ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

SCHOOL OF GRADUATE STUDIES

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

LABORATORY EVALUATION OF MOISTURE DAMAGE IN ASPHALT MIXES

By : Mizan Tekle Mengesha

A Thesis Submitted to the School of Graduate Studies in Partial Fulfillment of the Requirements for the Degree of Master of Science in Road and Transport Engineering

Advisor:
Habtamu Melese Zelelew (PhD,PE)

April, 2020

Addis Ababa, Ethiopia

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DECLARATION

I, the undersigned, declare that this thesis is my original work, has not been presented for a degree in any other University, and that all sources of materials used for the thesis have been duly acknowledged.

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Name: Mizan Tekle Mengesha

April, 2018

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ABSTRACT

Moisture damage usually called stripping happen when water infiltrates between an asphalt film and the aggregate surface, and replaces the asphalt aggregate's coating. These situations cause a loss of bond between the aggregate and the asphalt cement. The most serious consequence of stripping is the loss of strength and integrity of the pavement which result in premature failure of asphalt pavement. Moisture damage contributes significantly to the premature deterioration of asphalt pavements, which, leads to an increase vehicle operating cost and maintenance costs of a pavement. These conditions also make driving dangerous, and driving comfort and safety are often compromised.

This thesis utilized laboratory evaluations to study the effects of moisture damage in asphalt mixes. Two different sources of aggregates and one type of asphalt binder were used. The aggregates quality test were conducted namely Los Angeles abrasion, soundness loss, sand equivalent, flakiness index, specific gravity and absorption. The asphalt binder conventional tests were evaluated which were flash point, ductility, penetration, thin film oven test, softening point and solubility. Marshal Flow and stability test were conducted. Moisture susceptibility using indirect tensile strength test and single wheel tester were performed.

The quality test results of the aggregates and the bitumen were met the specification set by ERA. From the aggregate test result the more strong, the higher weathering resistance and the lesser water absorbent aggregate found to be less moisture sensitive. The research conclude that moisture in asphalt mix has a significant effect on the performance of asphalt mix which is decrease in Marshal stability, decrease in tensile strength and increase the rutting of a mixture. The test result from ITS show that the tensile strength values were higher for the dry specimens than the wet specimens. From wheel tester result the rutting of a mix were higher for the wet samples than the dry one.

Key words: Hot Mix Asphalt, Moisture Susceptibility, Stripping, Tensile Strength, Rutting

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LIST OF ABBREVIATIONS

AACRA	Addis Ababa city road authority
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt cement\Asphalt concrete
APA	Asphalt Pavement Analyzer
ASTM	American Society for Testing and Materials
ECS	Environmental Conditioning System
ERA	Ethiopian Road Authority
ERSA	Evaluator of Rutting and Stripping
GLWT	Georgia Loaded Wheel Tester
HMA	Hot Mix Asphalt
HWTD	Hamburg Wheel Tracking Device
ITS	Indirect tensile test
LVDT	Linear Value Displacement Transducer
NAT	Net adsorption test
OBC	Optimum Binder Content
PG	Performance Graded
SHRP	Strategic highway Research program
TSR	Tensile Strength Ratio
Va	Air Void
VFA	Voids Filled with Asphalt
VMA	Voids in Mineral Aggregate

CHAPTER 1
INTRODUCTION

1.1 Background

The road network is an important element of the national infrastructure and its construction, operation, and maintenance constitute a large part of the national annual budget. Mostly, roads in Ethiopia are flexible pavement type. Flexible pavement typically consists of asphalt mixture placed over granular base layer supported by the compacted soil, referred to as the subgrade.

Environmental conditions such as temperature and water can have a significant effect on the performance of asphalt concrete pavements in addition to the traffic load distresses that can be considerable due to the heavy axle loads. The presence of water (or moisture) often results in premature failure of asphalt pavements in the form of isolated distress caused by deboning of the asphalt film from the aggregate surface or early rutting/fatigue cracking due to reduced mix strength.

One of the fundamental properties for good performances of bituminous pavement is proper adhesion between aggregate and bitumen. Moisture presence in bituminous pavements is a primary cause which results in weakening or eventual loss of adhesive bond of bitumen and aggregate. This phenomenon is known as stripping.

The existence of water in bituminous pavement is often one of the major factors affecting the durability of Hot Mix Asphalt (HMA). For prevention of damage, adequate drainage must be provided. Nowadays, the roads are lacking in respect of proper drainage facilities, which is one of the main reasons for road damage due to stripping. Stripping can cause rutting, raveling, bleeding, cracking and formation of potholes and culminate with complete failure of the pavement.

Probably the most damaging and often hidden effect of moisture damage is associated with reduced pavement strength. Tensile strength plays an important role in the performance of a mixture under fatigue, rutting and moisture susceptibility. The damage due to moisture is controlled by the specific limits of the tensile strength ratios (TSR) or the percent loss in tensile strength of the mix. The moisture sensitivity of a mixture is evaluated by performing the AASHTO T-283 test . This test has a conditioning phase, where the sample is subjected to saturation and immersion in a heated water bath to simulate field conditions over time. Strength loss is then determined by comparing indirect tensile strengths of an unconditioned control group to those of the conditioned samples.

1.2 Statement of the Problem

Moisture damage has been a major concern to Asphalt pavement technologists for many years and remains a challenge still now. An asphalt mixture is a multi-phase material that results from combining aggregates (i.e., crushed rocks with different sizes and proportions) and asphalt binder. This material is used in the wearing course of flexible pavements and/or as part of base layers. Distresses in asphalt pavements are caused by the combined effects of dynamic traffic loading and environmental changes.

One of the root causes of moisture damage in Ethiopia is poor drainage system. During rainy season most of the capital roads flooded under water due to poor drainage system design and construction and due to damaged drainage, blockage of culverts and bridges. Poor drainage system in addition to the traffic load result in stripping and other asphalt pavement distress. Lack of proper design and construction of drainage system, lack of drainage master plan, the government little attention and lacks of experts are some of the causes of poor drainage system in Ethiopia.

The cold and rainy regions-related climatic and environmental factors can expose pavements to intense loading that can result in seasonal and long term loss of bearing capacity (structural shortage), or loss of surface smoothness due to differential frost action, crack propagation and surface raveling (serviceability shortage). Furthermore, these environmental factors, when accompanied with traffic loading, can result in premature failure of the road ways.

Stripping happen when water infiltrate between an asphalt film and the aggregate surface, and replaces the asphalt aggregate's coating. This situation causes a loss of bond between the aggregate and the asphalt cement. The most serious consequence of stripping is the loss of strength and integrity of the pavement. Stripping failures within the asphalt pavement structure can facilitate or translate into various types of pavement failure such as fatigue cracking, rutting, raveling and potholes. This condition makes driving dangerous, and driving comfort and safety are often compromised.

Moisture damage contributes significantly to the premature deterioration of asphalt pavements, which, leads to an increase vehicle operating cost and maintenance costs of a pavement.

1.3 Objective of the Study

The main objective of the study is laboratory evaluation of moisture damage in hot mix asphalt.

The specific objectives

1. To evaluate the volumetric properties of HMA.
2. To evaluate marshal flow , stability and rutting properties of HMA
3. To assess the effect of aggregate source on moisture susceptibility of the asphalt mixtures;
4. To evaluate the effect of moisture in hot mix asphalt performance;

1.4 Scope and Significance of the study

These studies let to understand the effect of moisture on asphalt pavement and show the best methodology to test the effect of water damage. Identifying the moisture sensitivity of mixes will prevent a pavement from premature failure. It will let to know the material sensitivity to water that helps to use the appropriate remedies such as eliminating moisture susceptible material, providing adequate drainage or treating materials with anti-stripping agent. The scope of the study covers aggregates and binder conventional tests, volumetric properties of the mixes, Marshal Flow and stability test of the mixes, wheel track testing of rutting and indirect tensile strength test of the compacted mixes.

1.5 Limitations

- The thesis evaluated few aggregate source and binder type and limited samples were tested for the experiment.
- The study did not investigate field validation.
- This report did not look into anti-stripping agents.
- The study failed to conduct chemistry tests.
- The research did not conduct Fatigue test of asphalt mixture.
- The rheological binder tests using Dynamic Shear Rheometer (DSR) were not conducted in the study

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

This chapter intends to review several literatures written in the areas of this study concern. The first parts discuss the meaning and problems of moisture damage. Continues in elaborating the major failure called stripping, moving on, the mechanism and factors influencing of stripping will be discussed. In the second part different test methods for evaluating moisture damage will be shown and finally literatures on preventing of moisture damage will be reviewed.

2.2 Moisture Susceptibility

Sources of moisture in an asphalt pavement can be either external or internal in both liquid and vapor form. Water can enter the pavement through infiltration of surface water by gravity, tire action, or irrigation; permeation of water vapor; capillary rise of subsurface water; and through the mixture itself (Wet aggregates) (Arambula Mercado, 2007; Santucci, 2010). Moisture susceptibility is a primary cause of distress in HMA pavements. HMA should not degrade substantially from moisture penetration in to the mix. HMA mixtures may be considered susceptible to moisture if the internal asphalt binder-to aggregate bond weakens in the presence of water. This weakening, if severe enough, can result in stripping. Moisture susceptibility of asphalt concrete paving materials indicates the proclivity for specific combinations of binder and aggregate to sustain damage or a loss in functionality due to the detrimental effects of moisture under repetitive traffic loading. Moisture damage defined as “the progressive functional deterioration of a pavement mixture by loss of the adhesive bond between the asphalt cement and the aggregate surface and/or loss of the cohesive resistance within the asphalt cement principally from the action of water.”By Kiggundu and Roberts (1988). Loss of adhesive bond which is stripping converts high strength asphalt treated pavement layer to a much weaker untreated aggregate section. When it occurs in isolated spots throughout the pavement, It can rapidly develop in to potholes. Over more extensive areas, premature fatigue cracking or rutting may develop due to the reduced support strength of the overall pavement structure. Water in the pavement may simply weaken the asphalt mix by softening or partially emulsifying the asphalt film without removing it from the aggregate surface. During this weakened state, the asphalt pavement layer is subjected to accelerated damage from applied traffic.

2.3 Adhesion Failure

Adhesion is defined as that physical property or molecular force by which one body sticks to another of another nature (Hicks1991)."The breaking of the adhesive bond between the aggregate surface and the asphalt cement" in an asphaltic pavement or mixture is known as stripping (Asphalt Institute, 1981). Several factors affect the adhesion of the asphalt binder to the aggregate, including: interfacial tension between the asphalt binder and the aggregate, chemical composition of the asphalt binder and aggregate, binder viscosity, surface texture of the aggregate, aggregate porosity, aggregate cleanliness, and aggregate temperature and moisture content at time of mixing (Hicks1991). Stripping is one of the major distresses with in asphalt concrete pavements caused due to penetration of water with in the interface of asphalt aggregate matrix.

The force of water intrusion with in asphalt and aggregate can destroy the pavements. Bituminous mixture derives its strength from the bond between the binder and the aggregate of a mixture. If a good bond exists, failure of the mixture should occur within the binder (Majidzadeh and Brovold, 1968). If the bond is poor, the failure may occur at the binder-aggregate interface and may result in premature failure of the mix. Failure caused by stripping occurs in two stages: the first stage is stripping failure, and the second stage is failure of the pavement under traffic (Department of Scientific and Industrial Research, 1962). Many asphalt. Pavements experience stripping failure with in the mix without structural failure of the pavement. If stripping within the pavement becomes excessive, loss of strength may result in excessive deformations caused by repeated loading; this can lead to complete disintegration of the pavement, often in the form of potholes (Department of Scientific and Industrial Research, 1962). Failure caused by stripping can also result in cracking and surface raveling of the pavement (Fromm, 1974). Wearing courses over stripped asphaltic bases are likely to exhibit adhesion failure by raveling and pothole formation (Scott, 1978). In some cases of stripping, a HMA mix has lost so much adhesion between the aggregate and asphalt that a core cannot be remove d in one piece.[Kennedy, McGinnis, and Roberts 1983; Roberts et.al, 1996]. It has been found that the damage will be minimal if stripping is restricted to the coarse aggregate (Kennedy, Roberts, and Lee, 1982). If the fine aggregate in the mixture strips, severe damage will result because the fine aggregate constitutes the basic matrix of the mixture (Kennedy, Roberts, and Lee, 1982). Numerous investigators have observed that if a stripped asphaltic mixture is exposed to a dry environment, the stripping process is reversed and the mixture will heal itself (Fromm (1974), Scott (1978), Gzemski, McGlashan and Dolch (1968), Schmidt and Graf (1972)). Failure of a stripped pavement due to traffic is not reversible, however, and prevention is the best and only cure. Hicks demonstrate four general theories of adhesion to explain the adhesion of asphalt binder to aggregates. These include the Mechanical Interlocking Theory, the Chemical Reaction Theory, the Surface Energy Theory, and the Molecular Orientation Theory. The actual nature of adhesion is not fully explained by anyone of these theories, but is partially explained in each theory (Hicks1991).A brief description of each theory follows.

2.3.1 Mechanical Interlocking Theory

Mechanical interlocking assumes in the absence of chemical interaction between binder and aggregate. The bond strength is assumed to be derived from the cohesion in the binder and interlocking properties of the aggregate particles which include individual crystal faces, aggregate porosity, absorption, surface coating, and angularity (Kiggundu and Roberts1988).

2.3.2 Chemical Reaction Theory

The chemical reaction suggested that the bond strength depend on the chemical interaction between binder and aggregate. This theory arises from an observation that stripping is more serious in acidic aggregate mixtures as compared to basic aggregate mixtures. It is suggested that the chemical reaction between most asphalt binders and acidic aggregates is not as strong as the reaction between most asphalt binders and basic aggregates (Hicks1991).

Surface Energy Theory

When asphalt spreads over and wets an aggregate surface, a change in energy takes place. This change of energy, known as adhesion tension, is a surface phenomenon that depends on the closeness of contact and mutual affinity of the asphalt binder and aggregate (Hicks 1991). The adhesion tension for water to aggregate is higher than that for asphalt binder to aggregate, and consequently water has a tendency to displace the asphalt binder from the aggregate.

2.3.3 Molecular Orientation Theory

The molecular orientation theory states that when asphalt binder comes into contact with an aggregate surface, the molecules in the binder orient themselves so as to satisfy the energy demands of the aggregate. Water molecules are dipolar. Asphalt molecules are generally non polar although they contain some polar components. Consequently, water molecules, being more polar, may more readily satisfy the energy demands of an aggregate surface (Hicks1991).

Cohesive Failure

It has been believed that cohesive failure of asphalt is less important factor in the problem of moisture damage but Bikerman (1960) suggested that the probability of cohesive failure was much greater than of adhesive failure. Kanitpong and Bahia (2002) also demonstrated that from test result of Tensile Strength Ratio (TSR) test they observed failure surfaces in asphalt mixtures in the binder coating without evidence of apparent loss of adhesion to the aggregate particles. This cohesive failure can be partially explained by emulsification of water in the asphalt phase, which is different to conventional emulsified asphalts in which the asphalt is emulsified in a water phase (Fromm1974). Fromm’s work showed that water could enter into the asphalt film and form a water-in-asphalt emulsion (Fromm1974). This emulsification of water in the asphalt film causes asphalt particles to separate from the asphalt film (cohesive failure) and ultimately leads to an adhesive failure at a critical time when this emulsification boundary propagates to the aggregate surface. However, since the mechanism of cohesive failure leads, ultimately, to an adhesive failure, most instances of cohesive failure may only be inferred rather than observed, and the final mechanism (i.e., adhesive) is reported as the cause (Terrel,R.L. and Al-Swailmi (1994)). Thus, even though the definition of moisture damage in HMA has been regarded as the failure of adhesive and cohesive bonds between the asphalt and the aggregates in the presence of water, it has proven difficult to distinguish between the two modes of failure in predicting failure mode unless the failure surface of HMA is visually inspected a posteriori

Mechanisms of Stripping

A mechanism is a process that leads to changes in the internal or external conditions of a system producing a new “state” or condition. When the final state of the system represents a reduction in its Original integrity, the process is considered a damage mechanism. Based on the literature (Taylor and khosla, 1983; kiggundu and Roberts, 1988; andTerrel and Alswalilmi, 1994) there are about seven different mechanisms of stripping: detachment, displacement, spontaneous emulsification, pore pressure, hydraulic scour, pH instability, and the effects of the environment on asphalt-aggregate material systems. It appears that these mechanisms may act individually or together to cause adhesion failure in bituminous mixtures.

2.3.4 Detachment

Detachment is the separation of an asphalt film from an aggregate surface by a thin film of water without an obvious break in the film (MajidzadraandBrovold, 1968). Where stripping by detachment has occurred, the asphalt film can be peeled cleanly from the aggregate, indicating a complete loss of adhesion (CollegePark, MD, 1981.). The ability of the asphalt to wet the aggregate is called wet ability. Detachment is happening because of the loose bond between the asphalt and the aggregate which is depend on the wet ability. The widely accepted theory of interfacial energy considers adhesion as a thermodynamic phenomenon related to the surface energies of the materials involved, namely, asphalt and mineral aggregates. Consider a three-phase system of aggregate, asphalt, and water. The surface tension of water is much lower than that of asphalt. The wettability of an aggregate increase as the surface tension (or free surface energy) of the adhesive decreases. Water is better than asphalt for reducing the free surface energy of the system to a thermodynamically stable condition of minimum surface energy (K.MajidzadehandF.N.Brovold, 1968).

The same theory emphasizes the effect of polarity of the molecules present at the surface of the two phases. Most aggregates have electrically charged surfaces. Asphalt, which is composed chiefly of high molecular weight hydrocarbons, exhibits little polar activity; therefore, the bond that develops between asphalt and an aggregate is primarily due to relatively weak dispersion forces. Water molecules, on the other hand, are highly polar and are attracted to aggregates by much stronger orientation forces (Department of Scientific and Industrial Research, London, 1962).

