PAVEMENT DESIGN WITH POROUS ASPHALT

A Dissertation
Submitted to
the Temple University Graduate Board

In Partial Fulfillment
of the Requirements for the Degree
DOCTOR OF PHILOSOPHY

---

by
Mehmet Cetin
May 2013

Examining Committee Members:

Robert M. Brooks, Advisory Chair, Civil and Environmental Engineering
Philip D. Udo-Inyang, Civil and Environmental Engineering
Felix F. Udoeyo, Civil and Environmental Engineering
Oleksandr Diloyan, Mechanical Engineering
Yuhong Guo, External Member, Computer and information Science

ABSTRACT

In this research study, a strategy was used for assigning stiffness values to various layers of the pavement system so that there are negligible tensile stresses in the subgrade and base layers. The stiffnesses of the aggregate base layers were assigned double the values of the corresponding subgrade materials. The layer thicknesses were designed to achieve surface deflection values within the acceptable limit. An innovative design procedure was developed for designing pavement sections covering various layer thicknesses, material and environmental variables. The designed sections were compared with the American Association of State Highway and Transportation Officials (AASHTO) procedure and the differences were critiqued. Porous asphalt layer was used as the surface course for all pavement sections.

The calculations validated the general principle of pavement design, as the subgrade stiffness decreased the base thickness increased for the same surface course thickness and traffic. Structural design of 63 pavements sections was accomplished representing various temperature and materials including additives. Low Density Polyethylene, performance graded asphalt, different soils, aggregates, lime and cement were the component materials utilized in this research study.

An explanation on the mechanics of the mixtures is given in the results and discussion section.

Key Words: Deflection Limiting Criteria, Low Density Polyethylene, Porous Asphalt, Resilient Modulus, KENPAVE, Subgrade Vertical Strain.

Part of this dissertation work is published in the following peer reviewed international journals:

Mehmet Cetin, Robert M. Brooks and Philip Udo-Inyang. (2012). "An Innovative Design Methodology of Pavement Design by Limiting Surface Deflection". International Journal of Research and Reviews in Applies Sciences (IJRRAS), Volume 13, Issue 2, S. No: 26, November 2012, ISSN: 2076-734X, EISSN: 2076-7366, page 607-610.
www.arpapress.com/Volumes/Vol13Issue2/IJRRAS_13_2_26.pdf

Mehmet Cetin, Robert M. Brooks and Philip Udo-Inyang. (2012). "A Comparative Study Between the Results of an Innovative Design Methodology by Limiting Surface Deflection and AASHTO Design Method". International Journal of Research and Reviews in Applies Sciences (IJRRAS), Volume 13, Issue 2, S. No: 27, November 2012, ISSN: 2076-734X, EISSN: 2076-7366, Page 611-616.
www.arpapress.com/Volumes/Vol13Issue2/IJRRAS_13_2_27.pdf

I would like to dedicate my research to those who make a difference in the world and make change in the environment for a good sustainable and healthy life.

ACKNOWLEDGMENTS

I would like to thank THE REPUBLIC OF TURKEY, MINISTRY OF NATIONAL EDUCATION SCHOLARSHIPS for providing a scholarship to me.

I would sincerely like to thank Dr. Robert M. Brooks for his continuous guidance and wisdom during my doctoral degree. He helped me develop knowledge and ability to understand transportation engineering. He opened the door of Transportation Engineering for me. I felt comfortable on exploring this area because of Dr. Brooks' efforts facilitated my learning process. This dissertation does not exist without his help and guidance.

I would like to thank Dr. Naji Khoury, my first advisor and former member of my advisory committee for his help and guidance during my research. He gave me support and guidance to keep on track with the research timeline. This dissertation would not exist without his help and guidance.

I would like to thank my advisory committee, Dr. Philip D. Udo-Inyang, Dr. Felix F. Udoeyo, Dr. Oleksandr Diloyan, and Dr. Yuhong Guo for their help and suggestion in improving my research.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

Highway engineers have been trying to develop a rational method for estimating the projected life of a given pavement through full-scale road tests, model studies, laboratory experiments and theoretical analysis. Unlike steel and concrete, whole structural properties are well defined. Little was understood the extremely complex behavior of crushed rock, water bound macadam (WBM) and other granular material until about 40 years ago resulting. This led to slow progress in the development of an empirical method for finding the relationship between the life span of pavements and the design thickness of their constituent layers (Cetin et al., 2012a).

Flexible pavement design methods are still empirical or semi-empirical, based on past experience with similar subgrades, pavement materials and traffic loads. These methods are satisfactory as long as materials, traffic loading conditions and layer thicknesses do not differ from those for which the methods were developed. Because of the considerable increase in frequency and intensity of axle loads and the use of new pavement materials the semi-empirical design methods have become irrelevant.

1.1 Development of Pavement Design Methods

During the last 40 years considerable progress has been made in theoretical approaches to the development of fundamental methods for designing road pavements. Burmister (1943), Burmister (1945), Burmister (1958), Fox (1948), and Hank & Scrivner (1948) give tables and charts of influence values for determining stresses and deflections. Acum & Fox (1950), Jones (1962), and Peattie (1962) present tables and graphs for three layer systems. In all these investigations, Poisson's ratio is given as 0.5, pertaining to an incompressible body and influence values have been given for points along the axis of symmetry. Verstraeten (1968) gives generalized expressions for determining stresses and deflections in a flexible pavement. Computer programs like BISAR (DeJong et al., 1973), CHEVRON (Michelow, 1963), ELSYM (Ahlborn, 1972), EPVAE (Doddihal, 1982), MPAVE (Doddihal, 1982) and NPAVE (Doddihal, 1982), etc. have been developed by various researchers for analyzing stresses and deflections in multilayer pavements. All these computer programs assume the pavement to be linearly elastic. Duncan et al. (1968), Barksdale (1969) and Crawford (1971) analyzed pavements that have materials with non-linear structural properties. These rigorous analytical methods have gone a long way in bridging the gap between theory and practice in pavement design (Cetin et al., 2012a).

Pavement surface deflection represents the inversely combined strength of all the layers. On the whole, the higher the deflection the weaker the pavement and vice versa. It is important to limit pavement surface deflection not only for new construction but also for rehabilitated pavements because pavement performance depends on surface deflection. The American Association of State Highway and Transportation Officials (AASHTO) design method requires obtaining a weighted structural number as a function of the sum of the products of layer thicknesses and their respective strength coefficients (Cetin et al., 2012b).

1.2 Objectives

The objectives of this research are:

(1) To use a strategy for assigning stiffness values to various layers of the pavement system such that there exist negligible tensile stresses in the subgrade and base layers,

(2) To design the layer thicknesses so that the surface deflection is within acceptable limits,

(3) To develop an innovative design procedure for designing several pavement sections covering various layer thicknesses, material and environmental variables,

(4) To compare the designed sections with the American Association of State Highway and Transportation Officials (AASHTO) procedure and to critique the differences.

1.3 Scope of Research

There are seven types of plastics that are used by industries; however, the scope of this research is limited to Low Density Polyethylene (LDPE) plastic. There are several types of performance graded asphalt used in the industry. This study is limited to performance grade 64-22. The subgrade stiffness is limited to 1180 psi through 7570 psi.

The gap grading of the aggregates in the surface layer allows freezing and thawing of water in extreme lower temperature regions. Therefore, this study is limited to the areas where water does not freeze for a considerable time of the year and design period of the pavements.

