.

Peterson (67) indicated that there is some evidence which suggests that mixtures suffering oxidative aging are moisture susceptible. Construction cracks together with air voids should be more conducive than air voids alone in circulating air inside the pavement. Air circulation inside the pavement causes excessive oxidative aging and hence hardening and its associated problems. One specific problem that directly influences stripping and is associated with oxidative aging is discussed in Section 3.9. Oxidation of thin bituminous films usually leads to the formation of asphaltenes. In the presence of water, piling of asphaltenes results in the asphalt cement losing its adhesiveness which promotes stripping.

4.1.3. STRESS CONCENTRATION

The existence of construction cracks during the build up of stresses due to traffic loading or due to thermal action can cause stress concentration at the tip of these cracks. This may result in widening of cracks and may also lead to crack propagation towards the bottom section of the asphalt concrete layer. In both instances, more water from the surface will enter the pavement increasing the degree of the pavement vulnerability to the detrimental effects of stripping. Freezing of water inside these cracks during the cold season results in approximately 9% increase in volume of saturation water. This will result in the build up of pressure on the walls of the cracks.

4.2 OTHER FACTORS

Factors other than the construction cracks that can directly or indirectly influence stripping and caused by compaction are discussed in the following section.

4.2.1 COMPACTION ENERGY

This is the energy used in densifying the mixture and creating the adherence of asphalt cement to aggregate which is necessary to resist stresses from traffic loads and thermal stresses generated by expansion and contraction. Although chemical adhesion does help in the development of resistance to such stresses, lack of adequate compaction energy will result in unfavourable physical and mechanical properties and thus directly affect the resistance of a mixture to stripping.

The geometry of the conventional steel roller is responsible for the inability of the laid asphalt concrete to benefit from the work done by the steel roller. This is reflected in the two conditions discussed below.

Resistance to Compaction

The geometry of the steel roller causes shoving of the material in front of the roller where part of the work done is being used to move the roller ahead against the resistance of the material wave created due to the geometry and the heavy weight of the compactor. This heavy load is applied along the roller side and the width of the contact area, called the arc width. The arc width decreases with increased resistance from the mixture due to densification and also due to the increase in the viscosity of asphalt cement. The creation of such a wave in front of the roller can actually cause pulling of the bottom of the compacted mat from the underlying layer, an observation made in three field trials used by this study. The type of separation caused by the wave action is shown in Figure 4.2. Such a separation can also result in water accumulation between existing and newly laid layers. The vibration by the steel roller can also contribute to the creation of such waves. The pressure from the steel roller increases with the increased resistance because

**FIGURE 4.2: SEPARATION OF NEWLY PLACED
ASPHALT CONCRETE LAYER DURING COMPACTION**

of the reduced contact arc until the load is practically a line load reaching a value of 350 lb/linear inch. Such high stresses on the pavement can actually cause some destruction. This is why Kennedy (66) and others noticed a reduction in density when vibratory steel rollers continued beyond a certain density level. On the other hand, the new compactor, AMIR, has a favourable geometry when compared with steel rollers and the area in contact with the pavement is much larger which results in less stresses with no material wave shoved ahead of the compactor. Therefore, a larger portion of the work done is used in compacting the asphalt concrete. The compactor faces relatively little resistance compared with steel rollers. The confinement factor also determines how much of the work done is used to effectively compact the mixture. Due to the limited degree of confinement under the steel roller, particles under pressure will follow the least path of resistance and move laterally rather than vertically, especially during the low viscosity state. From the above discussion, it is clear that of the total work done by the steel roller, only a portion actually constitutes the compaction energy.

Pressure Duration

Roller speed is one of the most critical factors in the compaction operation. To achieve an adequate asphalt density in the pavement, a very slow speed should be maintained during compaction. At the same time, and in order to finish compaction with less resistance from the material, compaction must be done fast enough so that it is compacted before the mixture cools and the viscosity increases. The economy of the compaction operation also favours a fast rolling operation. To strike a balance between slow speed and the limited period of time available for compaction under low material resistance, AMIR offers a solution where the compaction pressure is maintained over a pavement section for a relatively longer period depending on the length of the belt in contact with the pavement. The actual speed of the compactor is maintained high enough to finish compaction in as short a period as possible. A long period of time during which pressure is maintained over a section is necessary to allow the particles enough time to reorient themselves and for the asphalt cement to adhere to the aggregate surfaces. The steel roller geometry does not provide sufficient time period during which compaction

pressure would be maintained.

The limitation imposed on the steel roller by its geometry, limiting the contact area, explains the failure of the attempts to employ heavier compactors and larger drums to achieve higher densities. The important element seems to be the compaction energy which is dependent on the length of time the compaction pressure remains over a section in the pavement.

4.2.2 AGGREGATE DISTRIBUTION

Since the asphalt concrete mixture contains a relatively constant percent of bitumen, fine and coarse aggregates, the distribution of large aggregate particles determines the thickness of the bituminous cover. The thickness of the bituminous cover has a significant influence on stripping resistance as discussed earlier in Chapter 3. The uniformity of particle distribution and the degree of particle orientation influence physical as well as mechanical properties of composite materials like asphalt concrete. Researchers and pavement engineers are aware of the necessity for a uniformly distributed aggregate and a great deal of effort has been made in developing the mixture spreader presently used. The aggregate in the mixture behind the spreader has been found to be uniformly distributed, but after compaction, the uniformity of the aggregate distribution is usually lost. The lack of uniformity happens in three dimensions due to the action of the steel roller and its vibration. The lack of uniformity is reflected in the high variations experienced in both physical and mechanical properties of the compacted mixture. The shoving of material causes aggregate movement laterally, as discussed in the previous section, resulting in different aggregate distribution laterally. Gellrer (68) and many other investigators observed the effect of vibration during compaction. As a result of vibration, segregation of material occurs with finer materials migrating to the surface while the coarser material occupying the lower section. It seems that investigators are comfortable with this situation because it has been considered an improvement against surface water penetration. However, the situation is not a favourable one because of the following reasons:

- 1- The reduction in permeability due to the migration of fines to the surface does not help when surface cracks created during compaction are present. The water freely enters through these cracks and travels beyond the so called impervious layer.
- 2- Migration of fines to the surface results in a coarser lower section due to accumulation of large aggregate particles in the bottom section of the asphalt concrete layer. This will result in thinner asphalt films around aggregate particles making such a location more susceptible to stripping. This explains why stripping has been observed in the bottom sections of the asphalt concrete layer.

Preliminary observations made on sections compacted by the two compaction methods confirms the difference in aggregate distribution in the vertical section. AMIR compacted sections showed uniformity of aggregate distribution along the vertical section while the steel roller compaction resulted in accumulation of large size aggregate in the bottom section as shown by Figure 4.1.

Another observation made during construction of the field trials concerns the pavement surface texture. Coated aggregate particles are visible on the pavement surface compacted by the AMIR roller. This surface condition created by AMIR can serve other purposes particularly in maintaining frictional and durable surface characteristics. On the other hand, the steel roller results in migration of fines to the pavement surface which can result in slippery surfaces. In Ontario, studies are being carried out to design separate friction course mixtures to be used on existing highways after safety requirement called for such an action.

4.2.3 PARTICLE ORIENTATION

Particle orientation affects physical and mechanical properties of composite materials. The extent to which particle orientation influences these properties has not yet been investigated. Orientation of low-temperature cracks may be affected by the orientation of the long axis of the large size aggregate as suggested in Section 3.7. This thesis will

report particle orientation as observed on sections cut from field compacted mixtures.

Particle orientation at the micro level is not possible to investigate because of the presence of large aggregate particles in the mixture. In general, aggregate particle orientation depends on the compaction energy and in particular on the duration of pressure application. Orientation to a preferred level is resisted by material on the aggregate sides and by the underlying material whose resistance increases with time because of increased viscosity following temperature drop with time. Therefore, orientation of aggregate must take place at an early stage during the compaction operation with long enough pressure application period. This could be achieved using AMIR compaction method. Preliminary observations agree with this assumption where AMIR compacted sections showed more orientation towards a preferred level while the steel roller compacted section did not indicate any particular orientation.

4.3 SUMMARY

In general, compaction can influence stripping in many ways and the most visible factor is the presence of surface cracks that occur during construction and are associated with the conventional compaction method using steel rollers. Construction cracks allow surface water to enter the pavement. Water presence is of great significance to the stripping phenomenon. A continuous supply of water means stripping will continue to destroy the adhesive bond between aggregate and the asphalt matrix. Reducing the flow of surface water into the pavement is a feasible and effective solution to the stripping problem. Compaction quality directly influences physical and mechanical properties of compacted mixtures. These properties determine the material resistance to stripping damage.

CHAPTER 5

EXPERIMENTAL INVESTIGATION

5.0 INTRODUCTION

The results and findings of the analytical investigation led to the development of a comprehensive experimental investigation. The main objectives of the developed experimental investigation were as follows:

1. To verify the main findings of the analytical phase of this research,
2. To evaluate the existing laboratory test method and confirm the deficiencies identified earlier, and
3. To develop a more reliable testing procedure for the evaluation of stripping susceptibility of field compacted asphalt concrete mixtures.

In order to achieve these objectives the experimental program was designed and implemented based on the following principles:

1. Laboratory testing was performed on asphalt concrete cores recovered from actual field compacted test sections. The field trials were conducted employing three different asphalt concrete mixtures.
2. Influence of field compaction was provided through the utilization of a prototype of the new compactor "AMIR". In all trials, the conventional and AMIR compaction were tested side by side on the same asphalt concrete mixture.
3. Existing laboratory test methods were included in the broader and more comprehensive testing program adopted and implemented in this investigation.
4. The parameters identified in the analytical investigations were considered as independent variables. The parameters included the differential contraction between the asphalt matrix and the aggregate, construction cracks, temperature

range, time of exposure to moisture and freeze-thaw cycles.

5. The evaluation of the stripping susceptibility of the investigated mixtures was carried out using the indirect tensile strength test and visual observations during the period of the experimental program.
6. Prediction of field performance was based on the performance curve that relates the measured strength to the independent variables employed in the test. The use of a single ratio to accept or reject a given asphalt concrete mixture was shown to be unreliable.

The above mentioned objectives and principles resulted in the development and implementation of the experimental program shown in Figure 5.1. As shown in the figure, the current testing approach adopts a single soaking temperature, 140°F, and only one soaking duration, 24 hours. Clearly, this testing method does not consider the effect of cold climate on the stripping problem.

5.1 OUTLINE OF THE EXPERIMENTAL INVESTIGATION

The objectives and principles discussed in the previous section presented the main phases and important tasks of the experimental investigation. The following subsections present three main phases and details of each task.

5.1.1 VERIFICATION OF THE ANALYTICAL APPROACH

The most significant findings of the analytical investigations were the identification of the main variables governing the acting mechanisms of stripping. Specifically, the analysis suggested that thermal incompatibility between the asphalt matrix and the aggregate can result in thermal stresses at the interface between these two components of the asphalt concrete mixture. Thus, in order to verify this result the experimental program must include a wide temperature range as well as freeze-thaw cycles. The second finding was the role construction cracks play in the stripping mechanism. This finding resulted in the selection of field compacted asphalt concrete test sections, where two significantly different compaction methods were employed. Third, the results of the analytical investigations suggested that bond enhancement occurs when

FIGURE 5.1: MAJOR ELEMENTS OF THE EXPERIMENTAL PROGRAM

asphalt mixtures are exposed to cold temperatures. In the other hand, it was speculated that longer durations or very cold temperatures may cause serious damage to the mixture. Accordingly, the developed experimental program included the following variables:-

1. Temperature conditioning
2. Moisture conditioning
3. Field trials

The main variables are discussed in more details in the following subsections.

5.1.2 TEMPERATURE CONDITIONING

Soaking a partially saturated sample in a water bath at 140°F will not reveal mixture performance as discussed in Chapter 2. Therefore, the designed experimental program included a wide range of temperatures. Different regions are expected to adapt different temperature ranges to simulate the prevailing climate conditions. Instead of concentrating on the conditioning that produces stripping in the laboratory, samples were exposed to different temperatures and the response of the test cores to the specific conditioning was observed during and after the strength test.

To verify the different mechanisms that directly influence the adhesive bond or initiate the stripping damage, more than one conditioning procedure was necessary. The temperature conditioning procedures included:

1. Partially saturated cores were simply soaked at the designated temperature. Temperatures used were -20, 35, 65, 110 and 140°F. These temperatures cover both cold and warm pavement temperatures during a full calendar year in a region like southern Ontario.
2. Other groups were exposed to freeze-thaw cycles. This permits evaluation of the combined effect of exposure to both cold and warm temperatures. This was used to verify the hypothesized influence of cracks initiated during contraction as described by the results of the analytical investigation. This conditioning was expected to produce severe damage compared with other exposure conditions. The designed experimental program included three freeze-thaw cycles. All cores

belonging to this experiment were frozen at -20°F for 24 hours and then allowed to thaw at different temperatures for 24 hours and the cycle was repeated three times as mentioned earlier.

3. To analyze the influence of each conditioning level, groups consisting of three samples were tested at 75°F without conditioning. Each of these groups are referred to as a "control group" and it was selected from the same mixture and compaction method with relatively similar specific gravity. All of the indirect tensile strength tests were performed at 75°F in order to eliminate the influence of the mixture stiffness which is temperature dependent.

5.1.3 MOISTURE CONDITIONING

The term "moisture conditioning" is used to indicate exposure to water and is usually measured by the degree of saturation. Vacuum pressure have been used for laboratory moisture conditioning. A review of the literature indicated that vacuum saturation of asphalt samples is a rather simple and effective means of filling the voids with water. Subsequently, air void content has been used to determine the potential intensity of exposure to water.

The vacuum level needed for a certain degree of saturation differs from one mixture type to another. Different levels of vacuum pressure ranging from 14 inches to 24 inches of mercury were applied for a duration of 5 to 30 minutes as have been used in the past. The degree of saturation is usually calculated by expressing the volume of water in a saturated specimen as a percentage of the volume of air voids in the dry specimen. Arbitrary use of high vacuum pressure can result in an unknown degree of saturation and hence a different degree of exposure to water effects and the pressure may end up damaging the sample after all the available voids are filled. To avoid such a damage, a saturation level less than 100% has been suggested which limits saturation level to a degree between 50 and 80 %. A more cautious approach restricts the saturation level to 50% where only 50% of the voids are filled with water and those voids filled are called the permeable voids beyond which the sample will be blown up and more voids will be created and saturation level will exceed the 100% maximum level.

Presently the level of overall saturation of a pavement mixture in its later life is not accurately known. The designed experimental program included more than one vacuum pressure level. To avoid any damage due to the vacuum operation, which may obscure the influence of the other factors under investigation, the conservative view calling for less than 50% degree of saturation was adopted. The use of vacuum saturation will obscure the effect of construction cracks which allow the free flow of surface water into the pavement. Therefore, a number of cores from both compaction methods were conditioned without vacuum saturation. These groups were employed to investigate the influence of construction cracks on the stripping resistance of mixtures compacted with different compaction methods.

5.2 FIELD TRIALS

Three field trials were designed and carried out to provide asphalt specimens to satisfy the first principle of the research investigation. The three field trials are described below.

TORONTO TEST

The first field test was carried out in Toronto June 1988. The test was designed to compare between the quality of compaction of AMIR compactor and a conventional static steel roller. The total weight of AMIR was slightly less than that of the steel roller. Thus, the pressure under AMIR was much less than under the steel roller. The section was selected on the unpaved parking lot at the site of the Lovat Tunnelling property. Ontario Ministry of Transportation (MTO) standard HL-3 asphalt mix was employed in this test. Mix design details are given in Appendix A. Each roller made three passes on its designated section. One week later, a total of 104 asphalt cores were recovered from the test sections for further laboratory evaluation.

OTTAWA TEST No.1 (NRC #1)

After the Toronto experiment, the need to investigate other variables lead to construction of the first of two more field tests carried out on The National Research

Council of Canada (NRC) campus in Ottawa. The first Ottawa test was carried out to compare the quality of AMIR compaction to the one obtained by the combination of a steel vibratory and rubber wheeled rollers. The test section was an old paved access road on the NRC campus in Ottawa. A standard MTO HL-4 asphalt mix was employed in the test. Mix design details are given in Appendix A. The AMIR compactor started its operation fully loaded. The finished sections were divided as follows:

1. Block 0, represents the part of the compacted asphalt under which the AMIR roller was fully loaded representing AMIR potential if such a weight is adopted in the commercially produced roller.
2. Block 2 and 3, represents the part of the compacted pavement where thick lifts were used. The conventional roller weight was constant and the number of passes it made were the same on each block.

Figure 5.2 shows the lay out plan of the paved section and the location of the different blocks mentioned earlier.

OTTAWA TEST No. 2 (NRC # 2)

The third field trial was conducted on the last day of the paving season in Ottawa. The temperature on that day, Nov. 30, 1989 was 5°F, and the test site was covered with snow. The paving contractor cleaned the surface and the paving operation was completed. A standard MTO HL-3 mix was laid on top of an old pavement in two lifts. The road had two lanes. The conventional compactors were similar to the combination used in the first Ottawa test. Thus, one can compare between two mixes, HL-4 and HL-3 compacted by the same conventional compactors versus AMIR technique. The cores recovered from the third field trial were divided into top and bottom cores.

It is important to indicate that the three field trials were conducted by three different contractors. Each contractor was briefed on the objectives of the test and was expected to do his best to achieve the following criteria:

1. Crack free surface
2. Highest densities
3. High uniformity

**FIGURE 5.2: LAYOUT PLAN OF OTTAWA FIELD TRIAL NO.1
(NRC TRIAL #1) CARRIED OUT ON THE NATIONAL RESEARCH
COUNCIL OF CANADA CAMPUS IN OTTAWA (1988)**

The designed experimental program considered the presence of construction cracks and the direction of compaction in general during samples preparation stage. Since indirect tensile strength test was included in the designed experimental program as the strength test, cylindrical samples were collected by coring in the prepared field sections. The direction of compaction was marked on the pavement before lowering the coring machine to the specified core position. This enabled the author to identify the compaction direction and to load all samples with respect to a common direction thereby reducing the high variation found in previous research work.

The majority of cores used in the study were 3.7 inches in diameter but a limited number of 4.7" and 2.7 inch diameter cores were also collected. The depth differed according to the lift thickness.

5.3 LABORATORY TESTING METHODS

The second objective of this investigation was to confirm the deficiencies identified in the present testing methods. This objective was met by including the existing laboratory testing procedure within the experimental approach followed throughout this investigation. New conditioning procedures are discussed in details in this section. For standard tests, only the problems encountered during experiments are discussed. All the cores used in the investigation were stored in the Geotechnical laboratory of Carleton University prior to testing. Drying of cores required for bulk specific gravity and percentage of air voids determination was performed using a desiccating material to avoid any heat treatment prior to the conditioning stage.

5.4 STRIPPING SUSCEPTIBILITY TESTS

The development of the testing program followed a pilot project designed and implemented as part of a report prepared for the Ontario Ministry of Transportation (MTO). A new testing procedure as well as the conventional procedure were used to

evaluate stripping susceptibility of the investigated mixtures (37). This section defines some of the terms used in the data analysis of the thesis. It also specifies the number of groups used and the specific exposure level associated with each experiment.

5.4.1 CONVENTIONAL APPROACH

A single conditioning temperature was used for the conventional approach. Saturation was achieved by applying 24 inches of mercury vacuum pressure. This was followed by one of the following temperature conditioning procedures;

1. Cyclic freezing-thawing (-20°F and 140°F) was used following vacuum saturation. The cycle consisted of freezing cores at -20°F for 20 hours followed by soaking cores in a water bath at 140°F for 24 hours. Three such cycles were used after which the cores were soaked in 75°F water bath for two hours before loading the core.

2. The second method used consisted of simple soaking in a warm water bath for 24 hours. The recommended water temperature of 140°F was maintained for 24 hours after which the cores were soaked at 75°F for two hours. This was followed by indirect tensile strength test.

The first method has rarely been used in stripping investigations because only the warm portion of the cycle was considered responsible for the observed stripping damage. Laboratory studies comparing between the two procedures could not offer any explanation when freeze-thaw cycles produced more damage than simple soaking (24).

The conventionally conditioned groups represent a single condition within the new approach and therefore the results were included on the same tables. The tables will list the different temperatures under investigation including the 140°F soaking temperature which constitutes the whole conventional temperature conditioning requirement.

5.4.2 DEVELOPMENT OF A FRAMEWORK FOR LABORATORY TESTING

The third and final objective of this investigation was to develop a more reliable testing approach for predicting stripping. Phase I of the MTO report mentioned earlier (37), concluded that the existing stripping susceptibility evaluation procedures are not sufficient to predict how asphalt concrete will perform in the field. Phase II (38) of the

study designed an alternative to the existing evaluation procedure. Wider ranges of conditioning temperatures were used in the report and the results offered a better opportunity to study the asphalt concrete performance (38). The laboratory test method can be used for evaluating the stripping susceptibility of asphalt concrete mixtures.

As shown in figure 5.3, the two hypothetical mixtures a and b show two significantly different strength performance. However, according to present testing methodology, both mixtures will be evaluated as having the same degree of stripping susceptibility. This is due to the ratio between the strength at both ends of the curve. Clearly this result is misleading. According to the curves shown, mixture a shows better performance over mixture b. Based on this analysis, the experiment necessary for proper evaluation of stripping susceptibility must consider a much wider temperature range than suggested by the conventional approach. Freeze-thaw conditioning is also an essential part of the suggested stripping susceptibility evaluation method. The steps followed in the investigation included:

1- Mixture Performance at Different Soaking Temperatures:

After saturation using 14 inches of vacuum pressure, each group was soaked at a different temperature (3 cores in each group). Three soaking periods were used which included 2 days, 7 days and 14 days. These periods are based on the drainage ratings from AASHTO Guide for Design of Pavement Structures, 1986. The 2 and 14 day exposure periods were carried out first and after one year the 7 day exposure program was executed. The -20°F group was wrapped and placed in an environmental chamber for the same duration as the other temperatures. At the end of the specified soaking period, each group was moved into a water bath at 75°F and kept for two hours before loading. Indirect tensile strength test was then performed on all cores and the average strength of the three cores representing a group was used to evaluate the effect of different exposure levels.