2.3.5 Displacement

Stripping by displacement involves displacement of asphalt at the aggregate surface because of the penetration of water to the aggregate surface through a break in the asphalt film (Tarrer and Wagh, 1991; and Fromm, 1974, K.Majidzadeh and F.N.Brovold, 1968). This break can be caused by in complete coating of the aggregate or by film rupture. The source of the break or disruption maybe in complete coating of the aggregate surface initially, film rupture at sharp aggregate corners or edges because the asphalt film at these locations is generally thinner and under tension, pinholes originating in the asphalt film because coating of a dusty aggregate, and so forth. Scott (1978) States that chemical reaction theory can be used to explain stripping as a detachment mechanism. Changes in the pH of the microscopic water accumulations at the mineral surface can alter the type of polar groups adsorbed, as well as their state of ionization/dissociation, leading to the build-up of opposing, negatively-charged, electrical double layers on the aggregate and asphalt surfaces (Scott, 1978). The drive to reach equilibrium attracts more water and leads to physical separation of the asphalt from the aggregate (Scott, 1978).

2.5.3. Spontaneous Emulsification

Water and asphalt combine to form an inverted emulsion, the production of such an emulsion leads to stripping in spontaneous emulsification and is further accelerated by the presence of emulsifiers such as mineral clays and some asphalt additives. (Fromm (1974), Asphalt Institute (1981), Scott (1978)) It has been observed that spontaneous emulsification occurs whenever asphalt films are immersed in water but that the rate of emulsion formation depends on the nature of the asphalt and the presence of additives (Fromm (1974). Fromm (1974) realized that when the emulsion penetrates to the aggregate surface results in a total loss of adhesion. The process of stripping in spontaneous emulsification mechanism has been found that reversible upon evaporation of the water from the emulsion returns the asphalt to its original condition (Fromm (1974).

2.5.4 Pore Pressure

In high void mixes water may circulate freely through interconnected voids. Upon densification of the mix from traffic loading, water may become trapped in impermeable voids and result in the formation of pore pressure. Further traffic may induce high excess pore pressures in the trapped water disrupts the asphalt film from the aggregate surface or can cause the growth of micro cracks in the asphalt mastic (Majid zadra and Brovold, 1968, Asphalt Institute, 1981). Most HMA mixture is compacted to the air void range about8% and10% which is pessimism air void. Below this range air voids are disconnected and are relatively impermeable and thus do not become saturated with water. Above this level air void become interconnected and moisture can flow out under a stress gradient developed by traffic loading. In the pessimism range, water can enter the voids but can’t move freely and results to pore pressure build upon repeated loading. (Terrel and Al-Swailmi, 1994).

2.5.5 Hydraulic Scouring

Hydraulic scouring is a mechanism of stripping that is likely to happen on the pavement surface results from the action of vehicle tires on a saturated pavement surface. This causes water to be pressed down in to the pavement due to traffic loading in front of the tire and immediately sucked away from the pavement behind the tire. Osmosis and pull back have been suggested as possible mechanisms of scour (Fromm,1974).Osmosis occurs in the presence of salts or salt solutions in aggregate pores and creates an osmotic pressure gradient that actually sucks water through the asphalt film. Still there exists an ambiguity among researchers on this process (Little D.N, D.R. Jones. 2002). Mack (1964) supports it, while Thelen (1958) feels it is too slow to be valid. However, several factors support the potential occurrence of this mechanism, including the fact that some asphalt is treated with caustics during manufacture, some aggregates possess salts (compositionally), and asphalt films are permeable. In fact, Cheng et al. (2002) have demonstrated that the diffusion of water vapor through asphalt cement itself is considerable and that asphalt mastics can hold a rather surprisingly large amount of water.

2.5.6 PH Instability

It has been demonstrated that asphalt-aggregate adhesion is strongly influenced by the pH of the Contact water (Scott (1978) and Yoon (1987)).Kennedy (1984) investigated the effect of various sources of water on the level of damage that occurred in a boiling test. Fehsendfeld and Kriech (undated) observed that pH of contact water affects the value of the contact angle and the wetting characteristics at the aggregate-asphalt interface region. Kiggundu and Roberts (1988) point out that these results indicate that stabilization of the pH sensitivity at the asphalt-aggregate interface can minimize the potential for bond breakage, provide strong, durable bonds, and reduce stripping. Tarrer (1996) concluded that the bond between asphalt and aggregate depends on surface chemical activity.

Water at the aggregate surface (in the field) is at a high pH, some liquids used as anti-strips require a long curing period (in excess of about 3 hours) to achieve resistance to loss of bond at higher pH levels, and it is possible to achieve a strong chemical bond between aggregate and asphalt cement that is resistant to pH shifts and/or a high pH environment. This strong chemical bond can be achieved by the formation of insoluble organic salts (such as calcium based salts) which form rapidly and are not affected by high pH levels or pH shifts.

2.5.7 Environmental Effects on the Aggregate–Asphalt System

Terrel and Shute (1989) Report that factors such as temperature, air, and water have a profound effect on the durability of asphalt concrete mixtures. In mild climates where good quality aggregates and good quality asphalt cements are available, the major contribution to deterioration is traffic loading and the resulting distress manifestations. Premature failure may result when poor materials and traffic are coupled with severe weather. Terrel and Al-Swailmi (1994) identify a number of environmental factors of concern: water from precipitation, ground water sources, temperature fluctuations (including freeze thaw conditions), and aging of the asphalt. They identify traffic and construction techniques, which are external to the environment, as important factors.

2.6 Evidence of Stripping

Preliminary evidence of stripping of asphalt pavement mixtures often occurs as patch bleeding, or flushing, and localized instability. Localized flushing occurs when stripped asphalt cement rises to the surface of the pavement, producing localized shiny areas of asphalt. This bleeding is not necessarily confined to the wheel paths but rather is often distributed randomly across the pavement surface. Deformations in the form of shoving and rutting may also develop due to the loss of structural Strength and stiffness and due to instability caused by the excessive amounts of asphalt which accumulate near the surface. Shoving can be expected in are as carrying only moderate traffic and rutting will begin to develop. In addition, it may be found that cores cannot be obtained due to the lack of cohesion and strength in the lower portion of the pavement layers. Examination of the asphalt aggregate mixture will often show that the aggregates are essentially clean, with minimal asphalt. (Thomas W. Kennedy and James N.Anagnos, 1984)

2.7 Factors Influencing Moisture Damage

As stated earlier, stripping of asphalt pavement is a complicated phenomenon related to a large number of variables. Those variables, which have been identified through years of study by different researchers those are; type and use of mix, asphalt characteristics, aggregate characteristics, environment, traffic, and construction practice, Mix design, including binder content, gradation, and dust-to-asphalt ratio, which can determine the film thickness on the aggregates and the permeability of the mix; binder selection, which determines the stiffness of the binder and the susceptibility to penetration of the asphalt film by water; and the use of additives, which can reduce the overall susceptibility of the mix;(Hicks 1991).

Asphalt Characteristics

Bitumen rheology during mixing and compaction seems to influence moisture sensitivity of bituminous mixtures. Viscosity is important because it may indicate higher concentrations of asphaltenes (large polar molecules).Viscosity needs to below enough to allow proper wettability of the aggregate by bitumen. During service, high viscosity offers better resistance to moisture damage than the converse. Lower viscosities, which may represent lower concentrations of asphaltenes, are generally more susceptible to stripping. (Majidz adeh etal.1968, Kiggundu etal.1988, and Hicks 1991). A high concentration of polar viscosity building bitumen components possibly increases resistance to moisture damage. Individual components in asphalt binder such as sulfur oxides, carboxylic acids, phenols and nitrogen bases can also affect stripping potential.

Aggregate Characteristics

In general, aggregates that are hydrophilic (attract water) are more likely to strip than aggregates that are hydrophobic (repulse water).To address this, either stripping susceptible aggregates can be avoided or an anti-stripping asphalt binder modifier can be used. Some of the aggregate physical properties that have been reported to influence moisture damage include surface roughness, porosity, shape, friability and presence and nature of adsorbed coatings. Good bonding is promoted by rough textured aggregate surfaces (Yoon etal.1988). As regards shape, sharp angular aggregates may rupture the bitumen or mastic creating avenues for water.

Surface Chemistry

Surfaces that can more readily form bonds with the asphalt binder are less likely to cause stripping. In general, a more acidic aggregate surface is more susceptible to stripping. Iron, magnesium, calcium and perhaps aluminum are considered beneficial, while sodium and potassium are considered detrimental (Hicks, 1991).

Porosity and Pore Size

Pore size is the critical factor. If pores are large enough to allow asphalt binder entry, they may be a contributor to moisture susceptibility. High porosity results in high absorption, which means that more asphalt binder, must be used to achieve the desired effective asphalt binder content. Conversely, if high porosity is not considered, for a given amount of asphalt binder, more will be absorbed and less will be available to create the asphalt binder film around aggregate particles causing faster aging and possibly stripping. (Hicks, 1991).

Environment

The environment of an asphalt pavement is largely responsible for whether or not stripping will occur. Variations in the environment, such as wetting and drying, freeze and thaw, and temperature fluctuations, have notable effects on the resistance of the pavement to moisture damage. It is helpful to examine the ways moisture can have access to the pavement in service so that provisions can be made during pavement design to minimize the amount of water available to the mixture. Schmidt and Grafnave shown that the rate and extent of moisture damage to an asphalt mixture is proportional to its water content (R.J.Schmidt and P.E.Graf,1972). If one can assume that the pavement surface is properly sealed to prevent infiltration of surface water, the movement of moisture in to the sub grade and base courses becomes of great concern with respect to stripping. It has been found that water can enter the subgrade on which an asphalt pavement is supported in one or more of the following ways: seepage from adjacent higher ground, rising of the water table, capillary rise from the water table (moisture suction), from the shoulder (moisture suction), and hydro genesis (vapor movement) (Asphalt Institute (1981),W.L. Hindermann (1968), H. Schmerl (1981)) Hydro genesis has been suggested as the primary cause of moisture entering granular bases in flexible pavements (W.L. Hindermann (1968)).Most stripping begins where the bottom most layer of an asphalt pavement meets a wet granular base (Asphalt Institute,(1981).It has been observed that asphalt pavements placed over un treated granular bases with well-designed and properly operating drainage have not stripped, even when mixtures were made with aggregates known to be prone to stripping (Asphalt Institute,(1981).

Traffic

If water is present in the HMA structure, increased traffic loading can accelerate moisture damage for two reasons: Pore pressure build up: If water is in the aggregate pores and cannot escape, traffic loading will tend to compress these pores and cause a pressure build up, which could push asphalt binder away from the aggregate surface. Hydraulic scouring: Wheel passes over a HMA pavement tend to move water in the pavement. This movement causes a scouring action that could remove asphalt binder from the aggregate surface. (Hicks 1991).

Construction Practice

Air Voids

Proper compaction of asphalt mixtures during pavement construction is a necessity in order to minimize the potential for stripping. The extent to which pores in the aggregate absorb asphalt binder affects the volume of air voids in the HMA mixture. When HMA air voids exceed about 8 percent by volume, they may become inter connected and allow water to easily penetrate the HMA and cause moisture damage through pore pressure. To address this, HMA mix design adjusts asphalt binder content and aggregate gradation to produce design air voids of about 4 percent. Excessive air voids can be either a mix design or a construction problem (Tunnicliff and Root,1982).

Weather Conditions during Construction

It has been shown that pavement construction in the late fall results in asphalt pavements that are more susceptible to stripping because of the likelihood of aggregates being damp and weather conditions being cool and wet (Tunnicliff and Root,1982). Regardless of the season, if rain immediately follows pavement construction, stripping of the pavement is more likely to occur because the asphalt viscosity remains low for several hours after paving operations cease.

Construction Variability

Including segregation, which can create areas with high air void contents and low binder contents, which permit water to enter and are more susceptible to moisture damage; variance from the job mix formula, which can create susceptible areas with less dense gradation and lower binder contents; and compaction, which can create areas with high air voids and therefore high permeability and low strength. (Tunnicliff and Root,1982).

2.8 Test Methods for Evaluating Moisture Susceptibility

2.8.1 Tests on Loose Mixtures

These are the tests conducted on asphalt-coated aggregates in the presence of water. Examples Include boiling water, film strip, and static/dynamic immersion tests. One advantage of these tests is that they are simpler and less costly to run than tests conducted on compacted specimens. Another advantage is that they require simpler equipment and procedures. The major disadvantage is that the tests are not capable of taking the pore pressure, traffic action, and mix mechanical properties in to account. The results are mostly qualitative, and interpretation of the results becomes a subjective matter depending on the evaluator's experience and judgment. There is also not much evidence correlating results from these tests to field performance of hot-mix asphalt concrete. Loose mixture tests are best used for comparison between different aggregate–asphalt mixtures in terms of compatibility, strength of adhesion, and stripping. Mixtures failing in these tests, on the basis of some established criterion, have the potential to strip and should be avoided. However, good results should not mean that a mix can be used, since the effects of the other contributing factors are not considered in these tests. Defining a pass/fail criterion is not an easy task for most of these tests. For example, visual evaluation is used in the static immersion test to determine the degree of stripping below or above 95 percent, a criterion that is not very repeatable between different operators and different laboratories.

Boiling Water Test

The Boiling Water Test (ASTM D3625) is a national test standard from the first category that visually evaluates a loose mixture after boiling in water for ten minutes. Numerous prior research studies have indicated that the Boiling Water Test is not an ideal test method since the results are subjective, but some reasonable correlation to field performance has been shown for Alabama mixtures even though, it failed to identify moisture susceptible aggregates for Alabama AC mixes. (Parkerand Wilson, 1986). Additionally, the Boiling Water Test does not take in to consideration the void structure, permeability, or gradation of the asphalt concrete mixtures (Aschenbrener etal, 1995). As a result, the Boiling Water Test is not a popular test method.

Static Immersion Test (AASHTOT 182)

Although this test is still continued as a standard method under AASHTO, it is no longer available as an ASTM standard (originally ASTM Standard Practice D1664).The asphalt–aggregate mixture is cured for 2h at 60°C and cooled to room temperature .It is then placed in a glass jar and covered with 600 mL of distilled water. The jar is capped and placed in a 25°C water bath and left undisturbed for 16 to 18h. The amount of stripping is visually estimated on the basis of the established criteria. The total visible area of the aggregate is estimated as either less than or greater than 95%. This is a major limitation of the test because the results are decided purely on the basis of a subjective estimate of less than or greater than 95%. Test results have indicated that placing samples at 60°C bath rather than 25°C for18h increases the amount of stripping.

Texas Boiling Test

The Texas boiling test procedure was developed by Kennedy et.al (1982; 1984) on the basis of the earlier work. The procedure requires adding asphalt–aggregate mixture to boiling water and bringing the water back to boiling after this addition. After 10 min, the mixture is allowed to cool while the stripped asphalt is skimmed away. The water is drained, and the wet mixture is placed on a paper towel and allowed to dry. Visual rating is conducted to assess the level of stripping. This test procedure is a quick method for evaluating the moisture sensitivity of an asphalt–aggregate mixture. However, it does not account for mechanical properties of the mix, and it does not include the effects of traffic action. The test is also subjective and qualitative, and results are judged on the basis of a visual rating. A useful application of the test could be for quick evaluation of various asphalt aggregate combinations as a relative measure of the bond quality and stripping resistance. The procedure has been standardized as ASTM D 3625 (Effect of Water on Bituminous-Coated Aggregate Using Boiling Water)

Net Adsorption Test

The net adsorption test (NAT) was developed under SHRP in the early1990s and is documented in SHRP Report A-341 (Curtisetal.1993). The test is used to determine the affinity and compatibility of an asphalt–aggregate pair and the sensitivity of the system to water. Therefore, it can be considered a screening test. The test comprises two steps. First, asphalt is adsorbed on to aggregate from a toluene solution, the amount of asphalt remaining in solution is measured, and the amount of asphalt adsorbed to the aggregate is determined. Second, water is introduced into the system, asphalt is desorbed from the aggregate surface, the asphalt present in the solution is measured, and the amount remaining on the aggregate surface is calculated. The amount of asphalt remaining on the surface after desorption is termed net adsorption. The net adsorption test offers a direct means of comparing the affinity of different asphalt–aggregate pairs. The test is relatively fast and easily performed. However, SHRP ReportA-341 provides mixed conclusions in terms of correlation between NAT results and moisture sensitivity results from indirect tension tests on compacted specimens. The NAT procedure was modified by researchers at the University of Nevada at Reno, and the test results were correlated with the ECS (Scholzetal.1994). The study by Scholz et.al (SHRP-A-402, 1994) indicates that predictions of the water sensitivity of the binder as proposed by NAT show little or no correlation to wheel-tracking tests on the mixes.

2.8.2 Tests on Compacted HMA Mixtures

These tests are conducted on laboratory compacted specimens or field cores or slabs. Examples include indirect tensile freeze–thaw cyclic with modulus and strength measurement, immersion–compression, abrasion weight loss, and Hamburg Wheel Tracking Device. The major advantage of these tests is that the mix physical and mechanical properties, water/traffic action, and pore pressure effects can be taken in to account. The results can be measured quantitatively, which minimizes subjective evaluation of test results. The drawback of these tests is that they require elaborate testing equipment, longer testing times, and more laborious test procedures are needed. Immersion–Compression Test ASTM D1075 (1949 and 1954) and AASHTO T165-55 (Effect of Water on Compressive Strength of Compacted Bituminous Mixtures) the immersion–compression procedure was originally published as ASTM D1075-49. Therefore, the test is among the first to be used for evaluation of moisture sensitivity. Revisions were made to the procedure in 1996. (Goode explains the test in detail in ASTM Special Technical Publication 252). Two groups of compacted Specimens are used in this test method. One group is submerged in a120°F water bath for 4days for conditioning, and the other group is maintained dry. An alternative approach to conditioning is to immerse the test specimens in water for 24h at 140°F. Compressive strength is measured on specimens of both groups at 77°F at a deformation rate of 0.05 in. /min per inch of height. For a 4-in.tall specimen, the rate would be 0.2in./min. The average strength of conditioned specimens over that of dry specimens is used as a measure of moisture sensitivity of the mix. Most agencies have used a 70% ratio as a passing limit.

Moisture Vapor Susceptibility

The moisture vapor susceptibility procedure was developed and has been used by the California Department of Transportation (California Test Method 307). Two specimens are prepared and compacted using the kneading compactor, as for mix design testing, except that they are prepared in stain less steel molds. The compacted surface of each specimen is covered with an aluminum seal cap, and a silicone seal antis applied around the edges to prevent the escape of moisture vapor. An assembly with a felt pad, seal cap, and strip wick is prepared to make water vapor available to the specimen by placing the free ends of the strip wick in water. After the assembly is left in an oven at 60°C with the assembly suspended over water for 75h, the specimen is removed and tested immediately in the Hveem Stabilometer. A minimum Hveem Stabilometer value is required, which is less than that required for the dry specimens used for mix design.