CHAPTER 2

REVIEW OF LITERATURE

2.1 Full Scale Road Test

2.1.1 Experimental Pavements in the United States

The Bates Experimental Road, constructed in Illinois in 1920, was the first major test road in the United States. This test road was constructed using various materials; including brick, asphaltic concrete, and Portland cement concrete. The results of this test road gave basic data that were used by design engineers for many years (Yoder & Witczak, 1975). Between 1923 and 1950 several research projects involving specially constructed test tracts produced significant advances in pavement design and construction. Among these the Bureau of Public Roads conducted the Arlington Test; The Bureau of public Roads, the Highway Research Board and the Asphalt Institute completed the Hybla Valley Test; Columbia Steel Company constructed the experimental stretches of Pittsburg Test; the U.S. Amy Crops of Engineers conducted the Stockton Test and the Lockbourne Air Force Base Test; and the Crops of Engineers continuously conducted research at the flexible pavement laboratory, Vicksburg, Mississippi.

The Western Association of state Highway Officials (WASHO) sponsored the WASHO Road Test in 1953-1954, consisting of a number specially built flexible pavements in Idaho. The object was to investigate the performance of experimental flexible road pavements constructed to a wide range of overall thicknesses and subjected to known number of repetitions and known axle loads. Construction of test facilities for the American Association of State Highway Officials (AASHO), AASHO Road Test (1961), AASHO Road Test (1962) began in August 1956. Test traffic run from October 15, 1958 to November 30, 1960, with $1.114*10^6$ axle load applications.

The Bureau of Public Roads (present name: Federal Highway Administration) and the AASHO have been responsible for several test roads in the United States. In addition, several state highway departments have constructed test pavements for the purpose of evaluating the effect of load and materials on pavement design.

2.1.2 The WASHO Road Test

The WASHO Road Test, constructed in southern Idaho, was sponsored by the WASHO and administered by the Highway Research Board. The objective was to investigate the performance of experimental flexible road pavements constructed to a wide range of overall thicknesses, when they were subjected to a known number of repetitions of known axle loads.

Two identical test tracks were constructed and five experimental sections, each 100 m long, were separated by 30 m transition lengths where changes in thickness were made. The subgrade was a silty clay of average liquid limit of 35% and average plastic limit of 25%. The average moisture content of the soil at time of construction was 22.7 per cent. The outer lane of one test track carried 3-axle trucks with two load axles each carrying 10,100 kg. The inner lane of the same track was trafficked by similar vehicles with two load axles each carrying 8160 kg. The outer lane of the second test track carried 5-axle trucks with two pairs of tandem load axles, the load on each pair being 18,100 kg. The inner lane of the second test track was trafficked by similar vehicles with 14,500 kg on each pair of tandem axles (WASHO Road Test, 1954; WASHO Road Test, 1955).

The subgrade was compacted to an average dry density of 1528 kg/m^3 at an average moisture content of 22.8%. The subbase and the road base were laid at an average dry density of 2102 kg/m^3 and 2127 kg/m^3 respectively. Trafficking at a uniform rate began in mid-June 1953 and continued until the end of May, 1954, with two small break periods when whilst the subgrade was frozen.

Measurements were made in the wheel tracks and between the wheel tracks at monthly intervals. Observations were made on the amount of cracking and of the elastic deflection measured by the Benkelman Beam. The performance of the pavements was judged mainly in terms of pavement deformation (rutting). The thinnest sections (those without any subbase) deformed rapidly under all axle loads. Increasing the thickness of the subbase in all cases reduced the level of permanent deformation, although the influence of subbase thickness was less under the 100 mm surfacing than under the 500 mm surfacing. The performance of the sections with 100 mm asphalt surfacing was for superior to that of the sections with the same total thickness but with 50 mm surfacing (WASHO Road Test, 1954; WASHO Road Test, 1955).

2.1.3 The AASHO Road Test

In 1951, at the recommendation of the Road Test Advisory Committee of AASHO, a Working Committee was appointed to represent possible participating agencies for an expanded road test. In June 1952 the Working Committee produced an "AASHO Road Test Project Statement" containing the basic concepts of the proposed test which included both rigid and flexible type test pavements and test-bridge spans on the more heavily-loaded loops. In January 1953 the Working Committee produced a second report entitled "AASHO Road Test Project Program" that contained more detailed information about the selected test site, the layout of the test facility, construction specifications and procedures, test operations, and proposed special studies.

In August 1954 The Working Committee produced a third report, entitled "Project Program Supplement", which included additional details on construction, instrumentation and the collection and analysis of data. Construction of the test facility began in August 1956, and test traffic was inaugurated on October 15, 1958. Test traffic was operated until November 30, 1960, at which time $1.114*10^6$ axle loads had been applied to the pavement test sections.

A special studies program was conducted in the spring and early summer of 1961 over some of the remaining test sections. Strains, deflections and pressures were measured in these studies under a wide variety of vehicle types, load suspension, tires and tire pressures. Special military vehicles, included at the request of the army, as well as highway construction equipment, were included in these tests. The field office for the project was closed in January 1962 (AASHO Road Test, 1961; AASHO Road Test, 1962).

2.1.4 The Co-operative Pavement Investigation Program in Canada

A Co-operative Pavement Investigation Program was started in Canada in 1958 by the Canadian Good Roads Association (1962) and extensive pavement inventory data was collected from 1959 to 1961. Both the AASHO Road Test and the Co-operative Pavement Investigation Program conducted regression analyses to relate flexible pavement performance with the principal variables affecting it.

2.1.5 Experimental Pavements in Britain

In Britain the first experiment to investigate the design and performance of road structures as a whole was laid in 1949 on Trunk Road A. North of Borough Bridge, Yorkshire (Lee & Croney, 1962). Among the full-scale flexible pavement tests, the Alconbury Hill experiment (flexible sections) can be regarded as typical.

2.1.6 The Alconbury Hill Experiment

The flexible sections of the Alconbury Hill experiment opened to traffic in November 1957 occupying about 2 km of the northbound carriageway of Trunk Road A.1 in Cambridgeshire. The experiment consisted of 33 flexible sections that were mostly 60 m long. In 1957 total daily commercial traffic was 4,000 vehicles, in both directions (Croney & Loe, 1965). The main objectives were:

(1) To compare the behavior of five road base materials (wet-mix slag, rolled asphalt, lean concrete, open-textured tarmacadam and soil-cement) under a 100 mm rolled asphalt surfacing and to determine the appropriate thickness of each in relation to the traffic carried,

(2) To investigate the effect of varying the type and thickness of surfacing on pavement life using the same road bases,

(3) To investigate the effect of varying the thickness of the sand sub-base on pavement life.

The main criterion used to assess the relative performance of the flexible pavements is the deformation, measured transversely across the road, caused by the passage of the traffic. Non-destructive testing techniques including the deflection beam and wave velocity methods were used throughout the life of the road. Mean transverse profiles were plotted at four levels of traffic for the rest sections with wet-mix road bases and graphs were plotted between permanent deformations (rutting) and observed or estimated cumulative standard axles for all the sections concerned (Croney & Loe, 1965).

2.2 Porous Asphalt and Plastic Connection to the Study

There have been many advances in porous asphalt. Volume retention of permeable pavements is shown in Table 2.1 and Table 2.2 taken from the website of National Pollutant Discharge Elimination System (NPDES, 2012). The tables show the extra ordinary advantages of porous asphalt. On the other hand overly improved sites, grounds and soils, and impervious surfaces (e.g., sidewalks, roads, rooftops, and pavements) have poor storm water infiltrate qualities. This not only decreases ground water levels but also increases erosion due to flooding, increases temperatures causing the Heat Island Effect, and decreases water quality in bodies of water.