FIGURE 5.3: ILLUSTRATION OF EVALUATION CRITERIA FOR STRIPPING SUSCEPTIBILITY

2- Freeze-thaw Cycles:

Freeze-thaw cycles conditioning program covered a wide range of thawing temperatures. It included thawing at 65°F, 75°F, 85°F and the conventional 140°F. The cycle started with vacuum saturation using either 14 or 24 inches of pressure in an effort to study the influence of saturation level on the potential moisture damage. The cores were then frozen for 20 hours followed by thawing at the selected temperature level for 24 hours. Three cycles were used for all groups investigated in this study. After the last cycle, the cores were then soaked in a water bath at 75°F for 24 hours prior to the indirect tensile strength test. The tables showing the test results include details such as the vacuum pressure level, thawing temperature, bulk specific gravity of each core and the permeable voids percent corresponding to the saturation level achieved by vacuum pressure.

5.5 EVALUATION OF THE INFLUENCE OF COMPACTION TYPE ON STRIPPING SUSCEPTIBILITY

Compaction type can influence physical as well as mechanical properties of asphalt concrete mixtures. Chapter Four discusses some of these properties and how they may influence stripping susceptibility of asphalt concrete. The influence of density, air voids content and indirect tensile strength on the stripping resistance of an asphalt concrete mixture have been investigated in the past. Since all previous studies were investigating the performance of one compaction method, those elements which are characteristic of the compactor used remained undetected. However, this study used two uniquely different compaction methods to produce cores for the investigation. Construction cracks was the first and most visible element that differentiate between the different compaction techniques. Therefore a separate experiment was carried out to investigate the influence of these cracks on the stripping susceptibility of compacted asphalt concrete mixtures. Four groups from each compaction method were used for investigating the influence of surface cracks on the stripping problem. These groups were not exposed to vacuum pressure and the saturation degree was only controlled by the ability of water to freely

enter through the surface pores and cracks. The sample sides and bottom were wrapped by plastic sheets as an attempt to stop water flow from the sides and bottom. This was expected to limit water entrance to the top section, only, and to permit assessment of the role the surface cracks play in causing stripping damage .

The wrapped cores representing each of the different conditioning temperature were soaked in a water bath at the specified temperature. Water bath temperatures included 35°F, 65°F and 110°F. Cores remained in the water bath at the specified temperature for 7 days. Following this treatment, the cores were relocated in a water bath for 2 hours at 75°F before indirect tensile strength testing. Prior to loading, the amount of water that managed to inter the cores was determined as a percent of the total weight of the core.

5.6 CONTROL GROUPS

Groups, each consisting of three cores, were tested to allow for comparison between the strength before and after conditioning. These control groups are representative of each specific location, mixture and compaction method. This was expected to allow true comparison between different compaction methods. The use of relatively equal bulk density was also attempted because of the density effect on strength. This was not possible all the time because AMIR compaction provided higher density in most cases.

Control groups for the experiment investigating the influence of temperature change were vacuum saturated by the same effort prior to testing to eliminate any possible difference due to vacuum pressure. For freeze-thaw cycles, control groups were tested dry following the conventional approach. Similar to conditioned cores, cores of the control groups were tested at 75°F.

5.7 STANDARD TESTS

In order to evaluate the influence of the parameters presented in the previous sections on the stripping susceptibility of the investigated mixtures, The following standard tests were adopted for all test samples.

5.7.1 BULK SPECIFIC GRAVITY

Bulk specific gravity tests were performed according to ASTM D2726-73 (79) using saturated surface dry specimens. According to this standard test:

$$\text{Bulk specific gravity} = A/(B-C)$$

where,

A = weight of the dry specimen in air, gm

B = weight of saturated surface dry specimen in air, gm

C = weight of specimen in water, gm

The denominator of the above equation contains the weight of saturated surface dry specimen and its accuracy is quite essential to the overall accuracy of the bulk specific gravity value. During tests in the laboratory it was noticed that water pouring out of the specimen through the cracks on the surface of steel compacted cores. Loss of water will result in artificially high value of bulk specific gravity of steel compacted cores. The same observation was made by Santucci (35) and it has been associated with mixtures having high air voids contents. Although the air voids in the two Ottawa trials were not as high as those investigated by Santucci, still water escaped through the cracks initiated during compaction using conventional rollers. An alternative available for such a condition was to use wax-coated procedure. The wax-coated procedure is known to expose the cores to high temperatures which may alter the nature of the core and seal the existing cracks obscuring their influence on the stripping problem. Direct measurements of core dimensions to determine volumes is not accurate because of irregularities in the cores caused by drilling operation. Therefore, and in spite of the fact that water was escaping through the surface cracks, bulk specific gravity as specified by ASTM D2726 was used in the investigation.

5.7.2 VACUUM SATURATION AND PERMEABLE VOIDS DETERMINATION

The Transport Canada method, described in Appendix B, was used to saturate cores in the laboratory. Permeable voids for partially saturated cores was calculated using the formula suggested by Transport Canada (see Appendix B).

The calculated permeable voids were only used to compare between the different compaction methods and not as an indication of the actual air voids in the mixture. A 100% saturation degree is necessary to calculate air voids percent using the same formula. But a 100% saturation degree is not possible without the imminent core destruction under high vacuum pressure required to achieve saturation. Lower saturation levels were therefore targeted using low vacuum pressures and the calculated permeable voids percent should always be associated with the vacuum pressure level used during the saturation stage. Therefore, the calculated permeable voids using 14 inches of mercury vacuum pressure were lower than the actual air voids content of the mixture since the degree of saturation was less than 50%. High vacuum saturation forces the water into the core's interior whether there are surface cracks or not thereby obscuring the influence of those cracks on the stripping problem. Based on the above discussion, the experiment which investigated the influence of construction cracks did not include any vacuum saturation.

5.7.3 INDIRECT TENSILE STRENGTH TEST (ITS)

Indirect Tensile Strength (ITS) has already gained a wide popularity and has been extensively used in moisture damage prediction worldwide (6) (7) (21) (24) (25). The loading configuration develops a relatively uniform tensile stress perpendicular to the direction of the applied load and along the vertical diametral plane, which ultimately causes the specimen to fail by splitting along the vertical diameter. The Transport Canada recommended procedure was used throughout the investigation. Figure 5.4 shows the actual arrangement of the instrumentation. Details of the test procedure are shown in Appendix B. Based on the evaluation presented in Chapter 2, a loading direction perpendicular to the compaction direction was used throughout the investigation.

5.8 REMARKS

The curing period affects the indirect tensile strength of the tested cores. A number of cores from the Toronto field trial were tested dry to establish the rate of increase in indirect tensile strength for the two compaction methods used in the investigation. Groups investigating the effect of temperature change on the stripping susceptibility of

a given mixture for a specific duration were collected, conditioned and tested at similar dates to avoid the effect of curing on the indirect tensile strength. As mentioned earlier, groups investigating the 2 and 14 day durations were tested at the same age . The other groups deigned to investigate the effect of 7 day exposure duration were tested one year after the first two exposure durations. The different in the cores edge was expected to lead to higher indirect tensile strength of the cores exposed for the 7 day period.

FIGURE 5.4: INDIRECT TENSILE STRENGTH TEST SETUP

CHAPTER 6

OBSERVATIONS

6.0 INTRODUCTION

The objective of this chapter is to report observations made during and after the construction of field trials, and other observations made during the laboratory investigations and after indirect tensile strength tests. The importance of these observations stems from the fact that this study is the first effort investigating the influence of compaction on stripping susceptibility and the concept of differential thermal contraction/expansion. Reporting these detailed observations will help future research work in the subject of asphalt concrete stripping to design the appropriate experimental program for further investigations. The relatively large number of tested cores from three different field trials provided an excellent opportunity to investigate a large number of potential variables associated with temperature and moisture conditioning. Observations made during each specific period are reported separately.

6.1 CONSTRUCTION PERIOD

Material properties associated with compaction such as aggregate distribution, particle orientation and physical as well as mechanical properties are not yet directly linked to the stripping resistance of asphalt concrete mixtures. Observations made during construction and during the laboratory testing period explained some elements of the stripping phenomenon. Construction induced cracks were found to directly influence material resistance to stripping.

6.1.1 COMPACTION OPERATION

During NRC trial # 1, the contractor increased the number of passes, using a vibratory steel roller and a pneumatic roller, in order to achieve the best possible compaction as instructed. The action of the steel roller was in conformance with previous experience, that is, the steel roller settled into the mix and lateral displacement of the mix occurred in the first pass. Surface cracks, or checking as it is usually named, were visible after the first pass. Then the pneumatic roller entered the pavement and made more than thirty passes. Those cracks appearing after using the steel roller continued to exist and the pneumatic roller failed to seal them contrary to what has been claimed in the past. Figure 6.1a shows the asphalt concrete surface compacted by the AMIR roller with no cracks. Figure 6.1b shows the steel compacted section.

The AMIR roller entered its section fully loaded in order to examine the maximum potential of the roller. After the first pass, it was clear that under such a heavy weight steering will be difficult and therefore it was taken out and emptied to remove the additional weight of water inside the drums. The location where AMIR used its full weight was marked as block O.

Another observation made after construction concerns the surface texture of sections compacted by the current compactors. The section compacted by the steel roller indicated more finer material on the surface than that on surfaces compacted by the AMIR roller. The AMIR compacted section showed relatively greater numbers of coated large aggregate particles.

6.1.2 CONSTRUCTION CRACKS

Construction cracks were visible along the full length of the sections compacted using a combination of vibratory steel roller and the pneumatic roller. The direction of some of these cracks was perpendicular to the direction of rolling and some cracks were positioned with different angles with respect to the rolling direction. No cracks were seen on the AMIR compacted section.

(a)

(b)

FIGURE 6.1: COMPACTED ASPHALT CONCRETE SURFACE
(a) AMIR COMPACTED SURFACE
(b) STEEL ROLLER COMPACTED SURFACE

6.1.3 CORING OPERATION

During the coring operation a large number of cores from the AMIR compacted section came out from their locations with the asphalt concrete part from the old layer attached to the part from the new layer. Almost all cores of the new layer from the steel compacted section came out unattached to the sections from the old asphalt concrete layer. Figure 4.1 shows typical cores from both sections with the steel compacted section indicating separation between the layers. A similar observation was made in NRC trial # 2. This indicated that the two uniquely different compaction techniques influenced the adhesion forces between the old and new layers in a different manner.

6.2 CONDITIONING PERIOD

Observations made during and immediately after the different conditioning methods are summarized as follows:

- 1- Soaking asphalt concrete core in the warm water bath (temperature between 110°F and 140°F) resulted in the separation of the asphalt cement in the compacted mixture into a dark colloid and an oily medium suggesting a possible structural change of the bituminous material.
- 2- Discolouring of the cores surfaces was observed when high water temperatures were used for moisture conditioning. The degree of discolouring depended on the temperature level and soaking duration.
- 3- Longer soaking periods at elevated temperatures also resulted in loss of the bituminous cover from fine aggregate surfaces.

6.3 OBSERVATIONS MADE AFTER STRENGTH TEST

Indirect tensile strength tests caused cores to fail by splitting along the vertical diameter. The failure surface has been used by investigators to estimate the degree of stripping since it is the only location within the core interior exposed after conditioning.

The following subsections present the observations made on the failure surfaces organized according to the specific field trial. Within each field trial, the different groups are identified by the type of compaction used and the soaking duration allowed during the conditioning stage.

6.3.1 OTTAWA TRIAL NO.1 (NRC # 1)

Cores From AMIR Compacted Section

2 Day Soaking Period

140°F:

Within this group, some coarse aggregate particles at the failure surface completely lost their thin bituminous cover while a limited amount of the fine aggregate lost its bituminous cover. Many other large aggregate particles were separated from their asphalt matrix cover but were still black in colour and had a glassy reflection on the surface indicating a mild degree of oxidation on the thin bituminous film. No pronounced discolouring took place due to the limited soaking period of two days. The fine aggregate particles on the surface remained firmly attached indicating a mild degree of adhesion loss.

110°F:

Cores in this group suffered a lower degree of stripping compared with cores from the 140°F group. Less separation of the thin bituminous cover was observed but the loss of the asphalt matrix cover was similar to the 140°F group. The imbedded portion of the aggregate particles close to the failure surface remained firmly held. Much less discolouring took place and aggregate prints were left on the asphalt matrix. Exposed part of coarse aggregate particles were less in size when compared with the 140°F group indicating a lower degree of stripping. Separation took place between coarse aggregate particles and the asphalt matrix while a thin bituminous film cover was left around aggregate surfaces.

65°F:

Some aggregate surfaces did not have their asphalt matrix cover intact as observed

on the failure surface. But when compared with the control group, the estimated loss of cover due to conditioning was very little. No discolouring was observed on the failure surface and on the core surface in general. Only sharp edges of aggregate particles were broken.

35°F:

Compared with the control group, very little loss of asphalt matrix cover was observed. The control group did not have complete coverage on the failure surface although it was not exposed to any temperature conditioning. No discolouring was observed due to the 35°F temperature conditioning.

-20°F:

No loss of cover was observed on the failure surface and it was quite similar to the previous group. A limited number of aggregate particles were broken during the load application period.

7 Day Soaking Period

These groups were selected for longer periods of exposure compared with the previous two days exposure level. The average bulk specific gravity of each group in this section was higher than that of the previous two day exposure groups. This resulted in generally higher ITS values compared with the two days but the loss of aggregate asphalt matrix coverage was much higher due to the long exposure period.

140°F:

Detachment of the asphalt matrix cover away from aggregate surfaces was quite clear. The loss of cover extended to include the thin bituminous film of some aggregate particles. The detachment of the asphalt matrix from aggregate surfaces was observed during the load application period. Detachment of the asphalt matrix from aggregate started long before complete failure of the sample. Figure 6.2 shows the detachment that took place prior to crack propagation through the asphalt matrix. This observation was possible because the bottom sections of the cores were removed by sawing to separate the new pavement section of the core from the old one. The interior of the cores indicated

an extensive loss of aggregate cover on one side of the failure surface and the aggregate prints on the other side . A small number of aggregate particles retained their cover as observed on the failure surface. The exposed portions of the large size aggregate particles indicated that the longer axis of those aggregates was approximately parallel to the horizontal direction of the pavement.

110°F:

The asphalt matrix cover loss was less than that observed during the 140°F exposure level. There were a number of exposed aggregate particles observed on the failure surface which lost their thin asphalt film but it was less than the 140°F group. Before failure, the asphalt matrix cover started to pull away from the aggregate surface prior to crack propagation through the rest of the core diameter. Less degree of discolouring was observed compared with the 140°F.

65°F:

Unlike the two days exposure group, a small number of aggregate particles lost their asphalt matrix cover. A small number of aggregate particles completely lost their cover. Separation similar to the one experienced during 110 and 140°F exposure levels was not repeated here. The unexposed portion of the aggregate particles remained tightly held in place and it was not possible to remove by hand.

35°F:

Although the indirect tensile strength test indicated a higher strength compared with the control group, actual inspection of the failure surface indicated a number of locations where a loss of asphalt matrix coverage was observed. When this group was compared with the control group, i.e. the unconditioned cores, the same degree of coverage was observed indicating that the loss of cover of this group was inherited like the control group and not due to temperature conditioning.

**FIGURE 6.2: DETACHMENT OF THE ASPHALT MATRIX
PRIOR TO CRACK PROPAGATION DURING INDIRECT
TENSILE STRENGTH TEST**

-20°F:

No loss of cover was observed, but this time the asphalt matrix cover was partially removed from around the sharp edges of the aggregate particles close to the failure surface. Other than the observed rupture of the asphalt matrix, the overall condition of the failure surface was not different than that of the control group. Although the ITS strength was high, no excessive aggregate breakage was observed on the failure surface.

14 Day Soaking Period

The 14 day soaking group represented extreme exposure conditions where partially saturated asphalt concrete mixtures remain exposed at different temperatures for extended periods of time. The mixture suffered different types of damage depending on the temperature level as described below.

140°F:

Excessive loss of cover was observed and large portions of the aggregate particles were exposed. Many aggregate particles lost their thin bituminous film and the loss of adhesion extended below the failure surface. Some of the aggregate particles were easily removed by hand. Fine aggregates were also stripped of their bituminous cover. The observed degree of discolouring was the highest among AMIR compacted cores exposed to warm water soaking. Loss of the adhesive between asphalt cement and fine aggregate particles led to easy removal of fines from the cores surfaces. The relatively large oil spots observed on the water surface in the warm water bath and the discolouring indicated a high degree of oxidation and the consequent structural change of the asphalt cement part of mixture.

110°F:

Loss of cover was observed at the failure surface. A small number of aggregate particles lost their thin asphalt film and the loss of asphalt matrix coverage around aggregate was less than that observed on the 140°F group.

65°F:

There was more loss of cover than that observed on the control group. Compared with the 2 and 7 days exposure level, it seems that the asphalt matrix cover loss was progressing with time. Some aggregate particles retained their asphalt matrix cover. No loss of cover from around fine aggregate particles was observed.

35°F:

No stripping was observed and aggregates retained their cover. One difference observed between this group and the 65°F group was the increase in the number of broken aggregate due to the application of the load during the indirect tensile strength test. The number of broken aggregate particles was more than that observed on the control group.

-20°F:

Partial loss of the asphalt matrix cover was observed at the sharp edges of many aggregate particles. Actually only those sharp edges could be seen on the failure surface while the rest was covered with the asphalt matrix.

Cores Compacted By The Steel And the Pneumatic Rollers***2 Day Soaking Period***

Five groups were exposed to two days soaking at different temperatures to examine the influence of short exposure duration. The response of the partially saturated asphalt concrete mixture to the different temperature conditions is described below. The average bulk specific gravity of each group was 2.39.

140°F:

More than 50% of the aggregate particles on the failure surface lost cover including the thin bituminous film. Some aggregate particles retained their thin bituminous film but lost the asphalt matrix cover. Slight discolouring was observed on the failure surface and on the core surface. Aggregate particles prints were observed on both sides of the failure

surface indicating adhesive failure between aggregate and the asphalt matrix cover. Also, during loading of the cores the first crack occurred on the surface, before propagating to the bottom section of the core. The failure surface of conventionally compacted cores followed existing construction cracks. This observation was made during indirect tensile strength tests as well as by inspection of the failure surface after the test. Dust and foreign material were observed on the failure surface. Figure 6.3 shows a failure surface and the dust near the cores surface indicating a construction induced crack.

There was no specific aggregate particle orientation on the failure surface. Some aggregate particles were oriented with their long axis parallel to the pavement surface while others show a random orientation. The conventional method was not used here since it estimate the degree of stripping by comparing the exposed aggregate area with respect to the total failure surface. The majority of large aggregate particles were located on the lower section of the cores. The aggregate particles in the lower section were so close to each other and some aggregate particles did not have asphalt matrix cover between the particles as shown in Figure 6.4.

110°F:

Relatively less damage was observed at this level of exposure compared with the 140°F group. Loss of cover observed at the failure surface ranged from loss of the asphalt matrix to total loss of both the asphalt matrix and the thin bituminous film in direct contact with aggregate surfaces. Aggregate particles exposed on the failure surface resisted removal by hand. A large number of coarse aggregate particles were located in the lower section of the cores. No discolouring was observed due to this exposure level.

65°F:

A limited number of aggregate particles experienced partial loss of the thin bituminous cover. A larger portion suffered partial loss of the asphalt matrix cover. Some aggregate did not have any cover indicating that these aggregate particles were broken during compaction or because of the load applied during the indirect tensile test.

**FIGURE 6.3: FAILURE ALONG CONSTRUCTION
INDUCED CRACKS**

**FIGURE 6.4: AGGREGATE SEGREGATION
OBSERVED ON THE FAILURE SURFACE**

35°F:

The first crack observed during the indirect tensile strength test occurred on the cores surface. Inspection of the failure surface revealed again that failure occurred along an existing crack initiated during compaction as indicated by the dust found inside the failure surface.

Two of the three cores experienced partial loss of the asphalt matrix cover and the third core's cover was intact. Comparing this group with the control group revealed a similar lack of cover around some aggregate particles on the control group.

-20°F:

Partial loss of the asphalt matrix covering some aggregate particles were observed on the failure surface. Aggregate particles were not oriented in a certain manner. At times they were very close to each other and the gaps were not filled with asphalt matrix. Some aggregates were broken due to the load applied during the test or during compaction.

7 Day Soaking Period

Cores used in this section were selected to represent the best compaction achieved by the conventional compaction method using a vibratory steel roller and a pneumatic roller. The average bulk specific gravity of the cores, 2.420, was higher than the average value for cores used in the two days soaking period. The cores were also chosen because they were free from construction cracks. This was expected to permit assessment of the surface cracks participation in producing stripping damage by comparing them with cracked cores.

140°F:

The number of coarse aggregate particles that totally lost their cover were much more than those of the two days exposure period under the same temperature conditioning level. Discolouring was also more evident and more aggregate marks on the failure surface were

present indicating more adhesive failure. Separation of the asphalt matrix cover from aggregate particles extended beyond the failure surface and removal of aggregate by hand was possible. The random orientation of large aggregate particles was quite evident and some of the larger aggregate surfaces were parallel to the failure surface, i.e. perpendicular to the pavement surface.

110°F:

The observed loss of the asphalt matrix cover and the thin asphalt film cover was less than the 140°F condition. The stripped aggregate were concentrated at the bottom section of the cores where large aggregate particles were positioned after compaction. Other aggregate particles retained their thin bituminous film cover but partially lost the asphalt matrix cover.

65°F:

A number of aggregate particles lost their asphalt matrix cover. The cover loss was more than that observed on the control group. Failure occurred by splitting along the vertical diameter confirming the absence of construction cracks. Adhesion failure occurred between the asphalt matrix and the thin bituminous film surrounding the aggregate particles.

35°F:

A limited cover loss was observed which was similar to that observed in the control group. Failure took place along the vertical diameter confirming the absence of construction cracks.

-20°F:

The cores condition after failure was similar to the 35°F group except for the presence of some broken aggregate particles. These aggregate particles were either broken during compaction or due to the load applied during the indirect tensile test.

14 Day Soaking Period

Samples for this section were from Block O where the layer thickness was less than Block 3. This soaking period was the longest exposure duration used in the investigation. The average bulk specific gravity was similar to that of groups used in the two day exposure duration.

