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Moisture Susceptibility Of Dense Graded Asphalt Concrete Using Cyclic Moistue Induced Stress Tester

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Dedication

I would like to dedicate this thesis to my parents.

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by

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ABSTRACT

Moisture damage in asphalt concrete causes loss in mechanical properties of asphalt material due to the presence of moisture in its microstructures in a liquid or a vapor state. Some of the common types of distress observed in asphalt pavements due to moisture damage are stripping (separation of asphalt binder and aggregate), raveling (dislodgement of aggregate particles in asphalt mixture from surface), and hydraulic scour (a process that occurs on a saturated surface by which the pavement material is eroded due to dynamic action of tires in the presence of water). This thesis focuses on the research on the effects of such moisture in dense graded HMA samples using an accelerated moisture conditioning technique in the laboratory. The laboratory work consisted of preparation of samples from a HMA mixture type commonly used by the Texas Department of Transportation. The samples were conditioned for moisture in a Moisture Induced Stress Tester (MIST) machine which applies repeated water pressure on samples to simulate a repeated traffic load on a moisture intruded pavement layer. Dynamic Modulus Test of the samples following AASHTO – TP 62-03 and Indirect Tensile Test following Tex-226-F were carried out both before and after moisture conditioning in MIST to see the changes in stiffness parameters. Different levels of such moisture conditioning were correlated with the Dynamic Modulus and Indirect Tensile strength of asphalt samples. Results showed that moisture causes reduction in stiffness properties with high variability, which was backed up by statistical analysis. Test results were used as inputs and simulated in KENLAYER program which showed that the fatigue failure is more significant in higher frequencies and lower temperatures while the rutting basically increased after moisture conditioning.

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CHAPTER 1: INTRODUCTION

Asphalt concrete is a mixture of asphalt binder and different sized aggregates in certain proportions used as the top-most layer in a flexible pavement. There are different types of asphalt concrete mixes based on the gradation, mix proportion, and use. Among numerous existing mix designs the most common used in the state of Texas are Dense Graded Mix, Permeable Friction Course (PFC), and Thin Overlay Mix (TOM). The scope and performance of such mix designs significantly vary from one another. Moisture susceptibility, as one of the performance criteria for such mixes, greatly depends on their mix designs.

Moisture damage in asphalt concrete can be defined as the loss in mechanical properties of the material due to presence of moisture in its microstructures in a liquid or vapor state. Intrusion of moisture in asphalt pavements normally leads to loss of adhesive bonding between aggregate and binder, resulting in separation of these materials. Damage due to moisture in asphalt mixtures were first recognized in the 1930s and have been studied extensively since then (Caro et al., 2008). Numerous studies have been carried out to understand and characterize the damage in asphalt due to moisture. The common phenomena observed in asphalt pavements due to the moisture damage are stripping (separation of asphalt binder and aggregate), raveling (dislodgement of aggregate particles in asphalt mixture from surface), and hydraulic scour (a process that occurs on a saturated surface by which the pavement material is eroded due to dynamic action of tires in the presence of water) (Mc Gennis 1984, Straut 1990 and Hicks et al. 2003). If stripping within the pavement becomes excessive, severe pavement deformation and failure may occur as a result of repeated loading. Stripping failure can take the form of potholes,

cracking, and/or surface raveling of the pavement. Wearing courses placed over stripped asphaltic bases are likely to exhibit adhesion failure by raveling (Hicks 1991). This leads to premature deterioration of asphalt pavements, which in turn, is reported to have an annual extra vehicle operating cost estimated as much as $54 billion (Copeland, 2005).

CHAPTER 2: LITERATURE REVIEW

This chapter of the thesis provides the introduction and various mechanism of moisture damage as well as the review of the studies conducted in the field of moisture damage in asphalt concrete. The first section discusses different modes of moisture transportation in asphalt pavements. Next, it explains different mechanisms of moisture damage and various factors causing moisture damage. Afterwards, the importance of void structures and forces in aggregate-binder interface is discussed. Later, different test methods and various mathematical models used to characterize the asphalt concrete are explained. Finally, the measures to control moisture damage and the test concerning the asphalt concrete and various test methods are discussed.

2.1 Modes of moisture transport

Moisture can intrude the asphalt pavement in various ways and forms. There are mainly three different modes by which water reaches the asphalt mixtures: (1) infiltration of surface water due to permeability in the mix, (2) capillary rise of subsurface water, and (3) permeation or diffusion of water vapor (Masad *et al*. 2007).

Infiltration of the water into the asphalt concrete from the surface is one of the major modes of moisture transportation in asphalt pavements. The amount of moisture that intrudes into the pavement depends on the permeability, type of mix used, and other environmental factors like amount of precipitation, humidity, and temperature.

Permeability of the mix mainly depends on the air void structure of the mix. Various functions have been formulated to characterize the permeability of the mix as the function of

total air voids. Masad *et al*. (2007) used properties like void structure distribution to model the porosity of air voids. The air void distribution in laboratory samples can be completely different from the air voids distribution in the field. Mohammad *et al*. (2003) and St Martin *et al*. (2003) found out that there is an inverse relationship between the permeability of asphalt mix and the lift thickness when the mix is compacted in the field with fixed amount of air void content.

Capillary rise is also one of the other important modes of moisture transportation in asphalt mixtures. This rise in water level is due to surface forces between liquid and solid. In asphalt concrete, capillary rise causes large amount of moisture transportation. In general, capillary rise is not expected to take place when water is in contact with the neat binder due to the hydrophobic nature of the binder, but capillary rise does occur in asphalt mixtures when water comes in contact with the mastics (Caro *et al*., 2008). There has not been much research to quantify the phenomenon of moisture damage due to capillary rise. Masad *et al*. (2007) used X-ray CT and image analysis technique to quantify the capillary rise in asphalt concrete, and also estimated the contact angle between water and asphalt mastic.

Vapor diffusion is another important mode of moisture transport in asphalt concrete. The amount of water vapor and rate at which it accumulates in an asphalt mixture depend on three primary factors: (1) relative humidity, (2) diffusion coefficients, and (3) storage rate and storage capacity (Caro *et al.,* 2008). Sasaki *et al*. (2006) analyzed vapor mass transfer and water storage mechanisms in asphalt mixtures that generate moisture damage. The result supported the idea that there is a relationship between vapor transport, storage capacity, and moisture damage. Kassem *et al.* (2006) studied moisture diffusion in asphalt mixtures by measuring suction (i.e. free energy state of water) using thermocouple psychrometers. The study reported that low

suction values were related to high relative humidity gradients in voids and presented higher levels of moisture damage. Cheng *et al.* (2003) demonstrated the differences in rates of moisture diffusion and water-holding capacity of different asphalt binders using gravimetric measurements in controlled relative humidity conditions. The study showed that the amount of water vapor absorbed by the asphalt binder was significant, and also presented the hypothesis that mixtures that contain binders with high water holding potential were more prone to moisture damage. Kringos and Scarpas (2005) used self-developed finite element routine, called raveling of asphalt mixes (RoAM), to conduct numerical simulations of diffusion that generates dispersion in mastic due to vapor diffusion.

2.2 Moisture Damage Mechanism

Water can access the asphalt mixture in different ways depending on the external environment and the air void structure. Interaction of water in asphalt mix usually results in reduction of cohesive strength of asphalt binder and/or reduction in adhesive strength of mastic-aggregate interface. The mechanisms of moisture damage that can be observed in asphalt concrete are desorption, detachment, dispersion, displacement, and spontaneous emulsification (Kiggundu and Roberts, 1988).

Among some common mechanisms of moisture damage, detachment and displacement are one of the most important ones. Detachment is the separation of asphalt binder film from the aggregate surface without any apparent damage to the asphalt binder film. Detachment is usually seen in aggregates that are covered with dust or are improperly dried during construction process, or when the asphalt binder coating is very thin allowing the passage of water through it

by diffusion (Arambula, 2007). Displacement, however, is the removal of asphalt binder from the aggregate surface due to moisture intrusion.

Another mechanism of moisture damage seen in asphalt concrete is advective flow, where water in relatively large amounts can flow at high speeds and mechanically wash away the asphalt binder from the aggregate. These are usually common in open graded asphalt mixes where the flow causes desorption of binder and results in failure of adhesive bond between asphalt mastic and aggregate.

Kringos (2007) divided the process of moisture damage into two categories; physical and mechanical process. The physical process includes the weakening of aggregate and mastic bond or the weakening of asphalt and mastic bond. The mechanical process, on the other hand, includes occurrence of intense water pressure fields inside the mix caused by traffic loads, referred to as pumping action.

When asphalt concrete is exposed to moisture, the water diffuses through the asphalt mastic and reaches the aggregate mastic surface causing the weakening of the bond between mastic and aggregate. During this process, no damage is seen in the mastic layer itself, but the mastic actually separates from the aggregate surface, which creates room for more moisture storage between the layers and increased weakening of the bond. The other physical phenomenon of moisture damage that causes weakening of the mastic bond is the actual washing away of the mastic when the asphalt concrete is exposed to a fast water flow field (in other words, called additive flow) (Kringos, 2007). The pumping action, which is a mechanical process of moisture damage, is a different phenomenon altogether. Normally when a vehicle

passes over the pavement surface, the asphalt concrete experiences stress, which is transferred to the water present in the voids. This phenomenon is usually seen in the dense graded mixes where the hydraulic pressure created by the water does not dissipate due to low permeability and further weakens the bonding mechanically. When the wheel load moves away, the hydraulic pressure goes back to normal. This process repeats every time the wheel load passes over the pavement, which causes the constant pumping effect within the asphalt structure and deteriorates the asphalt concrete.

2.3 Factors causing moisture damage

There are various factors that influence the behavior of asphalt concrete on moisture intrusion. Stuart (1990) described the following factors that affect the moisture damage in asphalt concrete. Each factor that contributes to the moisture damage is explained further in brief.

1. Type of aggregate
    - Composition
        - ✓ Degree of acidity or PH
        - ✓ Surface chemistry
        - ✓ Types of minerals
        - ✓ Source of aggregate
    - Physical characteristics
        - ✓ Angularity
        - ✓ Surface roughness
        - ✓ Gradation

        - ✓ Porosity
        - ✓ Permeability
    - Dust and clay coatings
    - Moisture content
    - Resistance to degradation
2. Type of asphalt
    - Grade or hardness
    - Chemical composition
    - Crude source and refining process
3. Mixture design and construction
    - Air void level and compaction
    - Permeability and drainage
    - Film thickness
4. Environment
    - Temperature
    - Freeze thaw cycles
    - Moisture vapor
    - Dampness
    - Pavement age
    - Presence of ions in water
5. Traffic
6. Antistripping additive properties

2.3.1 Types of Aggregates

The composition and physical characteristics of the aggregates influence the moisture susceptibility of asphalt. Even when asphalts are amphoteric, or capable of functioning as a base or an acid, they have generally been considered slightly acidic in most chemical bonding studies (Straut, 1990). Hence basic aggregates have better chemical adhesion to the asphalt binder compared to acidic aggregates; and thus, less chances of stripping.

Bagampadde (2005) investigated the influence of aggregate chemical and mineralogical composition on moisture sensitivity using eleven different types of aggregates from tropical and temperate climates and one bitumen type. The results showed that mixtures from aggregates containing alkali metallic elements, like sodium and potassium, exhibited relatively high moisture sensitivity, while no indication of moisture sensitivity was seen for aggregates containing calcium, magnesium, and iron. No significant correlation was seen for aggregates with alumina and silica. Moisture sensitivity was observed high in mixtures having aggregates with high contents of both quartz and alkali feldspars. Aggregates with practically 100% quartz showed high resistance to moisture damage.

Asphalt binder is generally composed of Asphaltenes and Maltenes with some acidic organic compounds like carboxylic acids (RCOOH) and acid anhydrides. The aggregates used in pavement mainly consist of calcareous aggregates like marble, limestone, basalt, and dolomite to siliceous aggregates like sandstone, granite, and quartzite. Since the basic ingredients are hardly present in asphalt binder, chemical adhesion or bonding of acidic siliceous aggregate with asphalt binder does not occur as opposed to alkaline aggregates (Petersen et al., 1982). In

addition to that, the chemical bonding between aggregates with high carbonate content and asphalt binder is usually stronger in the presence of water (Hicks, 1991). However, aggregates with higher calcium carbonate content are more wearable and are hard to pass the polish test compared to the siliceous aggregates. It is usually common to use siliceous material as fines in the mixes, so stripping in the fines is more of a problem.

There has been very little research on the effect of physical characteristics of the aggregates like shape, texture, angularity, porosity, surface area, cleanliness, moisture content, mineralogical composition, and gradation on moisture damage in asphalt pavements. Higher angularity and surface roughness increases the mechanical interlock of aggregates, and can help resist the moisture damage. Aggregate wetting and uniform film thickness may, however, be difficult to obtain with aggregates of high angularity. Asphalt film at sharp edges may be very thin and more susceptible to breaking due to higher angularity. Crushing increases the angularity and surface roughness of the aggregates, but the changes in surface energy factors must also be considered. The mechanical interlock and contact area between asphalt and aggregate is also varied by varying the aggregate gradation.

Aggregates with high porosity and adsorption capacity allow the asphalt binder to penetrate deeper into the pores, crevices, and capillaries at its surface and improves the mechanical interlock between the two materials. A large surface area also provides an improved bond with the asphalt binder because of the increased contact region between the two materials. Damp and dirty aggregates result in weaker bonding at the interface of asphalt mastic and aggregate.

2.3.2 Type of Bitumen

The properties of asphalt binder like thickness, viscosity, chemical composition, aging, and surface energy influence the moisture damage in asphalt concrete. Thicker asphalt film causes cohesive failure within the asphalt mastics, whereas in thinner films, the failure is mainly adhesive, which is at the interface between asphalt mastics and aggregate (Cheng et al. 2002, 2003).

Stiffness of asphalt binder may affect the moisture susceptibility of asphalt concrete. Stiffer binders are harder to detach from the aggregate at normal temperature and thus have more moisture resistance. The strength or cohesiveness of a very stiff mixture may not decrease significantly even when there is a significant amount of detachment (Straut, 1990). This is often seen in age-hardened asphalt binders.

The chemical compounds present in asphalt concrete also play an important role in moisture susceptibility. Compounds like carboxylic acids and sulfoxide are undesirable because they easily attach to the aggregate surface, but are also removed with ease in presence of water (Arambula, 2007). In addition, the unbalanced charges present at the aggregate and asphalt interface results in surface energies. The adhesive bond between the binder and aggregates depend on the surface free energy values.

2.3.3 Mixture Design and Construction

The air void content in asphalt mix is an important factor to be considered for the mixture to be less moisture susceptible. Higher compaction in the field causes low air void content, and

makes the pavement impermeable and less susceptible to moisture as seen in dense graded mixture. In an open graded mixture, however, the water flows completely through the asphalt layer into the base layer. It is desirable to have nitrogen and phenol bases in binders because of their lower desorption in the presence of water (Hicks 1991; Little and Jones 2003). Aging of asphalt binder, on the other hand, makes the asphalt concrete prone to moisture damage. Additionally, low binder content results in higher air void content; and hence, is more susceptible to moisture damage.

2.3.4 Other factors

Arambula (2007) described four external factors that influence moisture damage, namely: production, construction, traffic level, and environmental conditions. Improper mixing of asphalt concrete may cause segregation during the production of asphalt concrete. Similarly, improper drying of asphalt also needs to be avoided to prevent moisture damage. Insufficient or poor compaction should be avoided during the construction phase.