Table 2.1. Volume Retention of Permeable Pavements (NPDES, 2012)

Application	Location	Soil Type	Underdrain	Volume Retention
Pervious Concrete				
Parking lot	Kingston, NC	Clay	No	99.9%
Residential street and sidewalk	Sultan, WA	--	--	100%
Permeable Interlocking Concrete Pavers				
Parking lot	Swansboro, NC	Sandy soil	No	100%
Parking lot	Renton, WA	---	No	100%
Field and laboratory tests	Guelph, Ontario, Canada	--	--	90%
Driveway	Cary, NC	Clay	Yes	66%
Residential street	Auckland, New Zealand	Clay	Yes	60%
Parking lot	Kinston, NC	Clay	No	55%
Parking lot	United Kingdom	liner installed Impermeable	Yes	45% - 34%
Porous Asphalt				
Parking lot*	Durham, NH	Clay	Underdrain	25%
Street	France	--	--	96.7%
Parking lot	State College, PA	--	--	Retained the 25 yrs. - 24 hrs. storm

Table 2.2. Monitored Pollutant Removals of Permeable Pavement (NPDES, 2012)

Application	Location	TSS	Metals	Nutrients
Permeable Concrete				
Parking lot	Tampa, FL	91%	75-92%	--
Permeable Interlocking Concrete Pavers				
Parking lot	King College, ON	81%	Cu: 13% Zn: 72%	TP: 53% TKN: 53%
Parking lot	Goldsboro, NC	71%	Zn: 88%	TP: 65% TN: 35%/td>
Driveways	Jordan Cove, CT	67%	Cu: 67% Pb: 67% Zn:71%	TP: 34% NO_3-N: 67% NH_3-N: 72%
Parking lot	Renton, WA	---	Cu: 79% Zn: 83%	--
Porous Asphalt				
Parking lot	Durham, NH	99%	Zn: 97%	TP: 42%
Highway (friction course only)	Austin, TX	94%	76-93%	43%

2.3 Plastic Waste

Traditional asphalt used to build the porous asphalt surface layer of pavements cannot take high tensile stresses that develop at the bottom of the surface layers due to heavy traffic loads. However, plastic modified traditional asphalt can build enough tensile strength in the porous layers of the pavements.

According to the United States Environmental Protection Agency (US EPA, 2012) website, 31 million tons of plastic waste were generated in 2010, representing 12.4% of total the Municipal Solid Waste (MSW). In that year, 14 million tons of plastics as containers and packaging, 11 million tons as durable goods (e.g., appliances), and 7 million tons as nondurable goods (e.g., plates, cups) were generated in the country. It is important to note that only 8 percent of the total plastic waste generated in 2010 was recovered for recycling. As per the US EPA website, plastic waste is hard to degrade back into the environment because it takes several decades and even centuries to degrade. Meanwhile plastics release a wide variety of complex toxins into the environment. These toxins are harmful to plant, animal, and human life causing irreversible health effects. The above two paragraphs combined with testing and design parts of this research explain the plastic connection to the research topic.

2.3.1 Plastics Classification

The Society of The Plastics Industry Trade Association (SPI, 2011) identifies the following seven categories of plastics:

(1) Polyethylene Terephthalate (PETE, PET). Plastics meeting this classification provide resistance to moisture and heat and barriers to gas. This type of plastic is extensively used for manufacturing mouthwash bottles, beer bottles, film, peanut butter and salad dressing containers, soft drink and water bottles, etc.

(2) High Density Polyethylene (HDPE). Plastics in his category provide permeability to gas, stiffness, strength, resistance to chemicals and moisture, ease of forming and processing. The bottles are suitable for products that have a short shelf life. It is suitable for many industrial chemicals and household products such as detergents and bleach. It is also used for cereal box liners,

shampoo, yogurt and margarine tubs, juice, milk, water, dish and laundry detergent bottles, cosmetic, trash and retail bags.

(3) Vinyl, Polyvinyl Chloride (V, PVC). This plastic is used for flexible and rigid products. If a plastic meets this standard it has versatility, ease of blending, resistance to grease or oil, resistance to chemicals, clarity, strength and toughness. It is weather resistance, flow typical features constant electrical qualities, and chemical resistance. It is used in clear food and non-food packaging, wire and cable insulation, toys, medical tubing, shampoo bottles, film and sheet and in construction products such as fittings, pipes, siding, window frames, carpet backing, and flooring.

(4) Low-Density Polyethylene (LDPE). If a plastic meets this standard it can provide ease of sealing, strength or toughness, ease of processing, barrier to moisture, and flexibility. It is extensively used in making films because the plastic has approximate transparency, flexibility and toughness. LDPE is used in flexible lids and bottles. The wire and cable industry has many LDPE applications. It is also used as coatings for paper milk cartons and hot and cold beverage cups, frozen food bags, container lids, dry cleaning, squeezable bottles (e.g., honey, mustard), and toys.

(5) Polypropylene (PP). Plastics in this category provide resistance to grease, oil, chemicals, and heat, toughness, moisture barrier, and versatility. PP is strong. It is used to make containers for hot liquids because it has a high melting point. PP is suitable for large pattern parts for automotive and consumer products, flexible or rigid packaging, fibers, containers for margarine, takeout

meals and deli foods, medicine bottles, yogurt, bottles for ketchup, and bottle caps.

(6) Polystyrene (PS). PS is versatile, clear, has insulating properties, and is easily formed (Styrofoam). It is hard and brittle with it's a low melting point. PS is used in making food containers, food packaging, protective packaging, and bottles. PS combined with rubber forms High Impact Polystyrene (HIPS). HIPS is used for packaging in food-service applications (e.g., grocery store meat trays, egg cartons), compact disc cases, aspirin bottles, cups, cutlery, and plates.

(7) Other. This type of plastic is dependent on a resin or combination of resins, or is made of more than one plastic and used in a multi-layer combination. This plastics are used to make five gallon buckets, clear plastic baby bottles, some citrus juice and ketchup bottles, food containers, water pitchers, Nalgene brand water bottles, compact discs, oven-baking bags, cell phones, computers, automobile parts, three and five-gallon reusable water bottles, barrier layers and custom packaging.

2.4 Theoretical Analysis

This chapter provides a review of three areas pertaining to road tests. First it looks at various techniques used for the determination of stresses in layered pavement system. Second there is a brief review of various full-scale road tests in order to determine future directions for the analysis of different test sections. Lastly there is a review of the development of repeated load equipment.

2.4.1 Elastic Half Space Analysis

Boussinesq gave the first set of formulae for stresses in a semi-infinite mass due to a point load at the surface of a medium assuming the medium to be elastic, homogeneous and isotropic. Newmark (1947) developed influence charts based on Boussinesq formulae to determine the stresses in elastic soil masses. Newmark's findings were used by Foster & Ahlvin (1954) to prepare charts for determining stress and deflection factors for vertical deflection under circular loaded area, for a Poisson's ratio of 0.5 pertaining to an incompressible body. Ahlvin & Ulery (1962) produced comprehensive tables by extending the previous work for any value of Poison's ratio to determine various stresses and strains at any point in a semi-infinite elastic mass due to a circular loaded area. Boussinesq's solution was extended by Sanborn & Yoder (1968) to include a semi ellipsoidal area.

2.4.2 Multi-Layer Analysis

A typical flexible pavement consists of layers of different materials. The moduli of elasticity of these materials decreases with depth causing reduction in stresses and deflections in the subgrade, because of dispersion of wheel loads over wider areas. Flexible pavements were first analyzed as layered systems by Burmister (1943, 1945).

Tables and charts of influence values were created for two layer systems by Burmister (1943), Burmister (1958), Fox (1948), and Hank & Scrivner (1948). Acum & Fox (1950) and Jones (1962) created tables for determining stress in three layered systems. Jone's tables were represented graphically by Peattie (1962). Only stresses and displacements at the interfaces along the axis of symmetry of loading can be found by these charts. All these investigations assumed a uniformly distributed vertical surface load acting over a circular area and Position's ratio of 0.5 for all the layers. Schiffman (1963) and Westman (1962) generalized Burmister's theory based on the derivation presented by Muki (1960) to include either vertical or horizontal asymmetric surface loading. For a two layered system a table was calculated by Heukelom and Klomp (1967) showing the stresses at various points for a Poisson's ratio of 0.5 for all the layers assuming a uniformly distributed vertical surface load acing over a circular area.