140°F:

All cores from this group showed extensive loss of the asphalt matrix cover as well as the thin asphalt film due to the long exposure duration. The amount of cover loss was magnified because of the particle orientation. Large areas of aggregate particle surfaces were facing the failure surface and stripping resulted in the exposure of those surfaces. Therefore, using the exposed surfaces at the failure surface to evaluate the degree of stripping is not an accurate procedure since aggregate orientation differ from one core to another. Inspection of failure surfaces indicated that failure occurred along construction cracks.

Large aggregate particles were concentrated on the bottom section and very thin cover was observed between the adjacent particles. These aggregate particles were removed without any effort since their cover was completely lost.

110°F:

Less degree of cover loss was observed compared with the previous group. Removal by hand of aggregate particles was not easy compared with the previous group indicating that they were still partially held by the asphalt matrix beneath the failure surface. Aggregate orientation and distribution was similar to the previous group.

65°F:

Some aggregate particles lost their thin asphalt film. The asphalt matrix was also separated from some aggregates while the thin asphalt film was still intact. Marks indicating adhesive failure were also observed on the failure surface.

35°F:

Unlike other conditioning durations, exposure to 35°F for such a long time resulted in some adhesive failure indicated by aggregate marks on the sides of the failure surface. A limited number of aggregate particles lost the thin bituminous film.

-20°F:

Partial loss of the asphalt matrix cover was observed. Some aggregate marks were observed indicating adhesive failure in a limited scale although it was only soaked in water at room temperature for two hours. Failure started at the cores top surface where existing construction cracks acted as a predetermined failure zone.

6.3.2 TORONTO FIELD TRIAL

Cores from the section compacted by the steel roller were kept in the laboratory for more than one year before being tested to investigate the effect of moisture and temperature conditioning. Cores were soaked in the water bath at the specified temperature for a 2 day period before the indirect tensile strength test. In general, no discolouring was observed due to any of the exposure conditions attempted. The majority of cores collected from this field trial were used in freeze thaw investigations.

140°F:

This was the only group where a limited number of aggregate particles were separated from their asphalt matrix cover. No discolouring was observed at the location where separation took place. No loss of the thin bituminous film was observed from aggregate on the failure surface. Small size aggregate particles and finer material retained their asphalt material coverage.

110°F:

No difference in colour or shape was observed between this group and the control group. Locations where adhesion failure and its intensity observed were similar to that of the control group. Fine aggregates retained their cover.

65°F:

No change in the nature of failure surface was observed due to this exposure condition compared with the control group.

-20°F:

No difference was observed between this condition and the control group.

6.3.3 OTTAWA TRIAL NO. 2 (NRC # 2):

Due to the fact that this section was constructed during the winter, the mix was compacted at a low temperature. The mixture was transported during low air temperature and remained in the truck for a long period before the site was ready for construction. Cores from this site were exposed to two days of soaking in a water bath at different temperatures. The following are observations made during and after testing.

Cores From AMIR Compacted Section**140°F:**

The exposure to such high temperature did not result in a loss of the thin bituminous film cover. Some aggregate particle lost the asphalt matrix cover. Marks where aggregate particles were in contact with the asphalt matrix were observed on the failure surface. No discolouring was observed on the surface of cores.

110°F:

No change was observed in the cores interior based on the on the condition of the control group. The cores retained their original colour prior to warm water soaking.

85°F:

A limited number of aggregate particles lost their asphalt matrix cover. The thin bituminous film covering the aggregate particles remained intact. Aggregate exposed on the failure surface were easy to remove by hand indicating loss of adhesion.

65°F:

The group performed in a similar manner to the 85°F group as described earlier. Marks where aggregate particles previously adhered to the asphalt matrix represented locations where adhesion failure occurred.

35°F:

No change was observed on the failure surface and aggregate particles were held firmly on both sides of the failure surface. Particle orientation indicated that the larger axis was parallel to the core surface. Only sharp edges of aggregate particles could be seen on the failure surface.

-20°F:

This group performed like the 35°F group. The aggregate particles retained their asphalt matrix cover.

Cores From The Steel Roller Compacted Section

Although the asphalt concrete mixture compacted by the steel roller was the same as that compacted by the AMIR compactor and although the same air temperature prevailed, the steel roller produced lower compaction quality. Nuclear gauge measurements, performed immediately after construction, showed that the conventional compaction technique achieved very low density.

140°F:

Similar to cores compacted by the AMIR roller, cores compacted by the conventional roller did not suffer discolouring due to the 140°F conditioning. A limited number of aggregate particles lost their thin bituminous film cover. Some aggregate particles lost

their asphalt matrix cover. Exposed aggregate were not firmly held and was possible to move by hand

Cores started to disintegrate one week after testing as shown in Figure 6.5. Separation at this phase took place in the asphalt matrix and not close to the aggregate surfaces.

110°F:

The thin bituminous film remained intact after this exposure level. Cores started to collapse due to their own weight one week after the indirect tensile strength test. No stripping was observed on these new failure surfaces.

85°F:

The thin bituminous film cover was intact but a limited number of aggregate particles lost their asphalt matrix cover. This was indicated by the aggregate mark left on the asphalt matrix on the failure surface. These locations were found wet immediately after the indirect tensile strength test.

65°F:

This group performed like the 85°F group. The cores continued to disintegrate after the first week following the indirect tensile strength test. A limited number of aggregate particles separated from their asphalt matrix cover.

35°F:

The thin bituminous film cover remained intact. Marks indicating pull-out of aggregate particles from the asphalt matrix mix were observed indicating a limited loss of adhesion. The cores suffered less disintegration after the test compared with the previous four groups.

-20°F:

The observed surfaces of this group were similar to those of the group exposed to 35°F. Failure occurred along existing construction cracks as indicated by foreign material observed inside the failure surface.

AMIR COMPACTED CORES

STEEL ROLLER COMPACTED CORES

FIGURE 6.5: DISINTEGRATION OF STEEL COMPACTED CORES 7 DAYS AFTER INDIRECT TENSILE STRENGTH TEST

6.4 ANALYSIS OF OBSERVATIONS

Observations made during the experimental program led to important findings concerning the nature of the stripping problem. These findings include:

- 1- Separation due to stripping is not accurately defined.
- 2- Evaluation of stripping susceptibility by conditioning cores using warm soaking does not reflect all stripping mechanisms.
- 3- Estimates of the amount of stripping based on observed areas of exposed aggregates on the failure surface is not an accurate procedure.
- 4- Compaction can affect material resistance to stripping.

These findings are discussed in the following sections.

6.4.1 DEFINITION OF STRIPPING

Stripping has been defined in the literature as the physical separation of asphalt cement and aggregate produced by adhesion failure. The stripping phenomenon will be redefined here based on observations made during this study after testing a large number of cores from three different field trials. It has been understood from the existing definition of stripping that separation occurs between the asphalt cement and the aggregate. Observations made during this investigation revealed that separation occurred between the asphalt matrix (i.e. asphalt cement, sand and other fine material used as fill) and the larger aggregate particles. The type of failure was adhesive failure and aggregate prints left on the asphalt matrix were covered with moisture. The condition of aggregate particle surface after separation depended on the soaking temperature. After soaking at mild temperatures, i.e. lower than 85°F and higher than 35°F, aggregate particles retained a thin bituminous film cover. It was quite rare when aggregate particles lost this cover, leaving bare aggregate surfaces, at such mild soaking temperatures. The condition that resulted in complete removal of the thin bituminous cover was the exposure to elevated soaking temperatures such as 110 and 140°F. At high soaking temperatures the thin bituminous film disintegrated and was decomposed into an oily medium and another

composite material which is believed to be mainly clustered asphaltene. The structural change followed oxidation of the asphalt cement as indicated by the discolouring observed on cores surfaces.

Peterson's (9) (67) description of the chemical adsorption which occurs during the mixing process, provides an explanation for the behaviour of the thin bituminous film covering individual aggregate particles. When asphalt cement initially wets aggregate surfaces, approximately sixteen known chemical functional group types in asphalt are usually adsorbed by aggregate followed by a chemical reaction between these groups and the aggregate constituents. Therefore, the part of the asphalt cement that reacted with the aggregate constituents must be considered as part of the aggregate and it may require a strong action to be removed such as warm water soaking. Warm water soaking was proven to be effective in removing the thin bituminous film which results in easy separation of the asphalt matrix from aggregate surfaces. But separation of the asphalt matrix at mild temperatures occurred while the thin asphalt film was still intact as observed during and after indirect tensile strength tests. Therefore, the disappearance of the thin bituminous film by disintegration and not separation, must be considered as another mechanism leading to stripping. Stripping after disintegration of the thin bituminous film, similar to that caused by warm water soaking, could not be blamed for stripping in the relatively cold regions of North America.

Based on the above discussion, the stripping phenomenon is redefined as follows:

"stripping is defined as the physical separation of the asphalt matrix and aggregate produced by adhesion failure"

The definition in the literature has been based on observations made in the field and after the bituminous film disappeared from aggregate surfaces after extended periods of exposure to field conditions. The laboratory investigations, using warm water soaking at 140°F led to the existing definition where aggregate surfaces could be seen without a cover suggesting separation at the asphalt cement/aggregate interface. While the previous definition seems dictated by the results of laboratory warm water tests, the new definition

supports the existing adhesion theories where none of these theories imply the need for elevated temperatures to produce stripping. Based on observations made during the investigation, stripping occurred at all temperatures above freezing temperature.

6.4.2 INFLUENCE OF COMPACTION ON STRIPPING

The pilot laboratory investigation performed earlier as part of this study revealed many factors associated with compaction and it was found to be affecting the resistance of asphalt concrete to stripping damage. Construction induced cracks is one of the these factors and Chapter 4 discusses the influence of these cracks on stripping. Also observations made during the different experiments revealed the effect of these cracks on the ease of water penetration into pavement. Water was observed pouring out freely from saturated cores through construction cracks. Almost all steel compacted cores failed along construction cracks. The other factors associated with construction which may influence stripping are discussed below.

I. Adhesion Between The New Layer And The Existing Surface

The degree of adhesion between layers during compaction can be used as an indicator of how compaction may enhance the adhesion forces in general. The separation observed in the section compacted by the steel roller may be due to one or a combination of the following factors;

- 1- The wave of the mixture in front of the roller pulling the bottom section of the newly laid asphalt concrete layer.
- 2- The roller speed and geometry allowing very short period for adhesion to take place between the two layers.
- 3- Limited compaction energy available to create the required adherence since only a small portion of the total work done constitutes the compaction energy.

The lack of proper adherence between new and old layer was severe because no tack coat was used and also because the old pavement surface was relatively cold. However, the same factors may affect proper adherence between asphalt cement and aggregate within the newly laid asphalt concrete after compaction by the steel roller. Compaction by the

AMIR roller resulted in proper adherence between the layers in spite of the unfavourable construction conditions mentioned above.

II. Aggregate Distribution

The two uniquely different compaction methods produced different aggregate distribution. It has been known that variation of aggregate distribution in the lateral and longitudinal directions results in density variation and locations with low density possesses little resistance to stripping. However, variation in the vertical distribution of aggregate and its influence on the stripping resistance of a compacted mixtures has not yet been identified. Therefore, the following portion of this section will summarize observations on mixture performance associated with vertical aggregate distribution.

1- Inspection of the pavement surface in three field trials indicated the absence of large aggregate particles from the surface in two field trials where the vibratory steel roller was used for compaction (NRC trials # 1 & 2). Only coated fine material was left on the surface. The surface of pavement sections compacted by the AMIR roller contained coated large aggregate particles.

2- The above condition was verified by the observations made after the indirect tensile strength test was completed and the interior of the cores was exposed for inspection. Observations made on the failure surface revealed the high variation in the distribution of aggregate particles in the vertical direction in sections compacted by the vibratory steel roller. Large aggregate particles were concentrated in the lower section of the pavement while fine aggregate migrated to the surface.

The observed segregation was anticipated and was explained in Chapter 4. As a result of vibration, segregation of mixture components occurred where finer materials migrated to the pavement surface while the coarser material occupied the lower section. The effects of segregation on the stripping problem are described below.

a- Other than the effect of moisture gradient, the fact that large aggregate particles in pavements compacted by the vibratory steel roller occupy the lower section of pavements explains why all field observations in the past have been reporting that stripping starts at the bottom section of the asphalt concrete layer. Based on observations made in this

study, this factor is associated with compaction and it is not a stripping characteristic. This was confirmed by observations made on sections compacted by the AMIR roller where stripped aggregate particles were uniformly distributed over the failure surface because large aggregate particles were originally distributed uniformly in the vertical direction.

b- Concentration of the stripped particles in one location particularly in the lower section of the pavement reduces the pavement resistance to tensile stresses building up under traffic load.

c- The concentration of aggregate particles in the bottom section of the asphalt concrete layer means that these aggregate particles have thin or no asphalt matrix cover between the individual particles. A thin asphalt matrix results in less resistance to the damaging effects of stripping and faster rate of deterioration.

d- During construction, the accumulation of aggregate particles with thin separating asphalt matrix cover or in the case of particle to particle contact, may cause excessive aggregate breakage. Aggregate breakage was observed in Block O of NRC trial no.1 where a thin asphalt concrete layer was used to overlay the existing surface. Such locations represents weak zones since no adhesion exists between parts of broken aggregate particles.

III. Particle Orientation

The influence of aggregate particle orientation on asphalt concrete properties has not yet apparently been investigated. It could be argued that particle orientation and the void "path", as influenced by compaction, may be responsible for the differences in stripping resistance. The apparent reason seems to be the fact that the steel roller, which compacted all the existing asphalt concrete pavements, results in random aggregate orientation. Observations made during this study confirmed the random nature of particle orientation in sections compacted by the steel roller in three field trials.

The AMIR roller produced relatively an identifiable aggregate particle orientation during optimum compaction conditions. The particle orientation produced by the AMIR roller at optimum compaction conditions will be referred to as the preferred particle

orientation. The optimum compaction conditions included ideal mixture and air temperatures, adequate ground support and properly designed mixture. The two locations available for observing the aggregate particles orientation were the sides of slabs, thawed and removed from NRC trial no.1, and the failure surface of tested cores. Generally, the long axis of large aggregate particles in the AMIR section of Block O were parallel to the pavement surface. Some particles within this block had their long axis positioned differently with respect to the pavement surface. As mentioned earlier, the steel roller did not produce any identifiable particle orientation pattern.

The importance of particle orientation, as observed in this study, is associated with estimating the degree of stripping. Investigators have used the exposed aggregate surfaces to estimate the amount of stripping damage in the field and in the laboratory after conditioning. Using exposed surfaces of randomly oriented aggregate particles results in high variation in the estimated damage. Under similar conditioning, cores with similar densities showed relatively similar strength but large differences in the estimated stripping damage based on exposed areas of aggregate particles measured on the failure surface. Therefore, estimates of the stripping damage based on the total area of exposed aggregate surfaces would result in inaccurate conclusion. Using areas of aggregate surfaces exposed on the failure surface to estimate the extent of stripping damage resulted in the lack of progress towards solving the problem.

6.4.3 Mixture Performance at Different Temperature Intervals

Observations made during this study revealed that stripping occurred at all temperatures above freezing temperature. Stripping was observed after conditioning at 35°F when cores were soaked for 14 days. The literature review and the analytical approach mentioned two different mechanisms that can lead to the separation of the asphalt cement from aggregate. The first mechanism has been explained by the existing adhesion theories which require the presence of water in the interface between the aggregate and the asphalt. The later mechanism requires hot water submergence with no theoretical explanation. The second mechanism has been used because it forces stripping in the laboratory. No comprehensive laboratory investigation attempted the use of low

soaking temperatures to estimate the influence of the other factors influencing stripping. The structural change accompanying warm water soaking is usually quite severe and it can obscure other factors capable of influencing the stripping process at normal operating temperatures. Since warm water soaking does not allow incorporation of other factors, using it during stripping investigations can be considered partially responsible for not accomplishing a better understanding of the actual nature of stripping.

The above discussion is intended to emphasise that existing evaluation methods using warm water soaking (140°F) does not actually evaluate the stripping potential of asphalt concrete mixtures since it affects the structure of the thin bituminous film. Soaking at such elevated temperatures does not give investigators an opportunity to estimate how much other potential factors contribute to the problem. Laboratory investigations attempting to simulate field conditions must adapt soaking temperatures consistent with the region where the problem is being investigated.

CHAPTER 7

RESULTS AND ANALYSIS

7.0 INTRODUCTION

The analytical model presented in Chapter 3 suggested that initiation of stripping can occur during low temperatures. Specifically, the model presented two mechanisms that can influence the mixture performance. First, as a result of thermal cooling, compressive stresses caused by the contact pressure may enhance the strength of the bond between the asphalt matrix and the large aggregate particles. Second, the build up of thermally induced tensile stresses in the asphalt matrix close to the aggregate surface may result in the rupture of the asphalt matrix. The net effect of these two mechanisms depend on temperature, exposure duration, initial damage that may exist in the mixture and on the mixture characteristics. Accordingly the experimental program presented in Chapter 5 was designed to verify the analysis given in Chapter 3.

Results obtained from tests performed for this study are presented according to the same sequence as the testing program. Results of tests performed to investigate the influence of differential thermal expansion or contraction are presented in separate tables according to the soaking periods. Groups will be referred to by the soaking temperature used during the conditioning period. Conditioning details as well as test results of groups exposed to freeze-thaw cycles are also shown in separate tables.

Observations made during and after indirect tensile strength tests were necessary for interpretation of test results. These observations, described in Chapter 6, will be recalled to differentiate between separation due to adhesion failure and disintegration of the bituminous film due to structural changes caused by warm water soaking.

Results of tests performed to investigate the influence of differential thermal

expansion or contraction are analyzed in the following section. The discussion will include any other factors affecting the stripping of the mixtures under investigation.

7.1 VERIFICATION OF THE THEORETICAL ANALYSIS

7.1.1 NRC TRIAL NO.1

2 Day Soaking Period

Performance of cores compacted by the AMIR roller was typical to the analysis proposed in Chapter 3 which describes the influence of thermal expansion and contraction on the bond performance. Test results given in Table 7.1 confirmed the bond enhancement mechanism. Figure 7.1 shows the relationship between soaking temperature and indirect tensile strength. The measured indirect tensile strength increased from 100.6 psi for the control group to 112.5 psi for -20°F group. The measured strength for 35°F increased to 101.4 psi. The increase in strength of 11.8% and 1% in the -20°F and 35°F groups respectively occurred because the asphalt matrix contracted more than the aggregate particles after thermal cooling resulting in an improved adhesive bond. Cores with improved bond showed an increase in the indirect tensile strength since other factors affecting the strength were kept equal for all groups. The effect of the degree of compaction within a section constructed by the same compactor was eliminated by distributing cores into groups with equal densities. The average bulk specific gravity for all AMIR tested cores exposed to 2 days was equal to 2.415.

One cannot ignore the possibility of internal damage due to the -20°F exposure level. The 11.8% increase in strength is not necessarily an accurate estimate of how much contraction contributed to the bond strength since the indirect tensile strength is a function of both adhesion and cohesion. This is due to the fact that within one core some locations may experience bond improvement while others suffer some damage to the asphalt matrix due to differential contraction. The measured increase in strength represents the net gain of the two mechanisms. The possibility of partial damage and

partial improvement, as indicated by the analytical approach, must be considered throughout the analysis. This complication is caused by the existence of different aggregate particle sizes and different asphalt matrix thickness due to different aggregate distribution in the mixture. Based on the analytical model derived to estimate the pressure exerted on the aggregate walls, the aggregate particle diameter and the asphalt matrix thickness influence the pressure value. The shape of aggregate particles can also influence the tangential stress magnitude necessary to cause the asphalt matrix rupture due to stress concentration at the sharp edges of some aggregate particles. To investigate the possibility of any damage inflicted on the cores after exposure to -20°F , cores must be allowed to stay in the water bath at room temperature for a longer period, i.e. more than two hours. Such action complicates the interpretation of the data as have been experienced with freeze-thaw conditioning in the past since other factors may also contribute to stripping. One such factor is the presence of faults which may exist prior to conditioning. The influence of such faults will be discussed later.

AMIR compacted cores subjected to 110°F and 140°F exposure levels suffered a decrease in strength as compared with the control group. The strength of the 110°F group decreased by 17% (from 100.6 psi at 75°F to 83.2 at 110°F). The strength of the 140°F group decreased by 23% (from 100.6 psi at 75°F to 77.6 psi). The limited change in colour experienced by these two groups indicated a mild degree of oxidation which limited the adhesion loss to the degree expressed by a 23% decrease in strength in the case of 140°F group. Excessive oxidation followed by structural change as described in Chapter 3, would have resulted in a steeper slope, i.e lower strength values for soaking temperatures warmer than 75°F .

The loss of strength of only 23% could lead the investigators to conclude that this mixture had an adequate resistance to stripping since the strength ratio (TSR) is greater than 75%. Recent stripping evaluation test methods use warm water soaking at 140°F as the only exposure level necessary to estimate the stripping resistance of asphalt concrete mixtures. Longer exposure durations will show that such a conclusion can be misleading as will be explained later.