Original Lottman Indirect Tension Test

The original Lottman procedure was developed by Lottman at the University of Idaho in the late 1970s (Lottman1978). The procedure requires one group of dry specimens and one group of conditioned specimens. The specimens are 4in. in diameter and about 2.5in.thick. Conditioning includes vacuum saturation of specimens under 26in. of mercury vacuum for 30 min followed by 30min at atmospheric pressure. The partially saturated specimens are frozen at 0°F for 15 h followed by 24h in a140°F water bath. This is considered accelerated freeze–thaw conditioning. Lottman proposed thermal cyclic conditioning as an alternative. For each cycle, after 4h of freeze at 0°F, the temperature is changed to 120°F and maintained for 4h before being changed back to 0°F. Therefore, a complete thermal cycle lasts 8h. The specimens go through 18 thermal cycles of this type. Lottman concluded that thermal cycling was somewhat more severe than the accelerated freeze–thaw conditioning with water bath. Conditioned and dry specimens are both tested for tensile resilient modulus and tensile strength using indirect tensile equipment. The loading rate is 0.065 in./min for testing at 55°F or 0.150 in./min for testing at 73°F. The severity of moisture sensitivity is judged on the basis of the ratio of test values for conditioned and dry specimens.

AASHTOT283 (Modified Lottman Indirect Tension Test Procedure)

The AASHTO Standard Method of Test T 283, “Resistance of Compacted Bituminous Mixture to Moisture Induced Damage,” is one of the most commonly used procedures for determining HMA moisture susceptibility. The test is similar to the original Lottman with a few exceptions. One of the modifications is that the vacuum saturation is continued until a saturation level between 70% to 80% is achieved, compared with the original Lottman procedure that required a set time of 30min. Another change is in the test temperature and loading rate for the strength test. The modified procedure requires a rate of 2in./min at 77°F rather than 0.065in./min at 55°F. A higher rate of loading and a higher temperature were selected to allow testing of specimens with a Marshall Stability tester, available in most asphalt laboratories. The higher temperature also eliminates the need for a cooling system. Briefly, the test includes curing loose mixtures for 16h at 60°C, followed by a 2-h aging period at135°C. At least six specimens are prepared and compacted. The compacted specimens should have air void contents between 6.5% and 7.5%. Half of the compacted specimens are conditioned through a freeze (optional) cycle followed by a water bath. First, vacuum is applied to partially saturate specimens to a level between 55% and 80%. Vacuum-saturated samples are kept in a 18°C freezer for 16h and then placed in a 60°C water bath for 24h. After this period the specimens are considered conditioned. The other three samples remain unconditioned. All of the samples are brought to a constant temperature, and the indirect tensile strength is measured on both dry (un conditioned) and conditioned specimens.

State highway agencies report mixed success with this method. Several research projects have dealt with the method’s short comings, resulting in suggested “fixes”, but the test remains empirical and liable to give either false positives or false negatives in the prediction of moisture susceptibility. Major concerns with this test are its reproducibility and its ability to predict moisture susceptibility with reasonable confidence (Solaimanian and Kennedy 2000a).

The indirect tensile test a number of advantages were attributed to the test, as follows (Kennedy,T.W., and W.R.Hudson,1968)

- It is relatively simple,
- The types of specimens and equipment are the same as those used for compression testing,
- Failure is not seriously affected by surface conditions,
- Failure is initiated in a region of relatively uniform tensile stress,

- The coefficient of variation of the test results is low, and
- Mohr's theory is a satisfactory means of expressing failure conditions for brittle crystalline materials such as concrete.

Texas Freeze–Thaw Pedestal Test

The Texas freeze–thaw pedestal test was proposed by Kennedy et.al (1982) as a modification of the water susceptibility test procedure proposed by Plancher et.al (1980) at the Western Research Institute. The test is in the category of those evaluating the compatibility between asphalt binder and aggregate and the corresponding adhesiveness. The test is designed to minimize the effect of mechanical properties of the mix by using a uniform-sized aggregate. It prescribes the preparation of hot mix using a fine fraction of aggregate (passing the No.20 (0.85-mm) and retained on the No.35 (0.50-mm) sieve) and asphalt at a temperature of 150°C. The hot mix so prepared is kept in the oven at150°C for 2h and stirred for uniformity of temperature every hour. At the end of 2h, the mix is removed from the oven and cooled to room temperature, reheated to150°C, and compacted with a load of about 28kN for 15min to form a briquette 41mm in diameter by19mm in height (the procedure does not prescribe any tolerance for the dimensions). The briquette is cured for 3days at room temperature and placed on a pedestal in a covered jar of distilled water. It is then subjected to thermal cycling of 15h at12°C, followed by 9h at 49°C. After each cycle, the briquette surface is checked for cracks. The number of cycles required to induce cracking is a measure of water susceptibility (typically10 freeze thaw cycles). Pedestal test specimens are prepared from a narrow range of uniformly sized aggregate particles coated with 5% asphalt. This formulation reduces aggregate particle interactions in the mixture matrix, and the thin asphalt coating between aggregate particles produces a test specimen that is highly permeable and thus allows easy penetration of water into the interstices found between aggregate particles. Therefore, moisture induced damage in the specimen can easily arise either from bond failure at the asphalt–aggregate interface region (stripping) or from the fracture of the thin asphalt–cement films bonding aggregate particles (cohesive failure) by formation of ice crystals.

Loaded Wheel Testers

The Asphalt Pavement Analyzer (AASHTO TP63)

Utilizes a repeated loading test device on specimens in saturated conditions and compares them to unconditioned specimens tested dry. The test criteria is the ratio of conditioned rut depth to un conditioned rut depth, with values greater than 1 suggesting the mixture is moisture susceptible. Similar to the HWTT, the Asphalt Pavement Analyzer (APA) allows for a maximum rut depth. Bausano et.al (2006) indicated that the APA testing of saturated mixtures is capable of identifying moisture susceptibility, simulating the repeated hydraulic loading that pavements undergo with desirable testing efficiency.

Hamburg Wheel Tracking Device (HWTD)

The Hamburg Wheel Tracking Device (HWTD) is one of the load wheel testers in the United States that is used to evaluate rutting in Asphalt mixtures. It was originally developed by the City of Hamburg, Germany in 1970 based on a similar British device that had a rubber tire (Yetkin Yildirim etal.2006). Helmut-Wind Incorporated of Hamburg finalized the test method and developed specification requirements for rutting and stripping susceptibility (Yetkin Yildirim etal.2006). HWTD was used as a Specification requirement for some of the most traveled road ways in Germany to evaluate rutting and stripping (Cooley L.Allen Jr.etal 2000).The HWTD is used to measure the combined effects of rutting and moisture damage on asphalt mixtures by simulating the real pavement conditions such as heavy rain and heavy use in a controlled environment. It has two stainless steel wheels (8-in diameter x1.85 in wide) that simulate the road wear by rolling and pressing against the samples with158pounds of contact pressure (Harris A. Jeffery 2011).Tests in HWTD are conducted on a slab whose dimension is 260mm wide,320mm long, and 40mm high (10.2in x 12.6in x 1.6in). The slabs are normally compacted to 7±1 percent air voids using a linear kneading compactor. HWTD was slightly modified by the Superpose Construction, U.S. and was referred to as the Superfos Construction Rut Tester (SCRT).The SCRT used slab specimen like the HWTD with the same dimensions.

The difference between the two rut testers was the loading mechanism in which the SCRT used a 180-lb. vertical load on to a solid rubber wheel with diameter of 194mm and width of 46mm. Another slight modification of the HWDT is the evaluator of Rutting and Stripping (ERSA) equipment which was built by the Department of Civil Engineering at the University of Arkansas. Testing within the ESRA is conducted on either cylindrical or beam samples in dry or wet conditions Each wheel is connected to a separate Linear Value Displacement Transducer (LVDT) to measure the deformation of the samples with each wheel pass. Testing in HWTD is done under water with temperatures of 25 to 70 degrees Celsius 50 being the most common. About an inch of water is kept above the specimen during the test. The temperature is kept constant with a water bath that has two heaters at either side. Tests are typically run up to 20,000 loaded wheel passes at a rate of approximately 50passes a cross the specimen per minute [AASHTO T324-04,2005].The results obtained from HWTD include: rut depth, creep slope, stripping inflection Point, and Stripping slope. The creep slope is the inverse of the deformation rate within the linear region of the deformation curve after post compaction and prior to stripping (if stripping happens). The stripping slope is the inverse of the deformation rate within the linear region of the deformation curve, after the onset of stripping. The stripping, inflection point is the number of wheel passes corresponding to the intersection of the creep slope and the stripping slope. This point indicates the relative resistance of the HMA sample to moisture induced damage [Yetkin Yildirim etal.2006].

The Environmental Conditioning System (ECS)

During SHRP to more realistically simulate field conditions using repeated hydraulic loading and repeated load cycles. This ECS system was utilized with a retained resilient modulus ratio with and without multiple moisture conditioning cycles (vacuum saturation, hot water, and optional freeze cycle) (AASHTO P34) (Terrel and Al-Swailmi, 1994). This nondestructive test parameter was measured after each moisture conditioning cycle, and specifications require a minimum retained resilient modulus of 70 % of conditioned specimens to unconditioned specimens. Several modifications to the original ECS conditioning parameters and Mq measurement protocols have been made to provide a better correlation between test results and field performance (Aschenbrener etal., 1995; Alam etal., 1998). More recently the ECS was evaluated for use with another nondestructive test, the compressive dynamic modulus (Ex) test (AASHTO TP62), to again assess the effects of moisture conditioning on the same specimen (Solaimanian eta1.,2006). The E* stiffness ratio (ESR) with and without ECS conditioning at reported threshold between 75 and 80% showed good correlation to field performance for mixtures from multiple states, but further work to simplify and shorten the testing protocol and/or add an evaluation of the effects of moisture and load separately and/or possibly continuously monitor mixture response during conditioning was recommended (Solaimanian eta1., 2007). Nadkarni etal. (2009) also utilized the ESR but with and without AASHTO T283 moisture conditioning and recommended a minimum required retained ESR of 70% for conventional mixtures in Arizona. Bausano and Williams (2009) also utilized and recommended the ESR with AASHTO T283 moisture conditioning (with a minimum retained E*ratio of based on an equivalent percentage of AASHTOT283 results below the 80% limit for different mixtures from Iowa), but they tested the conditioned specimens in a saturated condition. A recently completed extensive laboratory testing program for the NLAt racked Et with up to 15 multiple F/T cycles (AASHTO T283) for mixtures from multiple states with arrange in performance and found that these results correlated with other moisture susceptibility results (Sebaaly etal.,2010).

Georgia Loaded Wheel Tester

Developed during the mid-1980s through a cooperative research study between the Georgia Department of Transportation and the Georgia Institute of Technology, the GLWT is capable of testing HMA beam or cylindrical specimens. Beam dimensions are generally125mm wide, 300mm long and75mm high. The cylindrical samples are generally 150mm in diameter and75mm high.

Both specimen types are commonly compacted to either 4 or 7 percent air void content. Testing consists of applying 100-lb. load on to a pneumatic linear hose pressurized to 100psi. The load is applied through an aluminum wheel on to the linear hose, which resides on the sample. The test specimens in GWLT are tracked back and forth under the applied stationary loading. Typically, testing in GLWT is run for 8000 cycles under temperatures of 35 to 60 degree Celsius (Cooley L.Allen Jr.etal 2000). Previous experimentation done by Elizabeth Rae Hunter and Khaled Ksaibati (2002) used GLWT to test 6in (150mm) diameter cores with in a concrete frame some cores unconditioned and others conditioned and concluded that testing the cores in a saturated and freeze-thaw conditioning with GWLT was not an effective measuring for moisture damage because found that Conditioning of the cores did not contribute significantly to performance of the cores in the GLWT.

DYNA-TRACK Single Wheel Tracker

The DYNA-TRACK Single wheel tracker was manufactured by Controls group using EN standard. It consists of a loaded wheel which bears on a sample held on a moving table. The wheel is fitted with a solid rubber tire of outside diameter 200mm. The wheel load under standard conditions is 700+10 N. The wheel trucker is fitted with a temperature controlled cabinet with a temperature range from environment to 65^{0}C±1.0^{o}C. The sample may be either a 200mm diameter core or a 300m by 400m slab of asphaltic mixture from 25mm to100mm thick. A 25mm stroke LVDT transducer is included for monitoring rut depth in the center of a sample during a test to better than 0.1mm. The deformation and sample temperature is recorded by the internal data acquisition and control system then sent to computer.

2.9 Prevention of Moisture Damage

There are some techniques for limiting moisture susceptibility of mixtures and alleviate distress due to stripping. Provide adequate compaction, eliminate the use of moisture-susceptible aggregates and asphalts, provide adequate drainage, seal the asphalt-aggregate mixture surfaces, and treating the moisture-susceptible aggregates and asphalt are some of them.(Kennedy, T.W., and J.N. Anagnos, 1983), providing adequate compaction will reduce the air voids and the continuity of the air void system. This prevents the penetration of moisture into the mixture, thus reducing the possibility for stripping to occur. The air void content should, ideally, be less than 7 percent. At void contents in excess of 7 percent, water can readily penetrate the mixture. Thus, compaction should achieve a relative density of at least 93 percent of the theoretical maximum density.

Eliminating Moisture Susceptible Material

It may be desirable to eliminate the use of certain moisture susceptible aggregates and, to a lesser extent, certain asphalts. Such an approach may be costly, especially in areas with limited aggregate and asphalt sources. Nevertheless, in view of the long-term maintenance requirements, reduced pavement life and performance, and, in some cases, the rapid and severe failure of the pavement, it may in reality be the most economical solution if adequate protection cannot be achieved or if the mixture cannot be adequately protected.

Providing Adequate Drainage

Drainage should be provided to eliminate moisture, which causes stripping to occur. This involves rapid removal of surface water and prevention of moisture movement into the mixture from the sub grade, sub base, and base by drainage of these layers and by maintaining an adequate pavement elevation above the water table. The use of open-graded friction courses has been found to cause stripping by allowing moisture to enter the underlying layers under the action of traffic, especially if the moisture cannot readily drain laterally.

Sealing Mixture Surfaces

Both the top and the bottom surface of the asphalt mixture can be sealed to prevent moisture penetration or may be sealed for other reasons. This approach requires that careful consideration be given to the source of moisture to avoid the possibility of trapping water in the mixture. A number of cases in Texas and other states have been reported in which a surface seal was placed on an existing road way resulting in subsequent rutting and deterioration due to stripping. Thus, surface sealing may prevent evaporation of moisture from underlying layers which is moving upward through the mixture, and similarly, sealing of the bottom surface may trap surface water by preventing drainage into the underlying layers. It should also be noted that surface sealing is generally only a temporary preventative measure since cracks will ultimately reflect through the seal. Highly moisture susceptible mixtures will tend to fail rapidly along the cracks. Thus, sealing is, at best, a temporary method of controlling moisture damage and may in fact cause the pavement to fail if moisture is entering the mixture from underlying layers.

Treating Materials

A number of additives have been proposed for treating the aggregate and the asphalt, with the Primary emphasis placed on treatment of the aggregate. These additives are Commercial liquid anti-stripping agents, Portland cement, and Hydrated lime.

Anti-Stripping Agents

Anti-stripping agents may be necessary if a particular mix design has been shown to be susceptible to moisture induced damage. Liquid anti-stripping agents and lime additives are among the most commonly used types of anti-stripping agents. However, if an additive is used when it is not needed or if it is used incorrectly, adverse effects may occur, including an increased economic cost and early maintenance and/or rehabilitation [TunnicliffandRoot1984].

Lime additives

The anti-stripping mechanism of lime additives is not well understood. However, lime additives are an accepted method of minimizing moisture susceptibility of a mix. The general practice is to add 1 to 1. 5 percent lime by dry weight of aggregate to the mix. If an aggregate has more fines present, it may be necessary to use more lime additive due to the increased surface area of the aggregate.

Three forms of lime are used: hydrated lime (Ca (OH) 2), quicklime (CaO), and Diplomatic limes (both types S and N) [Robertsetal.1996]. Several methods exist for adding lime to mixtures. Dry hydrated lime is added prior to the asphalt cement. Georgia DOT adds the dry hydrated lime immediately before the asphalt cement is added [Robertsetal.1996]. However, there is a problem maintaining the coverage until the asphalt cement is added. Using hydrated lime slurry will increase the amount of water needed and the fuel costs of production. Adding dry hydrated lime to wet aggregate has the same results as hydrated lime slurry. Hot (quick lime) slurry is equivalent in cost to hydrated lime, but when slaked, there is a 25percent higher hydrated lime yield. Also, the elevated temperature during slaking helps to evaporate some of the added moisture [Robertset.al1996].

To evaluate the properties of bituminous mixtures containing hydrated lime, Mohammad, Abadie, Gokmen and Puppala [2000] studied TSR values, rutting and resilient modulus. Mohammad, Abadie,Gokmen and Puppal a found that if the hydrated lime was added as mineral filler, the permanent deformation and fatigue endurance improved. Also, test results illustrated that adding lime increased the tensile strength of HMA mixtures. Field and laboratory testing conducted by Kennedy and Anagnos (A Field Evaluation of Techniques for Treating Asphalt Mixtures with Lime 1984) found that dry lime and lime slurry improved moisture resistance. However, lime slurry had a better performance than dry lime. Adding the lime in a drum mix plant was in effective because much of the lime was lost before mixing with the asphalt. Washing the aggregate before it was used, reduced the moisture resistance of mixture.

Liquid Anti-Stripping Agent

Liquid anti-stripping agents are chemical compounds that contain amines. Most anti-stripping agents reduce surface tension between the asphalt and aggregate in a mixture [Tunnicliff etal.1984]. When surface tension is reduced, increased adhesion of the asphalt to the aggregate is promoted. Thus, most liquid anti-stripping agents are surface active agents [Robertsetal.1996]. An economical method of mixing the liquid anti-stripping agent with the asphalt is by heating the asphalt to a liquid state. However, a more successful method of adding the additive is to apply it directly to the aggregate prior to the addition of the binder [Kennedy, Roberts, Lee1983]. It is important that the liquid anti-stripping agent is heat stable. The liquid asphalt commonly is mixed with the liquid anti-stripping agent prior to adding aggregate to the mix [Roberts’s etal.1996].