Verstraeten (1968) has given general equations based on Burmister's theory for determining stresses and displacements at a point in a multilayer system for any value of Poisson's ratio under a uniformly distributed vertical stress, uniform centripetal shear stress and uniform one directional shear stress over a circular area. Pavements assumed to be finite slabs resting on elastic foundation pavements were analyzed by Yettram et al. (1968) under arbitrary loading. Ullidtz & Peattie (1980) used programmable calculators to determine the stresses in multilayer pavements by converting the layered system into an equivalent half space.

2.4.3 Viscoelastic Analysis

Pavement materials are not elastic masses. Bituminous layers have properties which are dependent on time of loading and temperature. Solutions (Freudenthal & Lorsh, 1957; Pister & Monismith, 1960; Pister & Westmann, 1963; Hoskin & Lee, 1959) have been given for cases where pavement materials possess simple viscoelastic behavior.

Elastic analysis has been extensively used to find stresses in bituminous mixtures by assigning appropriate stiffness corresponding to the particular time of loading and temperature. Monismith & Secor (1963) and Coffman et al. (1964) used equivalent time-dependent modulus for the asphaltic courses in the corresponding elastic analysis. Seed et al. (1961) used resilient moduli from repeated load tests and used the elastic layered theory. Ashton & Moavenzadeh (1968), Huang (1967), Barksdale & Leonards (1967), Perloff & Moavenzadeh (1967), Ishihara & Kimura (1967), and Ku (1967) used the viscoelastic method to find the stresses in pavement layers.

2.4.4 Finite Element Solutions

The finite element method of stress analysis has been used by various researchers to determine stresses and strains in flexible pavements, because of its added advantage in dealing with nonlinear material properties and complicated boundary conditions. A finite element computer program was developed by Waterhouse (1967) to determine the stresses and strains at points away from the centerline of loading. Duncan et al. (1968) found that displacements and stresses computed by the finite element method compare well with those determined from the Boussinesq solution when the nodal points in the finite element continuum are fixed at a depth of 18 radii for the bottom boundary at distance of about 12 radii from the axis of symmetry. Duncan et al. (1968) also found that in order to obtain a reasonable comparison with Burmister's solution, it is necessary to fix the bottom at a depth of about 50 radii maintaining the same radial boundary as for the Boussinesq solution. The pavement was modeled as an axisymmetric solid by Duncan et al. (1968), Waterhouse (1967), and Barksdale (1969) and analyzed as a prismatic solid by Crawford (1971). Using the three-dimensional elastic theory, Crawford (1971) developed a finite element analysis to calculate the stresses under multiple wheel loadings. Doddihal & Pandey (1984) analyzed full depth granular pavements as no-tension material using the finite element analysis.

2.5 Development of Mixture Design Frame Work

2.5.1 AAMAS General Frame Work

The general framework in an Asphalt Aggregate Mixture Analysis System (AAMAS) as developed by ARE Inc., modified by Monismith et al. (1985), Monismith et al. (1987) (Figure 2.1) and further modified by the author is explained below. This system contains several subsystems including procedures for selecting an aggregate, asphalt and their relative proportions to produce a mixture.

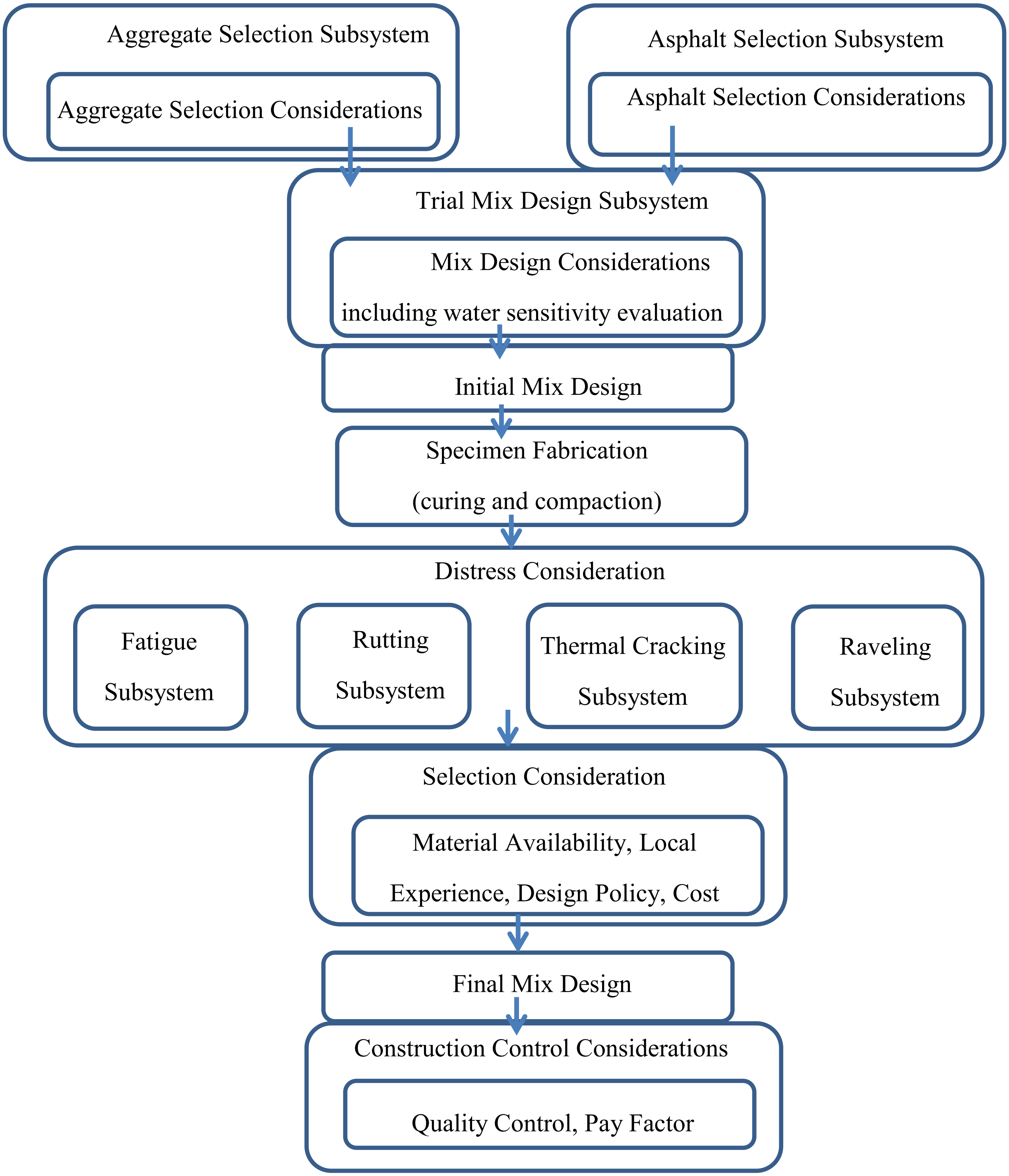

Figure 2.1. A Comprehensive Design System for Asphalt Concrete with or without Modified Asphalt (Monismith et al., 1985; Monismith et al., 1987).

2.5.2 Aggregate Selection Subsystem

In this subsystem several aggregate characteristics were considered including, but not limited to: absorption, shape, fraction, wettability, distribution, durability, texture, size, and the type and amount of filler. The characteristics can be divided into two types: natural and manufactured. Natural characteristics include absorption, texture, wettability, and durability. Manufactured characteristics include recycled fines, gradation, filler, interparticle friction, and shape. Tests were conducted as per ASTM C 33/C 33M-11a (ASTM-American Society for Testing and Materials) Standard Specification for Concrete Aggregates on both natural and manufactured characteristics. If the aggregates failed the tests further evaluation was conducted. Once the aggregates passed the tests they were accepted for trial mix design.