**TABLE 7.1: DIFFERENTIAL THERMAL ANALYSIS
(2 DAYS WATER SOAKING)
(NRC TRIAL NO.1-AMIR SECTION 0)
(TEMPERATURE -20 TO 65 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 1					
	A-1	2.460	1-2	114.3	
-20	A-2	2.368	3-4	96.1	112.50
	A-3	2.421	2-3	127.0	
Group # 2					
	A-1	2.434	3-4	95.0	
35	A-2	2.450	2-3	108.9	101.40
	A-3	2.366	4-5	100.3	
Group # 3					
	A-1	2.463	1.0	103.6	
	A-2	2.413	1-2	96.4	94.01
65	A-3	2.377	3-4	82.4	

**TABLE 7.1: DIFFERENTIAL THERMAL ANALYSIS
(2 DAYS WATER SOAKING)
(NRC TRIAL NO.1-AMIR SECTION 0)
(TEMPERATURE 85 TO 140 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 4					
	A-1	2.435	2-3	111.5	
85	A-2	2.446	2-3	103.5	101.2
	A-3	2.367	5-6	88.7	
Group # 5					
	A-1	2.435	1-2	88.4	
110	A-2	2.438	1-2	79.5	83.24
	A-3	2.376	3-4	81.8	
Group # 6					
	A-1	2.456	1-2	89.8	
140	A-2	2.358	-	-	77.56
	A-3	2.429	2-3	65.3	
Group # 7					
	A-1	2.351	4-5	102.0	
CONTROL GROUP VAC. AND TESTED	A-2	2.399	3-4	95.7	100.85
75	A-3	2.450	1-2	104.2

FIGURE 7.1: RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

Test results of cores from sections compacted by the steel roller and soaked in water for two days are presented in Table 7.2. Figure 7.2 shows the relationship between soaking temperature and indirect tensile strength. In contrast to the results obtained from the AMIR compacted section, the cores did not show any significant increase in their tensile strength. The -20°F exposure resulted in a small increase in strength from 93.0 psi at 75°F to 93.3 psi at -20°F (0.4%). No change occurred due to 35°F exposure level. Based on the differential contraction concept, thermal cooling should have resulted in an improved bond and higher strength. Since contraction must have taken place, other factors must have caused the reported strength with no increase after contraction. Figure 7.3 shows the test results for sections compacted by the two different compaction methods. The steel group and the previous AMIR compacted cores were from the same asphalt concrete mixture, constructed under the same field conditions and were compacted to approximately the same average density. The only difference was the compaction technique used during construction. Observations made during and after tests, presented in the previous chapter, indicated that the majority of cores from steel compacted sections failed along existing cracks (see Figure 6.3). Clearly, the loss of strength due to construction induced cracks outweighs any gains through bond improvement due to thermal cooling. The influence of construction cracks will be discussed in detail later.

The soaking temperature strength curve, Figure 7.2, shows relatively no change in strength for exposure temperatures ranging from -20 to 65°F . Above 75°F , the relationship indicated a sharp decrease in strength following stripping which was observed at the failure surfaces of tested cores following the expected behaviour described earlier in Chapter 3. A pronounced decrease in strength was experienced by cores exposed to 140°F soaking temperature following the excessive stripping observed on failure surfaces. The strength drop reached 41.8% due to 140°F exposure and 19.7% drop due to 110°F exposure level. The large drop in strength in the 140°F group compared with 110°F group was due to the higher degree of oxidation and the consequent structural change experienced after exposure to 140°F . Figure 7.3 shows the higher resistance offered by cores compacted by the AMIR roller after exposure to warm water soaking above 75°F .

The differences between the cores from the two compaction methods were discussed in Chapter 4 and it includes construction cracks, particle orientation and particle distribution. Construction cracks allowed warm water circulation reducing the resistance of steel compacted cores to the stripping damage.

**TABLE 7.2: DIFFERENTIAL THERMAL ANALYSIS
(2 DAYS WATER SOAKING)
(NRC TRIAL NO.1-STEEL SECTION 0)
(TEMPERATURE -20 TO 65 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 1					
	S-1	2.380	3-4	91.7	
-20	S-2	2.401	2-3	84.7	93.33
	S-3	2.414	2-3	103.7	
Group # 2					
	S-1	2.383	4-5	84.5	
35	S-2	2.399	4-5	101.2	93.03
	S-3	2.412	3-4	93.4	
Group # 3					
	S-1	2.418	2-3	95.41	
65	S-2	2.381	3-4	84.5	92.23
	S-3	2.396	2-3	96.8	

**TABLE 7.2: DIFFERENTIAL THERMAL ANALYSIS
(2 DAYS WATER SOAKING)
(NRC TRIAL NO.1-STEEL SECTION 0)
(TEMPERATURE 85 TO 140 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 4					
	S-1	2.385	4-5	84.9	
85	S-2	2.403	3-4	96.2	93.10
	S-3	2.411	3-4	98.2	
Group # 5					
	S-1	2.394	3-4	84.0	
110	S-2	2.395	2-3	68.3	74.70
	S-3	2.404	2-3	71.8	
Group # 6					
	S-1	2.383	4-5	36.7	
140	S-2	2.412	2-3	64.2	54.14
	S-3	2.400	2-3	61.5	
Group # 7					
	S-1	2.389	4-5	85.5	
CONTROL GROUP VAC. AND TESTED	S-2	2.395	2-3	95.5	93.01
75	S-3	2.407	2-3	98.0

FIGURE 7.2 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

FIGURE 7.3 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

7 Day Soaking Period

Asphalt concrete cores used for exposure duration were selected for both compaction methods to reflect higher densities and less surface cracking. The asphalt cores were left in the laboratory for a period of more than one year. The effect of construction quality and longer curing period made the comparison between these groups and the 2 and 14 day groups a difficult task.

Cores from the AMIR compacted section, with an average bulk specific gravity of 2.485, were used to study the performance of the compacted mixture after a 7 day soaking period. Test results are presented in Table 7.3. The relationship between soaking temperature and indirect tensile strength is shown in Figure 7.4. Between 140°F and 35°F soaking temperatures, the mixture performed in a manner similar to the previous 2 day soaking period. The strength increased from 150.1 psi at 75°F to 154.3 psi at 35°F (2.8%). Unlike the 2 day soaking period, thermal cooling at -20°F resulted in 1.2% decrease in strength (from 150.1 psi at 75°F to 148.3 psi at -20°F). The damage inside the cores caused by the rupture of the asphalt matrix, as observed on some sharp edges of aggregate particles on the failure surface, would negate the effect of bond enhancement due to differential contraction. Such rupture is expected when the tangential stresses set by differential contraction of the asphalt matrix exceeds its strength as explained by the analytical model. For the particular compacted mixture under investigation; seven day exposure at -20°F led to the observed deterioration. This suggests that significant damage occurred to the mixture as a result of differential contraction. This type of damage will allow the water to penetrate to the interface through the ruptured areas and start the stripping process. Stripping would have led to greater loss of strength, i.e., beyond that resulting from simple rupture caused by contraction, if cores were allowed more time in the water.

Other groups, exposed to temperatures warmer than 75°F, suffered stripping caused by loss of adhesion and some aggregate surfaces indicated disintegration of the thin bituminous film. This could be explained by the drop in indirect tensile strength in the case of the 140°F group from 150.1 psi to 89.3 psi (40%). The effect of longer exposure duration was reflected by the drop in strength beyond 23% caused by 2 day soaking

period at the same soaking temperature. The longer period resulted in the observed greater oxidative aging and the structural changes reflected by the oil spots observed on the surface of the water bath which in turn resulted in a pronounced loss of adhesion. This indicated that the structural change progressed with time following the observed increase in the degree of oxidation. Since separation caused by other known factors, such as interfacial tension between water and aggregate or repulsive forces developing between similarly charged asphalt cement and aggregate, progresses with time too, estimating the rate of structural change using the rate of loss in strength is not possible at this stage. Long exposure periods in the field to similar moisture and temperature conditions are expected in some locations. Therefore, soaking cores for periods as long as 7 days may be necessary to estimate the stripping susceptibility of an asphalt concrete mixture. The conventional 24 hour soaking period will not be enough for realistic evaluation of the stripping susceptibility of a given mixture.

Soaking cores at 65°F for a 7 day period caused a decrease in the strength from 150.1 psi at 75°F to 135.4 psi at 65°F (9.8% decrease). Some aggregate particles separated from their asphalt matrix cover. Faults were found in the control group of this experiment where some aggregate particles were found without a complete cover. Therefore the observed stripping can take place since the water has direct access to the aggregate/asphalt matrix interface.

Test results of cores compacted by the steel roller and soaked for a 7 day period are presented in Table 7.4. Figure 7.5 shows the relationship between soaking temperature and indirect tensile strength. Results indicated an increase in strength for the 35°F group. The strength increased from 115.9 psi at 75°F to 127.8 psi at 35°F (10.2% increase). The increase in strength after soaking at -20°F reached 1.8%. The limited increase in strength in -20°F group can be explained by the differential contraction concept. This performance was similar to the 7 day AMIR compacted group. According to this concept, after differential contraction due to thermal cooling, the asphalt matrix remained intact around some aggregate particles and ruptured in some other locations. Clearly, the absence of

**TABLE 7.3: DIFFERENTIAL THERMAL ANALYSIS
(7 DAYS WATER SOAKING)
(NRC TRIAL NO.1-AMIR SECTION 0)
(TEMPERATURE -20 TO 65 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 1					
	A-1	2.498	1.0	147.0	
-20	A-2	2.488	1-2	152.8	148.30
	A-3	2.467	1-2	145.1	
Group # 2					
	A-1	2.509	1.0	160.6	
35	A-2	2.484	1.0	153.0	154.34
	A-3	2.461	1-2	149.4	
Group # 3					
	A-1	2.504	1.0	149.5	
	A-2	2.486	1.0	127.7	135.43
65	A-3	2.463	1-2	120.1	
Group # 4					
	A-1	2.491	1-2	145.5	
85	A-2	2.489	1-2	141.6	137.93
	A-3	2.476	1-2	126.7	

**TABLE 7.3: DIFFERENTIAL THERMAL ANALYSIS
(7 DAYS WATER SOAKING)
(NRC TRIAL NO.1-AMIR SECTION 0)
(TEMPERATURE 110 TO 140 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 5					
	A-1	2.501	1.0	129.0	
110	A-2	2.487	1.0	94.9	104.17
	A-3	2.464	1-2	88.6	
Group # 6					
	A-1	2.495	1-2	107.9	
140	A-2	2.488	1-2	69.0	89.26
	A-3	2.472	1-2	90.9	
Group # 7					
CONTROL GROUP	A-1	2.495	1.0	155.8	
VAC. AND TESTED	A-2	2.491	1.0	148.5	150.09
75	A-3	2.461	1-2	146.0

FIGURE 7.4: RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

construction cracks in these cores made their performance very close to the one predicted by the model. However, the strengths measured for the AMIR compacted cores were generally higher than the steel compacted cores. This condition can be explained by the limitation of the conventional compaction method as will be discussed in more detail later.

Observations made after testing cores exposed to -20°F showed clear rupture of the asphalt matrix at some locations on the failure surface. Contraction not resulting in the rupture of the asphalt matrix cover improved the strength while ruptured locations off-set the effects of bond enhancement. Since the 35°F exposure level was not high enough to cause excessive contraction pressure leading to the asphalt matrix rupture, contraction resulted in an improved mechanical adhesive bond which led to an increase in strength. The absence of visible surface cracks, similar to those observed on cores from steel compacted sections used in the 2 day soaking experiment, influenced the performance of the tested cores.

To compare the 2 day and 7 day performance, normalized strength values were used. Figure 7.6 shows the calculated normalized values in terms of percentage change in indirect tensile strength with temperature for the 2 and 7 day soaking periods. Even though exposure duration was longer than the 2 day duration test, the drop in strength due to 140°F was 42.2% (from 115.9 psi at 75°F to 67 psi at 140°F). The drop was quite similar to that of the 2 day duration. This was because of the absence of construction cracks which are usually responsible for allowing more warm water into the cores resulting in loss of adhesion and a structural change of the bituminous film and both factors promote stripping. The strength loss after 110°F exposure level was not consistent with that of the 140°F group. The drop in strength reached 33.1%, i.e. 1.7 time more than that caused by the 2 day duration under the same exposure level. One possible explanation is the possibility that some of the cores from the 110°F group contained some internal cracks.

Overall, the increase in strength supported the suggested improvement in the bond after thermal cooling due to differential contraction. This particular group exposed for a 7 day

TABLE 7.4: DIFFERENTIAL THERMAL ANALYSIS
(7 DAYS WATER SOAKING)
(NRC TRIAL NO.1-STEEL SECTION 3)
(TEMPERATURE -20 TO 65 F°)

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 1					
-20	S-1	2.441	2-3	123.5	118.03
	S-2	2.428	3-4	130.1	
	S-3	2.390	4-5	100.5	
Group # 2					
35	S-1	2.446	1-2	137.6	127.83
	S-2	2.429	2-3	138.7	
	S-3	2.388	3-4	107.2	
Group # 3					
65	S-1	2.442	1-2	130.3	116.29
	S-2	2.417	2-3	100.6	
	S-3	2.403	3-4	118.0	
Group # 3					
85	S-1	2.430	2-3	118.8	109.32
	S-2	2.426	2-3	109.5	
	S-3	2.407	3-4	99.7	

**TABLE 7.4: DIFFERENTIAL THERMAL ANALYSIS
(7 DAYS WATER SOAKING)
(NRC TRIAL NO.1-STEEL SECTION 3)
(TEMPERATURE 110 TO 140 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 4					
	S-1	2.441	1-2	77.3	
110	S-2	2.416	2-3	80.2	77.62
	S-3	2.403	2-3	75.3	
Group # 5					
	S-1	2.430	3-4	68.7	
140	S-2	2.425	2-3	67.7	67.00
	S-3	2.408	3-4	64.6	
Group # 6					
	S-1	2.435	2-3	140.9	
CONTROL GROUP	S-2	2.413	2-3	104.9	115.98
VAC. AND TESTED					
75	S-3	2.393	3-4	102.1

FIGURE 7.5 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

FIGURE 7.6 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND PERCENTAGE CHANGE IN INDIRECT TENSILE STRENGTH

period do not accurately represent the general performance of a similar mixture compacted under the same conditions by the conventional method. But it was necessary to investigate the influence of construction cracks by investigating the performance of crack free steel compacted pavements if such crack free surface could at all be obtained using the existing steel roller.

14 Day Soaking Period

Cores compacted by the AMIR roller were exposed to moisture and temperature conditioning for a 14 day period. Test results are given in Table 7.5. The relationship between indirect tensile strength and soaking temperature is shown in Figure 7.7. The average bulk specific gravity, average percentage of air voids and the curing period of the different groups in this experiment were similar to those of the groups used in 2 day soaking period. This should make comparison possible between absolute values of indirect tensile strength for the two soaking periods.

Exposure of AMIR compacted group to -20°F temperature condition for such a long duration led to a decrease in strength. The observed 6.1% decrease in strength is five times greater than that caused by the 7 day exposure to -20°F . This decrease in strength is in contrast to the 11.8% increase in strength achieved after the 2 day exposure to -20°F . At this stage it became clear that the damage associated with tensile stresses, caused by differential contraction, progresses with time. The exposure to low temperatures is not expected to produce a change in the nature of the asphalt matrix. Therefore, to explain the continuing decrease in strength one can postulate that cracks, initially developed when thermally induced tensile stresses exceeded the tensile strength of the asphalt matrix, were propagating as a result of continued thermal cooling. Propagation of thermally induced cracks is outside the scope of this investigation.

Exposure to the 35°F temperature condition did not result in any noticeable change in strength and a limited number of aggregate particles separated from their asphalt matrix cover. The expected increase in strength due to contraction was off-set by the long exposure duration where the deterioration caused by stripping due to faults existing before

conditioning caused the observed decrease in strength and the off-set of the expected increase. The effect of long exposure periods in the presence of faults was magnified at the 65°F exposure level where the decrease in strength reached 15%.

For the AMIR compacted section, the highest decrease in strength was reached at 140°F exposure level where the strength decrease reached 59.4% (from 100.6 psi at 75°F to 40.8 psi at 140°F). This represents a drop in strength 2.6 times higher than that observed after 2 day exposure period. After exposure to 110°F the strength decreased from 100.6 psi at 75°F to 60.2 psi at 110°F (a decrease of 40.1%). The decrease after 14 day exposure is 2.4 times higher than that after the 2 day soaking period. The progressive nature of the damage associated with warm water soaking was discussed earlier during the analysis of data from the 7 day soaking period. Results from the 14 day period confirmed the effect of the structural change suggested by Mack (14). Observations made during the conditioning period and after the tests confirmed the high degree of oxidation expected after such long exposure period. Observations also confirmed the presence of relatively large oil spots on the water surface of the warm water bath originating from soaked cores.

Figure 7.8 shows the mixture performance after 2 and 14 day exposure periods. The influence of high temperatures on indirect tensile strength after exposure to both periods is similar. As the temperature increased the tensile strength decreased. Because of the progressive nature of the damage associated with stripping and as a result of excessive structural changes, the curve representing the 14 day exposure is steeper than the 2 day curve. On the other hand, as the temperature decreased, the effect of exposure duration was quite different. While the exposure to -20°F for 2 days led to 11.8% increase in strength, 14 day exposure led to 6.1% decrease in strength. These two conditions represents the two conditions described in Chapter 3, where one condition results in bond improvement and the other results in the rupture of the asphalt matrix.

TABLE 7.5: DIFFERENTIAL THERMAL ANALYSIS
(14 DAYS WATER SOAKING)
(NRC TRIAL NO.1-AMIR SECTION 0)
(TEMPERATURE -20 TO 65 F°)

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 1					
	A-1	2.415	2-3	94.0	
-20	A-2	2.397	3-4	98.7	94.53
	A-3	2.384	3-4	90.9	
Group # 2					
	A-1	2.444	2-3	105.9	
35	A-2	2.379	3-4	98.5	100.40
	A-3	2.369	4-5	97.5	
Group # 3					
	A-1	2.350	4-5	79.9	
	A-2	2.397	4-5	77.0	85.75
65	A-3	2.459	2-3	100.36	

**TABLE 7.5: DIFFERENTIAL THERMAL ANALYSIS
(14 DAYS WATER SOAKING)
(NRC TRIAL NO.1-AMIR SECTION 0)
(TEMPERATURE 110 TO 140 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 4					
	A-1	2.413	2-3	67.8	
110	A-2	2.405	2-3	62.6	60.25
	A-3	2.388	3-4	57.7	
Group # 5					
	A-1	2.409	4-5	46.3	
140	A-2	2.415	3-4	38.1	40.83
	A-3	2.366	4-5	38.1	
Group # 6					
	A-1	2.351	4-5	102.0	
CONTROL GROUP	A-2	2.399	3-4	95.7	100.65
VAC. AND TESTED					
75	A-3	2.450	1-2	104.2

FIGURE 7.7: RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

FIGURE 7.8 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

3 of/de 3

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS
STANDARD REFERENCE MATERIAL 1010a
(ANSI and ISO TEST CHART No. 2)

Partially saturated cores compacted by the steel roller were exposed to different temperatures for a 14 day period. Test results are shown in Table 7.6. The relationship between indirect tensile strength (ITS) and soaking temperature is shown in Figure 7.9. The average bulk specific gravity and percentage of air voids of the different groups in this section was similar to that of the groups used in 2 day soaking period. This should allow comparison between absolute values of indirect tensile strength for the two soaking periods. The response of the steel cores was quite similar to AMIR compacted cores exposed to similar conditions as described earlier. As shown in Figure 7.9, higher temperatures led to a decrease in strength. After exposure to 140°F the strength decreased from 102.8 psi to 36.6 psi at 140°F. The 64% decrease in strength due to 140°F soaking temperature indicated that beside expansion, which initially weakened the adhesive bond, oxidation aging of asphalt cement and the associated structural change resulted in the highest observed stripping damage and the consequent strength decrease. The degree of structural change was controlled also by the soaking temperature. Since higher temperatures causes higher degree of oxidation aging, higher temperatures led to more structural damage as reflected by all the results discussed in this section. Cores soaked at 110°F continued to strip and lose strength until the drop of strength reached 44.8%.

Test results of cores soaked at 35°F showed no change in indirect tensile strength. A number of aggregate particles separated from their asphalt matrix cover indicating that the occurrence of stripping due to existing faults off-set the expected gain in strength due to contraction. The 65°F soaking temperature for a 14 day period led to the highest decrease in strength under such temperature condition. Faults existing prior to conditioning, similar to those observed in the control group, are to blame for such deterioration by stripping.

To emphasise further the effect of longer exposure duration, results of 14 day exposure were compared with the 2 day exposure results. Figure 7.10 shows the relationship between soaking temperature and the normalized indirect tensile strength values for the 2 and 14 day exposure periods of steel compacted cores. A pronounced reduction in strength took place due to -20°F exposure level. The decrease in strength

after the long exposure period confirmed the progressive nature of the damage associated with thermal cooling. The strength dropped by 22% indicating that contraction due to thermal cooling inflicted serious damage on the asphalt matrix. Damage to AMIR compacted cores was much less (6.1%) and therefore not all the damage in steel compacted cores could be attributed to the effect of differential contraction. Construction induced cracks must also be considered as an additional factor contributing to the observed damage. Expansion of the frozen saturation water inside these cracks may result in additional damage. The damage to the asphalt matrix cover, observed at the failure surface after testing, is an alarming indicator that long term exposure to low temperatures, could lead to serious destruction. The effect of thawing on an asphalt concrete mixture after such damage is the penetration of water to the interface triggering stripping. Such long exposure periods to low temperatures is not rare in many areas in North America.

TABLE 7.6: DIFFERENTIAL THERMAL ANALYSIS
(14 DAYS WATER SOAKING)
(NRC TRIAL NO.1-STEEL SECTION 0)
(TEMPERATURE -20 TO 65 F°)

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
<hr/>					
-20	Group # 1				
	S-1	2.378	4-5	77.7	
	S-2	2.403	3-4	72.4	80.60
	S-3	2.425	2-3	91.7	
<hr/>					
35	Group # 2				
	S-1	2.417	3-4	110.4	
	S-2	2.388	3-4	94.8	102.80
	S-3	2.403	4-5	103.2	
<hr/>					
65	Group # 3				
	S-1	2.405	4-5	86.0	
	S-2	2.412	3-4	80.5	87.10
	S-3	2.396	4-5	94.8	
<hr/>					

TABLE 7.6: DIFFERENTIAL THERMAL ANALYSIS
(14 DAYS WATER SOAKING)
(NRC TRIAL NO.1-STEEL SECTION 0)
(TEMPERATURE 110 TO 140 F°)

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 4					
	S-1	2.397	3-4	50.8	
110	S-2	2.410	2-3	61.6	56.69
	S-3	2.405	2-3	57.7	
Group # 5					
	S-1	2.388	4-5	36.6	
140	S-2	2.402	4-5	34.6	36.50
	S-3	2.422	4-5	39.0	
Group # 6					
	S-1	2.405	3-4	101.9	
CONTROL GROUP	S-2	2.391	3-4	101.3	102.80
VAC. AND TESTED	S-3	2.416	2-3	105.3	
75

FIGURE 7.9 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

FIGURE 7.10 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND PERCENTAGE CHANGE IN INDIRECT TENSILE STRENGTH

7.1.2 TORONTO FIELD TRIAL

A limited number of steel compacted cores from the Toronto field trial were used to verify test results of NRC field trial no. 1. The short term exposure of 2 days was used and the test results are shown in Table 7.7. The relationship between indirect tensile strength (ITS) and soaking temperature is shown in Figure 7.11.