Amine-organic compound whose functional group containing a N atom with a lone pair of e-and at least one H atom replaced with an alkyl or aryl group (hydrocarbons)

Types of Amines in Liquid Anti-Strip Chemistry

Polyamines: compound with two or more amine functional groups. Heavies five or more functional groups per molecule, large molecules, vary in size. Many different types of polyamines differ in number & types of amine functional groups, size of hydrocarbon chain. Highly effective, lower odor Tetra ethylene pent amine (TEPA).Bis hexa methylene triamine (BHMT) polyamine, produced during nylon production. Commonly used compound in anti-strip in the past. Effective, but acrid odor. Fatty (tallow) amines derived from processing fatty deposits of animals. Tallow demine, tallow thiamine. Older type of amine anti-strip, engineered to have long chain hydrocarbon. Generally less affective compared to newer liquid anti-strip technologies amid amines created by reacting polyamines with fatty acids (carboxylic acid with hydrocarbon tail). Fatty acids derived from natural oils (coconut oil, tall oil) creates much larger molecule and substantially lengthens hydrocarbon chain of amine molecule .In some cases, performance equal to better than polyamines. Larger molecule can enhance heat stability. Different combinations of polyamines and fatty acids under varying reaction conditions yield amid amines with different anti-strip performance characteristics.

Table 1 some of available commercial anti- stripping agent

Manufacturer and supplier	**Anti-stripping agent**	**Remark**
Petrochem specialties	DERBO and WETBOND	liquid
AD-HERE	AD-here HP Plus	liquid
Opal	Super Bond	powder
HINCOL	HINCOL BITUGRIP	powder
Diamantes and chemical limited	Ethylene diamante	liquid

CHAPTER 3: METHODOLOGY

For this research the type of method that is used is experimental method. There are three main parts of this experiment. The first part includes investigation of properties of asphalt binder and aggregate. The second part consists of preparation of mix design for asphalt binder and aggregate. And the third part investigates the sensitivity of the mix for moisture.

3.1 Materials and methods

The experiment involved one asphalt binder type and two types of aggregates. The asphalt binders used were a penetration grade 85/100. The 85/100 penetration grade is a common type of asphalt used in Addis Ababa roads since the climate is relatively mild. The materials used in this study were obtained from Addis Ababa city road authority (AACRA) Kality asphalt plant and Fannuel asphalt plant site. Crushed gravel from natural basalt rock was the two aggregate types used for fabricating the asphalt concrete mixtures thus because of their origin and mineral composition their physical properties vary a great deal. The aggregate gradation and the asphalt mix designed to meet the requirements for manuals of Ethiopian Road Authority (ERA), Addis Ababa City Road Authority (AACRA), ASTM and AASHTO.

3.2 Aggregate Test and Preparation

3.2.1 Determination of Specific Gravity and Absorption of Aggregates

Specific gravity & Absorption of coarse aggregates (AASHTOT-85) both the mass and volume of the aggregates must be known in order to prepare the hot mix asphalt design. The mass and volume of the aggregates are related through the values of density or specific gravity. Thus, this test was conducted to determine the bulk specific gravity and absorption of the coarse aggregate. The bulk specific gravity of the coarse aggregate was calculated from equation.

$$\text{Bulk specific gravity} = \frac{A}{B - C} \quad \text{[3.1]}$$

Where: A = mass of oven dry specimen (g)

B = mass of saturated surface dry specimen (g);

C = mass of saturated specimen in water (g).

Similarly, based on the above data, the water absorption of the coarse aggregate was calculated using Equation

Absorption (%) = $\left[\frac{B-A}{A}\right] * 100$ …………………………………………………………… [3.2]

Specific gravity & Absorption of Fine aggregates (AASHTOT-84) this test was conducted to determine the bulk specific gravity and absorption of the fine aggregate or aggregate passing no 4 sieve. In this test, one kg of dry fine aggregate was submerged into water for 15 to 19hours. Then, the specimen was dried to a surface dry condition. The cone test was used to determine whether the fine aggregate is in the SSD condition. After cone tamping test, half part of fine aggregate was placed into a pycnometer. Then the pycnometer was partially filled with water and agitated using a vacuum pump to remove air bubbles. The bulk specific gravity of fine aggregate was calculated from equation

Bulk specific Gravity = $\frac{A}{(B+S-C)}$ ……………………………………..………………... [3.3]

Where: A= mass of oven dry specimen in air (g);

B = mass of pycnometer filled with water (g);

C= mass of pycnometer with specimen and water (g);

S= mass of saturated surface dry specimen (g).

Similarly, the percentage of absorbed water in to the pores aggregate was computed using Equation

Absorption (%) = $[\frac{S-A}{A}] * 100$…………………………..……………………… [3.4]

3.2.2 Soundness Test

The soundness test helps to describe the ability of aggregates to withstand the effects of weathering (or freezing and thawing) .the resistance to disintegration of aggregate is determined by soaking the specimen in saturated solutions of sodium sulphate. Based on AASHTO T 104, aggregate samples were washed and dried to a constant mass and then sieved under various sieve sizes. Then, the samples were repeatedly immersed in sodium sulphate solution followed by oven drying. Totally, the aggregate samples were undergo five cycles of immersion. After completion of the fifth cycle, immerse the test samples in a continuous flow of fresh water at 110^0F ± 10^0F until all of the sodium sulfate has been removed. Oven-dry and cool to room temperature. Sieve each sample to refusal over a sieve having square openings half the size of the sieve on which the aggregate was originally retained. Weigh the particles retained on this sieve and record the weight. The results were reported as the total percentage loss of material.

3.2.3 Los Angeles Abrasion Test

According to AASHTO T 96, The Los Angeles Abrasion test was conducted to measure the resistance to degradation of mineral aggregates resulting from a combination of actions including abrasion or attrition, impact and grinding. Abrasion value is the percentage of aggregate weights that passing sieve (No. 12 = 1.7mm) after application of standard abrasion by mechanical rotation parallel with standard iron balls for a dry aggregate.

3.2.4 Sand Equivalent Test

The sand equivalency test (AASHTO designation: T 176) was conducted to determine the amount of dust or claylike particles in the fine aggregate gradation. To conduct this test, fine aggregate was soaked in stock calcium chloride solution in a graduated cylinder. The sample was agitated or mixed thoroughly to separate the clay like or plastic fines from the sand-like particles. After sedimentation, the heights of sand and clay were measured and the ratio of sand to clay or simply sand equivalent value was expressed as a percentage.

3.2.5 Flakiness Index Test

(ASTM designation: D 4791) The flakiness index test was used to determine the percentage of flaky particles in the coarse aggregate. This test was important because flaky aggregates in hot mix asphalts have tendency to break along their thin and weak axis during compaction. The flakiness index of an aggregate sample is found by separating the flaky particles and expressing their mass as a percentage of the mass of the sample tested.

3.2.6 Aggregate Gradation Test

Gradation is one of the most important properties of an aggregate. It affects the stability and the durability of the HMA mixes. Therefore, gradation is a primary consideration in asphalt mix design. Gradation is usually determined by sieve analysis. According to AASHTO T 27, Sieve analysis was conducted by passing the aggregates through a series of sieves that have various sieve openings.

Then, the materials retained on each sieves were weighed, and the gradation was expressed as total percent passing at various sieve sizes. The aggregates used in this study were obtained from Addis Abeba city road authority (AACRA) kality asphalt plant and Fannuel asphalt plant site. Crushed gravel from natural basalt rock was the two aggregate types used for fabricating the asphalt concrete mixtures thus because of their origin and mineral composition their physical properties vary a great deal.

3.3 Conventional Tests on Asphalt Binder

3.3.1 Penetration Test

The consistency of an asphalt binder test is measured from penetration test. The instrument that is used for this test is called Penetrometer. According to AASHTO T 49, the asphalt binder sample was heated to an appropriate pouring temperature and poured into a test container and allowed to cool in air for 1 hour meanwhile the water bath will be maintained to a temperature of 25°C. After let the sample to cool in air, it was placed in a water bath for 1 hour and 30 minutes. After the specified conditioning period, a 100 gm weight was be attached to the standard test needle and allowed to penetrate the sample vertically at 25°C for 5 Seconds at 3 different locations 1 cm apart from each other. The Penetration value was taken as the average of the three values.

3.3.2 Softening Point (Ring and Ball Method)

The softening point of asphalt binder is the temperature at which the substance attains particular degree of softening. The test is useful in determining the consistency of asphalt binder. According to AASHTO T 53, the asphalt binder sample was heated until it has become sufficiently fluid to pour. The heated samples were poured into two rings, preheated to approximately the pouring temperature. The samples were then cooled for 30 minutes. After that, the excess material was then cleaned with a warm spatula. The apparatus was assembled with the rings, thermometer, and a steel ball weighing 3.5 gram centered in position. The beaker was then filled with distilled water to a height of 50 mm above the upper surface of the rings. Heat was applied to the beaker. The temperature at which the second plate touches the bottom plate was recorded. The average of the 3 samples were then taken and rounded to the nearest whole degree.

3.3.3 Ductility

Ductility of asphalt binder is its property to elongate under traffic load without getting cracked in road construction works. Some Asphalt binders having a high degree of ductility have also been found to be more temperature-susceptible and binder material having insufficient ductility gets cracked when subjected to repeat traffic loads and it provides pavement surface that is pervious. According to AASHTO T 51, the asphalt binder was melted at a temperature of 75°C to 100°C above the approximate softening point until it becomes thoroughly fluid. After stirring the fluid, it was poured in the mold assembly and placed on a brass plate. After about 30-40 minutes, the plate assembly along with the sample will be placed in a water bath; maintained at a temperature of 27°C for half an hour. Then the excess asphalt binder was trimmed with a hot spatula. The test specimen was then placed in the ductility water bath and conditioned to the desired test temperature. After the side pieces of the briquette were detached, the ductility machine was switched on to pull one end the specimen away from the other at a specified rate of speed. The distance in centimeters to which it elongates before breaking was measured. The average of three samples was taken.

3.3.4 Thin film oven Test

When asphalt cement is used in the production of asphalt concrete, it has to be heated to an elevated temperature and mixed with a heated aggregate. The hot asphalt mixture is then hauled to the job site, placed and compacted. By the time the compacted asphalt concrete cools down to the normal pavement temperature, significant hardening of the asphalt binder has already taken place. The properties of the asphalt in service are significantly different from those of the original asphalt. Since the performance of asphalt concrete in service depends on the properties of the hardened asphalt binder in service rather than the properties of the original asphalt, the properties of the hardened asphalt in service need to be determined and controlled.

According to AASHTO T 179, The Thin Film Oven Test (TFOT) procedure was developed to simulate the effects of heating in hot mix plant operation on asphalt cement. In the standard TFOT procedure, the asphalt cement sample is poured into a flat bottomed pan to a depth of about 3.2 mm. The pan with the asphalt sample in it is then placed on a rotating shelf in an oven and kept at a temperature of 163 ^{0}C for five hours. The properties of the asphalt before and after the TFOT procedure are measured to determine the change in properties that might be expected after a hot mx plant operation.

3.3.5 Flash and Fire point

Flash point is the temperature to which asphalt binder may be heated without the danger of causing an instantaneous flash in the presence of an open flame and Fire point is the lowest temperature at which a sample will sustain burning for 5 second. The apparatus that was be used is the Cleveland

Cup. According to AASHTO T 48, after the sample was heated between 75 to 100°C, a brass cup was filled partially with asphalt binder and was heated at a given rate. A flame was passed over the surface of this cup periodically and the temperature at which this flame causes an instantaneous flash was reported as the flash point. After Flash point, to determine the fire point heating the sample was continued so that the sample temperature increases at a rate of 5 to 6°C. The application of the test flame was continued at 2°C intervals until the sample ignites and continues to burn for at least 5 second. The temperature at that point was recorded as the fire point. The average of the 3 samples was taken and rounded to the nearest whole degree.

3.3.6 Solubility

Based on AASHTO T 44, the solubility test is used to detect contamination in asphalt cement. In the standard test for bitumen content, a small sample of about 2 g of asphalt is dissolved in 100 ml of trichloroethylene and the solution is filtered through a filtering mat in a filtering crucible. The material retained on the filter is then dried and weighed, and used to calculate the bitumen content as a percentage of the weight of the original asphalt. Specifications for asphalt cements normally require a minimum solubility in trichloroethylene of 99.0 percent.

3.4 Marshall Mix Design Method (AASHTO T-245)

The Marshall method of mix design is the pre dominant mix design method in Ethiopia. For a single selected aggregate gradation, five different asphalt contents are tested for various volumetric and strength criteria to select the optimum binder content.

3.4.1 Stability and Flow

Marshall Stability is the peak resistance load obtaining during a constant rate of deformation and Marshall Flow is a measure of the deformation of the specimen determined during the stability test. After measuring the bulk specific gravity of the mix, the mix specimens were immersed in a water bath at 60^0c for 35 minutes. Each specimen was then removed from the water bath and both stability and a flow test was measured using Marshall Stability machine.

3.4.2 Volumetric Analysis

Bulk Specific Gravity of a Compacted Mix (AASHTO T166) Gmb, is the mass of the sample divided by the mass (volume) of water it displaces. The bulk specific gravity of the mix (AASHTO T166) was determined by measuring the weight of each specimen in air, in water and in saturated surface dry condition.

$$Gmb= \frac{A}{B-C} \qquad [3.5]$$

Where

Gmb= bulk specific gravity of compacted specimen

A = mass of the dry specimen in air, g

B = mass of the saturated surface-dry specimen in air, g, and

C = mass of the specimen in water, g

Theoretical Maximum Specific Gravity (AASHTO T-209)

The theoretical maximum specific gravity of an asphalt mix is the specific gravity of the mixture at zero air void content. The theoretical maximum specific gravity was determined at each asphalt content. So that the percentages of air voids for each asphalt content were evaluated accordingly.

$$G_{mm} = \frac{A}{(A+B-C)} \qquad [3.6]$$

where

Gmm = theoretical maximum specific gravity of loose mix

A = mass of loose mix in air

B = mass of flask filled with water

C = mass of flask + sample + water

Bulk Specific Gravity of Total Aggregates (MS -2)

The bulk specific gravity of total aggregates was determined by considering the bulk specific gravities of individual coarse aggregate, fine aggregate and mineral filler fractions. The bulk specific gravity of total aggregate was computed as

$$G_{sb} = \frac{P1+P2+\cdots+Pn}{\frac{P1}{G1}+\frac{P2}{G2}+\cdots+\frac{Pn}{Gn}} \qquad [3.7]$$

Where,

Gsb = bulk specific gravity for the total aggregate

P1, P2, Pn = percentages by weight of aggregates 1,2,n

G1, G2, Gn = bulk specific gravities of aggregates 1,2,n

Effective Specific Gravity of Aggregate (MS -2)

The effective specific gravity of the aggregate, Gse, includes all void spaces in the aggregate particles except those that absorb asphalt.

$$\mathbf{Gse} = \frac{\mathbf{Pmm - Pb}}{\frac{\mathbf{Pmm}}{\mathbf{Gmm}} - \frac{\mathbf{Pb}}{\mathbf{Gb}}} \quad [3.8]$$

Where, Gse = effective specific gravity of aggregate.

Gmm = theoretical maximum specific gravity of loose mix (ASTM D2041) (no air voids).

Pmm = percent by weight of total loose mixture (= 100)

Pb = asphalt content (percent by total weight of mixture) at which ASTM D2041 test (Gmm) was performed.

Gb = specific gravity of bitumen

Percent Air Voids

The air voids, Va, in a compacted paving mixture consist of the small air spaces between the coated aggregate particles. The percentage of air voids in a compacted mixture can be determined by the following equation:

$$Va = 100- \frac{\mathbf{G_{mm} - G_{mb}}}{\mathbf{Gmm}} \quad [3.9]$$

Where:

Va = air voids in compacted mixture, percent of total volume.

Gmm = maximum specific gravity of paving mixture

Gmb = bulk specific gravity of compacted mixture.

The voids in the mineral aggregate, VMA, are defined as the inter granular void space between the aggregate particles in a compacted paving mixture, that includes the air voids and the effective asphalt content, expressed as a percent of the total volume.

$VMA = V_a - V_{be}$ or

$$\mathbf{VMA = 100 - \frac{G_{mb} P_s}{G_{sb}}} \quad [3.10]$$

Where:

VMA = voids in mineral aggregate (percent of bulk vol.)

Gsb = bulk specific gravity of aggregate.

Gmb = bulk specific gravity of compacted mixture.

Ps = aggregate, percent by total dry weight of mixture.

Percent Voids Filled with Asphalt, VFA (MS -2)

The voids filled with asphalt are the percentage of void spaces between the aggregate particles that are filled with asphalt. VFA does not include the absorbed asphalt.

$$\textbf{VFA} = \left[\frac{\text{VMA} - \text{Va}}{\text{VMA}}\right]*100 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots [3.11]$$

Where,

VFA = voids filled with asphalt, percent of VMA.

VMA = voids in mineral aggregate, percent of bulk volume.

Va = air voids in compacted mix, percent of total volume

Binder Content

Binder content is one of the most important characteristics of asphalt concrete. Use of the proper amount of binder is essential to good performance in asphalt concrete mixtures. Too little binder will result in a dry stiff mix that is difficult to place and compact and will be prone to fatigue cracking and other durability problems. Too much binder will be uneconomical, since asphalt binder is, by far, the most expensive component of the mixture and will make the mixture susceptible to rutting and shoving.

Typical asphalt binder contents range from 3.0% or less (for lean base course mixtures) to over 6.0% (for surface course mixtures and rich bottom layers), which are designed for exceptional durability and fatigue resistance

$$\mathbf{P_b = 100}* \left[\frac{M_b}{M_S + M_b}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots [3.12]$$

Where

Pb= Total asphalt binder content, % by mix mass

Mb= Mass of binder in specimen

Ms = Mass of aggregate in specimen

Binder Absorption

The percent binder absorption (P_{ba}) is the percentage by mass of binder that is absorbed into the aggregate.

$$P_{ba}= 100 * \frac{(Gse-Gsb)}{(Gse*Gsb)} * G_b \ldots\ldots [3.13]$$

Where,

Pba = absorbed asphalt, percent by weight of aggregate.