2.5.3 Asphalt Selection Subsystem

Asphalt’s physical properties include rheological characteristics, adhesion, safety (flash point temperatures), durability, and fracture (tensile) properties. The asphalt selection subsystem includes physical properties and chemical properties. The physical properties include safety, short-term durability, rheological properties, tensile fracture characteristics, long-term durability, and adhesion. The chemical properties include compatibility, composition, solubility, and reactivity with oxygen.

Tests were conducted as per ASTM D 6373-07e1 Standard Specification for Performance Graded Asphalt Binder. If the asphalt failed the tests further evaluation was conducted. Once the asphalt passed the tests it was accepted for trial mix design.

2.5.4 Mix Design Considerations

Currently used design methods include using: (1) the Hveem stabilometer, (2) the Marshall Test equipment, and (3) the Gyratory compaction. In this study the Marshall procedure was used to cast the samples. The desirable mixture properties are shown in Table 2.3.

Table 2.3. Mixture Properties (Monismith et al., 1987)

Property	**Definition**	**Examples of mix variables which have an influence**
Stiffness	Smix(t,T)=σ/ε relationship between stress and strain at a specific temperature and time of loading	Aggregate gradation Asphalt stiffness degree of compaction water sensitivity
Stability	Resistance to permanent deformation (usually at high temperatures and long times of loading—conditions of Smix)	Aggregate surface texture Aggregate gradation Asphalt stiffness Asphalt content Degree of compaction Water sensitivity
Durability	Resistance to weathering effects (both air and water) and to the abrasive action of traffic	Asphalt content aggregate gradation degree of compaction water sensitivity

Table 2.3. (continued)

Property	Definition	Examples of mix variables which have an influence
Fatigue resistance	Ability of mix to bend repeatedly without fracture	Aggregate gradation, Asphalt content, Degree of compaction, Asphalt stiffness, Water sensitivity, Note: selection of mix components dependent on structural pavement section design.
Fracture characteristics	Strength of mix under single tensile stress application	Aggregate gradation, Asphalt content, Degree of compaction Asphalt stiffness, Water sensitivity
Skid resistance (surface friction characteristics)	Ability of mix to provide adequate coefficient of friction between tire and pavement under 'wet' conditions	Aggregate texture and resistance to polishing, Aggregate gradation, Aggregate content
Permeability	Ability of air, water, and water vapor to move into and through mix	Aggregate gradation, Aggregate content, Degree of compaction

2.5.5 Distress Considerations

The major forms of distress are fatigue, rutting, thermal cracking and raveling. Data from four fundamental variables are needed to start with: traffic, materials, construction effects and environment. From the four fundamental variables trial designs are conducted with the help of structural analysis. The trial design that passes the limits of all the distresses is considered as the final design.

CHAPTER 3

MATERIALS AND METHODOLOGY

3.1 Material Properties

3.1.1 Recycled Low-Density Polyethylene (LDPE)

Recycled Low-Density Polyethylene (LDPE) is used extensively to produce tote bags for domestic goods. These bags become solid waste after their use and cause serious waste disposal problems. To solve this environmental problem, and at the same time to improve the drain down and other related engineering properties of the porous asphalt mixture, reclaimed LDPE bags were used in this investigation as additive in porous asphalt mixtures. Shredded LDPE material was used as an ingredient.

The supplier for recycled LDPE provided the test properties of the material with respect to density, tensile strength at break, elongation at break, impact strength, and melting point of the material as shown below Table 3.1 (Cetin et al., 2012b; LDPE, 2012; Asphalt, 2012; Punith & Veeraragavan, 2011). Recycled LDPE, which is identification number four, for this study was gathered, cleaned and shredded as shown below Figure 3.1.

Table 3.1. Recycled Low Density Polyethylene Features (LDPE, 2012)

Mechanical Properties	
Yield Strength	15-20 MPa
Elongation @ break	600-650 %
Bending Strength	10-40 MPa
Young's modulus (E)	200-400 MPa
Shear modulus	100-350 MPa
Tensile Strength (σ_t)	8-12 MPa
Physical Properties	
Density	910-928 kg/m^3
Thermal expansion	150-200 e-6/K
Water absorption	0.005-0.015 %
Melting Point	248 ^oF 120 ^oC
Thermal conductivity	0.3-0.335 W/m.K
Melting temperature	125-136 ^oC
Maximum Temperature	176 ^oF 80 ^oC
Minimum Temperature	58 ^oF 50 ^oC
Specific heat (c)	1800-3400 J/kg.K

Figure 3.1. Shredded for Recycled Low Density Polyethylene (LDPE).

3.1.2 Porous Aggregate

Crushed limestone was chosen as the course aggregate for mixing LDPE. Bulk samples were sieved in conformity with the sieve sizes for AASHTO No. 8 (American Association of State Highway and Transportation Officials). Figure 3.2 demonstrates the gradation for aggregates that were used. Porous aggregates conforming to the sizes 3/8 inches, Nos. 4, 8, 16 (AASHTO No. 8) were mixed with permeable plastic asphalt. Aggregates remained on each sieve were washed, dried for 24 hours in a 110^{o}C oven. This procedure provided regenerate samples that met AASHTO No. 8. This procedure allowed for greater control over each sample's gradation, an important factor since gradation has a significant impact on the engineering and physical features of mixed aggregates.

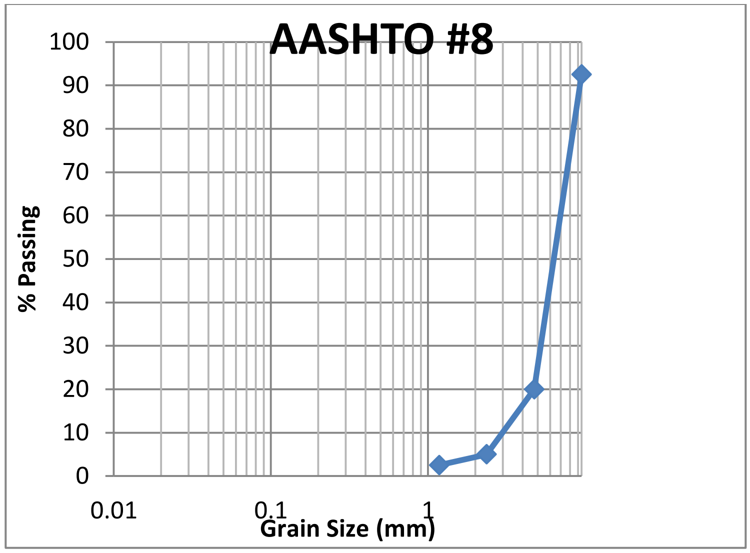

Figure 3.2. Grain Size Distribution of AASHTO No. 8.

3.1.3 Bitumen

"PG 64-22" was used in the porous asphalt mixture (Asphalt, 2012). Bitumen was mixed with Low-Density Polyethylene (LDPE) and porous aggregates. Samples were prepared for seven different percentages of bitumen (3.5%, 4%, 4.5%, 5%, 5.5%, 6% and 6.5% bitumen). Each sample was then prepared with 0%, 0.5% 1%, 2%, 3%, 4%, 5%, 6% and 7% LDPE. Seven types of mixtures with nine different percent LDPE and seven different percent bitumen were used at a mixing and compacting temperature of 160^{o}C. These are: Without LDPE, with 0.5% LDPE, with 1% LDPE, with 2% LDPE, with 3% LDPE, with 4% LDPE, with 5% LDPE, with 6% LDPE, and with 7% LDPE.