The average bulk specific gravity was 2.353 and percentage of air void was between 4 and 7 (14" vacuum saturation). The low density was caused by the relatively low compaction effort as the compactor made three passes only. Construction cracks were visible on cores surfaces. The experiment was designed to examine if the conclusions based on the performance of cores from the steel compacted section of NRC trial # 1 could be generalised to include other asphalt concrete mixtures.

The performance of cores from Toronto field trial was similar to that of steel cores from Ottawa trial no. 1 (NRC # 1) under the same exposure conditions. Figure 7.11 shows very little change in strength, compared with the control group, after exposure to temperatures below 75°F. Failure after cooling (between 65°F and -20°F) was controlled by the presence of cracks and the mixture did not benefit from bond improvement around individual aggregate particles caused by differential contraction. Almost all cores failed along existing construction cracks.

Unexpectedly, exposure to 110°F resulted in no change in strength. The absence of faults reduced the susceptibility of the mixture to stripping and structural change was not expected to be effective in such a short exposure period. Damage was limited to locations where separation occurred because of exposure to 140°F soaking temperature where structural changes forced the separation between the asphalt matrix and aggregate. The strength decreased from 83.5 psi at 75°F to 65.4 psi at 140°F. The 21% decrease in indirect tensile strength is low and highway organizations may accept this mixture since it possessed more than 75% TSR according to the conventional approach. This condition, i.e. exposure at 140°F for 2 days, is more severe than the conventional approach which requires only one day exposure.

Figure 7.12 shows the relationship between temperature and indirect tensile strength for Toronto as well as NRC trial no. 1. The performance of groups from Toronto

exposed to temperatures above room temperature indicated better resistance to stripping than that of NRC trial # 1 mixture. Aggregate coverage was better than that of NRC trial # 1 as observed on the failure surface (see Chapter 6). All groups tested indicated such proper coverage. The decrease in strength after exposure to 140°F was 50% less than that observed in NRC trial # 1 mixture after exposure to the same moisture and temperature conditioning and exposure period. Figure 7.12 reflect the effect of higher density of cores from NRC trial # 1 where absolute values of indirect tensile strength were higher than those from Toronto trial for the temperature range below room temperature. However, when the test results of both test sections were normalized, the results supported the analysis presented earlier. Figure 7.13 shows the relationship between percentage change in indirect tensile strength and temperature for Toronto and NRC field trial no. 1. As shown in Figure 7.13, there was no strength gain at -20°F while significant loss of strength was observed at 140°F. The general shape of the two curves indicates the consistency in the performance of the two mixtures compacted by the same compaction technique.

In summary, at low temperatures, both mixtures did not gain any noticeable strength while deterioration occurred at warmer temperature. The Toronto trial offered higher resistance to stripping after soaking at temperatures above 75°F while the NRC cores showed higher tensile strength at temperatures below 75°F. These interesting result can be explained as follows:

1. Density of asphalt cores plays a vital role in the strength of asphalt concrete when the material is elastic. At lower temperatures one should expect slightly denser cores due to shrinkage. On the other hand, as the temperature increases densities become less important due to the expansion of the mixture and due to flow of asphalt cement. Therefore, the higher density of NRC cores produced higher strength below 75°F compared with cores from Toronto field trial.
2. The proper coverage in Toronto mixture led to its higher resistance to stripping at temperatures above 75°F. Toronto mixture was compacted using a static steel roller while NRC mixture was compacted using vibratory steel roller followed by a rubber roller. The asphalt matrix coverage around individual aggregate particles

in the two mixtures was affected by:

a- Segregation due to vibration tends to push large aggregate particles to the bottom of the mixture while fines migrate to the surface. This results in a thin asphalt matrix cover around aggregate in the bottom of the compacted mixture. Vibration also causes excessive aggregate breakage which lead to the existence of many aggregate particles without complete coverage. The effect of these broken aggregate was noticeable at high temperature range.

b- The mixture used in NRC 1 is an MTO- HL-4 while the one used in Toronto is an MTO- HL-3 which is finer than HL-4. This affects the degree of asphalt matrix coverage as discussed earlier. A thick asphalt matrix cover will protect the thin bituminous film reducing the degree of structural changes following warm water soaking.

Based on the results of these two field trials, one can conclude that initial coverage during the mixing and compacting is an important factor for the resistance of the mixture to the damaging effect of stripping in spite of the density consideration. The above factors should explain the differences observed in Figure 7.12. While higher density results in higher strength at low temperatures, the effect of higher temperatures is more influenced by mixture proportions, aggregate distribution and other factors affecting asphalt matrix coverage.

**TABLE 7.7: DIFFERENTIAL THERMAL ANALYSIS
(2 DAYS WATER SOAKING)
(TORONTO TRIAL - STEEL SECTION)
(TEMPERATURE -20 TO 85 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 1					
	S-1	2.373	3-4	84.2	
-20	S-2	2.350	4-5	86.3	82.90
	S-3	2.337	4-5	78.2	
Group # 2					
	S-1	2.367	4-5	90.8	
35	S-2	2.354	4-5	82.6	83.70
	S-3	2.337	4-5	77.63	
Group # 3					
	S-1	2.365	4-5	82.2	
	S-2	2.342	4-5	83.4	86.09
65	S-3	2.343	4-5	92.7	

TABLE 7.7: DIFFERENTIAL THERMAL ANALYSIS
(2 DAYS WATER SOAKING)
(TORONTO TRIAL -STEEL SECTION)
(TEMPERATURE 110 TO 140 F°)

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 5					
	S-1	2.382	4-5	92.8	
110	S-2	2.361	4-5	81.0	84.30
	S-3	2.291	6-7	79.2	
Group # 6					
	S-1	2.366	4-5	67.2	
140	S-2	2.348	4-5	59.2	65.41
	S-3	2.341	4-5	69.8	
Group # 4					
	S-1	2.187			
CONTROL GROUP	S-2	2.363	4-5	88.0	83.50
75	S-3	2.240		79.0

FIGURE 7.11 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

FIGURE 7.12 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

FIGURE 7.13 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND PERCENTAGE CHANGE IN INDIRECT TENSILE STRENGTH

7.1.3 SUMMARY

The following is a summary of the test results for temperatures that represent the two extreme conditions:

1. Figure 7.14 shows the relationship between soaking period and normalized values of indirect tensile strength of steel and AMIR compacted cores after exposure to -20°F. The performance of AMIR compacted cores exposed to -20°F was in agreement with the expected performance described by the concept of differential thermal contraction. The steel compacted cores show little change in strength after exposure to -20°F for periods up to 7 days. After the 7 days the strength dropped significantly.
2. The presence of construction cracks on steel compacted cores explains the deviation from the differential contraction model for the short exposure period. Although the improvement in the bond around individual aggregate took place after cooling, the failure after loading occurred along the existing cracks. Therefore, any gain in strength due to cooling was negated by the failure along predetermined failure zones.

The significant drop in the strength of steel compacted cores compared with AMIR compacted cores after 14 days exposure as shown in Figure 7.14 was the result of the following factors;

- a. Differential thermal contraction after exposure to -20°F for 14 days resulted in part of the damage to the cores which took place in the asphalt matrix.
- b. Expansion of the water during freezing inside the cracks induced during construction applied stresses on the faces of the cracks. These stresses contributed to the observed drop in the strength which was supported by failure taking place along these cracks.
- c. The other factors associated with compaction include aggregate distribution which influence the asphalt matrix thickness. Compaction using the vibratory steel roller produced cores with the large aggregate particles concentrated in the bottom section of the core. This resulted in thin asphalt

matrix cover around individual aggregate particles which resulted in higher tensile stresses during thermal cooling as described by the concept of differential contraction. The damage following high stresses exceeding the matrix strength led to the higher drop in the strength of the steel compacted cores compared with AMIR compacted cores where aggregate distribution was more uniform.

This analysis explains the often reported field observation that stripping was found at the bottom portion of the asphalt layer. As explained earlier, the locations damaged due to differential contraction during thermal cooling are highly susceptible to stripping during warm periods.

- 3- Figure 7.15 shows the relationship between the soaking periods and normalized values of indirect tensile strength after exposure to 140°F. While AMIR compacted cores showed a gradual drop in strength, the exposure of steel compacted cores resulted in a significant drop after a short duration. Since compaction method was the only difference, a factor associated with compaction must be responsible for the observed performance. Construction cracks was a common factor among all steel compacted cores and failure took place along these cracks. Therefore, construction cracks contributed to the large difference between AMIR and steel compacted cores.

There is no difference in the normalized values of the strength after the 7 day period. The advanced state of stripping magnified by the structural changes reduced the strength of the two core types until there is no noticeable difference between them. Clearly, since both cores were from the same mixture, one should expect both to behave in the same manner after damage has occurred. This was the case at 140°F for the 14 day period.

FIGURE 7.14: RELATIONSHIP BETWEEN EXPOSURE PERIOD AND THE % CHANGE IN INDIRECT TENSILE STRENGTH

FIGURE 7.15: RELATIONSHIP BETWEEN EXPOSURE PERIOD AND THE % CHANGE IN INDIRECT TENSILE STRENGTH

7.2 INFLUENCE OF FREEZE-THAW CYCLES

The analysis presented in Chapter 3 suggested that cracks will develop when the tensile stresses resulting from differential thermal contraction exceed the asphalt matrix strength. These cracks can provide a path for water to reach the asphalt matrix/aggregate interface. This process will result in the separation of the asphalt matrix from the aggregate surface, i.e. causing stripping. The first step in this process was already verified in the previous sections. However, under Canadian climatic conditions, the effect of freeze-thaw cycles on the stripping is not fully understood. AMIR and steel compacted cores were conditioned using the freeze-thaw cycles described in Chapter 5 to investigate the influence of this parameter on the stripping problem. The difference between the cycle used and the previous simple soaking condition was the extended thawing period used in the freeze-thaw cycles. Freeze-thaw conditioning allowed cores to thaw for 24 hours after completing the freezing part of the cycle. This arrangement provided sufficient time period for frozen water to melt and the water to penetrate to the interface triggering stripping.

The freeze-thaw tests were carried out in two phases. Table 7.8 gives the test results of the first phase. As shown in the table, three freeze-thaw cycles, where 65°F was used as the thawing temperature, did not cause any damage to the asphalt specimens. In fact cores from both compaction methods showed improvement in the strength. However, as the thawing temperature increased, the effect of freeze-thaw became clear. The measured indirect tensile strength of steel compacted cores dropped by 41% (from 121.53 to 71.13) after thawing at 85°F and by 63% (from 121.53 to 45.40 psi) after thawing at 140°F. The AMIR compacted cores showed a drop of 37% (from 132 to 83.0 psi) and 63% (132 to 48.6 psi) for the same thawing temperatures. The results suggests that the differential expansion and contraction mechanism presented in this thesis is indeed a valid tool to explain the phenomenon of stripping. It was postulated that as the temperature change increases from the cold side to the warm side more damage to the pavement will occur.

Observation made after each element in the cycle indicated that freezing did not result in cracks on the surface or the bottom of the core. With the smaller dimensions of the tested cores, the damage indicated by the drop in strength, is definitely not due to the

conventional low-temperature cracking reported in the literature. The damage was clearly initiated by the thermally induced stresses as explained by the differential contraction concept. The interaction between the freeze-thaw cycles and thawing temperature is equivalent to the effect of the duration of exposure to temperature in the simple soaking tests discussed earlier. The only difference is the rate of deterioration observed in both tests.

However, the test results have also showed a significant loss of strength associated with a severe stripping condition when the thawing temperature used was 140°F. Based on the analytical investigation presented in Chapter 3, warm water soaking results in oxidative aging which makes the mixture more susceptible to the damaging effect of high tensile stresses building up due to differential contraction. The aged asphalt matrix will then fracture allowing the water to penetrate into the interface during the thawing period leading to stripping. Damage caused by stripping due to thawing at 85°F continued to accumulate after each cycle until the strength reached the magnitude shown in Table 7.8. No explanation was available in the past for indirect tensile strength test results after freeze-thaw cycles. This led investigators to adapt warm water soaking which deals with one conditioning element. The existing testing methods adopt the 140°F as the only soaking temperature necessary for evaluating the resistance of a given mixture to stripping. The 140°F is a relatively high temperature and will rarely occur during a pavement life in a region like Ontario. Moreover, warm water soaking evaluates one of the factors leading to stripping. Chapters 2, 3 and 6 revealed that this condition, i.e. warm water soaking, contribute most to the structural changes in the asphalt cement leading to the disintegration of the asphalt cement. Therefore, using warm water soaking at 140°F as the only method necessary for estimating the stripping susceptibility is neither accurate nor realistic.

The second phase of the freeze-thaw investigation involved the influence of temperature, freeze-thaw cycles, and the level of vacuum pressure used during saturation. Test results are shown in Table 7.9 and these results support the following analysis:

1. For groups 1 and 2, where high pressure and low thawing temperature were used, the AMIR compacted cores showed high resistance to stripping and a bond improvement at 65°F. On the other hand for the same testing conditions the steel compacted cores showed a significant drop when the temperature was increased from 65°F to 75°F. For a 10 degree increase the drop in strength reached 21%.
2. Using 140°F as the thawing temperature led to a significant loss in strength. This observation held true for low as well as high vacuum saturation pressure.
3. Changing the vacuum pressure level led to an interesting observation. While the temperature effect was dominant, increasing the pressure from 14 to 24 inches of mercury resulted in additional damage to the steel compacted cores. The higher pressure resulted in 18% drop in strength at the same temperature conditions. AMIR compacted cores showed no sensitivity towards the vacuum pressure adopted. The use of higher vacuum pressure did not result in any decrease in the strength. Instead the strength increased slightly. The only difference between these two core types was the compaction method and therefore, this significant difference in performance can be related to the compaction technique. Construction induced cracks were a visible difference explaining the difference in performance.
4. From comparison of results of Group 3 in Table 7.8 and Group 4 in Table 7.9 one can conclude that density does not play a significant role in the measured strength of asphalt cores when subjected to high temperatures. While the average density of Group 3 of Table 7.8 is 2.429 and 2.422 for Group 4 of Table 7.9, the average strength of Group 3 is 46.98 psi and for Group 4 the strength is 50.12 psi. This result is in agreement with the analysis presented in the previous section.

Table 7.10-a shows the percentage change in indirect tensile strength as affected by the thawing temperature. The values are selected from groups from Tables 7.8 and 7.9. Figure 7.16 shows the relation between percentage change in indirect tensile strength and thawing temperature. In general, stripping and consequent decrease in strength was the highest above room temperature reaching the maximum strength loss after thawing at 140°F. The decrease in strength exceeded that resulting from simple soaking at 140°F for

14 days.

Clearly the analysis showed that stripping induced at the 140°F is a "laboratory forced stripping" and not necessarily a field expected stripping. A more realistic approach to the evaluation of stripping susceptibility will be to adopt a procedure similar to the experimental approach presented in this section. Asphalt cores, recovered from the field after compaction, must be tested under different set of conditions (e.g. freeze-thaw cycles, different thawing temperatures, and simple soaking at different temperatures). Evaluation of stripping susceptibility of a given mixture should follow analysis of the results obtained from all tests and must be related to the expected conditions during the design life of the pavement in question.

These results emphasized the need to adopt a similar approach to the one developed in this study to evaluate stripping.

TABLE 7.8: FREEZE-THAW CYCLES
TEST RESULTS OF BULK SPECIFIC GRAVITY
AND INDIRECT TENSILE STRENGTH
(STEEL AND AMIR/BLOCKS 2 & 3)
(OTTAWA #1)

CONDITIONING ELEMENTS	SAMPLE NO.	BULK SPECIFIC GRAVITY	% PERM. VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVF. STRENGTH (psi)
Control Group	S-1	2.412	NA	139.6	
	S-2	2.389	-	113.0	
(Dry)	S-3	2.392	-	112.0	121.53
	A-1	2.433	-	135.3	
	A-2	2.438	-	133.1	132.03
	A-3	2.414	-	127.7	
Group #1					
- Vac. 24 in. Hg.	S-1	2.412	4-5	125.3	
- Freez. Temp. (-20°F)	S-2	2.371	6-7	115.9	122.56
- Thaw. Temp. (65°F)	S-3	2.412	4-5	126.5	
- 3 F/T cycles					
- Test Temp. (75°F)	A-1	2.440	3-4	142.3	
	A-2	2.456	3-4	146.8	141.60
	A-3	2.421	5-6	135.7	

continued.....

CONDITIONING ELEMENTS	SAMPLE NO.	BULK SPECIFIC GRAVITY	% PERM. VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVE. STRENGTH (psi)
Group #2					
- Vac. 24 in. Hg.	S-1	2.382	5-6	72.7	71.13
- Freez. Temp. (-20°F)	S-2	2.383	5-6	68.4	
- Thaw. Temp. (85°F)	S-3	2.412	4-5	72.3	
- 3 F/T cycles					
- Test Temp. (75°F)	A-1	2.452	3-4	89.3	83.00
	A-2	2.424	4-5	76.5	
	A-3	2.433	3-4	83.2	
Group #3					
- Vac. 24 in. Hg.	S-1	2.424	4-5	48.5	45.40
- Freez. Temp. (-20°F)	S-2	2.424	4-5	45.9	
- Thaw. Temp. (140°F)	S-3	2.421	4-5	41.8	
- 3 F/T cycles					
- Test Temp. (75°F)	A-1	2.435	3-4	49.2	48.57
	A-2	2.432	3-4	49.9	
	A-3	2.435	3-4	46.6	

TABLE 7.9: FREEZE-THAW CYCLES
TEST RESULTS OF BULK SPECIFIC GRAVITY
AND INDIRECT TENSILE STRENGTH
(STEEL BLOCK 0/AMIR/BLOCKS 2 & 3)
(OTTAWA #1)

CONDITIONING ELEMENTS	SAMPLE NO.	BULK SPECIFIC GRAVITY	% PERM. VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVE. STRENGTH (psi)
Control Group (Dry)	S-1	2.408	NA	115.7	125.20
	S-2	2.423	-	120.4	
	S-3	2.423	-	139.5	
	A-1	2.416	-	130.9	138.77
	A-2	2.429	-	137.8	
	A-3	2.445	-	147.6	
Group #1 - Vac. 24 in. Hg. - Freez. Temp. (-20°F) - Thaw. Temp. (65°F) - 3 F/T cycles - Test Temp. (75°F)	S-1	2.419	2-3	132.5	144.13
	S-2	2.417	3-4	151.0	
	S-3	2.425	3-4	148.9	
	A-1	2.443	2-3	149.1	151.00
	A-2	2.432	2-3	155.2	
	A-3	2.415	3-4	148.7	

CONDITIONING ELEMENTS	SAMPLE NO.	BULK SPECIFIC GRAVITY	% PERM. VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVE. STRENGTH (psi)
Group #2					
- Vac. 24 in. Hg.	S-1	2.417	3-4	109.5	114.55
- Freez. Temp. (-20°F)	S-2	2.419	2-3	119.6	
- Thaw. Temp. (75°F)	S-3	2.425	3-4	-	
- 3 F/T cycles					
- Test Temp. (75°F)	A-1	2.440	2-3	130.5	133.83
	A-2	2.439	2-3	136.8	
	A-3	2.415	3-4	134.2	
Group #3					
- Vac. 14 in. Hg.	S-1	2.404	4-5	47.5	52.40
- Freez. Temp. (-20°F)	S-2	2.429	3-4	57.3	
- Thaw. Temp. (140°F)	A-1	2.409	4-5	48.6	56.60
- 3 F/T cycles					
- Test Temp. (75°F)	A-2	2.430	2-3	64.6	
Group #4					
- Vac. 24 in. Hg.	S-1	2.415	3-4	42.7	43.15
- Freeze Temp. (-20°F)	S-2	2.412	3-4	43.6	
- Thaw. Temp. (140°F)	A-1	2.443	3-4	58.4	57.10
- 3 F/T cycles					
- Test Temp. (75°F)	A-2	2.421	4-5	55.8	

TABLE 7-10-a:

**% CHANGE IN INDIRECT TENSILE STRENGTH AFTER THREE
FREEZE-THAW CYCLES FOR DIFFERENT THAWING TEMPERATURE LEVELS
(BASED ON RESULTS OF TABLES 7.10 AND 7.11)
(OTTAWA TRIAL #1)**

THAWING TEMP. (°F)	% CHANGE IN ITS	
	AMIR	STEEL
65	+7.2	+0.8
75	-3.5	-8.5
85	-37.1	-41.5
140	-63.2	-62.6

TABLE 7-10-b:

THAWING TEMP. (°F)	% CHANGE IN ITS	
	AMIR	STEEL
65	+8.10	+15.19
75	-3.6	-8.5
140(1)	-59.2	-58.1
140(2)	-58.85	-65.5

FREEZE-THAW CONDITIONING

FIGURE 7.16 : RELATIONSHIP BETWEEN THAWING TEMPERATURE AND PERCENTAGE CHANGE IN INDIRECT TENSILE STRENGTH

7.3 INFLUENCE OF COMPACTION

In the past, compaction was not considered a factor when stripping phenomenon was investigated. As explained earlier, the main cause of this serious deficiency was the absence of any compaction method other than the conventional rollers. Yet, strong evidence suggesting that compaction has a significant influence on stripping did exist. The poor correlation between laboratory prepared samples and cores from field compacted sections is a testimony to this effect. Now with the availability of AMIR compaction method the significance of compaction could be established.

Many research findings dealing with the evaluation of this new method have been published by Halim (71), Sevec (72), Selvadurai (73) and Liliedahl (74). The main findings of their research work are summarized below:

1. The AMIR compaction method produced a crack free pavement with high and more uniform densities.
2. Conventional compaction methods have serious limitations that result in construction induced cracks and lower densities especially at the unsupported edges of the paved lanes.
3. Mechanical properties of AMIR compacted mixtures were more consistent and of higher quality than those compacted by the conventional rollers.