Gse = effective specific gravity of aggregate.

Gsb = bulk specific gravity of aggregate.

Gb = specific gravity of bitumen.

Effective Binder Content of the Mix

The effective binder content (P_{be}) of a paving mixture is the percentage by mass of binder that stays on the outside of aggregate particles and is not absorbed. It is effective or usable, as"glue" that binds the mix together and governs the performance of an asphalt paving mixture. The effective asphalt content of a bituminous mix is the total asphalt content minus the amount of asphalt lost by absorption into the aggregate particles.

$$\mathbf{P_{be}= P_b - [\frac{P_{ba}}{100} - P_s]} \ldots\ldots [3.14]$$

Where, P_{be} = effective bitumen content, percent by total weight of mix.

P_b = bitumen content, percent by total weight of mix.

P_{ba} = absorbed bitumen, percent by weight of aggregate.

P_s = aggregate content, percent by total weight of mix.

3.4.3 Optimum Binder Content

As an initial starting point it is recommended that the bitumen content giving 4% air voids is chosen as the design bitumen content. All of the calculated and measured mix properties at this bitumen content are determined by interpolation from the graphs .The individual properties are then compared to the mix design criteria as specified in ERA manual.

Table 2 Mix design criteria as specified in ERA manual

Mix property	MS-2 criteria
Va	3-5%
VMA	13%minimum
VFB	65-75%
Stability	8KN minimum

3.5 Indirect Tensile Test (ITS)

The indirect tensile test measures change in tensile strength resulting from effects of saturation and accelerated water conditioning of compacted HMA in the laboratory. The results may be used to predict long-term stripping susceptibility of bituminous mixtures and to evaluate the damage on tensile strength of the mix. The numerical indices of retained indirect tensile properties are obtained by comparing the retained indirect properties of conditioned laboratory specimens with the similar properties of dry specimens.

The Modified Lottman Test, which was standardized as AASHTO T283 (AASHTO 2000) “Resistance of Compacted Bituminous Mixture to Moisture Induced Damage,” is one of the most commonly used procedures for determining HMA moisture susceptibility. The indirect tensile test involves loading a cylindrical specimen with compressive loads which act parallel to and along the vertical diametrical plane, as shown in Fig1. To distribute the load and maintain a constant loading area, the compressive load is applied through a ha1f-inch wide stainless steel loading strip which is curved at the interface with the specimen and has a radius equal to that of the specimen. This loading configuration develops a relatively uniform tensile stress perpendicular to the direction of the applied load and along the vertical diametrical plane, which ultimately causes the specimen to fail by splitting or rupturing along the vertical diameter.

3.5.1 Equipment

The basic testing apparatus includes loading equipment capable of applying compressive loads at a controlled deformation rate, preferably 2inches (50mm) per minute, a means of measuring the applied load, and half-inch-wide curved face loading strips, which are used to apply and distribute the load uniformly along the entire length of the specimen.

Any loading equipment which is capable of applying compressive loads at a prescribed loading rate and can provide an accurate measure of the maximum load can be used, providing that the load capacity of the equipment is sufficient to fail the specimen. In addition, since it is necessary to apply the load to the specimen through steel loading strips which must remain essentially parallel, it is recommended that a guided loading head with loading strips attached to the upper and lower parallel platens be used

3.5.2 Test procedure

Sample preparation

Enough material is mixed to produce at least eight specimens approximately 1200gm each at the optimum binder content recommended for the mixture. Extra mixture will be needed for trials to establish the compaction required and for determining the maximum specific gravity of the mixture. After mixing, the mixture is placed in the pans and spread to about 1in. (25mm) thick. The mix is then cooled to room temperature for 2±0.5hours. The mixture is placed in the oven for 2hours at 275 ±5°F(135±3°C), and stirred every 60±5 minutes to maintain conditioning. Some experimentation will be needed to find the correct compactive effort that willyield7±0.5 percent air voids the specimens are compacted to avoid content corresponding to void levels expected in the field usually in the 6 to 8% range.

But ASTM D4867 suggested that in Marshall mixes with low crushing, 11 to 13 blows per side should yield and air void close to 7%. High crush mixes usually require 15 to 17 blows per side. The specimens are required to be compacted in accordance with AASHTO T312. After the specimens are removed from the molds, they are stored at room temperature for 24±3hours.

Evaluating of Specimens

After curing, the following tests and measurements of each specimen were done:

The maximum specific gravity (Gmm) in accordance with AASHTO T209 AASHTOT209-94 was used to determine the theoretical maximum specific gravity and density of each pavement mixture. The density is used to calculate values for percent airvoids in the compacted asphalt cores. The following equation was used with the procedure, Specific gravity = (B-A)/ (B-A)+(D-C) ……………………………………………………………[3.15]

Where,

A = Mass of the flask,

B= Mass of the flask with oven dry sample in air,

C= Mass of the flask filled with sample and water at 77^0F (25^0C),

D= Mass of the flask filled with water at 77^0F (25^0C),

And the theoretical maximum specific density is equivalent to the specific gravity multiplied by the unit weight of water (62.4lb/ft2). The thickness (t) and diameter (D) determine the specimen height and diameter with calipers in three locations and record the average to the nearest 0.1 mm (0.01”).

The bulk specific gravity (Gmb) in accordance with AASHTO T166,

Bulk Specific Gravity (Gmb) = [A/(B-C)]

A = Weight in grams of the specimen in air

B = Weight in grams, surface dry

C = Weight in grams, in water

Report the bulk specific gravity to the nearest 0.001.

Percent of water absorbed by volume = [(B-A)/(B-C)]x100

The volume (E) of the specimens is determined by subtracting the specimen weight in water from the saturated, surface dry weight. The percentage of air voids (Pa) is determined in accordance with AASHTO T269.Once determined; the specimens are separated into two subsets, of at least three specimens each, so that the average air voids of the two subsets are approximately equal.

For those specimens to be subjected to vacuum saturation, a freeze cycle, and a warm water soaking cycle, the volume of the air voids (Va) in cubic centimeters is calculated as follows:

Va =Pa E/100 ………………………………………………………….………...……. [3.16]

Where: Va=volume of air voids, cubic centimeters

Pa=air voids, percent volume of the specimen, cubic centimeters

The density=Gmb×62.4lb/ft3 ………………………………………………………...……… [3.17]

Percent air voids=100(1-A/B)

Where, A=bulk specific gravity (T-166),

B=theoretical maximum specific gravity (T-209).

Moisture Conditioning

The specimens in the conditioned group were vacuum saturated to 55-80% and then conditioned for 24hours at 60°C in a water bath. After removal from the 60°C water bath, the conditioned group specimens and the unconditioned group specimens were submerged in a 25°C water bath for an hour before being split diametrically at a loading rate of 5cm /minute. Moisture conditioning is an important step in the evaluation of moisture damage of HMA mixtures. Most research activities that evaluate the moisture damage of HMA mixtures rely on comparing the various properties of the mix before and after moisture conditioning. The properties prior to moisture conditioning are typically referred to as “dry” or “un conditioned” while the properties after moisture conditioning are typically referred to as “wet” or “conditioned”. The moisture conditioning process consists of saturating half of the samples to a level between 70 and 80%, then subjecting the saturated samples to a freeze thaw cycle consisting of freezing at 0^{o}F for 16hours followed by 24hours thawing at 140^{o}F and 2hours at 77^{o}F. It should be noted that versions of the AASHTO T-283 test prior to 2003 included the freeze cycle (i.e. freezing at 0^{o}F for 16hours) as an optional step. At the end of the curing period, the dry subset is wrapped with plastic in a heavy duty, leak proof plastic bag. The specimens are then placed in a 77±1°F (25±0.5°C) water bath for hours ± 10 minutes with a minimum of1in (25mm) of water above their surface. The other subset is conditioned as follows:

The specimens are placed in a vacuum container supported a minimum of1in.(25mm) above the container bottom. The container is filled with potable water at room temperature so that the specimens have at least 1in.(25mm) of water above their surface. A vacuum of 10-26in.Hg partial pressure (13-67kPa absolute pressure) is applied for approximately 5 to 10 minutes. The vacuum is removed and the specimen is left submerged in water for approximately 5 to 10 minutes.

The weight of the saturated, surface dry specimen after partial vacuum saturation (B1) is determined by Method A of AASHTO T 166. The volume of absorbed water (J1) in cubic centimeters is determined by the following equation.

$$J1=B1-A \quad \text{[3.18]}$$

Where:

J1 = volume of absorbed water, cubic centimeters

B1= weight of the saturated, surface dry specimen after partial vacuum saturation, g

A = weight of the dry specimen in air, g

The degree of saturation (S1) is determined by comparing the Volume of absorbed water (J1) with the volume of air voids (Va) using the following equation:

S1= 100J1Va …………………………………………………………………………………. [3.19]

Where:

S1=degree of saturation, percent

If the degree of saturation is between 70 and 80 percent, the conditioning by freezing may continue. If the degree of saturation is less than 70 percent, the vacuum procedure using more vacuum and/or time is repeated. If the degree of saturation is more than 80percent, the specimen is considered damaged and is discarded. After saturation is achieved immediately "treat" the saturated specimens by soaking them in distilled or deionized (minerals free) water at 60±1.0°C (140±1.8°F) for 24hrs.

Testing

Calculation of indirect tensile strength

The tensile strength is calculated using the following equation:

$$S_t = \frac{2000P}{\Pi t D} \quad \text{…[3.20]}$$

Where:

St = tensile strength, kPa

P = maximum load, Newtons

t = specimen thickness, mm

D = specimen diameter, mm

The numerical index or the resistance of asphalt mixtures to the detrimental effect of water can be expressed as the ratio of the original strength retained after freeze-thaw conditioning.

The tensile strength ratio is calculated as follows:

$$\text{Tensile Strength Ratio (TSR)} = \frac{S_2}{S_1} \quad \text{…[3.21]}$$

Where:

S1=average tensile strength of the dry subset, psi (kPa)

S2=average tensile strength of the conditioned subset, psi (kPa)

3.6 Wheel Tracker Rutting Test

This study was proposed to test moisture susceptibility of a compacted mix using Hamburg wheel tracker device (HWTD) since The HWTD is used to measure the combined effects of rutting and moisture damage on asphalt mixtures unfortunately it was difficult to find these type of wheel tracker and forced to use the DYNA-TRACK Single wheel tracker. This device was originally produced to measure deformation of a compacted asphalt mix but this study utilized it to test moisture susceptibility with simple modification. The rutting that occurred will be analyzed to test for moisture susceptibility. Marshal compacted Samples were produced using the same mix designs from ITS test. The Marshal compacted samples are placed in a concrete mold, shown in Figure, to maintain stability during the test. Also, the molds allow circular specimens to be used instead of beams and helps to use Marshal compacted samples instead of compacting 200mm diameter sample. Procedure B test method uses two samples conditioned at the test temperature for at least 1hour, and tracked for10,000 cycles under a solid rubber tire of width 50mm and load 700N at a rate of 26.5±1 cycles per minute, with the first 5 cycles used for conditioning. The deformation is measured as the mean of 25 equally spaced measurement points and the wheel tracking rate is calculated over the range of 5,000-10,000 cycles in mm/1000 cycles. Under this procedure, the proportional rut depth, as a percentage of the sample thickness, is calculated rather than the actual rut depth.

Figure 1 Single wheel Tracker

Figure 2Concrete mold for Single Wheel Tracker

3.7 Statistical Analysis

Following the laboratory procedures and data collection described in the previous topics, to evaluate the significance of conditioning of a mix, a statistical analysis was performed on tensile strength results performed using an Analysis of Variance (ANOVA). Initial observation of the TSR results indicates that the strength of each of the mixtures decreases after conditioning. A summary of the TSR values was given in Table11. The failure point for TSR used in this analysis was 70 percent. Three factors, without interacting, were analyzed for their significance. Those three factors are conditioning of the core, optimum binder content of the cores, and the aggregate type.

ANOVA allows one to determine whether the differences between the samples are simply due to random error (sampling errors) or whether there are systematic treatment effects that cause the mean in one group to differ from the mean in another.

Most of the time ANOVA is used to compare the equality of three or more means, however when the means from two samples are compared using ANOVA it is equivalent to using a t-test to compare the means of independent samples.

Hypotheses

The null hypothesis Ho which is the population means of all groups under consideration are equal and Ha: the population means are not all equal.

Ho: there are no differences in the average performance of tensile strengths within the conditioned and unconditioned groups

Ho: $\mu 1 = \mu 1 = \mu 2 .... = \mu k$

Ha: $\mu 1 \neq \mu k$ for some i, k

Independent Two Sample t-test: The independent two sample t-test which is used to test the null hypothesis that the population means of two groups are the same.

CHAPTER 4: RESULTS AND DISCUSSION

4.1Performance Tests of Aggregates

Table 3 presents the laboratory test results of the aggregates. Sand equivalent test result averaged 80.3% for aggregate A and 84.3% for aggregate B. the test determine the amount of dust or clay particle in the aggregate.80.3% of sand equivalent value mean that 80.3% of the aggregate was good material or sand and 19.7% of the aggregate was detrimental which is clay giving the pavement mixture the potential for cracking and moisture damage. Soundness test shows the resistance to disintegration of aggregate. The test result averaged 2.2% and 3% for aggregate A and B respectively which mean by 2.2% of the original sample aggregate washed by the sodium sulphate. The lesser the result show the better to resist weathering of aggregate. Flakiness index result of 30.2% of aggregate A and 25.1% of aggregate B was found to be flaky. As the flaky aggregate increase the tendency to break up easily will increase automatically. Los Angeles abrasion result shows the resistance of crushing, degradation and disintegration of aggregate. A result of 19% for aggregate A and 20% of aggregate B was found crushed and disintegrated. Which means the lesser the value the tougher the aggregate is. Specific gravity of aggregate is considered as an indication of strength. Specific gravity result of 2.74 for aggregate A has higher strength than aggregate B of specific gravity 2.65. Water absorption value of aggregate is considered as a measure of resistance to frost action and as a measure of sustaining weathering action.1.25% absorption for aggregate A and 1.52% for aggregate B was found which means aggregate A is better than aggregate B in sustaining weathering. The test results of each aggregate sources indicated that the results were within the required specification.

Table 3 properties of Aggregates

Test type		Test method	Specification(ERA)	Test result	
				Agg.A	Agg.B
Sand Equivalent		AASHTO T176	>75%	80.3	84.3
Soundness loss by sodium sulphate		AASHTO T-104	<12%	2.2	3%
Flakiness Index		BS812Part:105:1990	<45%	30.20%	25.1
Los Angeles Abrasion,		AASHTOT-96	<30%	19%	20%
Specific Gravity	Coarse	AASHTOT-85		2.74	2.65
	Fine	AASHTOT- 84		2.71	2.6
Absorption	Coarse	AASHTOT-85	<2%	1.25	1.52
	Fine	AASHTOT- 84		1.33	1.6

4.2Gradation

Gradation was determined by sieve analysis and the results were expressed as a total percent passing various sieve sizes as presented in Table 4 for aggregate A and Table 5 for aggregate B.

Table 4 Sieve Analysis of Aggregate A

(Sieve Dimension)	12.5-9.5mm	9.5-4.75 mm	< 4.75 mm	(Combination Grading)	Specification	
					Max	Min
25	100	100	100	100	100	100
19	100	100	100	100	100	100
12.5	99.8	100	100	100	100	100
9.5	62	99.8	99.7	91.8	100	90
4.75	1.3	29.1	98.3	56.5	75	45
2.36	0.1	3.8	71.7	35.6	58	30
1.18	0.1	1.9	53.4	26.2	44	20
0.6	0.1	1.2	36.7	18	32	13
0.3	0.1	1	24.4	12	23	9
0.15	0.1	0.8	16.6	8.2	16	6
0.075	0.1	0.7	11.4	5.7	8	4
RATIO	21%	31%	48%	100%	-	-

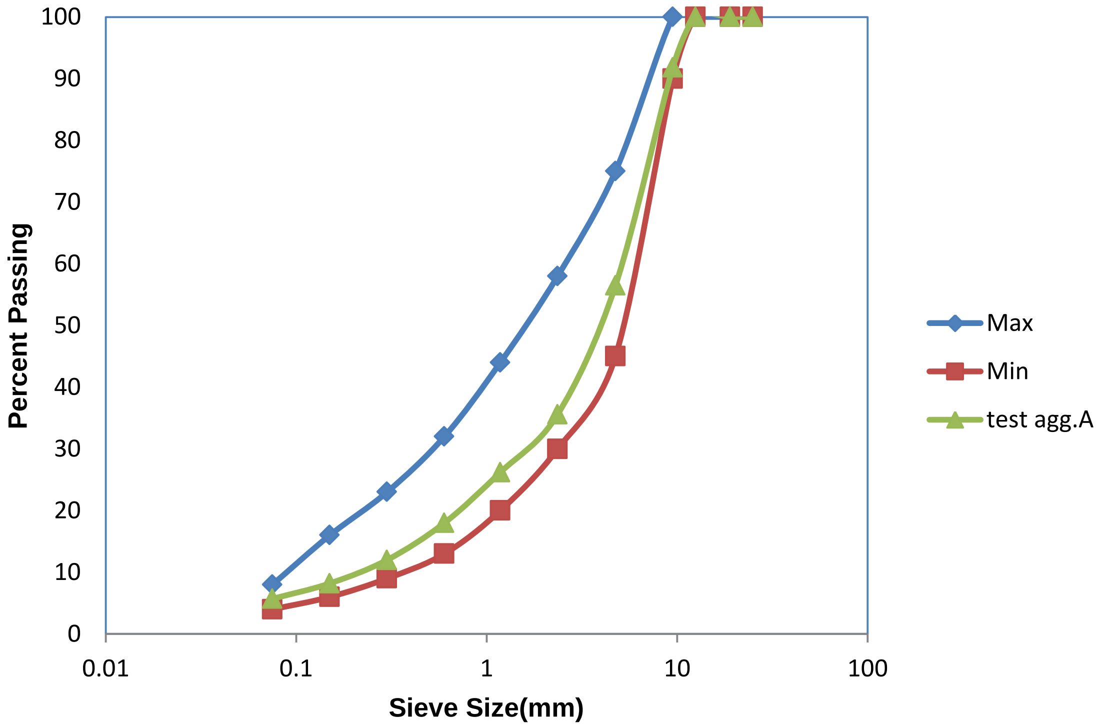


Figure 3 Gradation Chart for Aggregate A

The best gradation is the one that produces the maximum density. This would involve a particles are packed between the larger particles, which reduces the void space between particles. This creates more particle to particle contact, which in HMA would increase stability and reduce water infiltration. Gradation figure 3 and 4 is a gradation plotted on the 0.45 power gradation graph shows that aggregate B is coarser than aggregate A.