Among other factors, environmental conditions like freeze and thaw cycles, temperature variations dampness, and presence of moisture are some of the factors contributing the moisture susceptibility of pavements. Field experiences indicate that cool rainfalls and rapid drops in temperature while pavement mixture is being placed or cured can have harmful effects on adhesion (Straut, 1990). Pressure and water movements due to freezing and thawing can rupture asphalt films and thus may promote stripping. Aging, on the other hand, increases the stiffness of asphalt and thus may decrease the susceptibility of moisture damage.

Stresses from traffic and the effects of water interact to cause pavement failure. Sharp aggregate edges may be very susceptible to breaking. Because of these edges, the stress is high while the film thickness is low. In some cases, decreased pavement air voids and permeability due to traffic may reduce the susceptibility to moisture damage.

2.4 Importance of void structure

Void structure has an important role in characterizing the moisture damage in asphalt mixes. Even though the percentage of air voids is considered important, the structure of air voids also plays a significant role in characterizing moisture damage (Mohamed et al. 1993). The pore structures in an asphalt concrete can mainly be divided into three components: permeable air voids, dead end air voids, and isolated air voids (Tarefder and Ahmad, 2015). Arambula *et al*. (2007) and Masad *et al.* (2007) studied the void structure of asphalt specimens using X-ray CT scan method and explained the importance of properties of void structure in terms of size, distribution, connectivity, and tortuosity of the flow path to characterize the effect of air voids in moisture susceptibility of asphalt. The mixtures with same binder and same target air voids had different air void size distribution and connectivity (Arambula *et al*., 2007). Al-Omari et al. (2004) also used X-ray CT scan technique to determine the permeability in asphalt mixtures. He determined the tortuosity, which is the ratio of actual length of the void path in a sample to the length between extreme points. The study showed that the tortuosity of samples can be as high as four, which means that the void lengths in a sample can be up to four times the distance between these points.

Masad *et al*. (2006) used the ratio between fatigue life of a moisture-conditioned specimen after accelerated moisture conditioning and fatigue life of a dry specimen using Superpave Indirect Tensile Test. The results showed that there is an average size of air void, referred to as pessimum air void size, for which the moisture sensitivity of the mixture is maximum. Moreover, according to D'Angelo and Anderson (2003), most of the dense graded asphalt mixes are designed with air void content of around 5-10%, which is most likely to be in the range of pessimum air voids. Nonetheless, it is important to understand the differences in air void structures between laboratory samples and field cores to assess and predict moisture damage (Caro et al. 2008).

The size and distribution of air voids in asphalt mixtures depends mainly on the aggregate properties, mix design, and compaction processes (Brown, 2004; Masad *et al.* 2002). Cheng *et al.* (2014) classified air voids in asphalt mixtures into three categories: effective, semi-effective, and impermeable. Different methods were developed to determine the void structure of the porous specimens. Some of the common techniques are based on two-dimensional images of the cross-sections of the material acquired by Scanning Electron Microscopy (SEM), spectroscopic imaging techniques, and Atomic Force Microscopy (AFM) (Kosek *et al*., 2005).

2.5 Aggregate and binder interface

Asphalt concrete consists of aggregates bound together by mastics. The force that binds asphalt concrete together can be categorized into two types. The bond at the binder-aggregate interface, known as adhesive bond, is considered more susceptible to moisture damage than the bond within the binder, known as cohesive bond (Caro *et al*., 2008). Hefer *et al*. (2005)

described three types of interactive forces responsible for adhesion between the binder-aggregate interface: the electrostatic interactions (electrostatic forces between the charged particles), electrodynamic interactions (forces through Van der Waals forces), and the covalent bonds (forces through the sharing of electrons).

Hicks (1991), Kanitpong and Bahia (2003, 2005), Little and Jones (2003), and Hefer *el al*. (2005) explained the different theories to describe the adhesive bond mechanisms in asphalt concrete as follows: (1) weak boundary layers, (2) electrostatic forces, (3) chemical bonding, (4) mechanical bonding, and (5) adhesion due to surface free energy (Caro *et al*., 2008).

(1) Weak boundary layers:

Packham (2006) stated that adhesive failure in asphalt concrete may occur due to the presence of interface region of low cohesive strength between aggregate and mastic. This could be the result of the presence of dust on the surface of aggregates mixed with binders, or the dissolution of complexes at the aggregate surface after interaction with water.

(2) Electrostatic forces:

The bonding between aggregate and mastic or between mastic and mastic, due to columbic force of charges is electrostatic in nature. The zeta potential, which is the measure of electric potential at the shear plane between the solid and liquid media, has been used to analyze debonding of asphalt binder in the presence of moisture (Labib 1992). The zeta potential of aggregate and binder may be of the same polarity, depending

on the pH value, which leads to repulsive forces between them; resulting in net repulsion between the two materials (Labib 1992).

(3) Chemical bonding:

Unlike adhesive bond due to surface free energy or electrostatic interactions, which are also due to the chemical nature of these materials, chemical bonding is caused by the formation of a new material because of the chemical reaction between active functional groups from the binder and the aggregate (Caro et al., 2008). Aggregates with high carbonate content are easier to coat with binder than aggregates with silica. This is due to the presence of the hydroxyl groups in siliceous aggregates, which have greater affinity for water and carboxylic acid present in binder (Caro *et al*., 2008). However, the chemical bonds with carboxylic acid are more prone to displacement in the presence of water (Petersen et al., 1982).

Bagampadde *et al.* (2005) tested both moisture conditioned and unconditioned samples with eleven aggregates from different sources with one binder. He used AASHTO T 283 test method for moisture conditioning with the ratio of damaged to undamaged values of Dynamic Modulus and tensile strength. The study showed that mixtures with aggregates containing alkali metal elements or aggregates with high contents of quartz and alkali feldspars were more prone to moisture damage. On the contrary, mixtures with aggregates containing calcium, magnesium, and iron were found to be more moisture resistant. Bagampadde *et al.* (2006) later used similar methodology, but with four types of binders and different aggregates. The study showed that the

characteristics of asphalt binders such as acid number, penetration grade, and molecular size distribution are not statistically significant factors in determining the resistance of mixtures to moisture damage. Moreover, the results showed that it was the aggregate property, rather than the binder property, that influenced the moisture damage in the mixtures.

(4) Mechanical bonding:

The binder in asphalt mixtures is forced into the irregularities of the aggregate surface that produces the mechanical interlocks (Caro *et al.,* 2008). Aggregates with rough texture and a high amount of surface pores are more resistant to moisture damage (D'Angelo and Anderson 2003). Moreover, aggregates with high angularity are considered beneficial in producing better mechanical interlock (Sturart 1990).

(5) Adhesion due to free surface energy:

Moisture damage in asphalt mixtures is a thermodynamically favorable process. The free surface energy in an interface of a material is the amount of external work done on a material to create a new unit surface area in vacuum. The work of adhesion between two materials is the amount of work or energy required to separate the materials from their interface to create a new unit of area of each material in vacuum (Caro et al., 2008). Higher value of work of adhesion results in higher resistance of the interface to an adhesive failure. Moreover, higher reduction of free surface energy in the presence of water results is adhesive failure in the interface (Caro *et al*., 2008).

2.6 Characterization of Moisture damage

The quantification of problems associated with the effects of water in asphalt mixtures has been recognized since late 1960 (Caro *et al*., 2008). The first attempt used to evaluate the action of water in asphalt pavement was through visual inspection. Later, empirical tests to assess the effect of moisture in loose to compacted samples using a quantifiable performance parameter were developed. Kiggundu and Roberts (1988), Airey and Choi (2002), and Solaimanian *et al*. (2003) described that the test methods to characterize moisture damage in asphalt pavements can be classified based on the following factors:

- State of mixture (loose versus compacted mixtures)
- Mode of loading (static versus dynamic)
- Methodology used to induce moisture damage (i.e. moisture conditioning process), and
- Scale of performance measurement (macro-versus micro scale)

Caro *et al.,* (2008) proposed a new classification of test methods used to assess moisture damage as (1) Subjective qualification, (2) Quantification using a performance index during or after damage test, and (3) Ratio of parameter derived from tests on dry and moisture conditioned specimens.

1. Subjective qualification:

Tests on this group include the ones based on visual inspection or qualitative assessment of moisture damage, as opposed to a quantitative assessment. Table 1 includes the tests common to this group.

Table 1 List of laboratory tests for moisture evaluation with subjective quantification

Test	Reference	Assessment of damage
Rolling bottle method	CEN prEN 12697	Visual estimation of amount of binder retained in the mixture
Ancona stripping test	Bocci and Colagrande (1993)	Visual estimation of the total percentage of material stripped (Airey and Choi 2002)
Boiling water test	American Society of Testing and Materials, ASTM D 3625	Visual estimation of the amount of binder loss. Retained coating between 85 and 90% is considered acceptable
Film stripping	California test 302-1999	Visual estimation of the total percentage of material stripped
Static immersion test	American association of State Highway and Transportation Officials AASHTO T182 ASTM D 1664	Visual estimation of total area of the coated aggregate as above or below 95% (discontinued by ASTM)
Ultrasonic method	Vuorinen and Valtonen (1999)	Visual estimation of material stripped. It can also include the assessment of the weight of material released from the aggregate (a quantitative assessment of damage derived from this test is included in Table 2)
Chemical immersion test	Kennedy et al. (1983)	Concentration of sodium carbonate required to produce stripping (identified visually). This value is considered a measure of adhesiveness.

2. Quantification using a performance index:

Tests under this group include the use of performance parameters that are unique to each test and are used as the final value of a performance parameter. Some of the common tests under this category are summarized in Table 2 (Caro *et al*., 2008).

Table 2 List of laboratory tests for moisture evaluation with performance index

Test	Reference	Performance index (damage
First proposal of ultrasonic method	Vuorinen and Valtonen (1999)	Loss of weight of a polished stone test specimen of 2cm by 8cm coated with 2g of asphalt binder after the application of ultrasound under water (Airey and Choi 2002)
Ultrasonic accelerated moisture conditioning procedure (UAMC)	McCann and Sebaaly (2001)	Percentage loss in weight of original sample after subjecting it to an ultrasonic bath. The lost material corresponds to small particles of aggregates when ultrasonic energy is applied.
	McCann *et al*. (2005)	Recently the authors proposed the use of the rate at which the small particles are released as the asphalt recedes along the surface of the aggregate as a performance index that characterizes damage.
Net adsorption test (NAT)	SHRP Designation M-001, Curtis et al. (1993)	Net adsorption is defined as the amount of asphalt binder that remains on the aggregate after aqueous desorption. The value is measured based on the amount of binder that adsorbed onto the aggregate surface from a toluene solution and the amount of binder that is desorbed (i.e., removed) by the addition of water to the mixture (Airey and Choi 2002)
Modified net adsorption test	Walsh *et al*. (1996)	Same principle as NAT but with modifications in the preparation of the aggregates (Aiery and Choi 2002)
Pneumatic pull-off	Youtcheff and Aurilio (1997)	Adhesive bond of asphalt binders (based on the burst pressure required to debond the binder form a stone) as a function of time while exposed to water (tensile and bonding strength in different moisture conditions) (Solaimanian et al. 2003)
Texas freeze-thaw pedestal method	Kennedy and Anagnos (1984)	Thermal cycles required for causing crack initiation in an asphalt mixture.
Boiling water stripping test	Belgium Road Research Center (BRRC) Procedure ME 65-91	Stripping rate calculated from a calibration curve of acid consumption against percentage of uncoated material. The curve is obtained from mixtures with coated and uncoated aggregates in different ratios that are subjected to chemical attacks.

Hamburg wheel-tracking	Texas Department of Transportation TxDOT standard Tex-242-F, 2004	The test uses wheel tracking to simulate traffic conditions on compacted specimens that are immersed in water. Stripping is identified with the presence of a sudden inflection point in the plastic deformation during the application of cyclic load
Moisture vapor susceptibility (MVS)	California test 307, 1999	Stabilometer value at which the effect of moisture on mixtures is imminent. The stabilometer value is calculated following the test CT 366.
Methylene blue test	Ohio Department of Transportation (supplement 1052)	The test identifies the concentration of plastic fines (including clay and organic material) in the mix that can promote emulsification of asphalt in the presence of water (Kandhal *et al*., 1998)

3. Ratio of parameters derived from tests on dry and moisture conditioned specimens:

This approach uses the ratio of a parameter (or parameters) derived by testing dry or control specimens to the same parameter(s) derived by testing moisture conditioned specimens. Moisture conditioning of a specimen means the simulation of detrimental effects of moisture on the material during a short period of time (Caro *et al*., 2008). This ratio of parameters affected by moisture conditioning to the control state is referred to moisture damage ratio (MDR). Besides being a quantitative measure of moisture damage, MDR can also be used to: (1) evaluate the influence of different factors on moisture damage (e.g. effect of air voids or addition of antistripping agents) and, (2) select mixtures based on an acceptable level of damage (Caro *et al*., 2008). MDR can be derived simply as the ratio of a single material property, or single-parameter MDR and MDR derived as the ratio of a combination of multiple properties, or multiple-parameter MDR.

1. Single-parameter MDR

Typical examples of parameters used to determine MDR are stiffness, strength, resilient modulus, and number of cycles to failure. Although single parameter MDR captures the deterioration of an asphalt mixture property due to moisture conditioning on a macroscale, it does not provide explanations for the causes for such deterioration (Caro *et al*., 2007). Some of the common types of tests under single-parameter MDR are described in Table 3. The procedures described by Bhasin *et al*. (2006, 2007), Masad *et al*. (2006), and Little and Bhasin (2007) particularly employed thermodynamic material properties instead of mechanical properties to explain the loss of adhesion between aggregates and binder in the presence of water.

Table 3 List of laboratory tests for moisture evaluation with single parameter MDR

Test	Standard/reference	Material property used in the single-index performance function for MDR	Comments
Lottman	NCHRP 246, Lottman (1982)	Indirect Tensile strength and stiffness	
Modified Lottman	AASHTO T283	Indirect Tensile strength and stiffness	Commonly known as Tensile Strength Ratio (TSR)
Tunnicliff and root procedure	ASTM D 4867	Indirect Tensile strength	Commonly known as Tensile Strength Ratio
Duriez test	NFP 98-251-1	Unconfined compression strength	
Marshall stability test	AASHTO T245	Marshall stability	
Immersion compression test	AASHTO T165, ASTM D 1075	Compressive strength (wet over dry values)	The MDR in this case is referred to as index of retained strength (IRS)
Bitutest protocol	Scholz (1995)	Indirect stiffness modulus using the Nottingham asphalt tester	

Environmental conditioning system (ECS) with Dynamic Modulus	Al-Swalmi and Terrel (1992); NCHRP 9-19, 9-29 and 1-37	Dynamic Modulus using ECS (Solaimanian *et al.*, 2003, 2006)	
ECS with flow number		Flow number using ECS (Solaimanian et al. 2003, 2006)	
ECS with flow time		Flow time using ECS (Solaimanian *et al*. 2003, 2006)	
Flextural fatigue beams test with moisture conditioning	Shatnawi *et al*. (1995)	Fatigue life following AASHTO TP-8	
Disk-shape compact tension fracture test (DCT)	Apeagyei *et al*. (2006)	Fracture energy (Gf), Fracture strength (Sf)	
DMA on fine aggregate mixtures for analyzing moisture damage of mixtures	Song *et al*. (2005), Masad *et al*. (2006a)	Fatigue life, Nf (number of cycles to failure under strain control conditions)	Conducted on FAM using torsion strain controlled. There is not a threshold value that separates good and poor performance of mixtures
		Reduction in the Dynamic Modulus at fatigue life (G*/G')	Small values are desirable.
Static creep test	Al-Shweily (2007)	Accumulated microstrain	Comparision of accumulated microstrain after 60min of loading
Pull-off test	Copeland *et al*. (2007)	Tensile strength of the binder-rock interface	
Saturation aeging tensile stiffness (SATS)	Collop *et al*. (2004) and Airey et al. (2007)	Tensile stiffness modulus	Calculated at 20C using the Nottingham Asphalt Tester (NAT)
Surface energy measurements (Wilhelmey plate and universal sorption machine)	Bhasin *et al*. (2006a, 2006b, 2007)	Adhesive bond between aggregates and asphalt binders (ΔG_{AS}) and the potential of water to debond the asphalt (ΔG_{WAS})	High values of ΔG_{AS} and low values of ΔG_{WAS} are desirable

Although the modified Lottman test (AASHTO T283) is the most popular test in this category, there is little consensus regarding the most reliable procedure to evaluate moisture

damage. Kiggundu and Roberts (1988), Birgisson *et al*., Lu and Harvey (2006), and Solaimanian *et al*. (2006), among others, have pointed out the deficiencies of AASHTO T283 and other tests in terms of their ability to accurately predict the field performance of asphalt mixtures. Kiggundu and Roberts (1988) found out that AASHTO T283 had 67% success rate of performance prediction in the field. However, it is extremely difficult to find a single parameter that can be used to successfully characterize and predict moisture damage (Caro *et al*., 2008).