Observations made during this study, reported in Chapter 6, revealed the following differences between the performance of the two uniquely different compaction methods:

1. Construction cracks existed on the surface of the steel compacted sections while AMIR compacted sections were free from such cracks.
2. The new overlay compacted by AMIR adhered well to the existing pavement surface while the new overlay compacted by the steel roller did not adhere to the old layer.
- 3- Large aggregate particles were well distributed in sections compacted by the AMIR roller while the steel vibratory roller resulted in migration of fines to the surface and concentration of large aggregate particles in the bottom section of the

new layer.

- 4- Aggregate particle orientation in sections compacted by the AMIR roller indicated a relatively uniform orientation with respect to the pavement surface while the steel roller compaction resulted in a random particle orientation.

The findings from previous studies and observations made during this study suggest that AMIR compacted mixtures should perform better than steel compacted ones when the problem of stripping is considered. Accordingly, this part of the experimental investigation was designed and carried out to quantify the effect of compaction on stripping of asphalt concrete mixture. The following subsections present the results and analysis of the testing program.

7.3.1 EFFECT OF CURING PERIOD

In an effort to investigate the effect of curing time on the strength of mixtures compacted by the different compaction methods, cores from the Toronto trial were tested after one week, three weeks and three months. The test results of AMIR compacted section are given in Tables 7.11, 7.12, and 7.13 respectively. The test result of the steel compacted section are shown in Tables 7.14, 7.15 and 7.16. The increase in average strength of the steel compacted cores after three weeks was 8.5 %, and after three months the increase with respect to one week strength was 19%.

The increase in the strength of the AMIR compacted cores after three weeks was 6.7% and after three months the increase with respect to one week was 21.8%. The increase in strength of the AMIR compacted section due to aging is not substantially different from that of the steel compacted section. However, it should be emphasized that the initial strength of AMIR compacted cores was 23.4% higher than that of the steel compacted cores. Initial high strength reflects the capability of a pavement to resist traffic induced stresses or thermally induced stresses immediately after construction. Low initial strength will mean that the pavement material will be vulnerable to the damaging effects of these two factors. Damage due to stripping has been reported to occur at early stages of pavement life. The difference in measured initial strength between the AMIR and steel compacted cores could be related to:

1. The presence of construction induced cracks on the surface of steel compacted cores which results in stress concentration and consequently lower strength compared with cores free from cracks similar to those produced after AMIR compaction.
2. The capability of AMIR to provide high density due to higher compaction energy made possible by its geometry and the time spent on the pavement surface. In addition, the orderly oriented particles generally resulting from AMIR compaction seem to have contributed in the improved physical as well as mechanical properties of the compacted mixture. There are no definite means by which the degree of participation of particle orientation could be estimated.

The results of these tests provided a logical explanation for the higher strength measured for the asphalt cores tested after the 7 day exposure period. As noted earlier, these cores were kept in the laboratory for one year and subsequently the strength increased similar to the increase observed in the test results.

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**TABLE 7.11: TEST RESULTS OF BULK SPECIFIC GRAVITY AND DRY
INDIRECT TENSILE STRENGTH (AMIR SECTION / ONE WEEK)
(TORONTO TRIAL)**

SAMPLE No.	BULK SPECIFIC GRAVITY	INDIRECT TENSILE STRENGTH (psi)
1	2.352	71.4
2	2.324	66.1
3	2.361	74.3
4	2.368	72.1
5	2.344	73.8
6	2.358	65.6
7	2.340	63.7
8	2.323	62.1
9	2.367	67.4
10	2.368	75.5
11	2.376	64.8
12	2.330	73.7
13	2.352	74.7
Mean	2.351	69.63
Std. Dev.	0.0176	4.78

**TABLE 7.12: TEST RESULTS OF BULK SPECIFIC GRAVITY AND DRY
INDIRECT TENSILE STRENGTH (AMIR SECTION / THREE WEEKS)
(TORONTO TRIAL)**

SAMPLE No.	BULK SPECIFIC GRAVITY	INDIRECT TENSILE STRENGTH (psi)
1	2.342	83.3
2	2.303	80.4
3	2.324	76.0
4	2.368	76.0
5	2.344	80.8
6	2.361	64.8
7	2.365	64.4
8	2.325	62.8
9	2.340	80.1
Mean	2.3413	74.289
Std. Dev.	0.02152	8.06

**TABLE 7.13: TEST RESULTS OF BULK SPECIFIC GRAVITY AND DRY
INDIRECT TENSILE STRENGTH (AMIR SECTION / THREE MONTHS)
(TORONTO TRIAL)**

SAMPLE No.	BULK SPECIFIC GRAVITY	INDIRECT TENSILE STRENGTH (psi)
1	2.344	90.2
2	2.342	81.8
3	2.341	86.5
4	2.343	80.9
Mean	2.3425	84.85
Std. Dev.	0.0013	4.330

**TABLE 7.14: TEST RESULTS OF BULK SPECIFIC GRAVITY AND DRY
INDIRECT TENSILE STRENGTH (STEEL SECTION / ONE WEEK)
(TORONTO TRIAL)**

SAMPLE No.	BULK SPECIFIC GRAVITY	INDIRECT TENSILE STRENGTH (psi)
1	2.321	51.4
2	2.347	55.2
3	2.284	48.3
4	2.338	51.8
5	2.347	60.0
6	2.348	57.9
7	2.332	67.4
8	2.351	59.6
9	2.347	45.9
10	2.362	65.2
11	2.378	64.2
12	2.279	55.2
13	2.279	51.2
Mean	2.3318	56.41
Std. Dev.	0.0321	6.69

**TABLE 7.15: TEST RESULTS OF BULK SPECIFIC GRAVITY AND DRY
INDIRECT TENSILE STRENGTH (STEEL SECTION / THREE WEEKS)
(TORONTO TRIAL)**

SAMPLE No.	BULK SPECIFIC GRAVITY	INDIRECT TENSILE STRENGTH (psi)
1	2.321	63.3
2	2.346	72.0
3	2.345	60.1
4	2.250	61.3
5	2.345	74.1
6	2.335	64.8
7	2.323	56.5
8	2.288	57.6
9	2.309	54.3
Mean	2.318	61.24
Std. Dev.	0.0319	5.58

**TABLE 7.16: TEST RESULTS OF BULK SPECIFIC GRAVITY AND DRY
INDIRECT TENSILE STRENGTH (STEEL SECTION / THREE MONTHS)
(TORONTO TRIAL)**

SAMPLE No.	BULK SPECIFIC GRAVITY	INDIRECT TENSILE STRENGTH (psi)
1	2.336	68.5
2	2.317	66.0
3	2.309	61.6
4	2.353	72.6
Mean	2.3287	67.14
Std. Dev.	0.0197	4.606

7.3.2 CONSTRUCTION CRACKS

This section summarizes data previously discussed to clarify the role of construction cracks associated with the steel roller. In addition, it analyzes the results of an experiment specifically designed to investigate the role of construction cracks in promoting the penetration of surface water into the pavement and therefore reduce the material resistance to stripping. One major material property affected by the presence of these cracks was the tensile strength of the compacted mixtures.

I. Influence of Construction Cracks on Indirect Tensile Strength

The data from tests performed to investigate the influence of differential expansion/contraction, indicated that cracks influence indirect tensile strength at all conditioning levels. Almost all observed failure locations in steel compacted cores were along cracks which extended to different depths below the surface. Even when conditioning was expected to improve the asphalt matrix/aggregate bond of steel compacted cores, as a result of differential contraction, the indirect tensile strength test showed no improvement in the strength. On the other hand, similar conditions caused bond improvement which was reflected by an increase in the strength of the AMIR compacted cores. The strength of AMIR compacted cores increased from 100.6 psi to 112.5 after two days of exposure to -20°F while the strength of steel compacted cores did not change after the same exposure condition. When crack free steel compacted cores were intentionally selected and tested after 7 day soaking the results showed that the strength of AMIR compacted cores increased from 150.1 psi to 154.3 psi at 35°F and decreased to 148.3 psi at -20°F while the strength of the steel compacted cores increased from 116.0 psi to 127.8 psi at 35°F and also increased to 118.0 at -20°F.

Warm water soaking was also followed by failure along construction cracks resulting in steel compacted cores showing lower strength compared to that of AMIR compacted cores. Crack free steel compacted cores performed relatively similar to AMIR compacted cores as indicated by cores from NRC trial # 1 exposed to 7 day soaking period specifically within the low temperature range.

II. Construction Cracks as an Access for Surface Water

An experiment was designed to confirm that construction cracks provide an easy path for surface water to penetrate freely inside the pavement. As discussed in Chapter 5, the experiment was designed to investigate the influence of surface cracks during the soaking period. The water was expected to penetrate only through the surface. Waxing the core's sides and bottom section without affecting the core's surface was not possible. Heating cores, which is necessary for the waxing procedure, may change the original nature of the core and may also seal these cracks. The use of plastic wrapping procedure was the only method available to eliminate or reduce the possibility of water entering cores other than from the surface. This arrangement differentiated between the two compaction methods. In the other experiments where vacuum saturation was used, the amount of water entering cores of both compaction methods was relatively the same. The sides of the cores were not coated and therefore during vacuum saturation a large amount of water penetrated through the sides without regard to the surface condition. In this experiment when the cores were unwrapped, moisture in the sides and the bottom was present indicating that some water found its way into the cores through the sides. The test results indicated that plastic wrapping reduced the flow of water through the sides.

AMIR compacted cores allowed 0.9% of its weight in water to enter the cores, while steel compacted cores allowed 1.2% to enter the cores. The quantities of water calculated was not accurate because after unwrapping the cores, water escaped from the cores interior through voids and surface cracks.

Test results are shown in Table 7.17 for AMIR compacted cores and Table 7.18 for steel compacted cores. Relationship between soaking temperature and indirect tensile strength is shown on Figure 7.17 for AMIR compacted cores and on Figure 7.18 for steel roller compacted cores.

Strength of AMIR compacted cores increased 7.2% due to soaking at 35°F after a period of seven days. During the seven day period, surfaces of asphalt concrete cores were directly exposed to surface water in addition to a limited amount through the sides and the bottom of the asphalt concrete cores. The increase in strength was more than that observed following vacuum saturation and soaking for seven days. This was a direct

result of limiting the amount of water entering the core and little or no damage occurred as a result of faults that existed prior to conditioning. This increase could only be attributed to improved adhesion that followed differential contraction of the asphalt matrix.

No substantial change was observed due to the 65°F exposure. A limited decrease of 0.8% in tensile strength occurred. The decrease in strength was much lower than what followed soaking after vacuum saturation where the decrease in strength reached 9.8%. The limited drop in strength could be attributed to faults that already existed in the cores which allowed a limited amount of water to enter to the aggregate/asphalt matrix interface. The decrease due to existing faults was capable of off-setting the limited increase in strength expected after limited contraction due to thermal cooling from 75°F to 65°F.

The best achievement of AMIR compacted cores was observed after the 110°F exposure level. After seven days exposure, the AMIR cores lost 7.4% of their strength. Soaking cores for the same period after vacuum saturation resulted in 30.6% decrease in strength. This comparison indicates that the crack free surface of AMIR cores improves the mixture resistance to stripping by limiting the amount of water entering cores.

Cores from the steel roller compacted sections were also wrapped to seal the sides and bottom sections. Test results, shown in Table 7.24, indicate that steel compacted cores repeated its previous performance after vacuum saturation and soaking at 35°F. No increase in strength occurred after thermal cooling to 35°F. The same explanation offered before could be used here to explain the group performance. The expected increase in strength was not reflected in the indirect tensile strength because of the presence of construction cracks. This confirmed what was already emphasized earlier, that contraction and the associated bond improvement will not increase the strength if the cores were initially cracked.

The decrease in strength and the observed adhesive failure at 65°F reflected the relatively excessive water penetration through surface cracks resulting in adhesion failure. Separation at the aggregate /asphalt matrix interface must have taken place at locations where faults already existed in the compacted mixture allowing the water to enter into the

interface. The inflicted damage reflected by the 2.3% decrease in indirect tensile strength was more than that after vacuum saturation and soaking for two days at the same temperature (0.8%). The 7 day soaking period after vacuum saturation and exposure to the same soaking temperature did not cause a change in strength because the cores were crack free.

For the steel compacted cores soaked at 110°F, the influence of differential expansion and structural change of the bituminous film were expected to be similar to that of AMIR compacted cores under the same temperature condition. The differences between the two was the amount of water present to displace the asphalt film which ultimately determined the amount of stripping that can be observed after failure. Due to the free access provided by cracks for surface water penetration, steel cores lost 28.4% of its strength while AMIR compacted cores decrease in strength was only 7.4%. Figure 7.19 shows the relationship between soaking temperature and percentage change in indirect tensile strength for the two conditions. Cracks did not only allow water to penetrate freely into the cores, but also caused the expected stress concentration responsible for lower indirect tensile strength.

Therefore, when water was allowed to penetrate into the cores through the compacted surface of both compaction methods, i.e. no vacuum saturation was used to force saturation, the test results of this conditioning procedure support the following findings;

1. The AMIR compacted cores showed an improvement in the resistance of the mixture to stripping damage compared with the vacuum pressure condition. This indicates that crack free asphalt concrete surface is important to minimize damage associated with stripping.
2. No change was observed in the steel compacted cores resistance to stripping damage. This indicates that vacuum pressure is not necessary for conditioning of cores from conventionally compacted pavement surfaces.

**TABLE 7.17: DIFFERENTIAL THERMAL ANALYSIS
INFLUENCE OF CONSTRUCTION CRACKS
(7 DAYS WATER SOAKING)
(NRC TRIAL NO.1-AMIR SECTION 0)
(TEMPERATURE 35 TO 110 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% OF WATER IN AFTER SOAKING	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 1					
	A-1	2.402	1.1	104.9	
110	A-2	2.394	1.2	101.2	98.77
	A-3	2.371	1.4	90.2	
Group # 2					
	A-1	2.443	0.8	112.3	
110	A-2	2.439	0.9	119.9	112.93
	A-3	2.436	1.0	106.6	
Group # 3					
	A-1	2.447	-	101.8	
65	A-2	2.438	-	128.6	121.32
	A-3	2.436	-	133.6	

**TABLE 7.17: DIFFERENTIAL THERMAL ANALYSIS
INFLUENCE OF CONSTRUCTION CRACKS
(7 DAYS WATER SOAKING)
(NRC TRIAL NO.1-AMIR SECTION 0)
(TEMPERATURE 35 TO 110 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% OF WATER IN AFTER SOAKING	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 4					
	A-1	2.449	-	125.8	
35	A-2	2.436	-	141.1	130.84
	A-3	2.432	-	125.6	
Group # 5					
	A-1	2.460	-	128.1	
CONTROL GROUP TESTED DRY	A-2	2.422	-	115.9	122.02

INFLUENCE OF CONSTRUCTION INDUCED CRACKS

FIGURE 7.17 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

**TABLE 7.18 DIFFERENTIAL THERMAL ANALYSIS
INFLUENCE OF CONSTRUCTION CRACKS
(7 DAYS WATER SOAKING)
(NRC TRIAL NO.1-STEEL SECTION)
(TEMPERATURE 35 TO 110 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 1					
	S-1	2.411	1.0	89.6	
110	S-2	2.373	1.2	72.3	83.69
	S-3	2.377	1.3	89.2	
Group # 2					
	S-1	2.404	-	-	
110	S-2	2.380	1.3	85.7	86.11
	S-3	2.378	1.4	86.5	
Group # 3					
	S-1	2.382	-	124.0	
	S-2	2.364	-	108.2	117.5
65	S-3	2.410	-	120.3	

**TABLE 7.18: DIFFERENTIAL THERMAL ANALYSIS
INFLUENCE OF CONSTRUCTION CRACKS
(7 DAYS WATER SOAKING)
(NRC TRIAL NO.1-STEEL SECTION)
(TEMPERATURE 35 TO 110 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 4 35	S-1	2.406	-	141.3	
	S-2	2.383	-	120.0	120.29
	S-3	2.371	-	99.6	
Group # 5 CONTROL GROUP TESTED DRY	S-1	2.406	-	120.6	
	S-2	2.383	-	120.0	120.29

INFLUENCE OF CONSTRUCTION INDUCED CRACKS

FIGURE 7.18 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

INFLUENCE OF CONSTRUCTION INDUCED CRACKS

FIGURE 7.19 : RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND PERCENTAGE CHANGE IN INDIRECT TENSILE STRENGTH

7.4 COMPACTION IN COLD TEMPERATURES

In Canada the paving season ends in November. Field observations have suggested that pavement compacted late in the season will show failure one to two years later. However, the effect of compaction in cold temperatures was not evaluated in the context of the stripping problem. In order to carry out such an investigation, cores from NRC field trial no. 2 were collected. Field compaction was completed in the last day of the paving season in Ottawa (NOV. 30,1989). The air temperature during construction was 5°F. This was in contrast to air temperatures observed during the two other field trials which were in mid 80's. Asphalt cores were taken one week after construction and tested according to the procedure described for simple soaking. A period of two days was selected for conditioning because of the apparent weakness of the steel compacted cores as reflected by their bulk specific gravity. Table 7.19 shows the test results for the steel compacted cores and Table 7.20 for AMIR compacted cores. The test results show the following:

1. In general, the test result agree with the analysis carried out on Toronto and NRC no. 1 field trials.
2. The AMIR compacted cores showed an overall higher tensile strength than that shown by steel compacted cores.
3. Densities achieved by the Amir compaction method were significantly higher than those obtained from the steel compacted cores.

However, while the results confirmed the main findings of Section 7.1, the results showed an interesting phenomenon. As can be seen in Figures 7.20 and 7.21, the measured tensile strength showed a sudden drop when the soaking temperatures were close to room temperature, i.e. 65 to 85°F. The observed increase in strength after higher temperatures make these results more interesting. Clearly, one can argue that the drop observed at 65°F and 85°F may be the result of an experimental error. However, the fact that the same phenomenon was repeated on cores from both compaction methods makes the argument invalid. In order to explain this observation in the absence of large number of observations one can only speculate about the causes as follows:

1. The fact that compaction took place at a low temperature and therefore at a viscosity higher than recommended resulted in a compacted mixture with a response to temperature changes different from other mixtures compacted at higher temperatures.
2. The effect of curing at such low temperatures and at such a high cooling rate may have affected the nature of the mixture and its response to temperature changes compared with a mixture compacted at higher air temperatures.
3. The mixture may have contained a foreign substance which contributed to the observed performance. The mixture was delivered in the last day of the paving season.
4. Bond may have improved in the initial stage of exposure to 110°F and 140°F prior to the structural changes associated with such exposure. Improvement could have taken place because asphalt cement has flow properties at such high temperature levels and better coverage could have taken place. This improvement may include the flow of bitumen by expansion into faults that took place during compaction at low temperatures.
5. There may be a critical temperature after which the asphalt concrete loses its integrity. This critical temperature seems to depend on the temperature of the mixture during compaction, the temperature of the air and the type of the mixture. However, in the absence of sufficient data no further explanation can be offered.

7.5 SUMMARY

Finally, test results reported in this chapter satisfied the main objectives of the experimental program as stated in Chapter 5:

1. It verified the main findings of the analytical phase of this research,
2. It evaluated the existing laboratory test method and confirmed the deficiencies identified earlier, and
3. Proved that the developed testing method is a reliable testing procedure for evaluating stripping susceptibility of field compacted asphalt concrete mixtures.

TABLE 719: DIFFERENTIAL THERMAL ANALYSIS
(2 DAYS WATER SOAKING)
(NRC TRIAL NO.2-STEEL SECTION)
(TEMPERATURE -20 TO 85 F°)

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 1					
-20	S-1	2.319	3-4	80.4	72.90
	S-2	2.295	3-4	63.4	
	S-3	2.299	4-5	74.9	
Group # 2					
35	S-1	2.316	5-6	90.6	79.60
	S-2	2.283	6-7	70.2	
	S-3	2.297	5-6	78.1	
Group # 3					
65	S-1	2.320	4-5	62.8	57.08
	S-2	2.278	6-7	56.4	
	S-3	2.274	6-7	52.0	
Group # 4					
85	S-1	2.309	5-6	49.9	54.31
	S-2	2.303	4-5	59.6	
	S-3	2.287	5-6	53.4	

**TABLE 7.19: DIFFERENTIAL THERMAL ANALYSIS
(2 DAYS WATER SOAKING)
(NRC TRIAL NO.2-STEEL SECTION 0)
(TEMPERATURE 110 TO 140 F°)**

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 5					
	S-1	2.319	5-6	74.9	
110	S-2	2.289	6-7	62.6	66.20
	S-3	2.277	6-7	61.1	
Group # 6					
	S-1	2.337	3-4	63.9	
140	S-2	2.285	5-6	52.3	53.77
	S-3	2.262	6-7	45.1	
Group # 7					
	S-1	2.349	3-4	80.3	
CONTROL GROUP VAC. AND TESTED 75	S-2	2.277	6-7	-	69.23
	S-3	2.247	6-7	58.2	

TABLE 720: DIFFERENTIAL THERMAL ANALYSIS
(2 DAYS WATER SOAKING)
(NRC TRIAL NO.2-AMIR SECTION)
(TEMPERATURE -20 TO 85 F°)

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 1					
-20	A-1	2.426	1-2	103.6	102.02
	A-2	2.425	1.0	100.3	
	A-3	2.421	1-2	102.2	
Group # 2					
35	A-1	2.440	1.0	109.7	105.90
	A-2	2.422	1-2	101.9	
	A-3	2.413	1-2	106.1	
Group # 3					
65	A-1	2.432	1-2	92.4	89.50
	A-2	2.423	1-2	90.9	
	A-3	2.414	1-2	85.3	
Group # 4					
85	A-1	2.427	2-3	91.2	88.02
	A-2	2.425	2-3	92.3	
	A-3	2.419	1-2	80.5	

TABLE 720: DIFFERENTIAL THERMAL ANALYSIS
(2 DAYS WATER SOAKING)
(NRC TRIAL NO.2-AMIR SECTION)
(TEMPERATURE 110 TO 140 F°)

WATER BATH TEMPERATURE (F°)	SAMPLE No.	BULK SPECIFIC GRAVITY	% PERMEABLE VOIDS	INDIRECT TENSILE STRENGTH (psi)	AVERAGE STRENGTH (psi)
Group # 5					
	A-1	2.428	1-2	106.9	
110	A-2	2.423	1-2	102.8	105.80
	A-3	2.418	1-2	107.9	
Group # 6					
	A-1	2.426	1-2	104.0	
140	A-2	2.425	1-2	96.3	99.29
	A-3	2.420	1.0	97.6	
Group # 7					
CONTROL GROUP	A-1	2.427	1.0	98.97	
VAC. AND TESTED	A-2	2.424	1-2	106.8	105.42
75	A-3	2.418	1-2	110.5

FIGURE 7.20: RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

FIGURE 7.21: RELATIONSHIP BETWEEN SOAKING TEMPERATURE AND INDIRECT TENSILE STRENGTH

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.0 INTRODUCTION

The results and analysis of the analytical and experimental investigations presented in this thesis produced a number of new and important findings. The thesis presents a better understanding of the fundamental parameters that contribute to the problem of stripping of asphalt concrete mixtures in the field. More important, it presents a first step towards developing an analytical tool, testing and predicting methods, and a solution to the problem of stripping. Furthermore, the thesis provides verified explanations to a number of problems associated with the existing mechanisms and evaluation methods.