3.2 Samples Preparation and Compaction

3.2.1 Mixing of Shredded Waste Plastic (LDPE), Aggregate, and Bitumen

The aggregate and asphalt mix was heated to 160^{o}C in an oven. Shredded LDPE was mixed simultaneously with binder and aggregates and heated 160^{o}C for two hours so that a uniform mix was achieved. The mixture was taken out of the oven and compacted immediately to protect the moisture in the spacemens.

3.2.2 Porous Plastic Asphalt for Preparation and Compaction

Mixed plastic porous asphalt was poured and compacted into a mold, 4 inches in diameter and 2.5 inches high, using a steel shovel. The Marshall test procedure for designing porous mix by compacting the sample with 50 blows on one face by Marshall hammer, was used at varying binder contents. The mixture design trials used asphalt content in the range of 3.5 – 6.5 %, by total weight of the mixture, excluding the weight of the fibers, with 0.5% increments. The LDPE fibers were added to the porous mixtures at a dosage rate of 0, 0.5, 1, 2, 3, 4, 5, 6, and 7 % based on total mixture weight. The compacted samples were extracted from the mold when they had sufficiently cooled. After compaction, samples were kept in hot (conditioned) and cold (unconditioned) waters. Resilient Modulus tests were conducted. Samples were tested for indirect tensile strength. Appendix A shows the procedure for making samples. Figure A1 shows the preparation of the Porous plastic asphalt samples.

Figure A2 shows the plastic asphalt compactor. Figure A3 shows the mold used for making sample. Figure A4 is taken after compaction, extracting material from the mold. Figure A5 shows the Marshall Specimen Extractor. Figure A6 shows the sample in the water bath. Figure A7 shows the sample after being removed from the water bath. Figure A8 shows the sample prior o testing. Figure A9 shows the sample prior to ITS testing. Figure A10 shows the sample after the ITS test was completed. Figure A11 shows the computer used for testing with MTS machine.

Soil samples weight for determining moisture content was taken as shown in Figure A12. Figure A13 shows soil samples in the oven for taking moisture content. Figure A14 shows preparation of the soil sample. Water was added for preparing the soil sample as shown Figure A15. Figure A16 shows the soil sample after adding the water. Figure A17 shows the preparation and materials for soil compaction.

Figure A18 shows the preparations for weighing soil samples before compaction. Figure A19 shows the weighing before compaction of soil samples. The first layer of compaction is shown Figure A20. The first layer height was measured as shown in Figure A21. Figure A22 shows the soil compaction of the first layer. The second layer height was measured as shown in Figure A23.

Figure A24 is taken after all four layers were compacted. Figure A25 shows the preparations for taking out the samples. Figure A26 shows the sample with the membrane to obtain height and weight before the MTS test. Figure A27 shows the soil sample after removing the membrane. Figure A28 shows the procedure for obtaining the soil specimen weight. Figure A29 shows the preparing to put the new membrane. Figure A30 shows the arrangement of the new membrane for MTS testing. Figure A31 shows the sample after the MTS test. Figure A32 shows the samples after they are removed from the oven, before determining moisture content.

3.3 Laboratory Tests

3.3.1 Mix Design by Marshall Test

Processed plastic bags were used as an additive in bituminous concrete mixes. The processed plastic was used in different proportions (ranging from 3.5 to 6.5% by weight of bitumen) as an additive with heated bitumen and mixed well by hand, to obtain the modified bitumen. The properties of the modified bitumen were compared with ordinary bitumen.

Varying percentages of waste plastic by weight of bitumen was added to the heated aggregates. Marshall samples with varying waste plastic content were tested for stability. Maximum value of stability was the criteria for optimum waste plastic content.

The optimum modified binder content fulfilling the Marshall Mix design criteria was found to be 3.5%, %4, 4.5%, 5%, 5.5%, 6% and 6.5% by weight of the mix, consisting of 0%, 0.5%, 1%, 2%, 3%, 4%, 5%, 6%, and 7% by weight of processed plastic added to the bitumen. In order to evaluate the ability of the bitumen mix to withstand adverse soaking condition under water, Marshall Stability tests were conducted after soaking in water at $60^{o}C$ for 24 hours.

3.3.2 Indirect Tensile Strength (ITS)

Porous plastic asphalt samples were tested for indirect tensile strength. The indirect tensile tests/diametral tests (Appendix B) were conducted in accordance with the ASTM C 6931-07 test methods.

The indirect tensile test is a type of tensile strength test. It can yield significant information on the engineering properties of materials. The test has been run on asphalt-stabilized elements (Yoder & Witczak, 1975; Frocht, 1957; Hadley et al., 1971; Kennedy & Hudson, 1968). ITS was calculated from Equation 3.1 for use in the resilient modulus (M_R) test. In the diametral test, the tensile strength, S_t of the material is given by:

$$S_t = \frac{2Pmax}{\pi td} \qquad [3.1]$$

Where; S_t = Tensile strength

P = total applied load (lb)

t = specimen thickness (in.)

d = specimen diameter (in)

Repeated load tests on soils were conducted and the resilient modulus was measured through the Repeated Indirect Tensile mode for asphalt specimens. Statistical average of three samples was taken to evaluate the test results (Kuebler & Smith, 1976; Haldar & Mahadevan, 2000; Lipsey, 1990).

3.4 Test Results

The diametral test method was used to determine the tensile strength of the compacted bituminous mixtures. The porous plastic mix was tested as conditioned and unconditioned types. After compaction, samples were kept in the hot water (conditioned situation) and cold water (unconditioned situation). The objective of this test was to measure the water resistance of the mixture after immersion for 24 hours at 60°C. After that, the ITS was conducted in order to evaluate the resistance of porous plastic asphalt mix to deformation. The Tensile Strength Ratio (TSR) was calculated from Equation 3.2. TSR < 70% was considered susceptible to moisture. Moisture susceptibility of porous plastics was tested in accordance with ASTM C 6931-07. Samples that were four inched in diameter and 2.5 inches in height were cured at room temperature, 100°C and 160°C for 24 hours.

$$TSR = \frac{S1}{S2} \qquad [3.2]$$

Where:

S1 = conditioned set (wet)

S2 = unconditioned set (dry)

Table 3.2 shows the results of unconditioned ITS. The ITS for 5% binders mixed with 3% LDPE was 57 psi, while for 6% LDPE the ITS was 54 psi. As the percentage of LDPE increased, the results of unconditioned ITS decreased. On the contrary, the result of conditioned ITS increased from 62 psi to 68 psi with increased LDPE from 3% to 6% . Figure 3.3 shows the relationship between conditioned and unconditioned ITS. The last column of the table shows that the Tensile Strength Ratio, TSR is susceptible to moisture. The table proves that mixtures containing 3% or more LDPE are strong.

The results of k and ITS of porous plastic asphalt mixtures were within the approximately predictable ranked come up in the literature for conventional asphalt showing Table 3.3. It is an indication that porous plastic asphalt should be used as a sustainable alternative for pervious pavements.