2. Multiple-parameter MDR

The multiple-parameter MDR utilizes an analytically based function to quantify damage by combining more than one material property and considering dry and wet states (Caro *et al*., 2008). The main reason for needing to use multiple-parameter MDR to characterize moisture damage is that a single material property cannot simultaneously account for the physical, mechanical, and chemical changes that occur in materials during the development of moisture damage.

4. Other methods of characterization of moisture damage

Kringo *et al.* (2008) worked on the development of energy based computational framework, which incorporates moisture-induced damage by integrating physical and mechanical moisture-induced damage processes within the elasto-visco-plastic constitutive model. For the simulation of the models, governing equations are derived with parameters for several modes of moisture infiltration. Moisture diffusion into the mastic film, towards the

aggregate–mastic interface, and mastic erosion, due to high water pressures caused by the pumping action of traffic loading, are identified as the main moisture-induced damage processes and are implemented in a new finite element program named RoAM (Kringo *et al*., 2008).

Arambula *et al.* (2007) used crack growth method based on Paris's law for viscoelastic materials to evaluate moisture damage in asphalt concrete. The derivation of this method is based on principles of fracture mechanics, and it accounts for several mechanical and chemical material properties. Recently, a virtual two-dimensional model of microstructure of hot mix asphalt has been studied with ANSYS-FLUENT software to investigate moisture flow pattern in a typical air void structure in asphalt concrete (Chen and Williams, 2014). Similar research was conducted on the 3-D Finite Element Modeling of microstructural representation of asphalt concrete constructed from X-ray computed tomography images (Shakiba *et al*., 2015; Ghauch *et al*., 2015).

2.7 Measures to control Moisture damage

Moisture damage in asphalt concrete is mainly controlled in the production phase by the selection of proper mix with addition of additives. Proper selection of binder and aggregates, treatment of aggregates, and use of surface seals are some of the very general methods to minimize the moisture damage in pavements. However, use of chemical additives and lime are among the most common methods to minimize moisture damage (Emery and Seddik, 1997; Epps *et al*. 2003; Hicks, 1991). The use of chemical additives, or in other words liquid anti-strip agents, is to modify the chemical composition and electric charge of the asphalt binder, and to reduce the surface tension between the asphalt binder and the aggregate in order to obtain strong

and uniform aggregate coating. The use of lime, on the other hand, decreases the interfacial tension between the asphalt and the aggregate. The calcium ions in lime react with the carboxylic acids in asphalt and replace some cations on the aggregate and the nitrogen in the asphalt binder (Hicks, 1991). Lime also helps in the reduction in viscosity of asphalt binder with aging (Emery and Seddik, 1997).

According to a survey conducted by the Colorado Department of Transportation in 2002, along with fifty other departments of transportation, three FHWA Federal Land Offices, the District of Columbia, and one Canadian province, it was determined that 82% of the agencies required some sort of antistrip treatment, of which 56% treat with liquid anti-stripping agent, 15% with liquid or lime, and 29% with lime. Among different test methods used to characterize the moisture damage, 87% of the agencies tested for moisture sensitivity among which,

- 82% use a tensile test (AASHTO T283, ASTM D4867 or similar),
- 10% use a compressive test (AASHTO T115 or similar),
- 4% use a retained stability test, and
- 4% use wheel-tracking tests and tensile tests.

2.8 AASHTO T 283

Among several laboratory tests to access the moisture susceptibility of HMA, AASHTO T 283 is the most widely used method (Liang, 2008; Tarefder and Ahmad, 2014; Diab and You, 2013; Shrum, 2010). The test consists of preparation of 6 HMA samples using Marshall Impact compaction method. The air voids of the samples are between six and eight percent. The higher

air voids requirements are to help accelerate moisture damage to the specimens. The samples are divided into two groups: the first group is the control group or unconditioned group and the second one is conditioned group. The sample for conditioned group is vacuum-saturated at 50-80% (AASHTO T283-99) or 70-80% (AASHTO T283-03) with water and then placed in a freezer at 0^0F for 16 to 18 hours. The conditioned specimens are then placed in a water bath at 140^0F for 24 hours. Both the conditioned and unconditioned samples are tested in Indirect Tensile Test to determine the Tensile Strength Ratio (TSR) as the ratio of conditioned tensile strength to unconditioned tensile strength. Also visual estimation of the magnitude of stripping is done for the conditioned samples.

Buchanan *et al* (2004) discussed some of the limitations of AASHTO T283 as follows:

- The test is time consuming.
- Wide range of saturation level (50% - 80%) may result in substantial TSR variability.
- The conditioning procedure in T 283 does not simulate repeated generation of pore pressure under loads, which is believed to be a major cause of stripping in HMA pavements.

2.9 Other Test Methods

Cross *et al*. (2000) used the loaded wheel tester of Asphalt Pavement Analyzer (APA) to detect moisture susceptible mixtures. Samples were tested using four different preconditioning procedures: dry, soaked, saturated, and saturated with freeze cycle. Pan *et al*. (1998 and 1999) used Purdue Wheel tracking device (PURWheel) to predict the conditions that promote water

stripping of HMA specimens. Pan *et al.* (1998 and 1999) also pointed out the results from AASHTO T283 indicating that there is significant effect on stripping potential of the seven mixes tested. McCann *et al*. (2001) used ultrasonic moisture accelerated conditioning process to characterize the moisture sensitivity of HMA pavement. The results suggested that the potential of stripping by ultrasonic moisture accelerated conditioning is analogous to results established by tensile strength testing after 18 cycles of freeze-thaw conditioning. In Mississippi, Buchanan *et al.* (2005) used Moisture Induced Stress tester to evaluate the moisture susceptibility of asphalt samples. The damage was characterized by measuring the turbidity of the water after MIST conditioning. Birgisson *et al*. (2003) evaluated the use of a new performance-based fracture criterion. The study showed that moisture damage has an impact on the fracture resistance of the mixture. Khosla *et al*. (2000) used Superpave shear tester to evaluate the moisture sensitivity of the mixture instead of measuring Indirect Tensile Strength Ratio.

2.10 Past Moisture Conditioning in Moisture Induced Stress Tester

Cheng *et al.* (2007), conducted series of tests on dense graded Standard Marshall designed HMA mixtures to investigate whether SPT Dynamic Modulus Test or Superpave IDT combining with AASHTO T283 (ASTM D4867) or MIST conditioning were suitable to characterize the lab-measured moisture susceptibility. The tests were conducted with mixes having three different angularity levels (i.e. 0%, 50% and 100%) both with and without amine based antistrip additive. The specimens were conditioned for moisture damage with one and two cycles of Freeze and Thaw and 500 and 1000 cycles of MIST at 40psi and 40^{O}C and the performance tests were conducted to investigate the differences in performance parameters. The results showed drop in Dynamic Modulus, resilient modulus and Indirect Tensile strength but

increase in creep compliance after each of these moisture conditioning. Moreover, Superpave IDT tests and SPT Dynamic Modulus Test were found effective to characterize lab-measured moisture susceptibility for HMA mixes. The amine-based antistrip additive seemed to be effective to decrease moisture susceptibility; however, there was not much effect of aggregate angularity on moisture resistance.

Pinkham *et al*. (2013) tested eight field cores from projects under Maine Department of Transportation for changes in performance parameters after conditioning the samples under MIST at 30psi, 60^{O}C and 2000 cycles. Lower values of pressure and number of cycles were adopted as the field cores were obtained from older pavements. The changes in values of resilient modulus, bulk specific gravity and Indirect Tensile strength was observed after the moisture conditionings. The study concluded that MIST showed moisture damage in samples and suggested that further investigation should be conducted for its applicability.

Schram *et al*. (2012) in Iowa used plant produced surface mixes to perform Dynamic Modulus, flow number, AASHTO T283, Hamburg Wheel Tracking Device (HWTD) and moisture induced sensitivity test (MIST) to rank the mixes by their performance in the field. The study indicated that swelling from MIST conditioning and submersed flow number matched the performance ranking. Moreover, Hamburg testing parameters like stripping inflection point and ratio between striping slope, and creep slope was also effective to rank field performance. Dynamic Modulus with AASHTO T283, on the other hand, was found to be rather ineffective to rank the field performance.

Shrum (2010) performed tests on Standard Marshall Mix design warm mix asphalt and hot mix asphalt with various Reclaimed Asphalt Pavement (RAP) contents primarily to evaluate the moisture damage in Warm Mix Asphalt (WMA) mixes. Three different moisture damage tests were conducted; Superpave IDT with freeze thaw (F-T) and Moisture Induced Stress Test (MIST) conditioning, Hamburg Wheel Tracking Test, and Simple Performance Tests' (SPT) Dynamic Modulus with MIST. The results showed that both Hamburg Wheel Track tests and Dynamic Modulus ratio corresponds to the results with Tensile Strength Ratio. Moreover, it showed that MIST conditioning for 1000 cycles was adequate for Dynamic Modulus specimens indicating Dynamic Modulus is more sensitive to moisture damage than Tensile Strength Ratio.

Twagira and Jenkins (2009) at University of Stellenbosch at South America used MIST conditioning on bituminous stabilized material with maximum aggregate size of up to 19 mm with 2% bitumen content. The results of accelerated moisture conditioning in MIST was validated with a known laboratory model called model mobile load simulator.

2.11 Conclusion

Following is the summary of the points discussed in this chapter:

- Moisture intrusion in asphalt concrete leads to loss of adhesive bonding between aggregate and binder and results in separation of these materials.
- Moisture is transported to the asphalt mixture through following different modes: (1) infiltration (2) capillary rise and, (3) permeation or diffusion.

- The mechanisms of moisture damage observed in asphalt concrete are desorption, detachment, dispersion, displacement. and spontaneous emulsification.
- The factors that influence the moisture damage are mainly the type of aggregate, type of asphalt, mixture design and construction, various environmental factors, traffic and antistripping additive properties.
- Void structure is an important factor in characterizing the moisture damage in asphalt concrete. The size and distribution of air void is more accurate factor to characterize the moisture damage rather than just the amount of air voids.
- The theories to describe the adhesive bond mechanisms between asphalt binder and aggregate are as follows: (1) weak boundary layer, (2) electrostatic forces, (3) chemical bonding, (4) mechanical bonding, and (5) adhesion due to surface free energy.
- The test methods to quantify the moisture damage in asphalt concrete can be classified as (1) Subject quantification or qualitative quantification, (2) Quantification using a performance index during or after the damage test, and (3) Ratio of parameter.
- Various efforts have been made to quantify the moisture damage in asphalt samples using the moisture conditioning in Moisture Induced Sensitivity Tester (MIST). The results have shown that moisture conditioning in MIST can be correlated with the actual moisture conditioning in MIST.

It can be concluded form this chapter that moisture conditioning of asphalt concrete and characterization of moisture damage in the laboratory is a difficult task. Hence understanding the mechanism of moisture damage in detail is very important. Even when various tests have been devised to quantify moisture damage, the main problem occurs in the repeatability of the results.

Numerical approach to quantify moisture damage is gaining popularity but its limitation lies in poor correlation with the field results besides its modeling complications. Hence better and easier approaches of quantifying moisture damage in the laboratory are needed.

CHAPTER 3: RESEARCH METHODOLOGY

The objective of this research was to analyze the effect of moisture on one of the common mix types used in Texas. The method chosen for the purpose of moisture conditioning was moisture induced stress test. This chapter discusses about the information on selection of the material, mix design and the test methods used for the purpose of the research.

3.1 Material Selection

The type of aggregate used for the purpose of mix design was crushed limestone. A single mineralogical type was used for the purpose of this research with a view of narrowing the research outputs and to make the results usable for future research efforts. Crushed limestone aggregate used was brought from Ned Finney Quarry site at El Paso County, managed by Jobe Materials LP. The quarry was selected for the aggregate as it was one of the major sources of aggregates used in asphalt mixes in El Paso County and the surrounding areas in Texas. The aggregate was chosen from two stock piles - one of 3/8” size and another screenings. Aggregate was sieved in the laboratory to desired sieve sizes for the chosen mix. Similarly, the binder used for the research was PG 70-22 as one of the common binder types used in Texas and also due to easier availability for the research purpose. The binder was obtained from Western Refining which is a major supplier of fuel and asphalt binders especially in all the southwestern and western United States.

3.2 Mix Design

Among various dense-graded asphalt mixes used in the state of Texas, Type-C and Type-D are the most common ones (Walubita et al., 2013). The asphalt mix chosen for the research was TxDOT Performance Graded (Item 344) SP-Type D. The procedure for the obtaining optimum asphalt content for the mix design was from Part IV – Mix Design For Performance-Designed Mixtures Using the Superpave Gyratory Compactor of TxDOT test procedure - Design of Bituminous Mixtures (Tex-204-f). The design number of gyrations used for the mix design was assumed to account for a traffic load repetitions of about 0.3 million ESAL (Roberts et al., 1996). The initial, design and final number of gyrations for mix design process during the compaction was chosen as 7, 78 and 121 with the assumption of average design high air temperature of 41-42^{0}C (Roberts et al., 1996).

The design gradation of the aggregate used for the mix is shown in Figure 2. The maximum aggregate size used for the design mix was 3/8 inches while keeping the gradation within the specification limits. For the trial mixes, four different asphalt contents were used, one at 0.5% below, two at 0.5% above, and one at optimum asphalt content as specified in Tex-204-f. The maximum dry densities of the mixes were measured using CoreLok®. Two samples of 152 mm diameter were prepared for each of the four trial mixes at the final number of gyrations of 121. The weights chosen to prepare the samples were taken in such a way that the height of the samples at the design number of gyrations is 115±5 m. The accurate weight of the sample used to produce height of 115±5 mm at design number of gyrations was achieved by molding a trial sample at the design gyration with a trial weight. Figure 1 shows two samples for the mix design prepared at 4% asphalt content. After molding the samples, the bulk density of the samples were

measured using saturated surface drying method and the air void content was calculated. The heights of samples were measured and the actual air voids both at initial and design number of gyrations was calculated based on the calculated correction factors. In addition to obtaining the air voids of 4% for the design number of gyrations, it was made sure that the air voids at initial gyrations for the samples at optimum asphalt content was above 11% and final gyrations was above 2% to ensure that the mix is not tender. Moreover, the dust to asphalt ratio was kept between 0.6 and 1.6 as specified in TxDOT specification, Item-344.