The main objective of this research was to examine theoretically and experimentally the hypothesis that stripping of asphalt mixtures could be mechanistically explained by forces generated due to incompatibility of the coefficients of thermal expansion of the mixture components. The hypothesis has been verified by adopting a research plan that included the following phases:

1. Reviewing present definitions, theories and mechanisms dealing with asphalt concrete stripping.
2. Evaluating current testing methods and identifying their deficiencies.
3. Identifying the most important parameters which govern the susceptibility of asphalt concrete mixtures to stripping.
4. Developing and executing an analytical investigation that deals with the identified parameters.
5. Developing and conducting a comprehensive experimental investigation to verify the findings of step 4.

6. Developing a framework of analytical and experimental components for the evaluation of the problem of stripping.

The implementation of this comprehensive research plan have resulted in a number of important findings which are summarised in the following sections.

8.1 STRIPPING PHENOMENON

The results of the literature review carried out in this study have shown that existing theories and mechanisms failed to explain the phenomenon. Subsequently, the failure to identify the main causes of stripping led to the adoption of laboratory and testing procedures that are not relevant to the investigated problem. No adhesion theory describe the asphalt/aggregate bond response to high temperatures. Since warm temperature forces stripping in the laboratory, this conditioning method is widely used in the evaluation of stripping susceptibility of asphalt concrete mixtures. However, it was shown in this research that water soaking at lower temperatures causes stripping which can not be completely explained by any of the current theories. In addition, current theories can not explain stripping damage observed after freeze-thaw cycles. Another important observation was the definition of stripping used in the literature. The present definition was adopted to fit certain mechanisms and specific conditioning temperatures. The fact that stripping has been defined as the physical separation of asphalt cement and aggregate at the interface resulted in the most serious deficiency in the present research work. Observations made during this experimental program left no doubt that separation occurred between the asphalt matrix (i.e., asphalt cement, sand and other fine material used as filler) and larger aggregate particles in the mixture. The problem redefinition based on these observations helped in explaining the problem of stripping in more engineering terms and hence, the development of the analytical model that can explain it.

8.2 IDENTIFICATION OF DEFICIENCIES IN THE CURRENT TESTING METHODS

A major deficiency in the existing laboratory testing methods dealing with the problem of stripping has been the poor correlation between laboratory predictions and field performance. This finding is important since it addresses the possibilities of minimizing the stripping problem. Clearly, failure to predict the expected performance of a given asphalt concrete mixture can lead to costly decisions. It was shown in this research that the main deficiency of the laboratory evaluation methods were the result of inability to identify the main causes of the problem and the lack of precise definition. Another important deficiency was the use of the ratio between the indirect tensile strength of conditioned and dry specimens. As explained in this thesis, this single ratio can be misleading. Previously reported data showed ratios that were more than 1 indicating that moisture conditioning improved the asphalt mixture. When these mixtures were used in the field, stripping occurred. No explanation was available neither to the reported improvement in the laboratory treated specimens nor to the reported field observations.

One other deficiency associated with the current laboratory testing method has been the use of laboratory prepared cores to carry out moisture studies. This treatment completely ignores the influence of compaction on the problem of stripping. The influence of construction on the test results include direction of rolling, construction induced cracks, aggregate orientation, surface condition and its effect on permeability.

Clearly, with the lack of a sound mechanistic explanation to the problem, evaluation of stripping using laboratory testing procedure will become a sheer chance or at best a trial and error approach.

8.3 IDENTIFICATION OF MOST IMPORTANT PARAMETERS

The most important findings of this research were the identification of the most important parameters controlling the resistance of a given mixture to stripping. First, the interaction between the different components of the asphalt mixture under a wide range of temperatures was proven to be the most significant factor. Existing theories treat the

asphalt concrete pavement at the macroscopic level of the mixture. The asphalt mixture includes components which are very sensitive to temperature changes and the response to temperature changes of these component, in direct contact with each other, is different. This behaviour makes the need to investigate the problem at the microscopic scale a "MUST". In order to achieve this objective, it was found necessary to identify the main parameters that influence the behaviour of the asphalt mixture at the microscopic level. These parameters were found to be:-

1. Coefficients of thermal expansion and contraction of the mixture components, i.e., the asphalt matrix and aggregate particles,
2. Temperature changes,
3. Initial conditions of the asphalt pavement after compaction,
4. Temperature and moisture exposure duration, and
5. Type of asphalt mixture

In addition to these five parameters, test results carried out in this investigation suggested that the air temperature during compaction may affect the overall behaviour of the compacted mixture as demonstrated by the cores taken from the NRC field trial No.2.

While stripping has been recognized to be influenced by the degree of exposure to moisture, and by the type of mixture. This thesis has identified, for the first time, the influence of differential coefficients of thermal expansion and compaction technique on stripping.

8.4 THE ANALYTICAL MODEL

One of the main contributions of this research was the formulation of a new mechanistic model that recognized the problems of thermal incompatibility between the asphalt material and the large aggregates. Due to the fact that no data exists today about this parameter in the field of asphalt concrete, it was decided that the model should be simple as a first attempt to analyze the phenomenon of stripping. As a result, a number of limitations exist in the developed model. While these limitations may affect the accuracy of the model, the test results of this research verified the main assumptions and findings of the analytical model.

In general, the developed model assumed that due to the considerable difference in the coefficients of thermal contraction of aggregate and the asphalt cement, the asphalt matrix will tend to contract more than aggregate during thermal cooling. Since contraction of the asphalt matrix is limited by aggregate contraction, contact pressure will develop at the aggregate/asphalt matrix interface. The contact pressure produced by differential contraction results in the build up of compressive radial stress and tensile tangential stress in the asphalt matrix.

As a result of the thermally induced stresses described earlier, the material performance during thermal cooling may be affected in the following manner:

1- If the thermally induced tensile stresses exceeds the asphalt matrix fracture strength, cracks will be initiated close to the interface. These cracks are different than those associated with low temperatures which are described in the literature. Cracks caused by differential contraction starts at the asphalt matrix/aggregate interface while the low-temperature cracks described in the literature are initiated on the pavement surface. Cracks initiated by differential thermal contraction can lead to further damage as described below:

- a- The asphalt concrete strength will decrease depending on the amount of damage caused by thermally induced stresses.
- b- During the thawing period, and after melting of ice in the spring, water from pockets inside the compacted mixture will penetrate to the interface through the initiated cracks and cause stripping.

2- If stresses due to differential contraction does not exceed the breaking strength of the asphalt matrix, the bond at the interface will be improved and the tensile strength, which is a function of both cohesion and adhesion, will increase.

These analyses were confirmed through the implementation of a comprehensive experimental investigation specifically designed to verify these findings.

Finally, while the analytical model was based on a number of simplifications, the following points should be considered:-

1. The solution assumes that the asphalt matrix, at sufficiently low temperature will perform basically as a brittle elastic solid with very low creep or flow potential. Asphalt cement flow causing stress relaxation below an assumed critical temperature (T_c) will affect the accuracy of calculated stresses. However, accumulation of stresses at lower temperatures, far below the assumed critical temperature may exceed the fracture strength of the mixture under investigation since colder temperatures contribute larger portions of thermally induced stresses.
2. The accuracy of the calculated stresses depends on measured material properties. The derived formulas reflect the need for accurate measurements of the coefficient of contraction and Young's modulus of the asphalt matrix. Very little information is available about these two properties because in the past investigators were concerned with the asphalt concrete mixture which include large aggregate particles.
3. Aggregate size and asphalt matrix thickness are two important factors for estimating thermal stresses in the asphalt matrix. These two factors vary within one mixture depending on the mixture design and the compaction method used.

8.5 VERIFICATION OF THE THEORETICAL FINDINGS

Laboratory investigation of the problem of asphalt stripping has been shown here to be a matter of trial and error. The causes and reasons for this observation were discussed in detail throughout the thesis. This thesis developed and presented a more reliable and thorough experimental approach. The approach discussed in Chapters 2, 3, 4 and 5 develops a new testing procedure that could be adopted to evaluate the stripping potential of field compacted asphalt concrete mixtures. The testing methodology followed in this thesis was based on the following factors:

- a. An analytical model that explains the basic mechanisms of the stripping problem.
- b. Understanding of the parameters and their interactions with respect to the problem at hand,

- c. Recognition of the deficiencies that exist in current testing method, and
- d. The need to develop a more reliable testing methodology.

The developed method includes the following features:

1. Asphalt concrete core specimens should be recovered from the field. The cores should represent the conditions of the finished pavement before the given section is opened to traffic.
2. The testing program should cover temperature ranges similar to those experienced by the pavement in question.
3. Moisture conditioning should not include the use of high vacuum pressure levels.
4. Exposure duration should be determined based on actual environmental conditions, rainfall, freezing season and quality of drainage.
5. Freeze/thaw cycles should be included only when the pavement is expected to be subjected to such a condition.
6. Analysis of the test results should be performed taking into account the overall performance of the mixture relative to the tested set of conditions. The adoption of one single strength ratio may lead to false conclusions.
7. Acceptance or rejection of a given mix in terms of its resistance to stripping should be based on a minimum accepted strength value rather than a ratio. This minimum acceptable strength value should be incorporated in future design practice.

8.6 INTEGRATED FRAMEWORK FOR VALUATION OF STRIPPING POTENTIAL

One of the main problems associated with the investigation of asphalt stripping is the definition of the problem. The existing definition led to the adoption of stripping susceptibility tests using high temperatures for conditioning. This resulted in the development of unreliable laboratory test methods. Furthermore, the limitation forced by the stripping problem definition produced a number of theories and mechanisms which

failed to explain the phenomenon at hand.

As a result, the first step in developing a reliable approach to evaluate the problem was to redefine the term "stripping". The redefinition was based on actual observations taken over the course of this investigation. Hence, stripping has been redefined as:

"the physical separation of the asphalt matrix and aggregate produced by adhesion failure".

This definition integrated the influence of temperature, both high and low, with the effect of moisture.

The second step in the new approach was the development of the analytical model. As discussed in the previous section, the new model, while generally verified, more research effort remains needed to include other factors such as the geometry of the aggregate particle. Nevertheless, the model represents a useful analytical tool to analyze a given mixture with respect to the main causes of stripping.

The third and final step in the framework is the development of a more comprehensive laboratory testing procedure for evaluating the stripping susceptibility of compacted asphalt concrete mixtures. The testing methodology and test results presented in this thesis demonstrated the importance of understanding the influence of the governing parameters and their interactions on the investigated problem. Clearly, this framework requires more development in order to be adopted by highway agencies and researchers. The research was limited to three different types of asphalt mixtures and, therefore, the results may not apply to all other mixtures. However, the framework can still be used to investigate the stripping susceptibility of any other asphalt concrete mixture.

8.7 INFLUENCE OF CONSTRUCTION CRACKS ON STRIPPING

Construction cracks is the most visible difference between the two compaction methods investigated. The data from tests performed to investigate the influence of differential expansion/contraction indicated that construction induced cracks influence indirect tensile strength of steel compacted cores for all conditioning levels. Almost all observed failure locations in steel compacted cores were along cracks which extended to

different depths below the surface. Even when conditioning was supposed to improve the asphalt matrix/aggregate bond of steel compacted cores, test results indicated no change in the strength. Similar conditions led to bond improvement in AMIR compacted cores which was reflected by an increase in strength. After two days of exposure to -20°F, AMIR compacted cores showed 11.8% increase in indirect tensile strength (from 100.6 to 112.5 psi) and no change due to 35°F while steel compacted cores showed no change due to both exposure temperatures. When crack free steel compacted cores were used, test results showed that the strength of these cores after 7 days exposure increased by 10.2% at 35°F (from 116.0 to 127.8 psi) and increased by 1.8% at -20°F (from 116.0 to 118.0 psi) while the strength of AMIR compacted cores changed from 150.1 psi to 154.3 at 35°F and to 148.3 psi at -20°F).

Construction cracks also reduce stripping resistance of compacted mixtures by allowing more surface water into the pavement. Test results of an experiment investigating this factor and not using vacuum saturation, showed that AMIR compacted cores allowed an average of 0.9% of its weight in water to enter the cores, while steel compacted cores allowed 1.2% to enter inside the cores. After seven days exposure at 110°F, AMIR compacted cores lost 7.4% (from 122.0 to 112.9) of its strength and the steel compacted cores lost 28.4% (from 120 to 86.1 psi) of its strength. Construction induced cracks allowed more water into the cores which explains the large difference in the strength of cores from the two compaction methods.

8.8 FUTURE WORK AND RECOMMENDATIONS

It has been shown in this thesis that the problem of asphalt stripping is a complex one. Also, it was demonstrated that in order to fully understand the problem, one has to follow a comprehensive research plan that deals with both the analytical as well as the experimental parts of the problem. The results of the thesis also suggested that in order to eliminate or minimize stripping of field compacted asphalt concrete surfaces, the following should be considered:

1. Design of asphalt mixtures should be carried out to produce a more thermally

compatible mixture. This can be achieved by analyzing the different coefficients of contraction and expansion and determining the type and magnitudes of induced stresses.

2. Based on the results of the stress analysis, selection of certain types of polymers or additives should be based on the premise of reducing the effects of thermal incompatibility over the expected temperature range included in the analysis.
3. While crack free compacted mixes suggested significant improvement in the resistance of the mixture to stripping, the availability of a commercial AMIR roller at the present time is uncertain. Therefore, more effort is needed to either prevent these cracks or at least minimize their occurrence.
4. The results of the laboratory testing carried out on asphalt cores compacted during a period of low temperature suggested that asphalt properties may be improved under higher temperatures. However, the results were based on a single test and more research work is needed to confirm this conclusion.

While these recommendations are based on the results and observations of a comprehensive research program, more research work is needed in order to apply the principal findings of this thesis. Further research work would be needed towards:

1. Further development of the analytical model

A number of simplifying assumptions were used to arrive at the formulas needed for calculating thermally induced stresses. Accurate representation of the response of each component in the asphalt concrete mixture to temperature changes is necessary for accurate description of the state of stress. The effect of aggregate particle size and shape, the presence of faults and the actual coefficient of thermal contraction of the asphalt matrix must be clarified.

2. Consideration of initial conditions of field cores

Formation of faults during mixing and compacting must be investigated to determine how these faults form and what factors control its formation. These faults led to significant decrease in the mixture resistance to stripping during this investigation.

3. Structural changes of the Bituminous Film

Previous studies concentrated on investigating the chemical functional group types in asphalt cement and their relative tendency to be absorbed on mineral aggregate surface. In lieu of the newly suggested mechanism, first described by Benson (64), the structural change following warm water soaking requires further investigation to confirm the piling of asphaltene following the oxidation of the thin bituminous film in direct contact with the aggregate.

4. Development of adequate additives or polymers

The analytical approach identified two conditions where the use of admixtures may help increase asphalt concrete resistance to stripping. The first condition is low temperatures where differential contraction may result in damage to the asphalt matrix. The other condition is warm water soaking which may result in structural changes in the thin bituminous film around aggregate particles leading to its complete disintegration. Therefore research efforts should be invested to develop admixtures that have compatible thermal properties with the components of the asphalt concrete mixture and at the same time shows more consistency under high temperatures.

APPENDIX A

APPENDIX A**1. M.T.O. HL-3 ASPHALT MIX DESIGN**

Re: Asphalt Paving - Test Sections

The following is our proposal Asphalt Mix Design for the above noted project submitted for your approval.

H.L.3 Surface Course Mix Proportions

	<u>Aggregate</u>	<u>Mix</u>
H.L.3 Stone (McFarland)	- 50.0%	47.4%
Quarry Screenings (McFarland)	- 16.7%	15.8%
Blending Sand (McFarland)	- 33.3%	31.6%
Asphalt Cement (85/100 PEN)	- -	5.2%

Please find enclosed supporting data for the above proposal mix design.

I trust that this will meet with your approval, but if you require further information, please do not hesitate to contact the undersigned at 829-1770.

Yours truly,
H.J. McFarland Const. Co.

Clive Nadin

Clive Nadin,
Q.C. Technologist,
Ottawa Branch.

CN:cm

INSPEC-SOL (ONTARIO) LTD.

Unit 9, 620 Cataraqui Woods Drive, Kingston, Ontario K7P 1T8 Tel.: (613) 389-9812 Fax: (613) 389-5287

Member:

250

Canadian Testing Association

Association of Consulting
Engineers of Canada

Canadian Standard Association
Certified Concrete Testing Laboratory

Reference No. 8164-K2282

July 26, 1989

MCFARLANDS CONSTRUCTION LTD.
P.O. Box 11068
994 Moodie Drive, South
NEPEAN, Ontario
K2H 7T5

ATTENTION: MR. CLIVE NADINE

RE: M.T.O. HL-3 Asphalt Mix Design
Contract No. 89-72

Dear Sir:

As requested by yourself, we have conducted an asphalt mix design for the above-noted contract as per Special Provisions No. 310F16. Please find enclosed, all relevant M.T.O. documents.

We trust this information meets your requirements at this time. Please do not hesitate to contact our office should further questions arise.

Yours very truly,

INSPEC-SOL (ONTARIO) LTD.

Mike Baird
Prepared by: Mike Baird, C.E.T.

Rob Cole
Approved by: Rob Cole, C.E.T.

In Duplicate
Encl.
MB/lcp

2cc: Ministry of Transportation of Ontario

Associated with

Inspec-Sol Inc., Montréal, Québec (514) 333-5151
Inspec-Sol (Québec) Inc., St-Romuald, Québec (418) 839-0041
Desi-Tech Inc., Montréal, Québec (514) 333-5151
Dury Consultants Inc., Montréal, Québec (514) 333-5142

BITUMINOUS MIX DESIGN REPORT

251

CONTRACT NO.	89-72	HOT MIX TYPE / USE	HL-3	ITEM NO.	
HWY.	17	LOCATION	VARIOUS		
TESTING LAB.	INSPEC-SOL (ONT) LTD		JOB MIX FORMULA NO.		
LAB. MIX NO.	K2282-1		Issued by Qual Assurance		
TEST DATA CERTIFIED BY:			DATE SAMPLES REC'D.		
			DATE COMPLETED	JULY 25/9	

JOB MIX FORMULA - GRADATION PERCENT PASSING *

% AC	26.5	19.0	16.0	13.2	9.5	4.75	2.36	1.18	600	300	150	
5.2			100	98.7	82.7	50.0	45.0	40.4	34.5	15.2	3.7	2.
MARSHALL	REQUIREMENTS		SELECTED		% CA # 1		50	% RAP		-		
% VOIDS (min.)	4.0 ± 1.0		4.3 (3.950)		% CA # 2			% AC RAP		-		
FLOW (min.)	10.0		11.0		% FA # 1		16.7	RAP PEN		-		
STAB. (min.)	6700		9900		% FA # 2		33.3	BRIQ. BRD		2.406		
% VMA (min.)	14.5		14.5		% FA # 3			MRD		2.507		
					Gb		2.661	MRO (SD)		2.498		

ASPHALT CEMENT	
SUPPLIER	PENETRATION
SHELL	85/100

ADDITIVE		
SUPPLIER	TYPE	AS % OF.

AGGREGATE TYPE	SOURCE / INVENTORY NO.	AGGREGATE TYPE	SOURCE / INVENTORY NO.
COARSE AGG. # 1	MOODIE DR. 05-108-0	FINE AGG. # 2	BURNSIDE K3-76-0
COARSE AGG. # 2		FINE AGG. # 3	
FINE AGG. # 1	MOODIE DR. 05-108-0	RAP	

AGGREGATE TYPE	BULK RELATIVE DENSITY	ABSORP-TION	AGGREGATE GRADATION - PERCENT PASSING										
			26.5	19.0	16.0	13.2	9.5	4.75	2.36	1.18	600	300	150
HL-3 CA.	2.696	0.732			100	97.6	65.5	4.9	1.9	1.2	1.0	0.8	0.7
SCR.	2.630	1.969					100	94.7	47.4	44.5	28.7	18.9	12.9
BLSA.	2.625	1.670					100	98.8	98.1	96.5	87.2	35.3	4.3
RAP CA													
RAP FA													

* FINES RETURNED TO THE MIX (0.5%)

REMARKS:

BITUMINOUS MIX DESIGN: Contract No. 89-72

Mix Type / Use HL-3

252

Lab Mix No. K2282-1

% Voids.