Table 3.2. The Results of Indirect Tensile Strength Test (ITS)

Binder	LDPE	Unconditioned (psi)				Conditioned (psi)				TSR (con/uncon)
	0%	60	68	53	60	38	34	45	36	0.63
	0.5%	58	65	51	57	42	38	49	40	0.72
	1%	56	64	50	54	46	42	54	42	0.82
	2%	48	55	43	47	48	45	56	43	1
3.5%	3%	46	52	41	46	52	48	60	47	1.13
	4%	43	48	39	41	53	50	61	49	1.23
	5%	41	46	37	39	55	52	62	52	1.34
	6%	36	42	31	35	62	59	65	61	1.72
	7%	31	37	26	31	64	62	67	63	2.06
	0%	67	77	58	66	45	52	38	46	0.67
	0.5%	63	75	57	56	49	54	40	53	0.77
	1%	58	73	55	46	52	56	44	58	0.89
	2%	55	69	54	42	55	58	47	61	1
4%	3%	46	59	53	27	56	59	49	62	1.21
	4%	45	57	52	25	58	61	50	65	1.28
	5%	44	56	51	24	61	64	51	68	1.38
	6%	41	53	49	21	63	65	52	72	1.53
	7%	39	50	46	20	65	67	55	74	1.66
	0%	69	67	74	66	47	48	38	56	0.68
	0.5%	66	65	72	62	49	50	39	59	0.74
	1%	63	61	70	59	51	53	41	60	0.8
	2%	62	59	68	58	53	54	42	62	0.85
4.5%	3%	60	58	67	56	57	58	45	67	0.95
	4%	57	56	64	52	59	63	47	68	1.03
	5%	53	51	61	48	61	64	48	71	1.15
	6%	50	47	58	46	65	66	52	76	1.3
	7%	46	45	53	40	67	69	54	79	1.45

Table 3.2. (continued)

Binder	LDPE	Unconditioned (psi)				Conditioned (psi)				TSR (con/uncon)
	0%	71	74	79	59	54	51	44	67	0.76
	0.5%	65	66	74	55	57	52	46	72	0.87
	1%	62	61	71	53	59	54	49	73	0.95
	2%	60	60	69	51	60	55	51	74	1
5%	3%	57	57	68	48	62	56	53	77	1.09
	4%	56	55	66	47	64	58	55	79	1.14
	5%	55	54	64	46	66	59	58	80	1.2
	6%	54	52	63	46	68	61	61	81	1.26
	7%	51	49	61	43	70	62	65	82	1.37
	0%	70	64	71	76	55	61	53	52	0.78
	0.5%	69	62	70	74	59	64	54	58	0.85
	1%	67	59	69	72	62	66	58	63	0.92
	2%	63	56	65	69	66	69	62	67	1.04
5.5%	3%	60	54	62	65	68	72	64	69	1.13
	4%	58	52	59	62	70	73	67	71	1.2
	5%	56	51	55	61	73	77	69	74	1.3
	6%	54	49	53	59	76	79	73	76	1.4
	7%	51	47	51	56	78	82	74	77	1.52
	0%	73	67	74	78	63	71	57	62	0.86
	0.5%	71	65	73	75	65	74	59	63	0.91
	1%	69	62	72	73	67	76	61	64	0.97
	2%	67	60	70	71	69	79	63	65	1.02
6%	3%	66	58	69	70	71	81	64	67	1.08
	4%	64	57	67	68	72	83	65	68	1.12
	5%	62	55	65	66	74	86	67	69	1.19
	6%	61	54	64	65	75	87	68	71	1.23
	7%	57	51	62	59	79	91	72	74	1.38
	0%	77	73	78	80	68	85	71	70	0.88
	0.5%	75	71	75	78	72	80	69	68	0.96
	1%	71	68	74	70	74	82	72	69	1.04
	2%	68	65	72	67	77	85	75	71	1.13
6.5%	3%	63	62	66	61	79	89	77	72	1.25
	4%	61	60	63	59	81	91	79	74	1.32
	5%	59	58	62	57	85	92	83	79	1.44
	6%	56	55	60	54	88	94	86	83	1.57
	7%	54	52	57	52	91	95	89	89	1.68

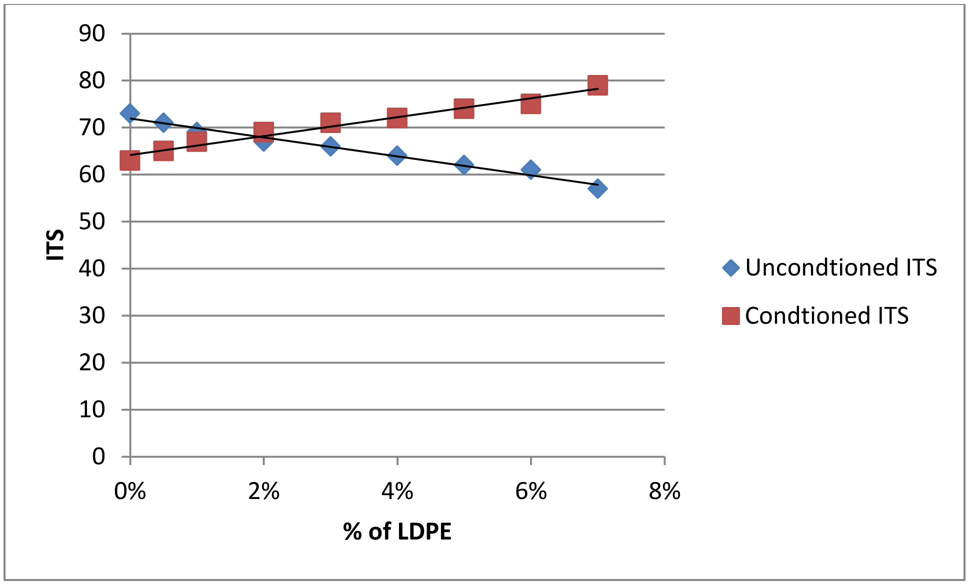

Figure 3.3. The Results of ITS with Mixing %6 LDPE.

Table 3.3. Summary of ITS Porous Asphalt from Literature

References	**ITS ranges (psi)**
Suresha et al. (2010)	29.29-69.76
Setiawan (2005)	18.17-38.82
Liu et al. (2010)	253.9-377.09
Shen et al. (2009)	29-65.26
Xiao et al. (2010)	26.1-130.53
Subagio et al. (2005)	18.30-55.90

3.5 Design with Computer Program

An iterative procedure was followed to predict surface pavement deflections under the wheel load. Using a computer program, "KENPAVE" (Huang, 2004) stresses, strains, and deflections were determined at all the important points in the pavement system. A strategy was designed to assign stiffness values to various layers of the pavement system such that there were negligible tensile stresses in the subgrade and base layers. The layer thicknesses were designed to keep targeting the surface deflection within acceptable limits. Several pavement sections were designed covering various environmental and traffic variables. The designed sections were compared with the AASHTO procedure and the differences critiqued (Cetin et al., 2012a; Cetin et al., 2012b).

Repeated load tests and repeated diametral tests were conducted on soils and surface course materials respectively.

3.6 Pavement Design by the Method of Limiting Surface Deflection

The following procedure was used (Cetin et al., 2012a).

1. Surface layer (h1) thicknesses were calculated by limiting surface deflection to 0.047 inches (Matthews & Pandey, 1991).
2. Two types of soils were selected.
3. Crushed stone was selected as the base material. Since crushed stone cannot take tensile stresses, their resilient moduli were calculated as double the value of subgrade resilient moduli.
4. Base thickness was taken as 6 and 12 inches.
5. Surface course stiffness was determined as a function of asphalt with LDPE content.

6. The required surface course thicknesses were determined by limiting the surface deflection to 0.047 (1 mm) inches. For this purpose KENPAVE (Huang, 2004) software was used. These values were cross checked by a procedure shown by Yoder & Witczak (1975).

CHAPTER 4

RESULTS AND DISCUSSION

The required surface course thicknesses are shown in Table 4.1. The vertical stress at the bottom of the surface layer, sigma z1 (Figure B1 in Appendix B), tensile stress at the bottom of the surface layer, sigma r1, and vertical stress at the bottom of base course, sigma z2 were calculated as shown in Table 4.2, Table 4.3, and Table 4.4. The results of KENPAVE (Huang, 2004) and the procedure shown in Yoder & Witczak (1975) agreed within 95% accuracy. It is important to note that the variation in soil stiffness used in this study could not affect the surface course thickness. In future, more study is needed involving a wide variety of soils for determining the effect of subgrade stiffness on surface course thickness.