Figure 1 Samples for trial mixes

The volumetric plots against the asphalt contents are as shown in Figure 3. The asphalt content for design percentage of air voids of 4.5% was chosen as the optimum asphalt content for samples with compaction at design number of gyrations. Moreover, the plot for the Voids filled with Mineral Aggregates (VMA) versus asphalt content was plotted and the optimum asphalt content percentage was verified for the minimum VMA value. The optimum asphalt content of the mixture was found out as 4.5% which has been used throughout the research experiments. The asphalt mixer used for preparation of mix is shown in Figure 4. The mix was prepared at 300^0F and left in the oven at 275^0F for conditioning for two hours before the mix could be used for compaction in Superpave Gyratory Compactor (SGC).

The samples used for mix designs and testing were molded using Superpave Gyratory Compactor with the Txdot test procedure – Superpave Gyratory Compacting of Test Specimens of Bituminous Mixtures (Tex-241-f). Samples compacted for mix design were 115±5 mm high and 152 mm in diameter with the air void content of 4%.

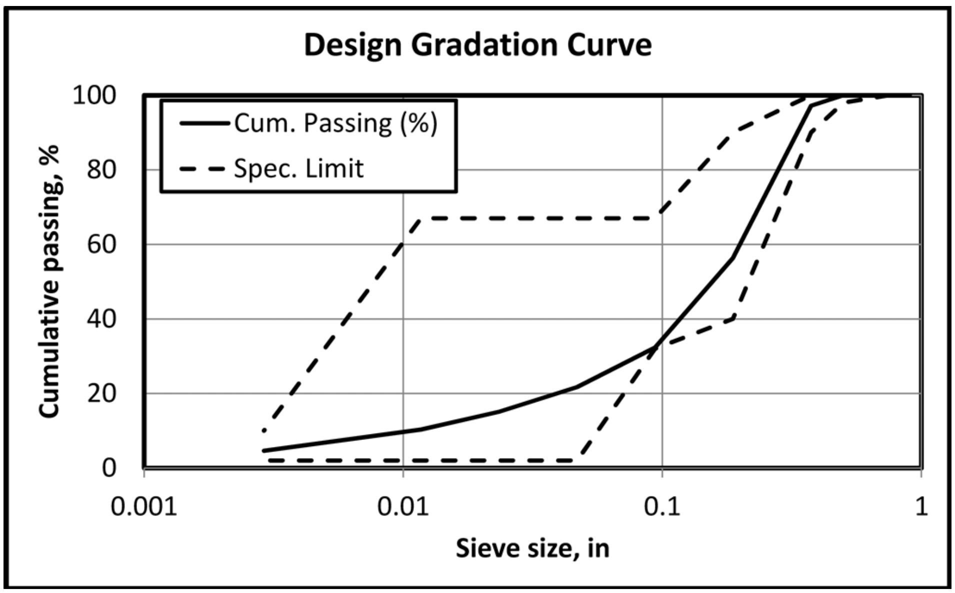


Figure 2 Aggregate Gradation Curve for the Design Mix

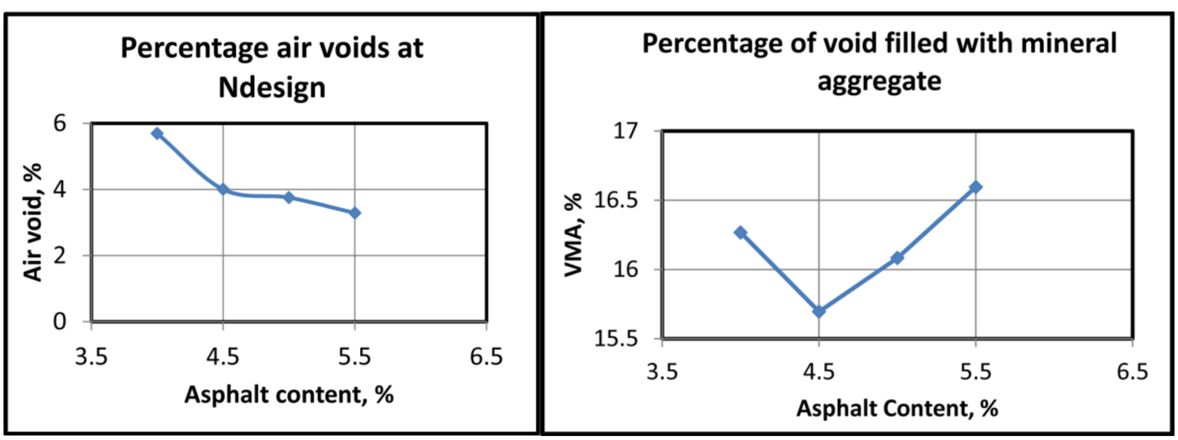


Figure 3 Mix Properties versus Asphalt Content

Figure 4 Asphalt Mixer used for the preparation of mixes in the laboratory

3.3 Specimen Preparation

Samples used for the laboratory tests were prepared in Superpave Gyratory Compactor. After preparing the mix at 300^{0}F, the loose mix was conditioned for two hours at 275^{0}F. The samples were compacted at 275^{0}F as the required temperature based on tex-241-f for PG 70-22 binder. As both the Hamburg Wheel Tracking Test and the AASHTO T283 moisture conditioning tests uses the samples with 7% air voids the samples were prepared at 7±1% air voids for conditioning in Moisture Induced Sensitivity Tester in order to be consistent with the contemporary test procedures. The average pessimum air void percentage for most of the dense graded asphalt concrete lies between 5-10%, for which the mix is more susceptible to moisture (D'Angelo and Anderson, 2003). To achieve the exact air void of 7±1% in the samples, trial

samples were prepared. The weight of the loose mix required to achieve 7% air voids were interpolated from the weights used for the trial samples. After compaction, the specimens were left overnight to cool off before further testing. The samples prepared for the Dynamic Modulus Test were six inches in height and four inches in diameter while those for the Indirect Tensile Test were two inches in height and 4 inches in diameter.

3.4 Test Methods

Two different test methods were used to measure the performance parameters for the purpose of the research; the Dynamic Modulus Test and the Indirect Tensile. The Moisture Induced Sensitivity Tester was used in order to condition the samples for moisture. The test procedures for these test methods are described further below.

3.4.1 Dynamic Modulus Test

One of the test methods used for the testing was Dynamic Modulus Test of asphalt concrete. As asphalt is a visco-elastic material, the modulus of elasticity of asphalt depends on the frequency and temperature of loading. In order to quantify the modulus of asphalt concrete, the test method used should incorporate both temperature and frequency. Instead of using a regular testing method to find the modulus of elasticity, a different test method called Dynamic Modulus Test is used. The test is performed on cylindrical asphalt samples by applying sinusoidal vertical loads and by measuring the deformation. The Dynamic Modulus, after testing, is the absolute value of modulus calculated by dividing the maximum (peak-to-peak) axial stress to the corresponding strain (AASHTO TP 61-03). The standard size of the samples used for the testing is four inches in diameter and six inches in height.

The standard method of test used to determine the Dynamic Modulus of asphalt samples was AASHTO – TP 62-03 "Determining Dynamic Modulus of Hot-Mix Asphalt Concrete Mixtures" for the purpose of this research. According to this method, the samples are tested at the temperatures of -10^{0}C, 4.4^{0}C, 21.1^{0}C, 37.8^{0}C and 54.4^{0}C and the frequencies of 25Hz, 10Hz, 5Hz, 1Hz, 0.5Hz and 0.1HZ for each of the temperatures. Loading of the specimens is carried out with a haversine load in cyclic manner so as to obtain axial strains between 50 and 150 microstrains. Usually a higher load is needed for lower temperatures and higher frequencies to reach such strain levels. The deformation is measured between the points 4 inches apart on the sides of the samples. Either two or three Linear Vertical Displacement Transducers (LVDT) is used to measure the deformation on the samples. In this research, two 20mil - LVDTs placed at 180^0 apart were used to measure the displacement on the samples. For the purpose of calculation, the last five peak stresses and peak strains are used. Dynamic Modulus is then calculated as the ratio of the average peak stress over the average peak recoverable axial strain.

As the test is conducted for five different temperatures and six different frequencies, there are thirty different values of Dynamic Modulus obtained from the test method. In order to compare the results, one of the variables is normalized. This is done by shifting the values of Dynamic Modulus for different temperatures to a single temperature using the principle of time-temperature superposition (Dougan et al., 2003). The equation used to shift the frequencies from one temperature to another as suggested by AASHTO TP 62-03 is,

$$loga(T) = \beta_0 + \beta_1(°\text{F})$$

Where, a (T) = shift factor as function of temperature,

β_0, β_1 = constants of regression line,

°F = temperature in degree Fahrenheit.

The above regression line was fitted for the case of this study with the minimum coefficient of determination (R^2) kept at 0.97. As proposed in "2002 Guide for the Design of Pavement Systems", currently under development in NCHRP Project 1-37A, the modulus of the asphalt concrete – at all analysis levels of temperature and time rate of load- is determined from a master curve constructed at a reference temperature, generally 70°F (21.1°C) (Dougan et al,2003). The master curve is mathematically modeled by using a sigmoidal function as follows:

$$Log|E^*| = \delta + \alpha/\left[1 + e^{\beta + \gamma(log t_r)}\right]$$

Where, t_r = time of loading at reference temperature

δ = minimum value of E^*

$\delta + \alpha$ = maximum value of E^*

β, γ = parameters describing the shape of the sigmoidal function

α = variable which is a function of gradation

The equation for the master curve was fitted with the minimum coefficient of determination (R^2) kept at 0.98. The Dynamic Modulus values obtained during the test were also interpreted individually without shifting the values as done for the master curve.

The machine used for the purpose of the test was Mechanical Testing and Sensing (MTS) system. It consists of the loading system inside an Environmental Chamber. The temperature of the chamber is adjustable from -10°C to 54.4°C. Typically, the duration to finish a test takes about five days including the testing time and the time to change temperatures.

Figure 5 Test set-up in the environmental chamber for testing Dynamic Modulus

3.4.2 Indirect Tensile Test

The Indirect Tensile Test is performed on cylindrical samples of asphalt concrete by loading it with compressive loading parallel to the vertical diametrical plane. Such loading develops uniform tensile stress on the vertical plane and causes the specimen to fail by splitting along the vertical plane. The test method followed for Indirect Tensile Test for the purpose of this research is Tex-226-F (Indirect Tensile Test) as used by the Texas Department of Transportation. The samples can either be six inches in diameter with 2.4 inches in height, or four inches in diameter with two inches in height. Samples are compacted to achieve the density of 93±1% of the theoretical maximum density, in other words, with the air void of 7±1%. Loading of specimens is carried out to achieve deformation at the rate of two inches per minute. The vertical load at the time of failure is noted to calculate the tensile strength of the sample. The

equations for determining the tensile strength of asphalt is derived with the assumption that asphalt concrete is homogenous, isotropic, and elastic. The equation used to determine the tensile strength of asphalt concrete is:

$$S_T = \frac{2F}{3.14x(hd)}$$

Where,

S_T = Indirect Tensile strength, psi

F = Total applied vertical load at failure, lb.

h = Height of specimen, in.

d = Diameter of specimen, in.

The samples were compacted in Superpave Gyratory Compactor with an air void of 7±1%. Samples were four inches in diameter and two inches in height. Figure 6 shows the equipment setup for testing the Indirect Tensile strength of the samples.

Figure 6 Test setup for Indirect Tensile Test

3.4.3 Moisture Induced Sensitivity Tester

Moisture Induced Sensitivity Tester (MIST) is designed to simulate the stripping mechanism of the asphalt samples in the laboratory. The samples in MIST are subjected to both static and dynamic moisture fields as opposed to AASHTO T283, which only has a static moisture field. Most importantly, the procedure simulates stripping due to the occurrence of hydraulic pressure in mixes with low permeability. The machine applies repeated hydraulic pressure in the sample in cycles placed in a triaxial chamber filled with water. The average rate of loading for the MIST is about 4 seconds per cycle. The machine can heat the sample along with water to temperatures up to 60^0C and apply hydraulic pressure in cycles from 0 psi to 40 psi. The number of cycles of loading in MIST can be varied as needed. For the purpose of this study, the samples were conditioned at 60^0C and for 500, 5000 and 10,000 cycles of loading with the maximum hydraulic pressure of 40 psi to induce maximum damage possible. After conditioning in MIST, the samples were air dried at room temperature overnight before further testing.

Figure 7 Moisture Induced Stress Tester (MIST) machine with sample in the triaxial chamber

3.5 Test Methodology

The main objective of the research is to investigate and characterize the moisture damage in asphalt samples using an accelerated moisture conditioning technique in laboratory. Different test methodologies were designed with the above mentioned test methods to fulfill the research objective. Figure 8 shows the summary of the flow chart of the test plan used for laboratory testing. The test methodologies are discussed further.

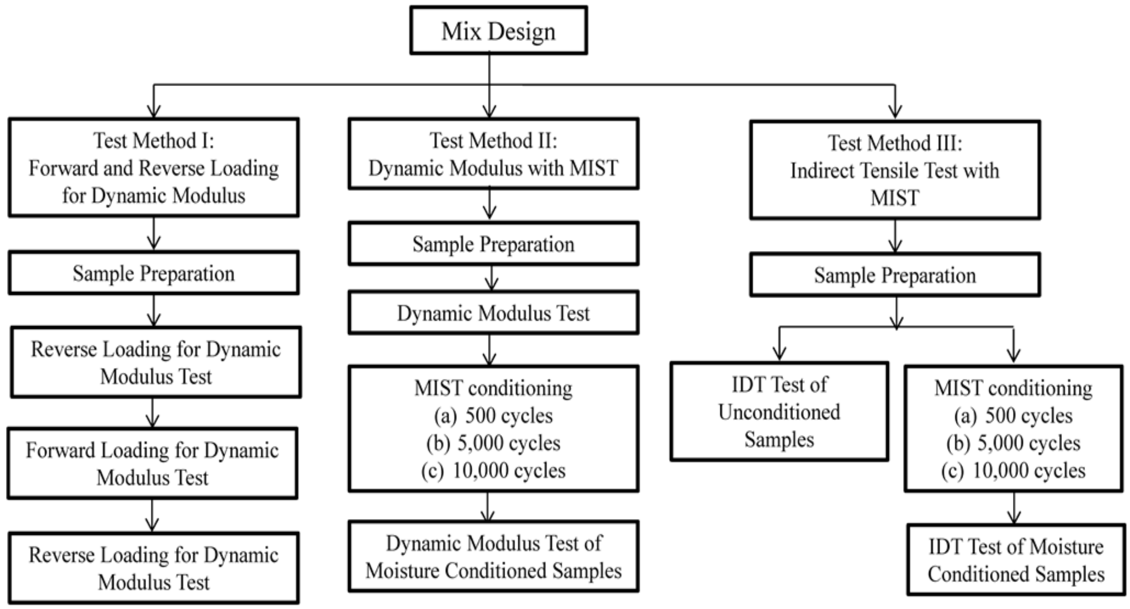


Figure 8 Flow chart of laboratory test plan

Eleven samples were prepared for the Dynamic Modulus Test with a total of twenty four Dynamic Modulus Tests. Similarly, twenty four Indirect Tensile Test samples were prepared with twenty four number of IDT tests in total. Figure 6 shows the Test Matrix with the number of batches of mix prepared and the number of samples used for each set of test conducted.