Marshall Stability (N) X 1000

Bulk Relative Density

Marshall Flow (0.25mm)

Maximum Relative Density

% VMA

4.2 4.6 5.0 5.4 5.8

A.C. %

4.2 4.6 5.0 5.4 5.8

A.C. %

VISUAL OBSERVATIONS: _____

SEMI-LOG GRADING CHART

MINISTRY SIEVE DESIGNATION

% VOIDS IN MINERAL AGGREGATE

254

DATE JULY 20/89

CONTRACT NO. 89-72

M A T E R I A L S	LAB. NO.	B. R. D.	WATER ABSORPTION
RAP Agg. - Retained 4.75 mm			
RAP Agg. - Passing 4.75 mm			
Coarse Aggregate #1	HL-3 C.H.	2.696	0.732
Coarse Aggregate #2			
Fine Aggregate #1	SCR.	2.630	1.969
Fine Aggregate #2	BL. SA.	2.625	1.670

MIXES SELECTED

	1	2	3	4
% RAP Agg. (Ret. 4.75 mm)				
% RAP Agg. (Pass. 4.75 mm)				
% Coarse Agg. #1	50			
% Coarse Agg. #2				
% Fine Agg. #1	16.7			
% Fine Agg. #2	33.3	...		
% A.C.	1. 4.6			
	2. 5.0			
	3. 5.4			

CALCULATIONS

1. Bulk Relative Density of Compacted Mix (Db) = $\frac{\text{Mass in Air}}{\text{Mass in Air} - \text{Mass in Water}}$

1	2	3	4
2.399			
2.402			
2.398			

2. B.R.D. of Aggregate (Gb) = $\frac{100}{\frac{\% \text{ Coa. Agg. \#1}}{\text{BRD Coa. Agg. \#1}} + \frac{\% \text{ Fine Agg. \#1}}{\text{BRD Fine Agg. \#1}} + \frac{\% \text{ RAP Agg. Ret. 4.75 mm}}{\text{BRD RAP Agg. Ret. 4.75 mm}} + \dots}$

1	2	3	4
2.661			

3. Bulk Relative Density of Compacted Aggregate (Gc) = $\frac{\text{Db} \times (100 - \% \text{ A.C.})}{100}$

1	2	3	4
2.289			
2.282			
2.269			

4. Percent Voids in Compacted Aggregate (% V.M.A.) = $\frac{(Gb - Gc)}{Gb} \times 100$

1	2	3	4
14.0			
14.7			

BITUMINOUS MIX FORM

TRANSPORTATION AND COMMUNICATIONS

2 6 M

		BRIQUETTE NUMBER			AVERAGE
		1	2	3	
A ₁	MASS OF COMPACTED SPECIMEN IN AIR	1251.0	1250.0	1248.6	
A ₂	SURFACE DRY MASS OF SPECIMEN IN AIR AFTER IMMERSION IN WATER	1251.6	1250.5	1249.1	
B ₁	MASS OF COMPACTED SPECIMEN IN WATER	728.4	731.4	727.5	
B ₂	VOLUME = $A_1 - B_1$	523.2	519.1	521.6	521.3
C	BULK RELATIVE DENSITY = $\frac{A_1}{B_1}$	2.391	2.408	2.394	2.398
D	MASS OF PLASK AND MIXTURE IN AIR	1404.0	1367.3	1393.2	
E	MASS OF PLASK IN AIR	400.5	421.8	406.4	
F	MASS OF MIXTURE IN AIR = D - E	1003.5	945.5	986.8	
F ₁	SURFACE DRY MASS OF MIXTURE IN AIR	1006.0	948.1	992.1	
G	MASS OF PLASK & MIXTURE IN WATER	822.4	799.3	815.8	
H	MASS OF PLASK IN WATER	220.1	232.2	224.0	
I ₁	MASS OF MIXTURE IN WATER = G - H	602.3	567.1	591.8	
I ₂	VOLUME = $F - I_1$	401.2	378.4	395.0	
I ₃	S.D. VOLUME = $I_2 - I_1$	403.7	381.0	400.3	
J	MAXIMUM RELATIVE DENSITY = $\frac{F}{I_2}$	2.501	2.499	2.498	2.499
J ₁	S.D. MAXIMUM RELATIVE DENSITY = $\frac{J}{I_2}$	2.492	2.488	2.478	2.486
K	PERCENT VOIDS IN MIXTURE = $\frac{J - C}{J} \times 100$	4.4	3.6	4.2	4.0
K ₁	S.D. PERCENT VOIDS IN MIXTURE = $\frac{J - C}{J} \times 100$	4.0	3.2	3.4	3.5
L	PERCENT V.P.A. = $\frac{V.M.A. - K}{V.M.A.} \times 100$				

S.D. MEANS SURFACE DRY
S.D. V.P.A. CALCULATED ON % VOIDS IN MINERAL AGGREGATE FORM

MARSHALL TEST VALUES

	BRIQUETTE NUMBER			AVERAGE
	1	2	3	
STABILITY (DIAL)	210	240	184	211
FLOW NR 1 (s)	3.9	4.3	4.0	
FLOW NR 2 (s)				
AVERAGE FLOW (s)				4.1

VISUAL OBSERVATIONS

MIX APPEARANCE	0.6 J.R. V.R.
BRIQUETTE APPEARANCE	0.6 J.R. V.R.
COATING	0.6 J.R. V.R.
STRIPPING	0.6 J.R. V.R.
C. AGG. FLACURE	0.6 J.R. V.R.

SUMMARY OF TEST RESULT

VOIDS (%)	4.0	3.5
FLOW (.25 mm)	11.0	
CORRECTED STABILITY (N)	10119	
% V.P.A. (%)	14.7	

CONTRACT NO. 09-72
DATE 10/19
SAMPLE NO. 9-1
REMARKS: AGGREGATE SAMPLE NO. 50
C/M
S/M
S/M

BITUMINOUS MIX FORM

Division of Transportation and Communications

MARSHALL TEST VALUES

	BRIQUETTE NUMBER				AVERAGE
	1	2	3		
A ₁ MASS OF COMPACTED SPECIMEN IN AIR	1251.7	1251.0	1248.8		
A ₂ SURFACE DRY MASS OF SPECIMEN IN AIR AFTER IMMOBILIZATION IN WATER	1251.9	1251.2	1249.0		
B ₁ MASS OF COMPACTED SPECIMEN IN WATER	729.3	729.1	728.7		
B ₂ VOLUME = $A_1 - B_1$	522.6	522.1	520.3		521.7
C BULK RELATIVE DENSITY = $\frac{A_1}{B_1}$	2.395	2.396	2.400		2.397
D MASS OF PLASK AND MIXTURE IN AIR	1436.2	1452.5	1373.6		
E MASS OF PLASK IN AIR	400.5	412.3	416.3		
F MASS OF MIXTURE IN AIR = D - E	1035.7	1040.2	957.3		
F ₁ SURFACE DRY MASS OF MIXTURE IN AIR					
G MASS OF PLASK & MIXTURE IN WATER	840.1	848.9	799.6		
H MASS OF PLASK IN WATER	220.1	227.6	222.1		
I ₁ MASS OF MIXTURE IN WATER = G - H	620.0	621.3	570.5		
I ₂ VOLUME = $F - I_1$	415.7	418.9	386.8		
I ₃ S.D. VOLUME = $F_1 - I_1$					
J MAXIMUM RELATIVE DENSITY = $\frac{F}{I_2}$	2.491	2.483	2.475		2.483
J ₁ S.D. MAXIMUM RELATIVE DENSITY = $\frac{F_1}{I_2}$					
K PERCENT VOIDS IN MIXTURE = $\frac{J - J_1}{J} \times 100$	3.7	3.5	3.0		3.5
K ₁ S.D. PERCENT VOIDS IN MIXTURE = $\frac{J - J_1}{J} \times 100$					
L PERCENT V.T.A. = $\frac{100(V.M.A. - S)}{V.M.A.} \times 100$					

* S.D. MEANS SURFACE DRY
 ** V.M.A. CALCULATED ON % VOIDS IN MINERAL AGGREGATE FORM
 *** V.F.A. VOIDS FILLED WITH ASPHALT

VISUAL OBSERVATIONS

MIX APPEARANCE	D.M.C.P.
BRIQUETTE APPEARANCE	D.M.C.P.
COATING (F.A.G.)	2.5
STRIPPING	60% M.M.
C. AGG. FRACTURE	M.M.

SUMMARY OF TEST RESULTS

VOIDS (%)	5.5
FLOW (1.25 mm)	14.8
CORRECTOR STABILITY (M)	73.6
** V.M.A. (%)	15.1

CONTRACT NO. 89-72

DATE 7/20/17

SAMPLE NO.

REMARKS:

NO. 3

% FINE 4.75 mm 5.0

BLENDED 2.7

AGGREGATE SAMPLE NO.

C/A

SAND

BAFFER

BITUMINOUS MIX FORM

MARSHALL TEST VALUES

	BRIQUETTE NUMBER			AVERAGE
	1	2	3	
STABILITY (DIAL)	174	225	201	200
FLOW NO 1 (s)	3.3	4.1	3.7	
FLOW NO 2 (s)				
AVERAGE FLOW (s)				3.7

VISUAL OBSERVATIONS

MIX APPEARANCE	D. 5.3, VR
BRIQUETTE APPEARANCE	D. 6.3, VR
COATING (S.G.G.)	1.28
STRIPPING	6.2, A, M
C. MOD. FRACTURE	6.2, A, M

SUMMARY OF TEST RESULT

VOIDS (%)	4.6	4.2
FLOW (1.23 mm)	10.9	
CORRECTED STABILITY (N)	9603	
% V.M.A. (%)	17.2	

CONTRACT NO. 85-72 ML 3 % A.C. 5.0
 DATE 10-4-78 % FLOW 4.73 mm 5.0
 SAMPLE NO. 2-7 % A.C. 5.0

REMARKS: MP
 C.A. MP
 SAND MP
 GRAVEL MP

	BRIQUETTE NUMBER			AVERAGE
	1	2	3	
A ₁ MASS OF COMPACTED SPECIMEN IN AIR	1251.6	1249.6	1248.5	
A ₂ SURFACE DRY MASS OF SPECIMEN IN AIR AFTER IMMERSION IN WATER	1252.3	1250.6	1249.8	
B ₁ MASS OF COMPACTED SPECIMEN IN WATER	729.1	728.8	734.0	
B ₂ VOLUME = $A_2 - B_1$	523.2	521.8	515.8	520.3
C BULK RELATIVE DENSITY = $\frac{A_1}{B_1}$	2.392	2.395	2.420	2.402
D MASS OF PLASK AND MIXTURE IN AIR	1325.2	1409.3	1425.7	
E MASS OF PLASK IN AIR	408.0	413.6	414.6	
F MASS OF MIXTURE IN AIR = D - E	921.2	995.7	1061.1	
F ₁ SURFACE DRY MASS OF MIXTURE IN AIR	923.7	997.4	1064.2	
G MASS OF PLASK & MIXTURE IN WATER	778.9	827.7	869.5	
H MASS OF PLASK IN WATER	225.2	227.6	228.1	
I ₁ MASS OF MIXTURE IN WATER = G - H	553.7	600.1	641.4	
I ₂ VOLUME = $F - I_1$	367.5	395.6	419.7	
I ₃ S.D. VOLUME = $F_1 - I_1$	370.0	397.3	422.8	
J MAXIMUM RELATIVE DENSITY = $\frac{F_1}{I_1}$	2.586	2.517	2.528	2.517
J ₁ S.A. MAXIMUM RELATIVE DENSITY = $\frac{F_1}{I_1}$	2.496	2.510	2.517	2.508
K PERCENT VOIDS IN MIXTURE = $\frac{J - J_1}{J_1} \times 100$	4.5	4.8	4.3	4.6
K ₁ S.D. PERCENT VOIDS IN MIXTURE = $\frac{J - J_1}{J_1} \times 100$	4.2	4.6	3.9	4.2
L PERCENT V.P.A. = $\frac{100 \times V.M.A. - 5}{V.M.A.} \times 100$				

Agency or
Transportation and
Construction

BITUMINOUS MIX FORM

MARSHALL TEST VALUES

	BRIQUETTE NUMBER			AVERAGE
	1	2	3	
STABILITY (DIAL)	228	174	200	201
FLOW NO 1 (s)				
FLOW NO 2 (s)	3.5	3.1	3.6	
AVERAGE FLOW (s)				3.4

VISUAL OBSERVATIONS

AIR APPEARANCE	0.68, VR
BRIQUETTE APPEARANCE	0.68, VR
COATING (C, AGG)	1.28
STRIPPING	0.68, VR
C. AGG. FRACTURE	0.68, VR

SUMMARY OF TEST RESULTS

VOIDS (%)	5.6
FLOW (1.32 mm)	9.3
CORRECTED STABILITY (N)	9652
% V.M.A. (%)	14.0

CONTRACT NO. 89-72
DATE 12/14/81
SAMPLE NO. 2-1
REMARKS: MCF
AGGREGATE SAMPLE NO.
C.M.
S.M.S.
S.A.V.S.R.

	BRIQUETTE NUMBER			AVERAGE
	1	2	3	
A ₁ MASS OF COMPACTED SPECIMEN IN AIR	1250.1	1247.8	1257.2	
A ₂ SURFACE DRY MASS OF SPECIMEN IN AIR AFTER IMMERSION IN WATER	1251.9	1249.0	1252.5	
B ₁ MASS OF COMPACTED SPECIMEN IN WATER	730.2	731.7	728.6	
B ₂ VOLUME = $A_2 - B_1$	521.7	517.3	523.9	521.0
C BULK RELATIVE DENSITY = $\frac{A_1}{B_1}$	2.396	2.412	2.388	2.399
D MASS OF FLASK AND MIXTURE IN AIR	1362.7	1324.5	1361.3	
E MASS OF FLASK IN AIR	406.4	413.3	405.2	
F MASS OF MIXTURE IN AIR = D - E	956.3	911.2	956.1	
F ₁ SURFACE DRY MASS OF MIXTURE IN AIR	958.7	913.6	958.7	
G MASS OF FLASK & MIXTURE IN WATER	804.5	781.1	801.1	
H MASS OF FLASK IN WATER	224.0	222.3	222.9	
I ₁ MASS OF MIXTURE IN WATER = G - H	580.5	553.8	578.2	
I ₂ VOLUME = $F - I_1$	375.8	357.4	377.9	
I ₃ S.D. VOLUME = $I_2 - I_1$	378.2	359.8	380.5	
J MAXIMUM RELATIVE DENSITY = $\frac{F}{I_2}$	2.545	2.530	2.530	2.542
J ₁ S.D. MAXIMUM RELATIVE DENSITY = $\frac{J - J_1}{J_1} \times 100$	2.535	2.539	2.520	2.531
K PERCENT VOIDS IN MIXTURE = $\frac{J - C}{J} \times 100$	5.9	5.4	5.6	5.6
K ₁ S.D. PERCENT VOIDS IN MIXTURE = $\frac{K - K_1}{K_1} \times 100$	5.5	5.0	5.2	5.2
L PERCENT V.P.A. = $\frac{100 - V.M.A. - V}{V.M.A.} \times 100$				

* S.D. MEANS SURFACE ONLY
n = V/V_a CALCULATED ON % VOIDS IN MINERAL AGGREGATE FORM

MARSHALL TEST VALUES

Transportation and Communications

	A	B	BRIQUETTE NUMBER			AVERAGE
			1	2	3	
A ₁	MASS OF COMPACTED SPECIMEN IN AIR	1350.7	1299.8	252.0		
A ₂	SURFACE DRY MASS OF SPECIMEN IN AIR AFTER IMMERSION IN WATER	1353.5	1252.8	1254.9		
B ₁	MASS OF COMPACTED SPECIMEN IN WATER	719.5	731.1	731.7		
B ₂	VOLUME = $A_2 - B_1$	524.0	521.7	523.2	523.0	
C	BULK RELATIVE DENSITY = $\frac{A_1}{B_1}$	2.387	2.396	2.393	2.392	
D	MASS OF PLAS AND MIXTURE IN AIR	1256.4	1274.1	1378.4		
E	MASS OF PLAS IN AIR	413.6	416.3	400.5		
F	MASS OF MIXTURE IN AIR = D - E	842.8	857.8	977.9		
F ₁	SURFACE DRY MASS OF MIXTURE IN AIR	740.4	744.6	796.3		
G	MASS OF PLAS & MIXTURE IN WATER	2276	229.1	220.1		
H	MASS OF PLAS IN WATER	512.8	520.5	576.2		
I ₁	MASS OF MIXTURE IN WATER = G - H	330.0	337.3	371.7		
I ₂	VOLUME = F - I ₁					
J ₂	S.D. VOLUME = $F_1 - I_1$	255.4	254.3	255.0	254.9	
J ₁	MAXIMUM RELATIVE DENSITY = $\frac{F_1}{I_1}$					
K	S.R. MAXIMUM RELATIVE DENSITY = $\frac{F_1}{I_1}$					
L	PERCENT Voids IN MIXTURE = $\frac{J_2 - C}{J_1} \times 100$	65	5.8	62	62	
L ₁	PERCENT Voids IN MIXTURE = $\frac{J_2 - C}{J_1} \times 100$					
L ₂	PERCENT V.P.A. = $\frac{100 \times V.M.B. - 1}{V.M.B.}$					

0.30 MEANS SURFACE ONLY

	BRIQUETTE NUMBER			AVERAGE
	1	2	3	
STABILITY (DIAL)	212	246	240	233
FLOW NO 1 (g)	3.1	3.5	3.5	
FLOW NO 2 (g)				
AVERAGE FLOW (g)				3.4

VISUAL OBSERVATIONS

MIX APPEARANCE	60 M. R. VR
BRIGHTNESS APPEARANCE	60 M. R. SP. P.
COATING	1. AGG 2. AGG 3. AGG
STRIPPING	40 M. M. M
C. AGG. FRACTURE	60 M. M. M

SUMMARY OF TEST RESULTS

VOIDS (%)		6.2	0.0
FLOW (.25 mm)		90	
CORRECTED STABILITY (N)		10697	
0.0 % M.A. (%)		14.9	

89-72

3 sec 4.2

3816167115

11 AUG 6 73 AM 530

ON 1 NOV 63

new 2-1

730

ON 1 JAN 1968
114720Z

REMARKS:

1

1400	20
1400	20

APPENDIX A

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2- M.T.O. HL-4 ASPHALT MIX DESIGN

CONTRACT NO.		HOT MIX TYPE / USE	HL4/	ITEM NO.	
HWY.		LOCATION			
TESTING LAB.			JOB MIX FORMULA NO.		
LAB. MIX NO.			Issued by Qual Assurance		
			DATE SAMPLES REC'D.		
			DATE COMPLETED		
TEST DATA CERTIFIED BY:

JOB MIX FORMULA - GRADATION PERCENT PASSING *

% AC	26.5	19.0	16.0	13.2	9.5	4.75	2.36	1.18	600	300	150	75
5.0			99.4	90.7	73.7	54.3	47.8	42.4	34.9	17.5	6.1	4.6

MARSHALL	REQUIREMENTS	SELECTED	% CA #1	46.1%	% RAP	
% VOIDS (min.)		3.8	% CA #2		% AC RAP	
FLOW (min.)		10.4	% FA #1	18.0 %	RAP PEN	
STAB. (min.)		12500	% FA #2	35.9 %	BRIQ. BRD	
% VMA (min.)		15	% FA #3		MRD	
			Gb		MFD (SD)	

ASPHALT CEMENT

SUPPLIER	PENETRATION
ASHWARREN	85/100

ADDITIVE

SUPPLIER	TYPE	AS % OF AC

AGGREGATE TYPE	SOURCE / INVENTORY NO.	AGGREGATE TYPE	SOURCE / INVENTORY NO.
COARSE AGG. #1	BOYCE HL4 STONE	FINE AGG. #2	RYPER SAND
COARSE AGG. #2		FINE AGG. #3	
FINE AGG. #1	BOYCE SCREENING	RAP	

AGGREGATE TYPE	BULK RELATIVE DENSITY	ABSORP- TION	AGGREGATE GRADATION - PERCENT PASSING											
			26.5	19.0	16.0	13.2	9.5	4.75	2.36	1.18	600	300	150	75
CA. 1	2.164	0.568			98.0	72.9	42.9	4.5	2.2	1.8	1.6	1.4	1.3	0.9
FA. 2	2.635	0.969						98.4	97.0	93.1	78.5	33.4	2.8	1.5
FA. 1	2.751	0.898						93.5	66.6	45.3	33.6	26.6	12.2	15.1
RAP CA														
RAP FA														

* FINES RETURNED TO THE MIX ()

SEMI-LOG GRADING CHART

MINISTRY SIEVE DESIGNATION

H.L. - 4

APPENDIX B

APPENDIX B

1- VACUUM SATURATION AND PERMEABLE VOIDS DETERMINATION

The following is the procedure followed to saturate cores in the laboratory and the formula used to calculate permeable voids as suggested by Transport Canada.

1. Selected dry asphalt cores were placed in a deep water vacuum saturation jar. The jar was then filled with distilled water to a height of one inch above the top of the core.
2. Vacuum saturation was achieved by the application of the desired pressure, 14 or 24 inch of mercury vacuum pressure, to the cores submerged in the distilled water at 75°F for a duration of 30 minutes.
3. After the 30 minutes, the vacuum was removed and the inside of the jar was allowed to reach ambient atmospheric pressure. The cores were kept submerged in the jar under atmospheric pressure for another 30 minutes.

$$\text{Permeable Void \%} = 100[(w'_1 - w_1) + (w'_s - w_s)] / (w'_s - w_1)$$

where,

w_1 : submerged weight of initially desiccated dry core section immersed in distilled water at 75°F for 3 minutes

w_s : weight in air of desiccated (dry) core

w'_1 : submerged weight of vacuum saturated core in distilled water at 75°F

w'_s : weight in air (saturated surface dry) of core section after obtaining w_1 .

2- INDIRECT TENSILE TEST

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Procedure used for determining tensile strength followed Transport Canada Method as described below:

a- Dry and conditioned Cores were tested at 75 F.

b- One inch flat loading block was used and the core was placed under a loading head of a compressive testing machine, set at the vertical deformation rate of 2 inches per minute at 75 F test temperature. The maximum compression load of the core was recorded. The load was then decreased rapidly to zero.

c- The core was then measured for flattening (corresponding to maximum load condition) to the nearest 0.01 inch.

d- After recording maximum load and measuring flattening, cores were further deformed at the prescribed deformation rate until a vertical tensile crack opened. The cores were then separated in half and their interfaces examined for visual moisture damage, i.e. stripping.

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

$$St = \frac{S10}{10,000} \cdot \frac{P_{max}}{L}$$

where,

St = Tensile strength in psi

Pmax = Maximum compressive load on core in lab.

L = Thickness of core in inches.

S10 = Maximum tensile stress, in psi produced by a load P of 10,000 lb in a 4 inch diameter solid cylinder and a thickness of one inch.

$$S1 = 1591 + 437a - 1889a^2 + 2854a^3 - 2474a^4 + 885a^5$$

a = Amount of core flattening, in inches under Pmax (0 to 1 inch).

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