Table 5 Sieve Analysis of Aggregate B

	Specification			12.5-4.75	4.75-2.36	2.36-0	Combination
Sieve	Max	Min	Middle	A	B	C	Test agg. B
25	100	100	100	100	100	100	100
19	100	100	100	100	100	100	100
12.5	100	100	100	99.7	100	100	99.9
9.5	100	90	95	87.8	99.7	100	97.4
4.75	85	55	70	13.3	89.2	98.8	77.9
2.36	67	35	51	6.9	27	86.4	51.3
1.18	44	20	32	6.1	20.8	59.5	36.3
0.6	32	13	22.5	5.6	18.2	38	25
0.3	23	7	15	5.4	17.2	22.6	17.3
0.075	10	2	6	5.1	1.3	11.6	7
				21%	31%	48%	100%

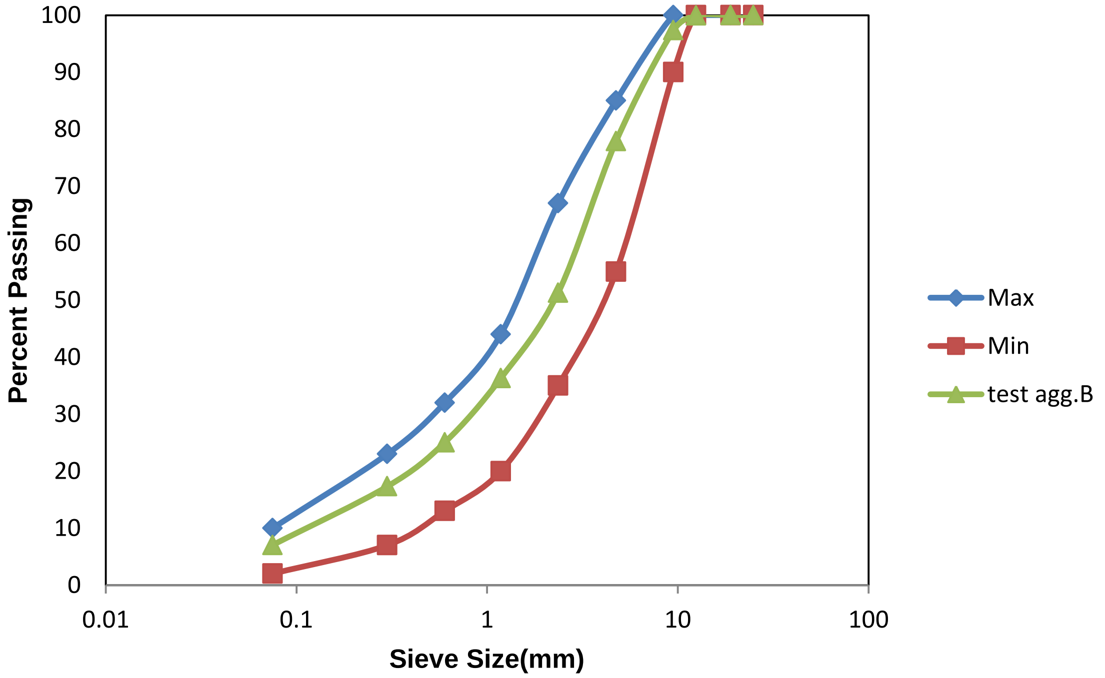


Figure 4Gradation Chart for Aggregate B

4.3 Bitumen characterization

The laboratory test results for the quality assurance of 85/100 penetration grade bitumen were presented in Table 6. The lowest temperature at which vapor of substance quickly catches fire in the form of flash averaged 280^{o}c. It means the safe temperature value of bitumen grade for mixing. The ductility test result averaged 146cm. It is a distance to which it elongates before breaking. Ductility of bitumen shows that its property to elongate under traffic load without getting cracked in road construction. Binder material having insufficient ductility gets cracked when subjected to repeated traffic loads and it provides pervious pavement surface. The penetration test result averaged 90mm at 25^{0}c test temperature which means 90mm depth was measured on a standard 100g loaded needle penetrated vertically in 5 seconds. The higher the penetration value, the softer the asphalt binder will become. Higher penetration grades are used in colder regions to prevent the occurrence of excessive brittleness. Penetration test is commonly used for asphalt grade determination. In this laboratory test result, since the penetration was found to be 90, it was confirmed that the bitumen used for this study was 85/100 penetration grade. The bitumen attains particular degree of softening at a temperature of 50^{0}c. 99.62% solubility of bitumen in trichloroethylene means that there are only 0.38% of impurities (mineral matter) in the bitumen. Laboratory tests results on 85/100 Pen Ac grade signify that all the quality tests were within AASHTO specification.

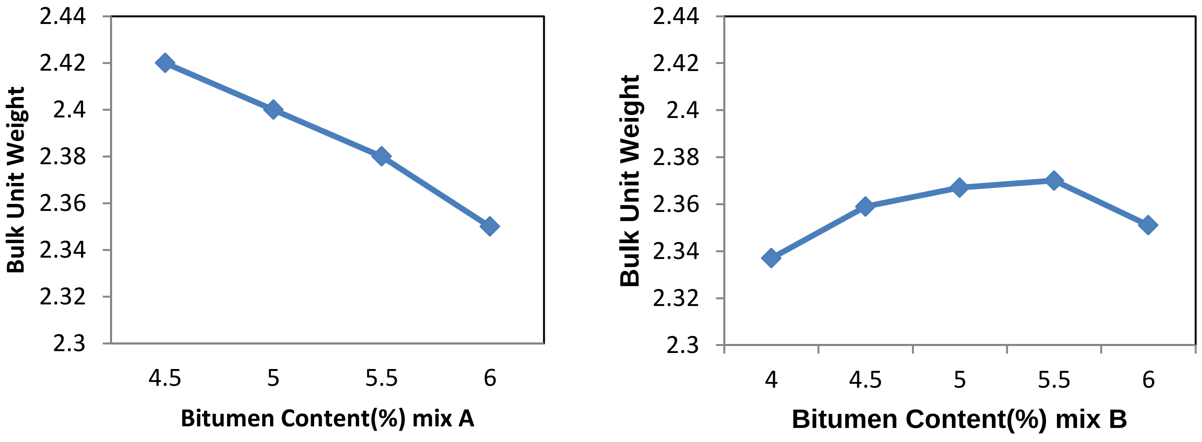
Table 6 Laboratory Test Result of Penetration grade 85/100 Asphalt Binder

Test type	Test method	Specification	Test result
Flashpoint ^{0}c	AASHTO T48	232 min	280
Ductility cm at 25^{0}C	AASHTO T51	100+	146
Penetration at 25^{0}c	AASHTOT49	85-100	90
Thin film oven test% loss	AASHTOT179		0.024
Solubility in tri chloro ethylene%	AASHTOT44	99min	99.62
Penetration of residue cm at 25^{o}c	AASHTOT49	50min	90
Penetration of residue % original	AASHTO T49	50min	136
Softening point	AASHTO T53	42-51	50
Ductility of residue cm at 25 ^{o}C	AASHTO T51	75 min	100

4.4 Summary of volumetric and Marshall Data

4.4.1 Bulk Specific Gravity of a Compacted Mix, Gmb

Figure 5 and 6 shows the bitumen content versus bulk unit weight of the mixture. As shown in the graph the bulk unit weight increases with increasing bitumen content up to certain point, after which it decreases. The bulk specific gravity of a mixture refers to the specific gravity of compacted mixture, including the volume of air voids within the mixture. The voids in an asphalt mixture are directly related to density. It is apparent that the bulk density increase as the amount of proportional binder content increases in the mixture up to some point and then decreases. This is due to an increased amount of binder will decrease the voids in the mix which subsequently lower the bulk density, thus, density must be closely controlled to insure that the voids stay within the acceptable range.

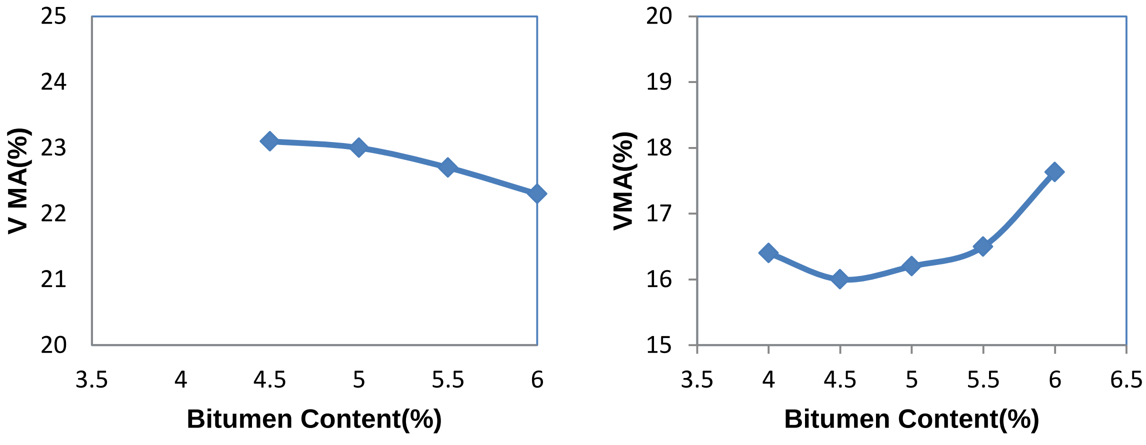


(a) (b)

Figure 5 Bitumen content versus unit weight (a) A mix (b)B mix

4.4.2 Voids in Mineral Aggregate, VMA

The VMA decreases to minimum value then increases with increasing bitumen content as shown in the figure 7 and 8. The mixtures with higher VMA should be preferred because more asphalt can be incorporated in the mixture to increase durability, lower sensitivity to variation in asphalt content during production and mixtures with low VMA will flush if slightly excessive in asphalt content, and will be dry and brittle if slightly deficient in asphalt content. Excessive VMA can cause unacceptably low mixture stability.

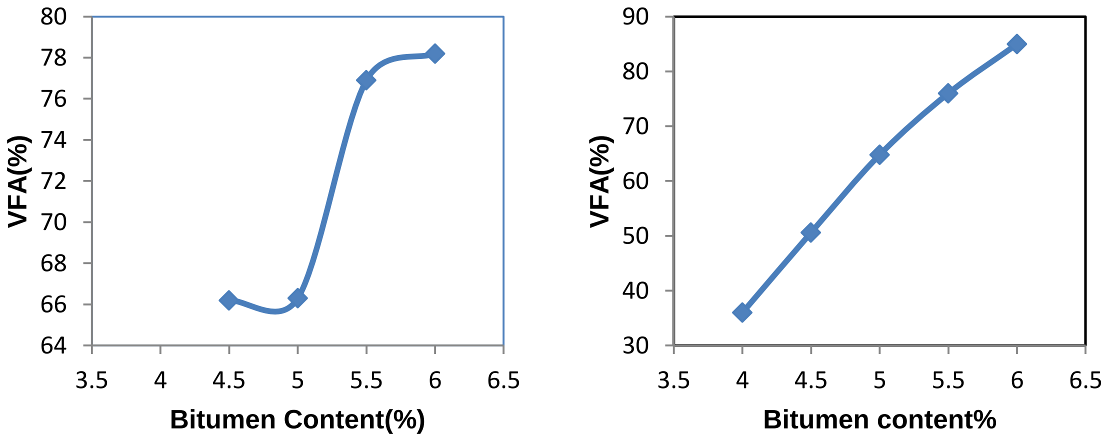


(a) (b)

Figure 6 Bitumen content versus VMA (a)A mix (b) B mix

4.4.3 Voids Filled with Asphalt (VFA)

Figure 9 and 10 show the result of VFA versus asphalt content. The VFA increases with increasing asphalt content. VFA represents the volume of effective asphalt content. The decrease of VFB indicates a decrease of effective bitumen film thickness between aggregates, which will result in higher low temperature cracking and lower durability of bitumen mixture since bitumen perform the filling and healing effects to improve the flexibility of mixture.

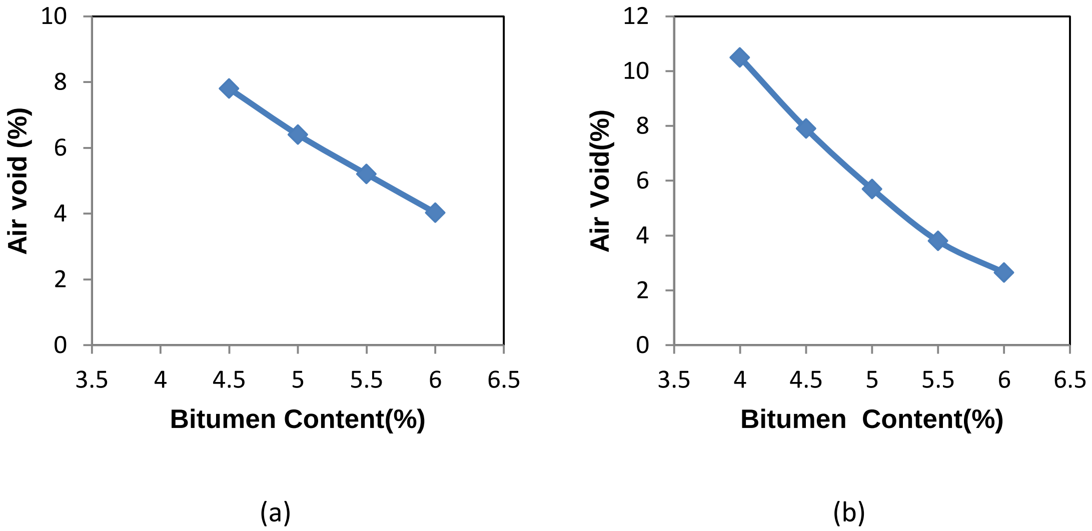


(a) (b)

Figure 7 Bitumen content versus VFB (a) A mix (b) B mix

4.4.4 Percent Air Voids (Va)

As shown in the figure11 and 12 the percent of air voids decreases with increasing asphalt content. Voids are the air spaces within the mixture. It is important that a mixture contains sufficient voids to provide spaces for expansion of bitumen and a slight amount of additional densification (compaction) under traffic. Aggregate size, shape and gradation have an effect on the amount of voids developed in a mixture and in the amount of bitumen that a mixture can contain.

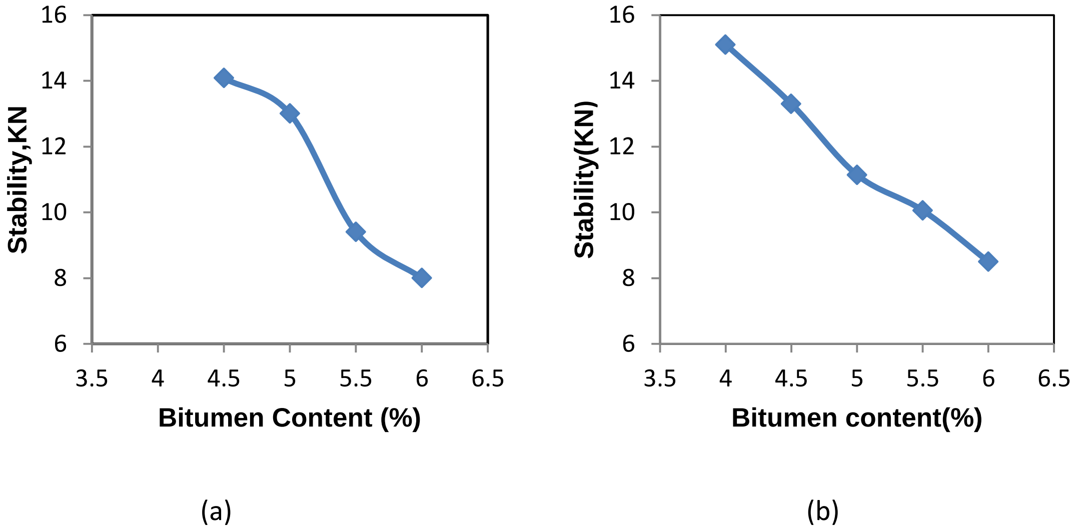


(a) (b)

Figure 8 Bitumen content versus Air void (a) A mix (b) B mix

4.4.5 Stability

The stability value increases with increasing asphalt content up to a maximum after which the stability decreases. Cases are not uncommon where no stability peak is obtained. Stability is a measure of resistance to deformation. It is necessary to have sufficient stability to meet the requirements of traffic without mat distortion or displacement. There are two forms of resistance, frictional or interlocking and cohesive resistance. Frictional or interlocking resistance is dependent on the aggregate framework. Cohesive resistance develops in the bitumen binder portion of the mixture. It depends on the rate of loading, load and temperature. High stability is undesirable if it is due to high density and low voids. Mixtures of this type have an excess of filler and are deficient in bitumen. Such surfaces will have low resistance to cracking, are brittle in the winter, and tend to ravel under traffic. Bitumen holds the aggregates in position, and the load is taken by the aggregate mass through the contact points. If all the voids are filled with bitumen, the one to one contact of the aggregate particles may lose, and then the load is transmitted by hydrostatic pressure through bitumen, and hence the strength of the mix reduces. That is why stability of the mix starts reducing when bitumen content is increased further beyond a certain value.



(a) (b)

Figure 9 Bitumen content versus Stability (a) A mix (b) B mix

4.4.6 Flow

As shown in figure 15 and 16 the flow value increases as the bitumen content of the mixture increases. Flow is an index of plasticity or the resistance to distortion. The amount of bitumen that fills the aggregate voids affects the flow. Mixtures that contain high air voids usually develop excessive flow value before reaching the bitumen content which will produce a satisfactory density. Flow value will increase rapidly with small increases in asphalt in mixtures which contain a large amount of filler.