Table 4 Test Matrix for preparation of samples and conduction of all laboratory tests

Sample	Number of Samples			
	Unconditioned	**500 cycles of MIST**	**5000 cycles of MIST**	**10,000 cycles of MIST**
Batch 1 - Dynamic Modulus	3	3	-	-
Batch 2 – Dynamic Modulus	2 samples repeated 3 times	-	-	-
Batch 3 - Dynamic Modulus	3	-	3	-
Batch 4 - Dynamic Modulus	3	-	-	3
Batch 5 – Indirect Tensile Test	4	-	4	-
Batch 6 – Indirect Tensile Test	4	-	-	4
Batch 7 – Indirect Tensile Test	4	4	-	-

3.5.1 Forward and Reverse Loading in Dynamic Modulus

The Dynamic Modulus Test following AASHTO TP 62-03 covers testing the asphalt concrete sample of four inches in diameter and six inches in height at -10^{o}C, 4.4^{o}C, 21.1^{o}C, 37.8^{o}C and 54.4^{o}C. The samples are tested at the frequencies of 25Hz, 10Hz, 5Hz, 1Hz, 0.5Hz and 0.1Hz. The axial strain level during the test is always kept between 50 to 150 micro strains varying the axial loads which may be as high as 2500lbs to as low as 4lbs in order to simulate the same range of strain levels in the field.

Such extensive loading of the samples during the testing may result in the change in the void structure of the sample and hence change in the value of Dynamic Modulus. Because the main test plan includes repeating the Dynamic Modulus Test more than one time on the same sample, it is important to ensure that there is not any significant change in the value of Dynamic Modulus even after the samples are tested repeatedly.

In order to verify that there is no significant change in the Dynamic Modulus value even after repetition of the test on the same sample, a test plan is devised where the samples are tested repeatedly for the Dynamic Modulus and results are compared for any significant differences in the values of Dynamic Modulus after each repetition. The test method repeated the Dynamic Modulus Test three times using the same sample. The first phase consisted of testing the samples starting from 54^0C and moving to -10^0C, referred to as Reverse Loading in this thesis. In the second phase of the test, the sample was tested starting from -10^0C and moving to 54.4^0C, referred to as Forward Loading. The third phase of testing consisted of testing the same sample starting again at 54.4^0C and moving back to -10^0C, referred to as Reverse Loading II. A higher temperature was chosen in the beginning of the test, keeping in view that there would be more damage in the sample at a higher temperature. This is important because the second test will represent the damage as already occurred at a higher temperature in the first phase of the test.

3.5.2 Dynamic Modulus Test with MIST

In order to see the change in the Dynamic Modulus values before and after the moisture conditioning in MIST, the Dynamic Modulus Test was conducted on the same sample both before and after the MIST conditioning. The test procedure included preparing the mix and conditioning in the oven for two hours. Three samples were prepared from each batch of mix using Superpave Gyratory Compactor. The air void percentage of the sample was measured using the Saturated Surface Dry method. Samples were then left to dry overnight at room temperature before the test. All three samples from the batch were tested for Dynamic Modulus according to AASHTO – TP 62-03. After the first set of Dynamic Modulus Test, the samples

were conditioned in MIST. The samples were then taken out of the MIST and were left for surface drying at room temperature overnight. Finally the samples are tested again for Dynamic Modulus after the conditioning. Three batches of mix were prepared with three samples from each batch. The number of cycles of MIST conditioning was chosen as 500, 5000 and 1000 cycles for each batch of the samples. In other words, eighteen Dynamic Modulus Tests were conducted from nine samples tested two times each one before and one after the MIST conditioning. The results are interpreted further in Chapter 4 of this thesis.

3.5.3 Indirect Tensile Test with MIST

The test procedure followed for Indirect Tensile Test (IDT) with MIST conditioning was similar to that for Dynamic Modulus. The samples were four inches in diameter and two inches in height with an air void of 7±1%. Eight such samples were prepared in a single batch using Superpave Gyratory Compactor. The bulk specific gravites of the samples were determined using the Saturated Surface Dry method and the air voids were calculated. Samples from each batch were divided into two groups with four samples on each group. All four samples from one group were conditioned in MIST for 500 cycles. The conditioned samples were left overnight to dry at room temperature. Finally all eight samples from the batch were tested for Indirect Tensile Strength. Similar procedure was followed for two additional batches of samples with 5000 cycles and 1000 cycles of MIST conditioning. The results of the tests are discussed in Chapter 4 of this thesis.

CHAPTER 4: RESULT AND DISCUSSION

As described in Chapter 3, the overall test plan is mainly divided into three methodologies; forward and reverse loading, Dynamic Modulus with MIST, and Indirect Tensile Test with MIST. This chapter describes all the results obtained from the tests procedures as described in Chapter 3. The results are further analyzed and interpreted in order to substantiate the research objectives. The results in this chapter are mainly explained in four major topics. First the test results from Dynamic Modulus Tests with and without the MIST conditioning is discussed. The result of change in Dynamic Modulus due to moisture damage on performance of a typical asphalt pavement is discussed later. The third topic discusses statistical analysis performed on the Dynamic Modulus Test results in order to validate the test results. Finally, the results of Indirect Tensile Test with and without moisture conditioning are discussed and interpreted.

4.1 Dynamic Modulus with MIST

The results from the Dynamic Modulus Test conducted before and after the moisture conditioning showed that there is an overall decrease in the values of Modulus after moisture is introduced in the asphalt sample. Figure 9 shows the average Dynamic Modulus values from the samples tested before and after the MIST conditioning.

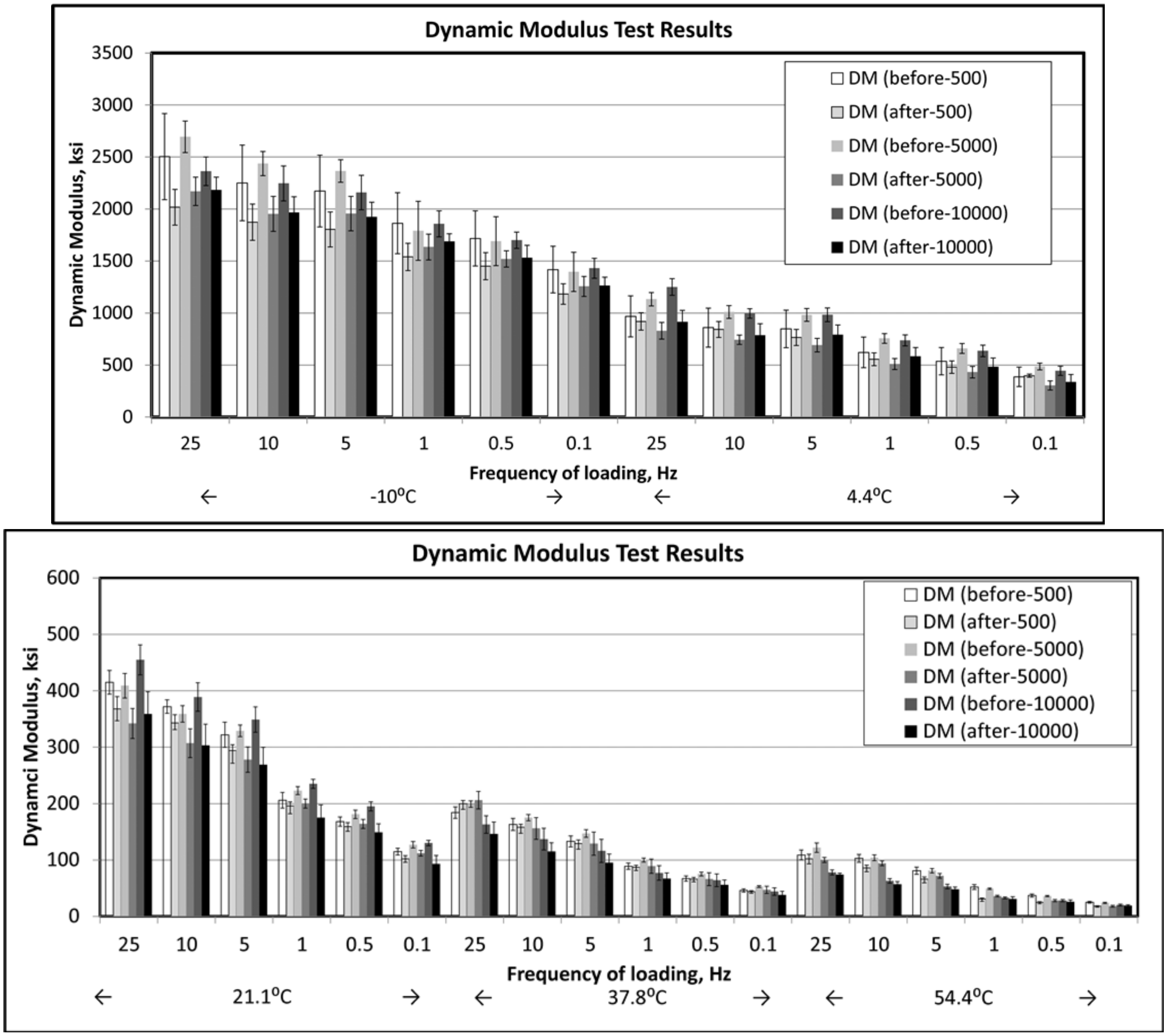


Figure 9 Test Result of Average Dynamic Modulus Test of the samples

As a single Dynamic Modulus Test following AASHTO TP 62-03 involves thirty different test combinations with five different conditions for the temperature and six different conditions for the frequency, comparing the results before and after the moisture conditioning involved comparing all the thirty values of modulus separately. Analyzing the results in such a

manner makes the comparison incomprehensible as it involves comparing thirty separate values discretely. In order to make the analysis more comprehensible, the results were compared using the master curve plotted for each set of tests both before and after the moisture conditioning. Figure 10 shows the master curve plotted for different cases of loading both before and after moisture conditioning. There was a decrease in the value of Dynamic Modulus after moisture conditioning as seen in each of the curves. There was a higher drop in the modulus value for 5000 cycles than 500 cycles however it was hard to conclude whether the drop for 10,000 cycles was higher than that for 5000 cycles. The decrease in modulus value after 10,000 cycles of MIST seemed to be more for very low and very high frequencies but the result did not have any significant application in that range of frequencies of loading.

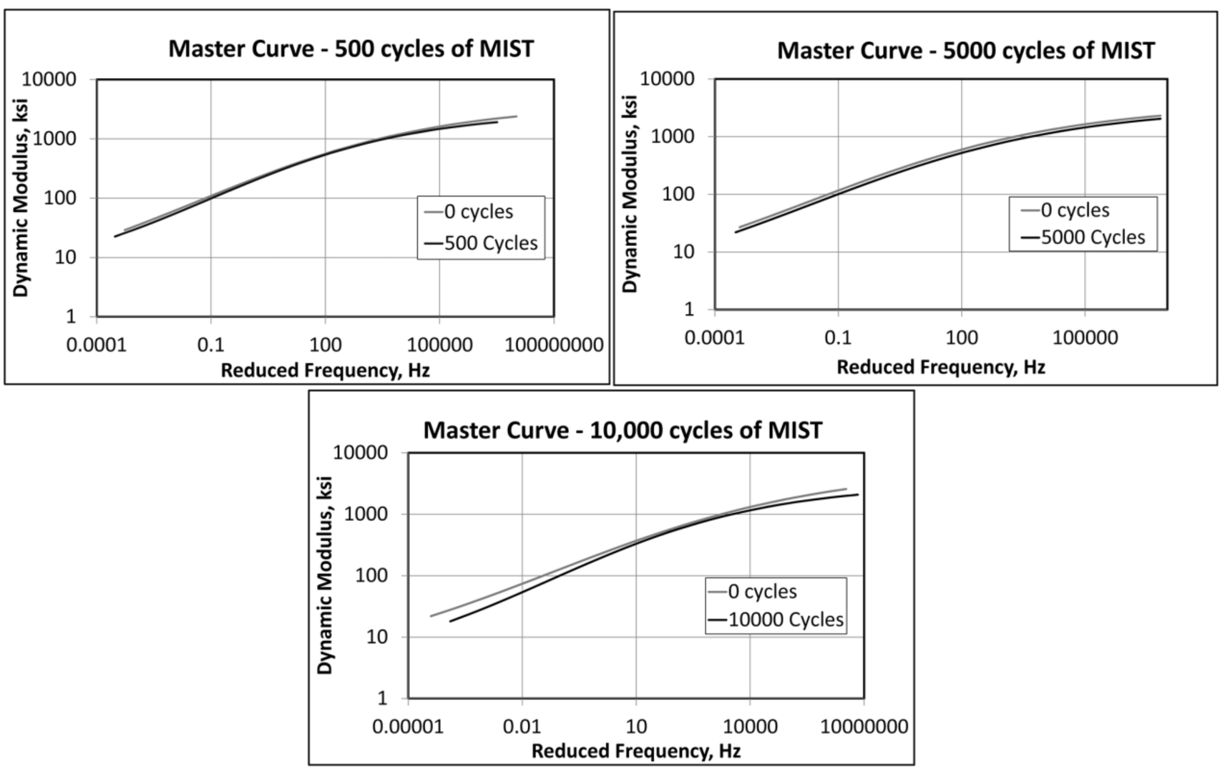
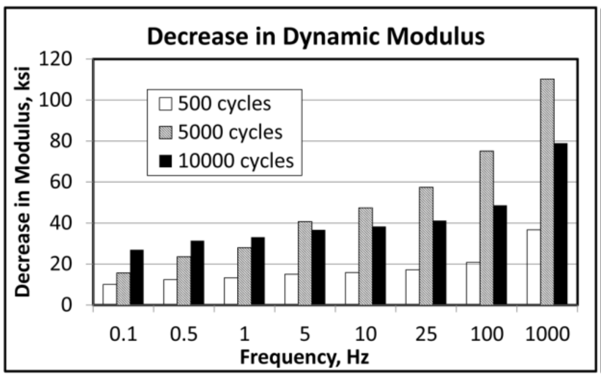


Figure 10 Master Curve before and after moisture conditioning for different MIST cycles

In order to compare the decrease in Dynamic Modulus with increase in the number of MIST cycles, the values of modulus for certain frequencies was chosen from the master curve developed using the test results. The frequencies chosen were 0.1, 0.5, 1, 5, 10, 25, 100 and 1000 Hz in order to capture different vehicular speeds in field. Figure 11 shows the decrease and percentage decrease in the value of Dynamic Modulus before and after the moisture conditioning.



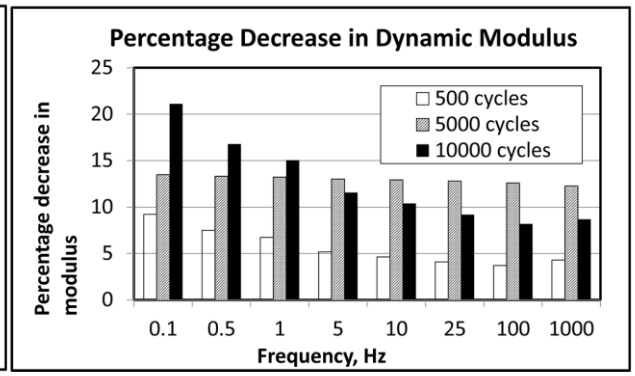


Figure 11 Decrease and percentage decrease in Dynamic Modulus after moisture conditioning in MIST

As it can be seen from the figure, the Dynamic Modulus value decreases after the asphalt sample was conditioned with moisture. The percentage decrease in modulus with respect to the number of cycles conditioned in MIST seemed to vary with the frequency of loading and the number of moisture conditioning cycles. From Figure 12 it was observed that increase in MIST cycles causes decrease in Dynamic Modulus of asphalt concrete for lower frequency loading. For the case of higher frequency of loading, the increase in moisture conditioning cycle however causes a decrease in Dynamic Modulus value up to a certain limit after which the modulus increases again. The reason for the increase in the Dynamic Modulus value of asphalt for higher frequencies after certain cycles of MIST conditioning may be due to the change in chemical

composition of the binder under aging which is not under the scope of the current research and can be further investigated in future research efforts. Moreover, the maximum number of cycles of conditioning in MIST was just 10,000 cycles while, the actual number of load repetitions in case of a pavement structure in the field is at least above 0.3 million equivalent single axle load. Hence the number of conditioning cycles used in this research can only predict the short term behavior of moisture influence in a recently compacted asphalt concrete. In order to simulate the long term effect of moisture damage in asphalt, it is recommended to increase the number of cycles higher with combinations of different pressures and temperatures of conditioning.

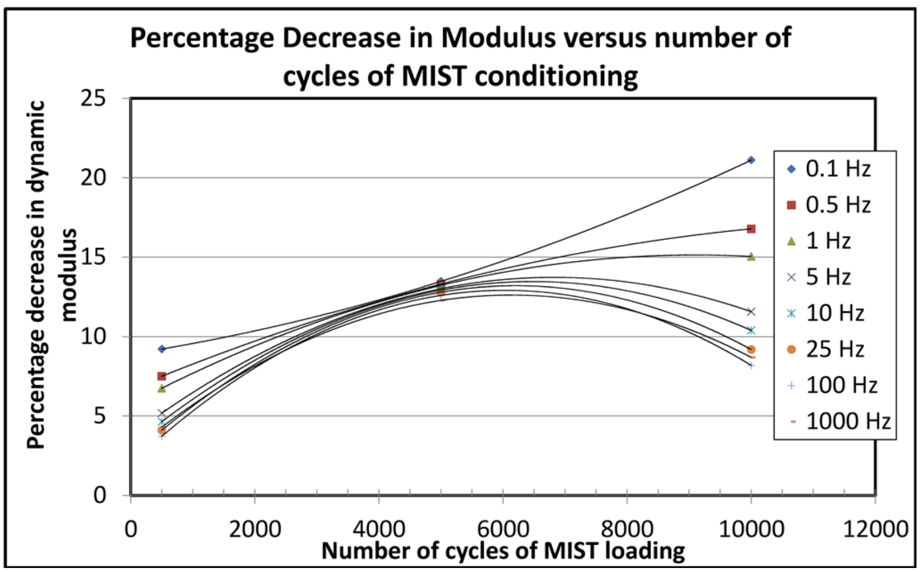


Figure 12 Percentage decrease in Dynamic Modulus plotted against the number of cycles of MIST conditioning

4.2 Performance Evaluation for Dynamic Modulus Using KENLAYER

In order to see the effect of moisture damage in asphalt concrete on the performance of a typical pavement system, KENLAYER computer program was used. The analysis in the program

was run in order to calculate the critical strains for the pavement layer. The model used for the analysis considered asphalt concrete layer as a linearly elastic material with a single value of Dynamic Modulus and the base layer as a granular non-linear material. The thickness for asphalt concrete layer was assumed as 4 inches and the unbound aggregate layer was assumed as 6 inches. The model used to characterize the unbound aggregate layer was a K1- K2 model. The constitutive relationship for the value of modulus of the granular layer as used in the program is expressed as,

$$E = K_1 \theta^{K_2}$$

A typical value of K1 ranges from 2900 to 7750 psi and K2 ranges from 0.46 to 0.65 for an in-service base and sub-base material according to a study conducted by Monismith and Witczak (1980). For this thesis, the value of K1 was assumed at 4000 psi and K2 at 0.5. The subgrade was assumed as relatively soft linear elastic material with the value of modulus as 7000 psi. The contact radius of the tire was assumed as 5.5 inches with the contact tire pressure of 80 psi for a single tire wheel load. The analysis was done for sixty different cases, thirty for the modulus value obtained from samples before the moisture conditioning and thirty for the values of modulus obtained after the moisture conditioning in MIST for all 500, 5000 and 10,000cycles of MIST conditioning separately.

The tensile strain at the bottom of the asphalt layer and the compression stress at the top of the subgrade were obtained from KENLAYER for all the cases. The Asphalt Institute (Asphalt Institute, 1982) failure criteria were used as the transfer functions to find out the number of allowable load applications for both fatigue and rutting failure. The allowable number

of load repetitions for fatigue failure and the allowable number of load repetitions for rutting failure as used by the Asphalt Institute are expressed in following equations,

$$N_f = 0.0796(\in_t)^{-3.291}|E^*|^{-0.854}$$

$$N_d = 1.365 \times 10^{-9} (\in_c)^{-4.477}$$

Figure 13 shows the calculated values of allowable number of load repetitions for fatigue failure for different cases both before and after the moisture conditioning. It was seen that the number of allowable load applications for the similar pavement configuration at lower temperatures decreased after the moisture was introduced in the pavement. The case was different for higher temperatures and lower frequencies as can be seen in the figure. The number of allowable load applications for lower frequency and higher temperature after moisture conditioning was actually greater than that of the sample before moisture conditioning. This meant that the decrease in Dynamic Modulus due to moisture conditioning for lower frequencies and higher temperatures actually improved the fatigue performance of the pavement but not so for lower temperatures and higher frequencies. There seemed to be very little change in the fatigue performance in the model for temperatures of 21.1 and 37.8^{0}C.

Similar analysis was done for the rutting performance of the pavement model. Figure 14 shows the plot of allowable number of load repetitions for rutting failure against various cases of temperature and frequencies. The number of load repetitions for rutting failure unlike fatigue seemed to decrease for each case after moisture was introduced into the asphalt.

The results from these performance analysis suggested that the decrease in the Dynamic Modulus due to moisture damage in asphalt causes decrease in the allowable load repetitions for both fatigue and rutting failure. It was also observed that for lower values of Dynamic Modulus as seen in cases of higher temperatures and lower frequencies, the pavement behaved better in fatigue after moisture conditioning. This was because as the Dynamic Modulus of the asphalt layer decreases, the base layer becomes stiffer. After a certain limit of decrease in Dynamic Modulus of asphalt layer, there is a decrease in tensile strain at the bottom of the asphalt layer. This results in an increase in the fatigue performance of the pavement. On the other hand, a decrease in modulus of pavement after the moisture damage always causes a decrease in rutting performance of the pavement.

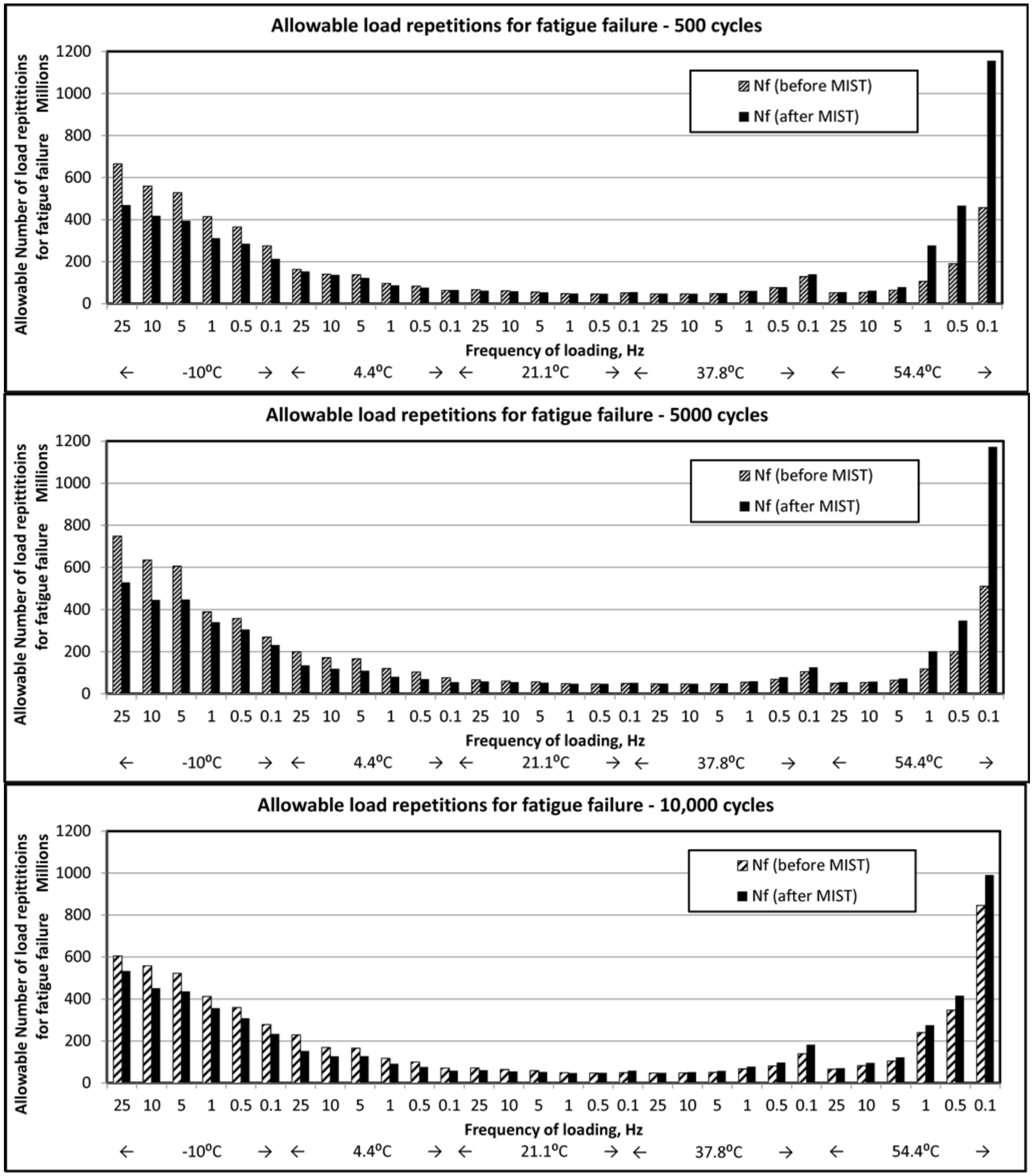


Figure 13 Allowable number of load repetitions for fatigue failure before and after the MIST conditioning

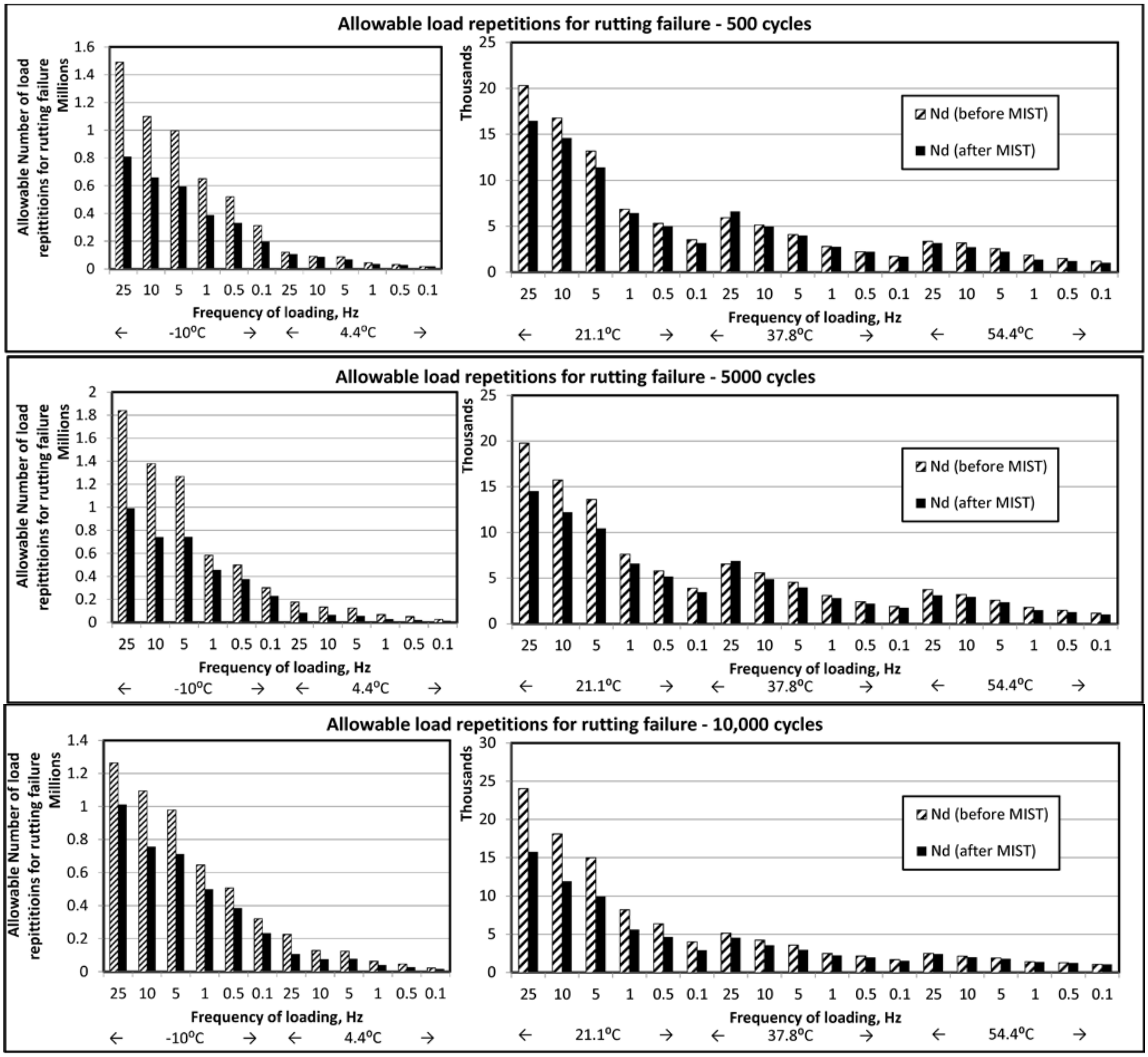


Figure 14 Allowable number of load repetitions for rutting failure before and after the MIST conditioning

4.3 Statistical Analysis for Dynamic Modulus Test

Most of the moisture characterization approach uses the comparison of parameter derived by testing dry or control specimens and the same parameter derived by testing moisture

conditioned specimens. If the test method is non-destructive, the control specimen and the conditioned specimen can either be the exact same sample or separate samples with similar properties. In order to decide whether to use the same sample or different but similar samples as control specimen and conditioned specimen for the purpose of this research, the Coefficient of Variation of the Dynamic Modulus values with similar test conditions were compared among the specimens in two separate ways; by comparing the Dynamic Modulus on same specimen with repeated testing and by comparing the Dynamic Modulus on different but similar specimens. The approach which gives lower variability in the modulus value would be chosen as the approach followed for comparing the control specimen and the conditioned specimen. For the first case, the test was repeated three times on two similar samples. For the second case, the modulus was compared from test results from six different but similar specimens. Table 5 shows the approach used to calculate the Coefficient of Variation using both of these techniques.