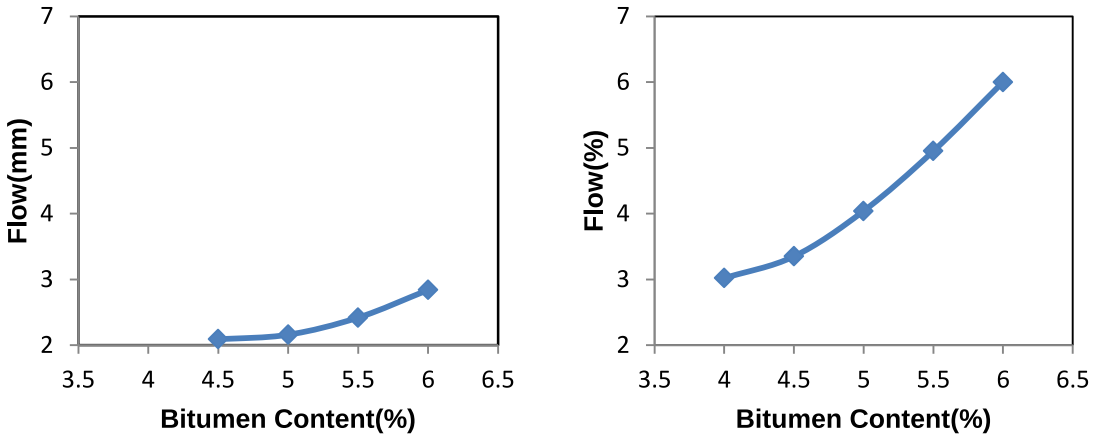


Figure 10 Bitumen content versus Flow (a) A mix (b) B mix

4.4.7 Optimum Asphalt Content

The optimum asphalt binder content is finally selected based on the combined results of Marshall stability and flow, density analysis and void analysis. The asphalt binder content that corresponds to the specification median air void content which is 4% is the optimum asphalt binder content. Table 4 determines properties at this optimum asphalt binder content by referring to the plots. Compare each of these values against specification values and if all are with in specification, then the preceding optimum asphalt binder content is satisfactory.

Table 7 Optimum Asphalt Content

	OBC=6%	OBC=5.5%	
Mix property	Mix A	Mix B	Specification(ERA)
AV(%)	4	3.8	3-5%
VMA(%)	22.3	16.5	>15%
VFB(%)	78.2	76	70-85%
Stability(KN)	8	10.06	>7KN
Flow(mm)	2.84	4.95	>2mm

4.5 Indirect Tensile Strength (ITS) Results

Tensile strength values determined using AASHTO T283 procedures as previously stated are Summarized in Table 8 and 9. It can be observed that, in all cases, tensile strength values were higher for the dry specimens than the wet specimens as expected. The moisture and thaw conditioning resulted decrease in tensile strength for both mixtures.

Table 8 ITS Result for dry samples

	Dry sample	Gmm	Gmb	AV	t(mm)	P(KN)	Ts(kpa)	Avg.Ts	SD
	A	2.39	2.22	7.1	63	6.24	630.53		
mix A	B		2.225	6.9	62.8	6.39	647.73		
	C		2.213	7.4	63	5.93	598.73	625.66	24.86
	A	2.43	2.259	7	63.2	7.50	755.56		
mix B	B		2.25	7.4	62.5	7.14	727.62		
	C		2.255	7.2	63	7.62	770.2	751.13	21.63

Table 9 ITS Result for wet samples

	wet sample	Gmm	Gmb	AV	t(mm)	P(KN)	Ts(kpa)	Avg.Ts	SD	TSR
	Aw	2.39	2.21	7.5	63	4.98	503.45			
mix A	Bw		2.222	7.0	63	5.15	520.26			
	Cw		2.227	6.8	63.1	5.50	554.48	526.06	26.01	0.84
	Aw	2.43	2.262	6.9	63.2	5.41	545.43			
mix B	Bw		2.247	7.5	63	4.92	496.82			
	Cw		2.252	7.3	62.8	5.19	525.64	522.63	24.44	0.7

The tensile strength test results of dry and wet samples of both mixtures are shown on figure 17.The tensile strength results of conditioned samples have lower values as compared to unconditioned samples. The average tensile strength of mix A dropped from 625.66kpa to 526.06kpa when it threatened by moisture. These result illustrated that an asphalt mixture with fulfilling all design criteria decrease tensile strength due to the effect of water. This is because moisture gets in between the asphalt and aggregate and breaks the bond between them. Bituminous mixture drives its strength from the bond between the binder and the aggregate of a mixture. The loss of the bond resulted in decrease of tensile strength.

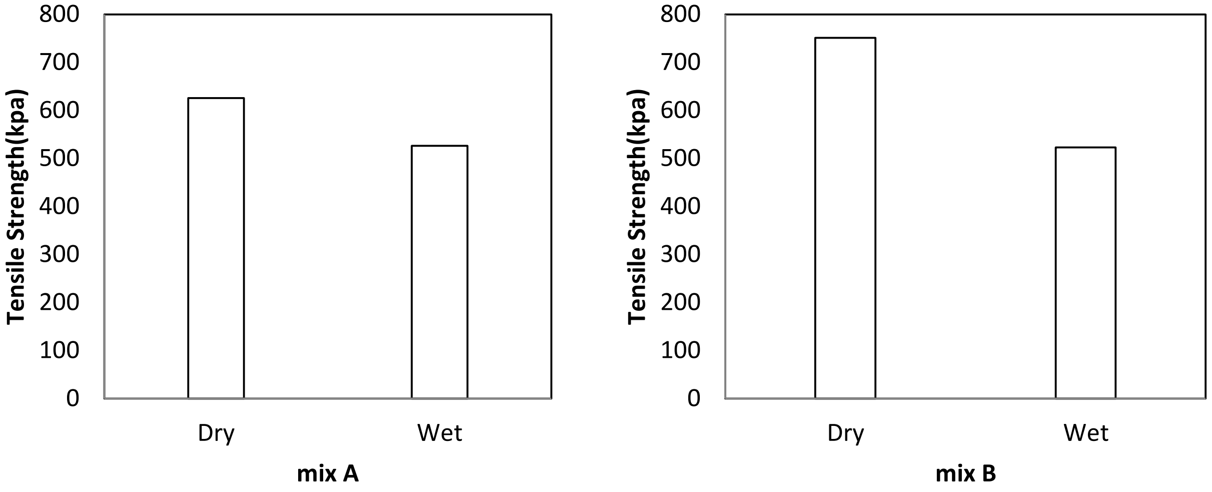


Figure 11 Tensile Strength Result

4.6Tensile Strength Ratio (TSR)

In this study the relative change of the indirect tensile strength was used to evaluate moisture damage by the tensile strength ratio (TSR), which is the ratio of conditioned to unconditioned tensile strength of an asphalt mixture. This test method uses a certain value of TSR to determine mixture susceptibility to moisture; for this study threshold is set at 70%.As shown in the figure 18 the TSR values for Mix A is 0.84 and for Mix B is 0.7.which means that aggregate A mix is not susceptible to moisture whereas aggregate mix B is an average moisture sensitive mix. The gradation of the aggregate, the property of the aggregate and the mix property have a great deal on this variation of the two mixture in moisture variation

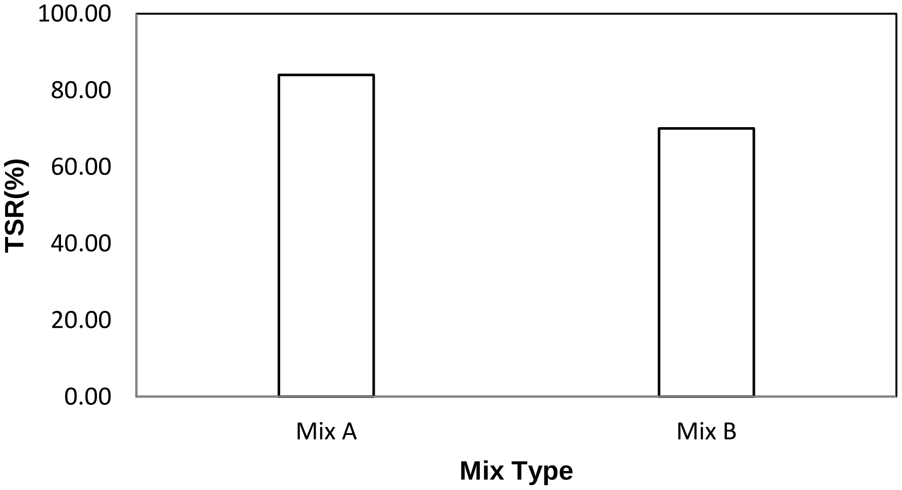


Figure 12 Tensile strength Ratio (TSR)

4.7 Rutting test results

The cores from Phase II were tested for rut depth in the loaded wheel tester. Summarizes rut depths measured after 5000 cycles. After the 5000 cycles losing of aggregates from the cores was observed especially for the conditioned cores, the cores after conditioning found that soft, deformed and lose pieces of aggregate from the edges and forced the test to stop on 5000 cycle. The measured rut depths for mixtures are shown in Table at Appendix A. The wheel trucker test result shows that the conditioned sample cores for both mixtures exhibits a higher rut depth than the unconditioned samples. The average rutting data for the conditioned and unconditioned laboratory fabricated specimens are shown in the figures.

Figure19 present the deformation as the loading cycle increase for mixture type A. Both the dry and wet specimen deformation increases as the loading increase but the wet samples deformed more in every loading than the dry samples. This is because when the compacted samples exhibit to water loses the bond between the asphalt and aggregate. It became soft and easy to crumble that makes rut more when it loaded than the dry one.

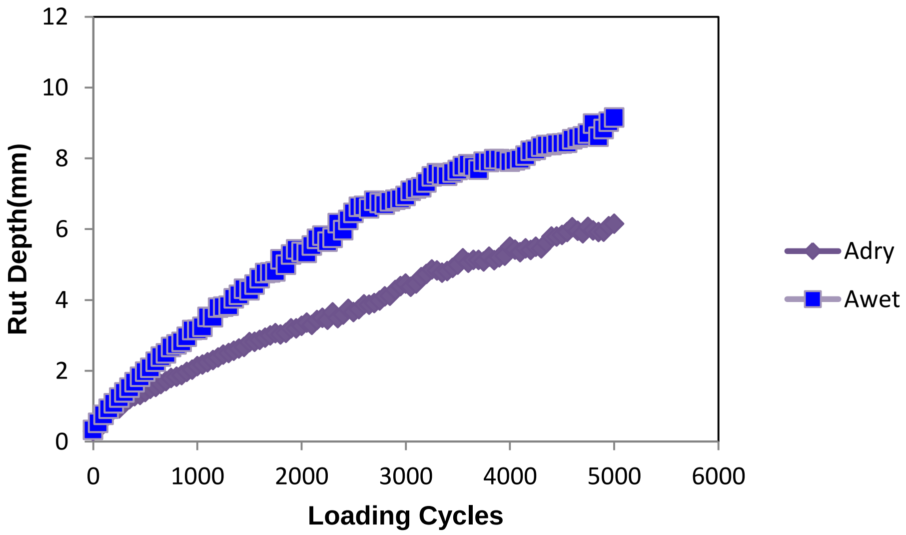


Figure 13 Rut depth of mix A

The rut depth versus the loading result of wheel trucker loading tester of mix B is shown in figure 20. Both the dry and wet specimen rut depth increases as the loading increase but the wet samples rut more in every loading than the dry samples.

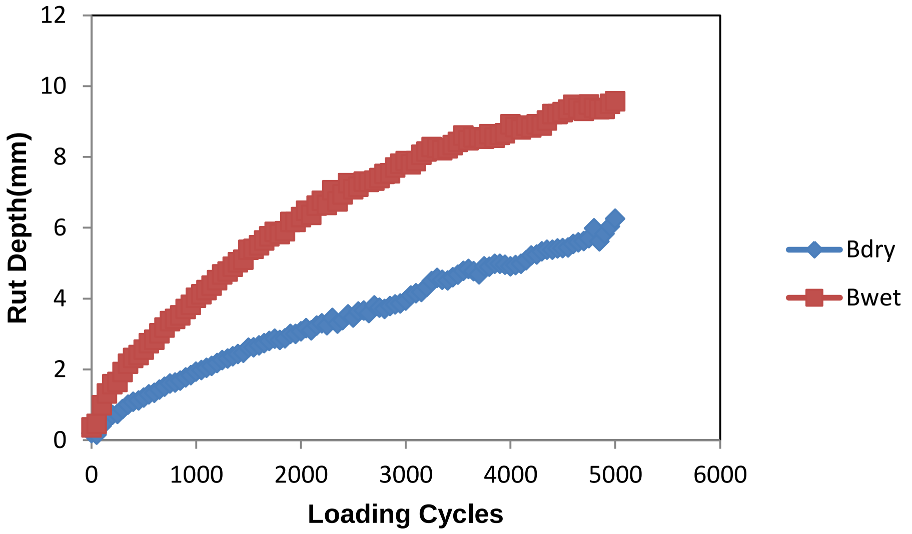


Figure 14 Rut depth of mix B

Figure 21 compares the rutting depth as the loading increase for both mixtures. As shown in the figure both mixtures exhibits an increase rutting depth as the loading increase for dry and wet samples but the wet samples rut more than the dry samples for both mixture types. The figure also shows that mixture type B wet samples were more deformed than mixture type A. this shows that mixture type B were more water sensitive than type A because as we can refer back to the performance test of aggregate. Aggregate A water absorption was a little high and has a coarser gradation as compared to aggregate B and also mix type A has a little more asphalt content than mix B that helps mix type A to resist the moisture damage a little longer.

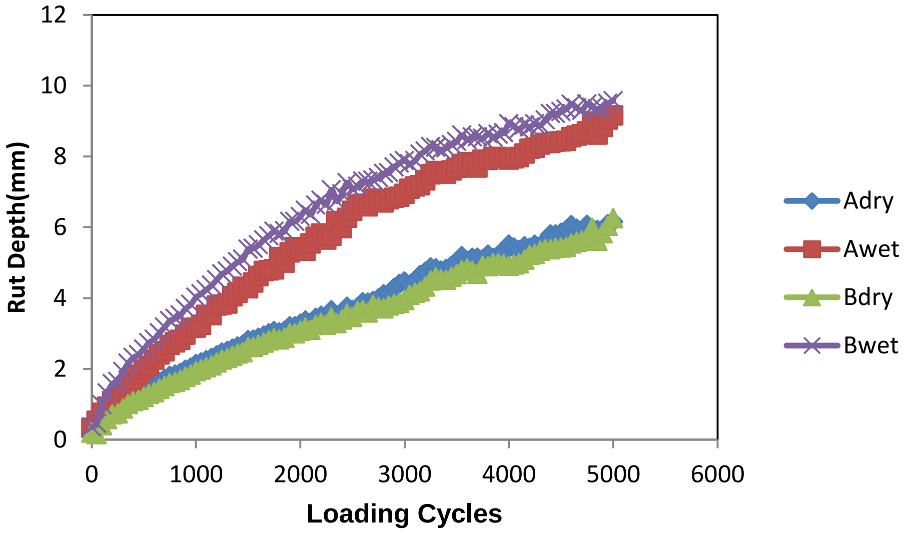


Figure 15 Rut depth for both mixes

Rut Ratio

Figure22 shows the wet/dry rut ratios for each mixture; a ratio greater than 1 indicates more rutting happen under the wet testing.

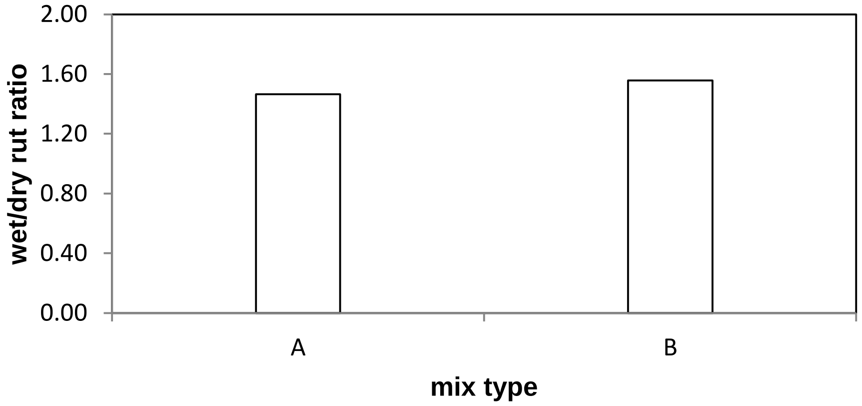


Figure 16 : Wet/dry rut ratio

4.8 Statistical Analysis

The decision to accept or reject the null hypothesis (Ho) is made by comparing the test statistics (computed t value) with critical value from the table. If the computed t value is exceeds the critical value, the hypothesis is rejected; if not, the hypothesis is not rejected.Table 15, The ANOVA result of tensile strength data collected in laboratory on both aggregates before and after conditioning indicates that, the observed value of the test statistics, the t value exceeds the critical value at 0.05 level of significance; therefore the null hypothesis is rejected. Hence this research accepted that conditioning on HMA have an effects on the tensile strength of HMA.The result t calculated compared to the t values from the table for confidence level of 0.05.

Table 10 Statistical Analysis of Tensile Strength Using ANOVA

	A dry	**A wet**	**B Dry**	**B wet**
	630.53	503.45	755.56	545.43
	647.73	520.26	727.62	496.82
	598.73	554.48	770.2	525.64
n	3.00	3.00	3.00	3.00
$\bar{X}$	625.66	526.06	751.13	522.63
S	24.86	26.00	21.63	24.44
s=25.43 **t=4.7976**			**S=23.077** **t=12.1269**	

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

This thesis utilized laboratory evaluations to study the effects of moisture on the Hot Mix Asphalt (HMA) mixtures by evaluating the tensile strength and rutting performance on different two asphalt mixtures using indirect tensile strength loading apparatus and Loaded wheel Tester The laboratory testing was accomplished with the production of laboratory Marshall compacted specimen from the two aggregate mixtures to an air void content corresponding to void levels expected in the field usually in the 6 to 8% with the design Marshall mix. 3 of them conditioned (wet) and the others 3 specimens unconditioned (dry) are tested for their indirect tensile strength. In the wheel trucker test the specimens are prepared in the same manner as the ITS test and measured their rutting performance using the loading wheel tester.

Conclusions

- From ITS test it’s observed that, tensile strength values were higher for the dry specimens than the wet specimens. This is true for both mixture types.
- Comparing the TSR values of the mixes, Mix A has higher TSR value than mix B. the TSR values of mix B is 0.7 which suggest that mix B is an average moisture sensitive mix .
  From Mix A which has higher binder content than mix B Exhibit a better performance in TSR shows that an increase in binder content can lead to an increase in the TSR value.
- From the statically analysis of ITS , this thesis accepted that conditioning on HMA have an effects on the tensile strength of HMA.
- The conditioned sample cores for both mixtures exhibits a higher rut depth than the unconditioned samples
- Wet/dry rut ratios from wheel trucker testing which is greater than 1 indicate that both of the mixtures are susceptible to moisture damage although TSR results from the same specimens didn’t show the same trend.
- During wheel trucker testing, losing of aggregates from the cores was observed and the cores after conditioning was found soft, deformed and edge breaking of the mixture were observed.

Recommendations

- Moisture sensitivity tests should be a requirement in asphalt mix design of ERA manual since stripping is one of a major problem result in different types of distress.
- The TSR values of mix B is 0.7 which suggest that mix B is average moisture sensitive mix further investigation needs for the application of anti-stripping agent.
- Samples with different types of aggregate and binder mix should be tested for moisture sensitivity since the material properties has a great influence on moisture susceptibility of a mix.
- Further research should be needed for the remedies of moisture damage. The usage of anti-stripping agent for moisture susceptible mixture should be studied and used by Ethiopian road construction agencies and authorities.
- Further research should be performed to study effectiveness of the Dayna truck single wheel trucker at measuring moisture susceptibility.
- Due to the complexity of the effects of moisture damage on asphalt pavements, further research is needed to evaluate resistance to moisture damage caused by a particular combination of variables.

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APPENDIX-A:

TABLES AND FIGURES

Table A1 Summary of volumetric and Marshall data of mix A

BC(%)	Gmb	Gmm	VIM(%)	VMA(%)	VFB(%)	Stablity(KN)	Flow(0.25mm)
4.50%	2.23	2.42	7.8	23.1	66.2	14.08	2.09
5%	2.246	2.4	6.4	23	66.3	13	2.16
5.50%	2.265	2.39	5.2	22.7	76.9	9.4	2.42
6%	2.29	2.39	4.03	22.3	78.2	8	2.84

Table A4: Maximum theoretical specific gravity of mix

Asphalt Content	4.5		5		5.5		6	
Sample no.	1	2	1	2	1	2	1	2
(A) Weight of Flask	460	460	460	460	460	460	460	460
B) Weight of Flask + Water	1688.5	1689	1688.5	1688.5	1688.5	1689	1688.5	1688.5
C) Weight of Sample	500	500	500	500	500	500	500	500
D) Weight of Flask + Sample + Water	1996	1997	1992.5	1992	1994.5	1995	1989.5	1989
(E) Temperature of Water	25	25	25	25	25	25	25	25
(G) Weight of Replaced Water (B+C) – D	192.5	191.5	196	196.5	194	194	199	199.5
(H) Max. Sp. Gravity of Material	2.43	2.41	2.42	2.38	2.38	2.4	2.37	2.42
Average Max Specific Gravity of Material	2.42		2.4		2.39		2.39	

Table A5 Wheel trucker output for mix A

Wheel trucker output(rutting depth in mm) for mix A for conditioned and unconditioned specimens								
cycle	A Dry	A Wet	cycle	A Dry	A Wet	cycle	A Dry	A Wet
1	0.3	0.33	**1700**	3	4.78	**3400**	4.82	7.51
50	0.35	0.53	**1750**	3.06	4.81	**3450**	4.91	7.6
100	0.61	0.76	**1800**	3.03	5.15	**3500**	5	7.67
150	0.78	0.93	**1850**	3.07	5	**3550**	5.18	7.78
200	0.91	1.08	**1900**	3.2	5.29	**3600**	5.05	7.83
250	0.94	1.25	**1950**	3.2	5.43	**3650**	5.13	7.77
300	1.08	1.39	**2000**	3.27	5.36	**3700**	5.13	7.68
350	1.2	1.53	**2050**	3.36	5.33	**3750**	5.09	7.91
400	1.28	1.68	**2100**	3.3	5.54	**3800**	5.22	7.9
450	1.32	1.83	**2150**	3.43	5.73	**3850**	5.12	7.98
500	1.4	1.98	**2200**	3.5	5.82	**3900**	5.2	7.98
550	1.49	2.09	**2250**	3.44	5.65	**3950**	5.25	7.95
600	1.54	2.26	**2300**	3.65	5.76	**4000**	5.5	7.91
650	1.63	2.4	**2350**	3.49	6.17	**4050**	5.42	7.94
700	1.71	2.49	**2400**	3.59	5.97	**4100**	5.35	7.98
750	1.8	2.67	**2450**	3.75	6.26	**4150**	5.46	8.08
800	1.83	2.75	**2500**	3.66	6.46	**4200**	5.41	8.21
850	1.88	2.82	**2550**	3.74	6.63	**4250**	5.5	8.24
900	1.97	2.96	**2600**	3.88	6.66	**4300**	5.46	8.33
950	2.03	3.15	**2650**	3.87	6.59	**4350**	5.61	8.38
1000	2.13	3.16	**2700**	3.91	6.8	**4400**	5.79	8.38
1050	2.18	3.24	**2750**	3.98	6.74	**4450**	5.79	8.41
1100	2.24	3.52	**2800**	4.1	6.7	**4500**	5.84	8.42
1150	2.3	3.51	**2850**	4.12	6.79	**4550**	5.92	8.44
1200	2.38	3.79	**2900**	4.28	6.83	**4600**	6.05	8.54
1250	2.46	3.81	**2950**	4.39	6.86	**4650**	5.95	8.58
1300	2.5	3.84	**3000**	4.46	6.94	**4700**	5.88	8.62
1350	2.57	4.05	**3050**	4.37	7.08	**4750**	6.06	8.7
1400	2.63	4.15	**3100**	4.46	7.15	**4800**	5.96	8.98
1450	2.67	4.32	**3150**	4.64	7.18	**4850**	5.92	8.61
1500	2.81	4.27	**3200**	4.73	7.32	**4900**	5.93	8.83
1550	2.82	4.44	**3250**	4.86	7.49	**4950**	6.08	9.04
1600	2.87	4.62	**3300**	4.83	7.58	**5000**	6.15	9.15
1650	2.94	4.76	**3350**	4.77	7.53			

Table A6 Wheel trucker output for mix B

Wheel trucker output(rutting depth in mm) for mix B for conditioned and unconditioned specimens								
cycle	Bdry	Bwet	cycle	Bdry	Bwet	cycle	Bdry	Bwet
1	0.2	0.36	**1700**	2.8	5.76	**3400**	4.51	8.24
50	0.15	0.46	**1750**	2.86	5.88	**3450**	4.6	8.33
100	0.41	0.98	**1800**	2.83	5.82	**3500**	4.67	8.42
150	0.58	1.32	**1850**	2.87	5.9	**3550**	4.78	8.6
200	0.71	1.58	**1900**	3	6.16	**3600**	4.83	8.47
250	0.74	1.64	**1950**	3	6.16	**3650**	4.77	8.55
300	0.88	1.92	**2000**	3.07	6.3	**3700**	4.68	8.55
350	1	2.16	**2050**	3.16	6.48	**3750**	4.91	8.51
400	1.08	2.32	**2100**	3.1	6.36	**3800**	4.9	8.64
450	1.12	2.4	**2150**	3.23	6.62	**3850**	4.98	8.54
500	1.2	2.56	**2200**	3.3	6.76	**3900**	4.98	8.62
550	1.29	2.74	**2250**	3.24	6.64	**3950**	4.95	8.67
600	1.34	2.84	**2300**	3.45	7.06	**4000**	4.91	8.92
650	1.43	3.02	**2350**	3.29	6.74	**4050**	4.94	8.84
700	1.51	3.18	**2400**	3.39	6.94	**4100**	4.98	8.77
750	1.6	3.36	**2450**	3.55	7.26	**4150**	5.08	8.88
800	1.63	3.42	**2500**	3.46	7.08	**4200**	5.21	8.83
850	1.68	3.52	**2550**	3.63	7.16	**4250**	5.24	8.92
900	1.77	3.7	**2600**	3.66	7.3	**4300**	5.33	8.88
950	1.83	3.82	**2650**	3.59	7.29	**4350**	5.38	9.03
1000	1.93	4.02	**2700**	3.8	7.33	**4400**	5.38	9.21
1050	1.98	4.12	**2750**	3.74	7.4	**4450**	5.41	9.21
1100	2.04	4.24	**2800**	3.7	7.52	**4500**	5.42	9.26
1150	2.1	4.36	**2850**	3.79	7.54	**4550**	5.44	9.338
1200	2.18	4.52	**2900**	3.83	7.7	**4600**	5.54	9.47
1250	2.26	4.68	**2950**	3.86	7.81	**4650**	5.58	9.37
1300	2.3	4.76	**3000**	3.94	7.88	**4700**	5.62	9.3
1350	2.37	4.9	**3050**	4.08	7.79	**4750**	5.7	9.48
1400	2.43	5.02	**3100**	4.15	7.88	**4800**	5.98	9.38
1450	2.47	5.1	**3150**	4.18	8.06	**4850**	5.61	9.34
1500	2.61	5.38	**3200**	4.32	8.15	**4900**	5.83	9.35
1550	2.62	5.4	**3250**	4.49	8.28	**4950**	6.04	9.5
1600	2.67	5.5	**3300**	4.58	8.25	**5000**	6.25	9.57
1650	2.74	5.64	**3350**	4.53	8.19			

APPENDIX-B:

PHOTOGRAPHS

Caption 1: Sample aggregates

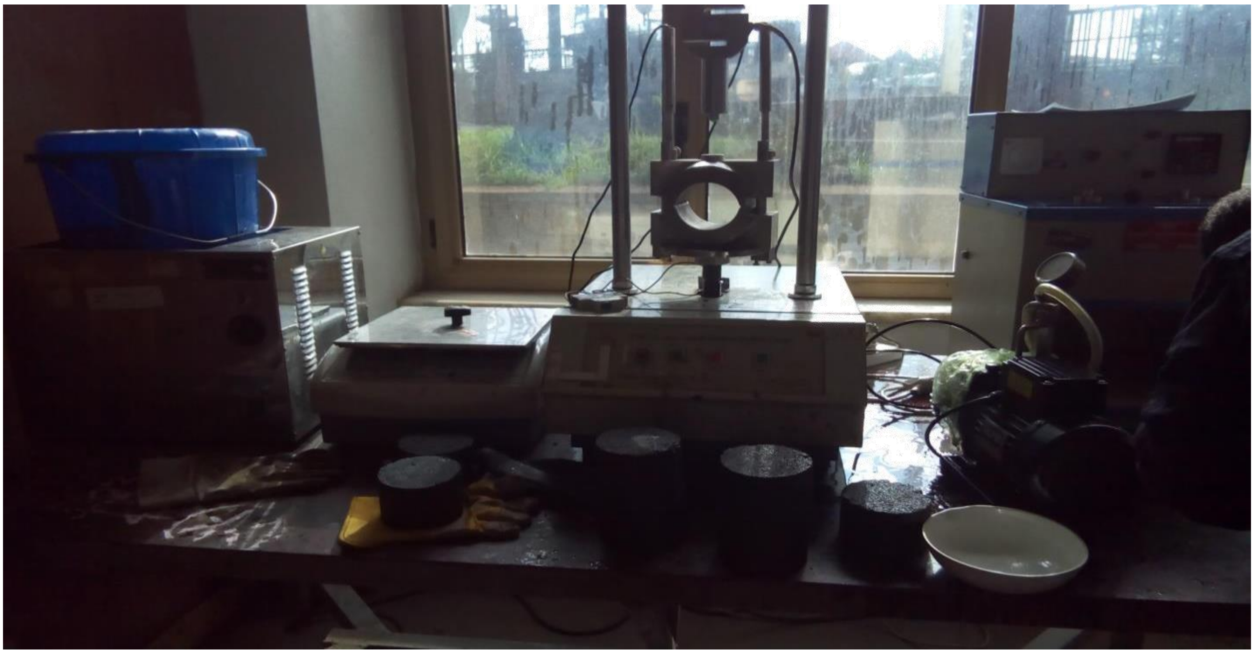
Caption 2: Weighting of the sample aggregates

Caption 3: Attuning the required temperature of the mixture

Caption 3: Mixing of aggregate and bitumen

Caption 4: Compacting the mixture with Automatic Marshall Compactor

Caption 5: Unmolding the compacted Marshall specimen



Caption 6: Marshall Stability apparatus

Caption 7: Weighting specimen in water

Caption 8: Conducting Maximum Theoretical Density of the mix

Caption 9: preparing specimens for Dyna track wheel load tester

Table A: Test Result of Marshall Mixture for aggregate A

Specimen	% Asphalt Content AC	Specimen height	Wt. Of Specimen in air (gm)	Wt. Of Specimen in Water (gm)	SSD Wt. Of Specimen in air (gm)	Vol. Of Specimen (D-C) cc.	Bulk Density (gm/cc) B/E	Gmm	% Air voids (G–F) x100 G	% VMA = 100 – (Fx%AC) Gsb	% VFB (I-H)x 100 I	Stability in KN	Corrected Coeff.	Corrected Stability in kgfLxM	Flow value (mm)
	A		B	C	D	E	F	G	H	I	J	K	M	N	O
1A	4.5%	63.0	1196.0	669.0	1199.5	530.5	2.254	2.420	6.8	22.3	69.3	11.80	1.04	12.3	2.3
1B		63.5	1195.5	665.8	1199.5	533.7	2.240		7.4	22.8	67.3	12.68	1.04	13.2	2.2
1C		62.5	1196.0	655.0	1199.5	544.5	2.197		9.2	24.3	61.9	13.67	1.04	14.2	1.8
Avg.							2.230		7.8	23.1	66.2			13.2	2.09
A	5.0	63.0	1190.0	660.0	1198.2	538.2	2.211	2.400	7.9	20.2	61.0	12.63	1.0	12.6	2.2
B		63.5	1194.0	666.0	1195.2	529.2	2.256		6.0	18.6	67.7	13.13	1.0	13.1	2.39
C		62.5	1194.5	668.0	1194.0	526.0	2.271		5.4	18.0	70.2	13.21	1.0	13.2	1.88
Avg.							2.246		6.4	18.9	66.3			13.0	2.16
A	5.5	63.0	1189.0	668.0	1192.0	524.0	2.269	2.390	5.1	22.6	77.6	9.63	1.04	10.0	1.9
B		63.5	1192.2	667.0	1196.0	529.0	2.254		5.7	23.1	75.3	9.03	1.04	9.4	2.3
C		62.5	1190.0	670.0	1194.0	524.0	2.271		5.0	22.5	77.9	8.46	1.04	8.8	3.0
Avg.							2.265		5.2	22.7	76.9			9.4	2.42
A	6.0	63.7	1194.0	677	1196.5	519.35	2.299	2.390	3.8	22.0	77.2	9.63	1.04	10.0	1.9
B		62.8	1186.5	671.78	1189.0	517.22	2.294		4	22.2	78.0	9.03	1.04	9.4	2.8
C		62.4	1194.5	671.7	1194.0	522.3	2.287		4.3	22.4	79.6	8.46	1.04	8.8	3.8
Avg.							2.29		4.03	22.3	78.2			9.4	2.84

AC%	trials	Specimen height	Wt. Of Specimen in air (gm)	Wt. Of Specimen in Water (gm)	SSD Wt. Of Specimen in air (gm)	Vol. Of Specimen (D-C) cc.	Bulk Density (gm/cc) B/E	Gmm	% Air voids (G–F) x100 / G	% VMA = 100 – (Fx%AC) Gsb	% VFB (I-H)x 100 / I	Stability in KN	Corrected Coeff.	Corrected Stability in kgf LxM	Flow value (mm)
4	A		1252.4	715.7	1253.5	537.8	2.329	2.611	10.8	16.4	36	15.54	0.93	14.5	3.1
	B		1248.3	714.4	1248.9	534.5	2.335		10.6			15.13	0.96	14.5	2.86
	C		1250.4	718.5	1251.2	532.7	2.347		10.1			16.83	0.96	16.2	3.11
	AV		1250.4	716.2	1251.2		2.337		10.5					15.1	3.02
4.5	A		1260.2	727.5	1261.5	534	2.36	2.562	7.9			13.62	0.96	13.1	3.29
	B		1258.4	725.6	1259.2	533.6	2.358		8			14.71	0.96	14.1	3.44
	C		1256	724.5	1256.7	532.2	2.36		7.9			13.37	0.96	12.8	3.32
	Av						2.359		7.9	16	50.6			13.3	3.35
5	A		1262.7	728.4	1262.8	534.4	2.363	2.511	5.9			11.85	0.96	11.4	4.12
	B		1257.9	729.9	1258.2	528.3	2.381		5.2			11.94	0.96	11.5	3.91
	C		1259.3	725.5	1259.8	534.3	2.357		6.1			11.32	0.93	10.52	4.08
	Av						2.367		5.7	16.2	64.8			11.14	4.04
5.5	A		1269.1	733.9	1269.3	535.4	2.37	2.463	3.8			10.54	0.96	10.1	4.83
	B		1270.3	734.5	1270.4	535.9	2.37		3.8			10.68	0.96	10.25	4.92
	C		1271.5	734.8	1271.6	536.8	2.369		3.8			10.26	0.96	9.85	5.11
	Av						2.37		3.8	16.5	76			10.06	4.95
6	A		1271.3	730.4	1271.7	541.3	2.349		2.74			9.17	0.93	8.5	5.74
	B		1270.7	730.6	1270.9	540.3	2.352	2.415	2.63			9.48	0.93	8.8	5.89
	C		1272.6	732.2	1272.9	540.7	2.354		2.54	17.63	84.96	8.83	0.93	8.2	6.24
	Av						2.351		2.65					8.5	6