Table 5 Different cases used to calculate Coefficient of Variation to compare the repeatability of the results

First Case						
Sample 1, Test 1	Sample 1, Test 2	Sample 1, Test 3	COV			
Sample 2, Test 1	Sample 2, Test 2	Sample 2, Test 3	COV			
Second Case						
Sample 1, Test 1	Sample 2, Test 1	Sample 3, Test 1	Sample 4, Test 1	Sample 5, Test 1	Sample 6, Test 1	COV

As a single Dynamic Modulus Test consists of thirty different test conditions which gives thirty different values of Dynamic Modulus, sixty values of Coefficient of Variation was

obtained from the two samples from the first case and thirty values of Coefficient of Variation was obtained from the second case. As the coefficient of variation is independent of both the units of measurement and the magnitude of data (Lovie, 2005), the average value of the Coefficient of Variation was obtained for each case. The average Coefficient of Variation for the first case was obtained as 7.4 with a standard deviation of 3.7. This means that the probability of getting the Coefficient of Variation of the test results below 15% was 97.98%. The probability was calculated with an assumption that the coefficient of variation is distributed normally. Similarly, the average value of Coefficient of Variation for the second case was obtained as 13 with the standard deviation of 4. This means that the probability of getting the Coefficient of Variation of the test results below 15% was 69.15%. Comparing the probabilities for both of the cases, it was concluded that the variance of the Dynamic Modulus in the first case is less than that in the second case. Hence it was more reasonable to use the first case with the same sample both as the control and the moisture conditioned specimen to compare the modulus value before and after the conditioning.

In addition to that, it was important to check if the same sample subjected to Dynamic Modulus repeatedly undergoes any damage or change in its modulus value. In order to check that, the forward and reverse loading test methodology as referred in this paper was followed as described earlier. The test result showed that there is not much change in the Dynamic Modulus on the same sample even after the sample is tested three times. The slope of the fitted regression line was found close to one in all cases as seen in Table 6 with high Coefficient of Correlation. Figure 15 shows that the fitted line is very close to the line of equality which suggests that the

values of modulus obtained both before and after the repetition of the test on the same sample are very close to each other.

Table 6 Coefficient of correlation and slope of the fitted line

Loading Sequence	Coefficient of Correlation (R)	Slope of regression line
Forward versus Reverse	0.999	0.9118
Reverse II versus Forward	0.998	0.9691
Reverse II versus Reverse	0.999	0.8851

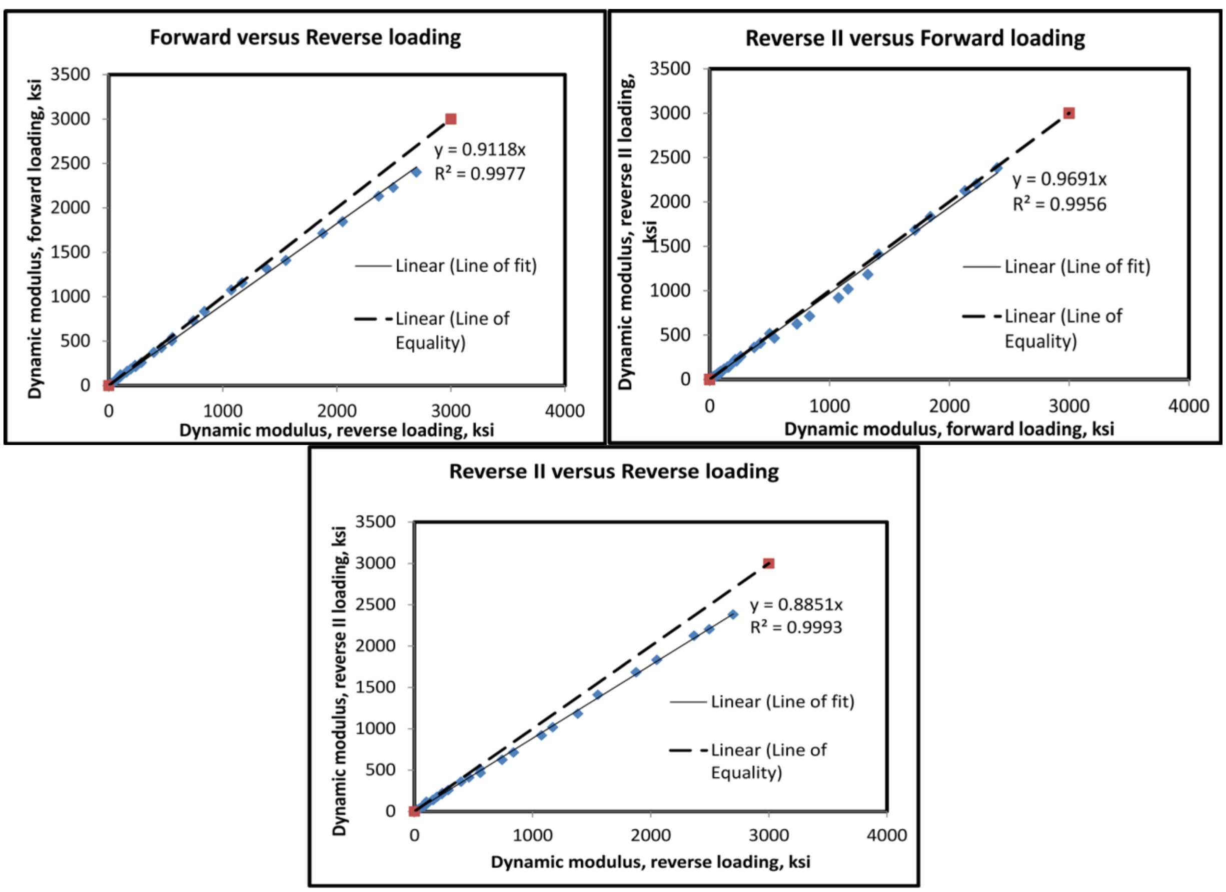


Figure 15 Comparison of test results after repetition of the test with the line of equality

In order to check the validity of the test results, the variability of the Dynamic Modulus was measured in terms of Coefficient of Variation. The Coefficient of Variation was obtained among the samples that were tested with similar conditions. Table 7 shows the approach used to obtain the Coefficient of Variation for similar test conditions. Thirty values of Coefficient of Variation were obtained for each case as shown in Table 7. The average value of Coefficient of Variation and its standard deviation is presented in Table 8.

Table 7 Approach to calculate the Coefficient of Variation of the test results for result validation from Dynamic Modulus Test

Before MIST SPD-018	Before MIST SPD-019	Before MIST SPD-020	COV 1
After 500 cycles SPD-018	After 500 cycles SPD-019	After 500 cycles SPD-020	COV 2
Before MIST SPD-021	Before MIST SPD-022	Before MIST SPD-023	COV 3
After 5000 cycles SPD-021	After 5000 cycles SPD-022	After 5000 cycles SPD-023	COV 4
Before MIST SPD-026	Before MIST SPD-027	Before MIST SPD-028	COV 5
After 10,000 cycles SPD-026	After 10,000 cycles SPD-027	After 10,000 cycles SPD-028	COV 6

Table 8 Coefficient of Variation of results from Dynamic Modulus Test of samples tested with similar conditions

Combination	Average COV (%)	Standard Deviation (%)	Probability of getting COV below 15%	Probability of getting COV below 25%
COV 1	12	7	66.64	96.86
COV 2	6	3	99.87	100
COV 3	8	3	98.98	100
COV 4	12	6	69.15	98.5
COV 5	8	4	95.99	100
COV 6	12	4	77.34	99.94

As seen in Table 8, the variability for the three samples tested before 500 cycles of MIST conditioning seem to be higher as the probability to get the coefficient of variation below 15% is only 66.64%. Similar condition is seen for the three samples tested after 5000 cycles of MIST conditioning and the samples tested after 1000 cycles of MIST conditioning where the probability of obtaining the Coefficient of Variation below 15% are 69.15% and 77.34% respectively. This showed high variability in these test results among different samples tested with the similar test conditions. However, for the other three cases, the probability to obtain 15% of Coefficient of Variation was well above 95%. Further, with the acceptance limit of Coefficient of Variation increased to 25%, all the test results show that the probability to get the results below 25% was well above 95%. This suggested that the results from the samples can further be analyzed keeping in view that the test results are repeatable with the Coefficient of Variation kept below 25%. The reason for higher Coefficient of Variation may be because of variation in the air voids as the target air voids were chosen as 7% with the tolerance of 1% or because of the equipment error itself.

4.4 Indirect Tensile Test with MIST

The results from Indirect Tensile Test showed that there was a decrease in tensile performance of asphalt concrete after moisture intrusion. The Coefficient of Variation of the peak tensile strength was calculated before and after the moisture conditioning for each four samples with similar test conditions. It showed that the Coefficient of Variation of the peak tensile strength of the sample varies from 4.11% to 17.07%. Higher coefficients of variations are seen for the samples which are tested for Indirect Tensile Test after the moisture conditioning.

Table 9 Coefficient of Variation of peak tensile strength before and after moisture conditioning

Test Conditions	Average Peak Tensile Strength (psi)	Standard deviation of peak tensile strength (psi)	Coefficient of Variance (%)
IDT before 500 cycles	60.46	2.48	4.11
IDT after 500 cycles	53.26	4.3	8.07
IDT before 5000 cycles	41.66	1.95	4.68
IDT after 5000 cycles	36.97	3.06	8.29
IDT before 10,000 cycles	48.44	5.72	11.81
IDT after 10,000 cycles	35.7	6.09	17.07

Figure 16 shows the values of peak tensile strength after Indirect Tensile Test in MIST. There was a small drop in the peak tensile strength after moisture conditioning. The Tensile Strength Ratio for the average peak tensile strength of the samples before and after the moisture

conditioning for 500, 5000 and 10000 cycles of MIST was 0.88, 0.89 and 0.74 respectively.

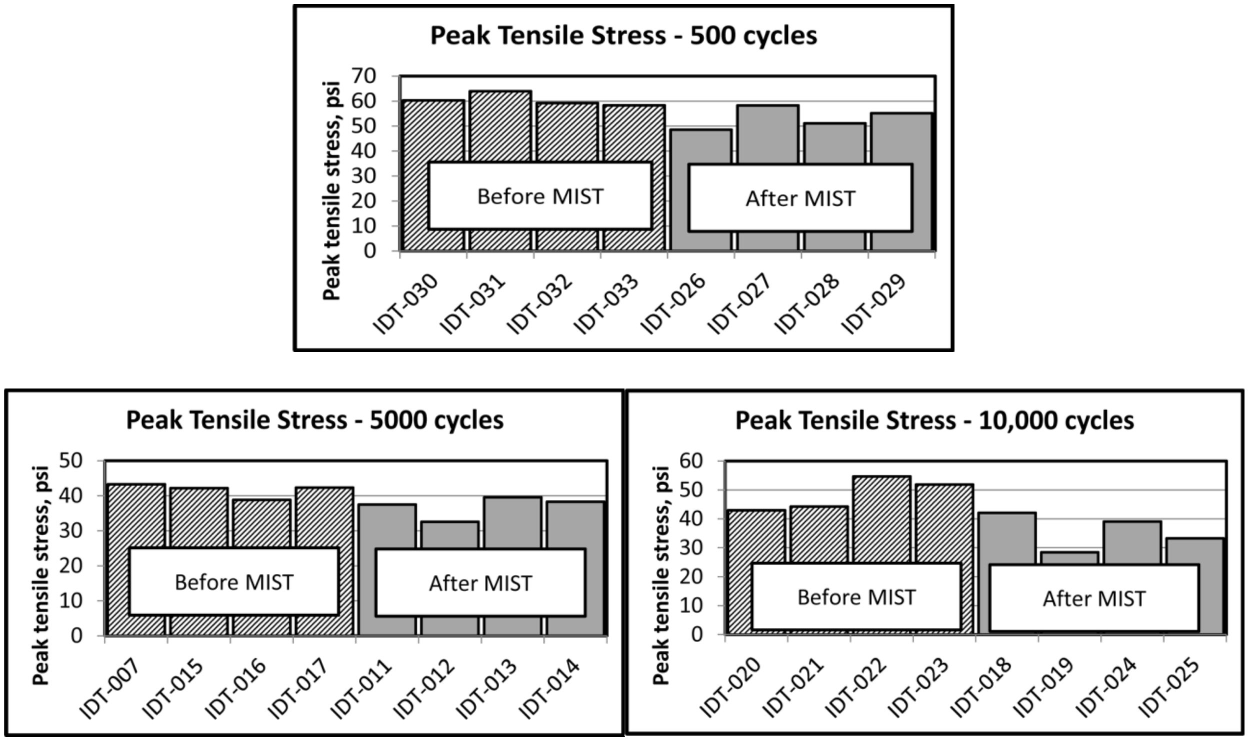


Figure 16 Peak Tensile Stress before and after MIST conditioning

Figure 17 shows the plot of strain versus stress for all eight samples tested for Indirect Tensile strength for each of 500 cycles, 5000 cycles, and 10000 cycles of MIST conditioning. The plot clearly shows that there is a decrease in the peak Indirect Tensile strength due to moisture conditioning in MIST. However, a better distinction appears in the nonlinearity of the stress-strain curves. The samples without MIST conditioning seemed to show more linear behavior on the stress vs. strain plot compared to the samples with MIST conditioning before failure.

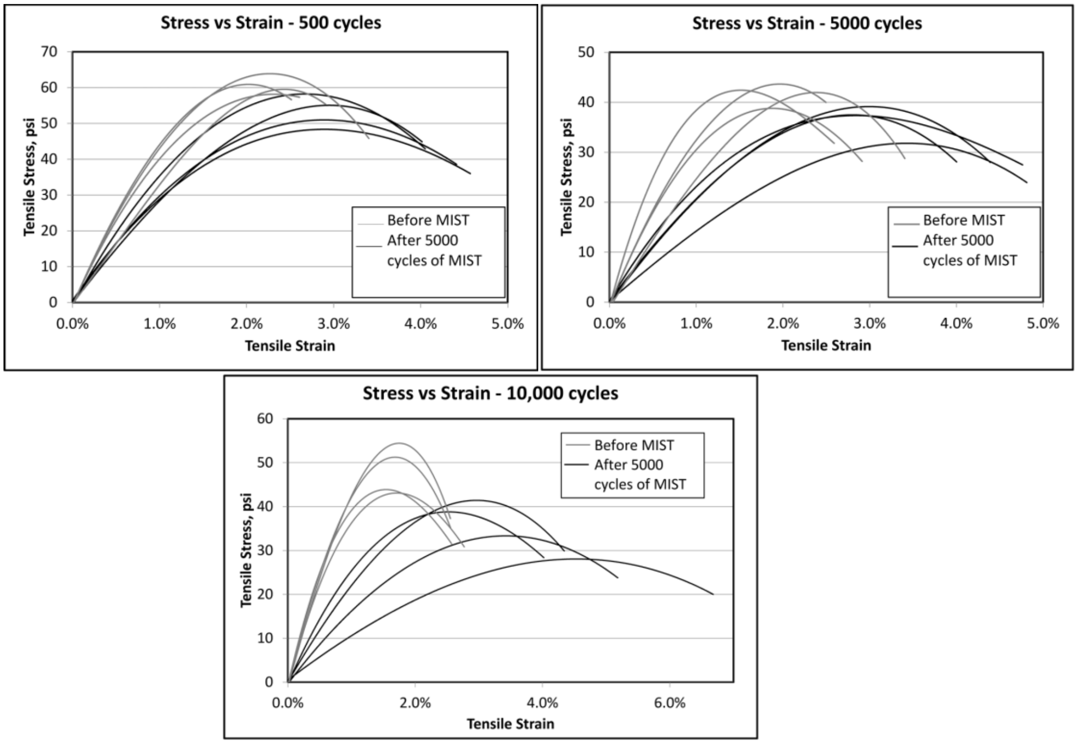


Figure 17 Indirect Tensile Stress versus Strain before and after 500, 5000 and 10,000 cycles of MIST conditioning

In order to quantify the observed nonlinearity in the stress-strain curve, the values of secant modulus were found out for 1% strain and for the peak value of stress. The plots for secant modulus at 1% strain in Figure 18, and for the secant modulus values for the peak stress in Figure 19 show that the values of modulus for samples without moisture conditioning is much higher than the samples with moisture conditioning. The average value of the secant modulus at 1% strain level decreased to 77%, 63% and 46% of their unconditioned value for 500, 5000 and 10000 cycles of MIST conditioning respectively. Similarly, the average value of secant modulus at peak stress decreased to 70%, 54% and 39% of their unconditioned value for 500, 5000 and 10000 cycles of MIST conditioning respectively. This showed that there is a significant drop in

the secant modulus at peak and secant modulus at 1% strain level due to moisture damage which are useful parameters to characterize moisture susceptibility of asphalt.

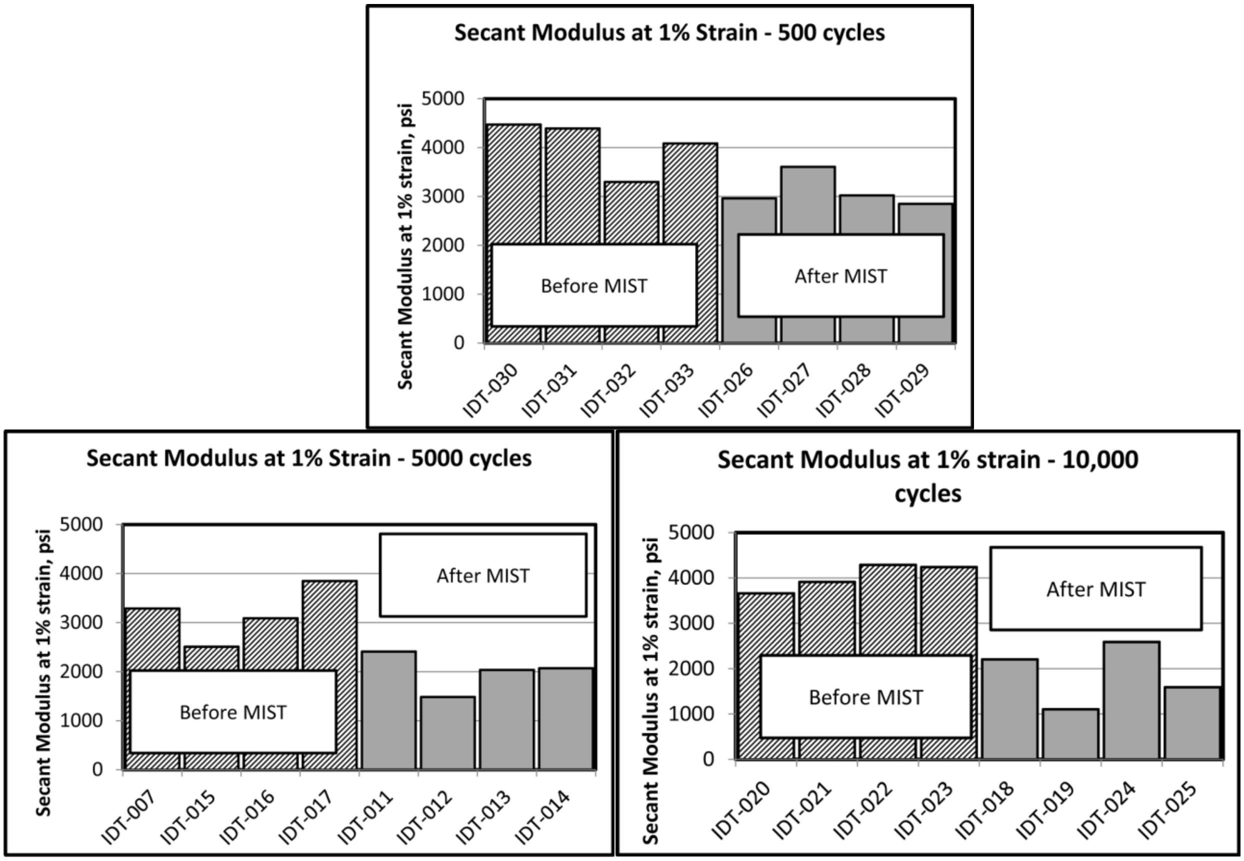


Figure 18 Secant Modulus at 1% strain level before and after moisture conditioning in MIST

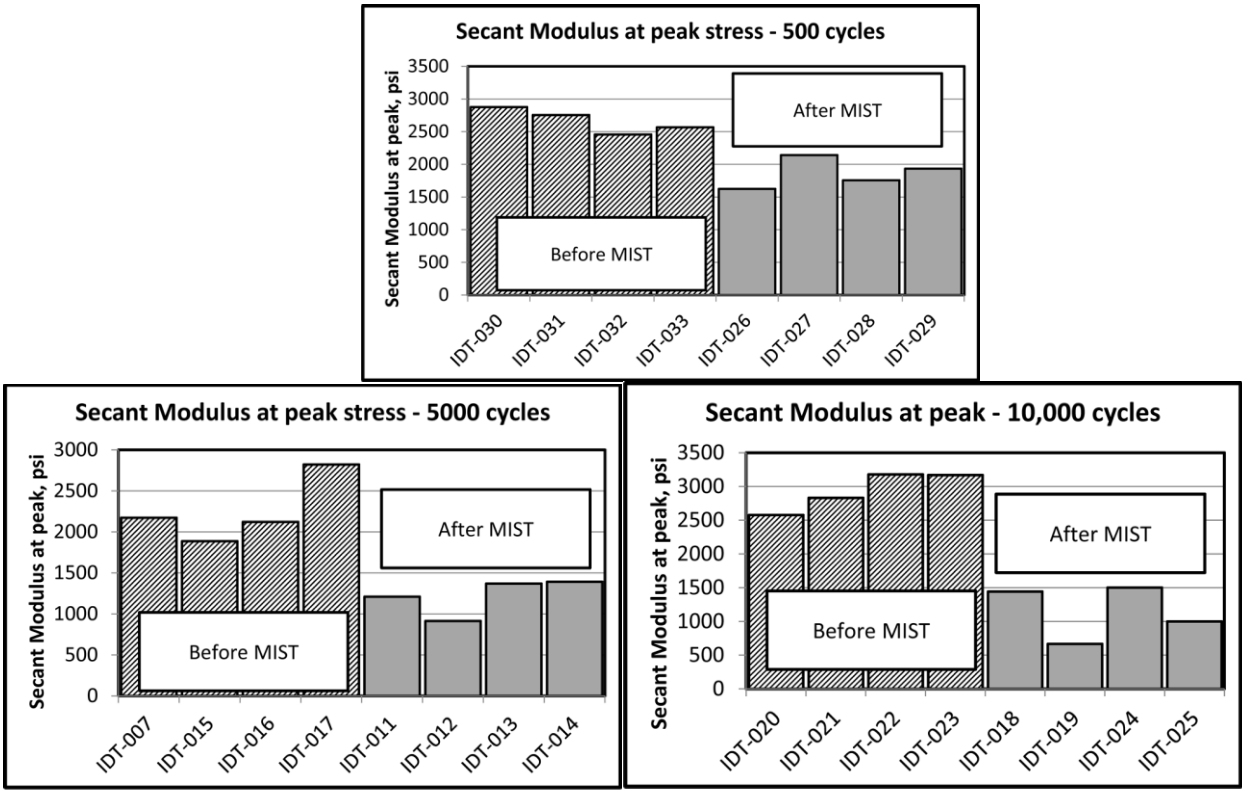


Figure 19 Secant Modulus at peak stress before and after MIST conditioning

For the measure of the sample's nonlinearity, the resilient strain and the strain at failure was found out from strain versus stress plot. The nonlinearity of the sample, as opposed to the nonlinearity of the stress-strain curve, was defined as the ratio of strain at failure to the resilient strain which is the highest value of strain for the linear portion. Table 10 shows the values of nonlinearity calculated for different samples for 500, 5000 and 10,000 cycles of MIST conditioning. The average value of nonlinearity after moisture conditioning changed from 4.67 to 3.32 for 500 cycles, from 2.51 to 3.62 for 5000 cycles and, from 2.81 to 3.96 for 10000 cycles. This shows that the nonlinearity of the samples increases significantly after moisture is introduced in the samples for longer time. The average percentage increase in nonlinearity for the case of 5000 cycles of MIST was 44.2% and the same for the case of 10,000 cycles of MIST

was 40.9%. The nonlinearity actually seemed to decrease for the case of 500 cycles of MIST conditioning. Although the nonlinearity of the sample is important to show the damage in asphalt due to moisture, this parameter is not significant in determining the extent of moisture damage as the percentage of change in the nonlinearity for both 5000 cycles and 10,000 cycles of loading were almost the same while it increases for moisture conditioning for a shorter time.

Table 10 Average values of secant modulus and nonlinearities of the samples with respective Coefficient of Variation before and after MIST conditioning

Test Conditions	**Secant Modulus at 1% strain**		**Secant Modulus at peak stress**		**Nonlinearity of the test**		**Peak Tensile Strength**	
	Average secant modulus at 1% strain (psi)	**COV (%)**	**Average secant modulus at peak (psi)**	**COV (%)**	**Average Nonlinearity**	**COV (%)**	**Average Peak Tensile Strength**	**COV (%)**
IDT before 500 cycles	4061.98	13.2	2663.51	7.04	4.67	13.91	60.46	4.11
IDT after 500 cycles	3108.03	10.9	1862.63	12.03	3.32	30.1	53.26	8.07
IDT before 5000 cycles	3184.12	17.37	2251.04	17.8	2.51	15.25	41.66	4.68
IDT after 5000 cycles	1999.73	19.17	1220.94	18.01	3.62	13.26	36.97	8.29
IDT before 10,000 cycles	4025.65	7.32	2938.87	9.88	2.81	7.9	48.44	11.8
IDT after 10,000 cycles	1871.89	35.02	1151.98	34.19	3.96	25.96	35.7	17.1

Table 11 shows the ratio of various parameters of tensile strength between conditioned and control specimens. It was seen that the average percentage change in the ratio of the secant modulus at peak was the highest among all the other parameters. In other words, the secant modulus at peak for Indirect Tensile Testing of asphalt concrete was more sensitive to moisture

conditioning compared to any other parameters including the Tensile Strength Ratio. Hence secant modulus can be a better tool to characterize moisture damage in asphalt concrete against the Tensile Strength Ratio. The nonlinearity, on the other hand, does not seem to have a trend in order to be used as a tool to evaluate the moisture damage in asphalt.

Table 11 Summary of ratio of tensile strength, secant modulus at 1% strain, secant modulus at peak and nonlinearity

No of MIST cycles	**Before and after moisture conditioned ratio of**			
	Peak Tensile Strength	**Secant Modulus at 1% strain**	**Secant Modulus at peak**	**Nonlinearity**
500	0.88	0.77	0.7	0.71
5000	0.89	0.63	0.54	1.44
10000	0.74	0.46	0.39	1.41

CHAPTER 5: CONCLUSION AND RECOMMENDATION

Moisture conditioning of asphalt concrete and characterization of moisture damage in the laboratory is a difficult task. The main objective of the research was to understand the effect of moisture damage in the mechanical properties of asphalt concrete and its effect on the performance of a pavement. Hence understanding the mechanism of moisture damage in detail is very important. Even when various tests have been devised to quantify moisture damage, the main problem occurs in the repeatability of the results. Numerical approach to quantify moisture damage is gaining popularity but its limitation lies in poor correlation with the field results besides its modeling complications. Hence better and easier approaches for quantifying moisture damage in the laboratory are needed.

One of the most common moisture conditioning methods used is AASHTO T-283 (Freeze and Thaw) method. This test method shows better correlation in conjunction with peak indirect tensile strength but variability is a big issue with this technique. In addition to that, the mechanical mechanism of moisture damage is not simulated during the moisture conditioning process. Hamburg Wheel Tracking is another method of evaluating mixes for potential moisture damage widely used in TxDOT. The major limitation of this method is that it is not a moisture conditioning method rather a performance test which makes it impossible to characterize moisture damage with other performance tests. Besides, this technique does not confirm saturation of samples much less simulate various mechanisms of moisture damage. Moisture Induced Stress Tester (MIST), on the other hand, simulates both physical and mechanical

moisture damage mechanism. The samples can be used in conjunction with other test methods to characterize moisture damage phenomenon.

The test methods used for performance testing were dynamic modulus, and indirect tensile strength. Selection of a dynamic performance test over static is more relevant as asphalt concrete is a visco-elastic material. Moreover, recent research in asphalt concrete is focused more on the applicability of dynamic responses of asphalt concrete against static techniques. Indirect tensile test, on the other hand, renders the adhesive failure mechanism in asphalt which is the most failure mechanism is moisture damaged asphalt concrete.

The conclusions of the study can be summarized in the following points:

- The effect of moisture causes decrease in the Dynamic Modulus in asphalt concrete. The decrease in Dynamic Modulus depends on the extent of moisture conditioning and frequency of loading while keeping the strains at constant levels. There seemed to be a decrease in Dynamic Modulus with increase in moisture conditioning for lower frequencies while increase in Dynamic Modulus with increase in moisture conditioning for higher frequencies for a type-d dense graded hot mix asphalt concrete in Texas.
- Moisture intrusion causes drop in peak tensile strain of asphalt concrete. However, the main difference was seen in the stress-strain curve which becomes more non-linear with increase in moisture damage. There is higher decrease in peak tensile stress, secant modulus at 1% strain level, and secant modulus at peak for samples with longer moisture conditioning.

- Ratio of secant modulus at peak for Indirect Tensile Test is more sensitive to moisture damage than Tensile Strength Ratio which can be a better index parameter to characterize the moisture damage in asphalt concrete.
- There is a decrease in fatigue performance in pavement after moisture intrusion for higher frequencies of loading and lower temperatures while an increase in fatigue performance for lower frequencies of loading and higher temperatures. Moisture damage causes decrease in rutting performance in a flexible pavement.

There is a high variability in both Dynamic Modulus and tensile test used as the parameters for characterizing moisture damage in asphalt. Moisture conditioning itself causes higher variability in the test results but the results from the study is significant for further research efforts. The variability may be minimized by decreasing the tolerance of air voids from 7±1% to 7±0.5%. Controlling the quantity of air voids in laboratory itself causes rejection of samples affecting time and budget which is a completely different scenario to account for. Despite the variability in results which is somewhat inevitable, the result in increase in the strain at failure as shown in the stress-strain curve for samples loaded with indirect tension is an important finding. Moreover, the sensitivity of moisture affected asphalt concrete with low frequency – high temperature dynamic modulus is another important aspect to look into. The characterization of moisture damage in asphalt concrete using such results is crucial in order for a correct field simulation. The research has a very wide scope in future efforts for characterizing moisture damage in asphalt in pavement engineering.

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