4.1 Importance of Limiting Surface Deflection

One of the important functions of the surface course is to provide reasonably good friction and steering qualities for the vehicle riding the surface. Over time, if the surface course in the transverse direction is deformed, dangerous conditions may result. Water may accumulate in the deformations. Depending on the depth of the water, speed and weight of the vehicle travelling on the road surface, aquaplaning or hydroplaning can occur on the standing water. This phenomenon makes the wheels lose contact with the road and they simply surf on the water. This is because the instantaneous stiffness of water increases considerably to support the vehicle. When the driver loses contact still travelling at high speeds, severe accidents may occur. This research shows the importance of limiting surface deflection when designing and constructioning roads.

Table 4.1. Calculation of Surface Course Thicknesses for Soil 1, 2, and 3

Soil 1 Stiffness psi	Soil 2 Stiffness psi	Soil 3 Stiffness psi	Base Stiffness psi	Base Thickness inches	Surface Stiffness psi	Surface Course Thickness inches
7570	1180	4375	15140	6	250000	7
7570	1180	4375	15140	6	305000	11
7570	1180	4375	15140	6	277000	9
7570	1180	4375	15140	6	495000	19
7570	1180	4375	15140	6	490000	18
7570	1180	4375	15140	6	500000	20
7570	1180	4375	15140	6	100000	2
7570	1180	4375	15140	6	121000	6
7570	1180	4375	15140	6	131600	7
7570	1180	4375	15140	12	250000	7
7570	1180	4375	15140	12	305000	11
7570	1180	4375	15140	12	277000	9
7570	1180	4375	15140	12	495000	19
7570	1180	4375	15140	12	490000	18
7570	1180	4375	15140	12	500000	20
7570	1180	4375	15140	12	100000	2
7570	1180	4375	15140	12	121000	6
7570	1180	4375	15140	12	131600	7

Table 4.2. Calculation of Sigma z1, Sigma r1, and Sigma z2 for Soil 1

Soil 1 Stiffness psi	sigma z1	sigma r1	sigma z2 = sigma z3
7570	25.94	-97.79	10.74
7570	7.99	-93.40	4.99
7570	20.14	-95.98	10.05
7570	1.90	-30.85	1.70
7570	2.03	-30.81	1.90
7570	1.71	-31.14	1.60
7570	97.44	-73.22	44.88
7570	53.35	-70.38	16.67
7570	43.75	-79.98	14.65
7570	24.00	-94.56	7.10
7570	6.00	-93.40	2.30
7570	11.45	-81.14	5.60
7570	1.40	-30.73	0.85
7570	1.60	-30.62	1.20
7570	1.14	-31.07	0.71
7570	82.30	-82.59	14.20
7570	46.68	-72.07	10.30
7570	38.10	-80.66	8.25

Table 4.3. Calculation of Sigma z1, Sigma r1, and Sigma z2 for Soil 2

Soil 2 Stiffness psi	sigmaz1	sigma r1	sigma z2 = sigma z3
1180	20.1	-88.51	8.33
1180	2.15	-84.12	2.58
1180	14.3	-86.7	7.64
1180	1.66	-21.57	-0.71
1180	1.79	-21.53	-0.51
1180	1.47	-21.86	-0.81
1180	97.2	-63.94	42.47
1180	47.51	-61.1	14.26
1180	37.91	-70.7	12.24
1180	18.16	-85.28	4.69
1180	0.16	-84.12	-0.11
1180	5.61	-71.86	3.19
1180	1.16	-21.45	-1.56
1180	1.36	-21.34	-1.21

Table 4.3. (continued)

Soil 2 Stiffness psi	sigma z1	sigma r1	sigma z2 = sigma z3
1180	0.9	-21.79	-1.71
1180	76.46	-73.31	11.79
1180	40.84	-62.79	7.89
1180	32.26	-71.38	5.84

Table 4.4. Calculation of Sigma z1, Sigma r1, and Sigma z2 for Soil 3

Soil 3 Stiffness psi	sigma z1	sigma r1	sigma z2 = sigma z3
4375	24.52	-89.09	9.71
4375	5.92	-100.07	3.96
4375	10.39	-95.6	9.02
4375	0.1	-106.45	0.67
4375	0.55	-105.48	0.87
4375	-0.8	-106.91	0.57
4375	95.23	-38.41	43.85
4375	46.93	-67.72	15.64
4375	43.92	-76.41	13.62
4375	24.8	-90.21	6.07
4375	4.13	-73.82	1.27
4375	12.58	-89.41	4.57
4375	-0.9	-74.94	-0.18
4375	-0.8	-74.78	0.17
4375	-1	-75.05	-0.33
4375	86.19	-68.57	13.17
4375	46.15	-68.85	9.27
4375	41.04	-73.96	7.22

4.2 Pavement Design by AASHTO, Untreated Subgrade and Crushed Stone Base

(1) For the existing conditions and demands Structural Number (SN) needed is calculated from the monograph as sown in Table 4.5.

   a. for E3 (Stiffness) of 7,570, the soil support (S) value is 5.4.
   b. Traffic is 10 million standard axles.
   c. SN (Structural Number) bar is determined.
   d. Regional factor is taken a 1.0.
   e. SN weighted (SN needed) is calculated.

(2) For the existing conditions and demands SN existing is calculated from AASHTO (American Association of State Highway and Transportation Officials) equation as shown in Table 4.5, Table 4.6, and Table 4.7.

   SN existing = h1 * 0.44 (Plant Mix) + h2 * 0.14 (Crushed Stone)

(3) Comparison of Surface deflection and AASHTO methods is shown in Table 4.5, Table 4.6, and Table 4.7.

The comparison is for 5 sections for E3 = 7,570, 2 sections for E3 = 1,180, and 3 sections for E3 = 4,375 have failed.

Table 4.5. Soil 1 Pavement Design by AASHTO, Untreated Subgrade and Crushed Stone Base

E3:Soil	**inch**	**E=ff(Asp %) psi**	**inch**	**SN needed**	**SN existing**	**Pass/Fail**	**Recommendation**
	H2	**E1, Surface**	**H1='**				
7570	6	250000	7	4.2	3.92	**fail**	**Increase base by 1 inch**
7570	6	305000	11	4.2	5.68	pass	
7570	6	277000	9	4.2	4.8	pass	
7570	6	495000	19	4.2	9.2	pass	
7570	6	490000	18	4.2	8.76	pass	
7570	6	500000	20	4.2	9.64	pass	
7570	6	100000	2	4.2	1.72	**fail**	**Treat subgrade (SG) with lime**
7570	6	121000	6	4.2	3.48	**fail**	**Treat subgrade with lime**
7570	6	131600	7	4.2	3.92	**fail**	**Increase base by 1 inch**
7570	12	250000	7	4.2	4.76	pass	
7570	12	305000	11	4.2	6.52	pass	
7570	12	277000	9	4.2	5.64	pass	
7570	12	495000	19	4.2	10.04	pass	
7570	12	490000	18	4.2	9.6	pass	
7570	12	500000	20	4.2	10.48	pass	
7570	12	100000	2	4.2	2.56	**fail**	**Treat SG with lime**
7570	12	121000	6	4.2	4.32	pass	
7570	12	131600	7	4.2	4.76	pass	

Table 4.6. Soil 2 Pavement Design by AASHTO, Untreated Subgrade and Crushed Stone Base

E3:Soil	inch	E=ff(Asp %) psi	inch	SN needed	SN existing	Pass/Fail	Recommendation
	H2	E1, Surface	H1='				
1180	6	250000	7	3.4	3.92	pass	
1180	6	305000	11	3.4	5.68	pass	
1180	6	277000	9	3.4	4.8	pass	
1180	6	495000	19	3.4	9.2	pass	
1180	6	490000	18	3.4	8.76	pass	
1180	6	500000	20	3.4	9.64	pass	
1180	6	100000	2	3.4	1.72	**fail**	**Treat SG with Cement**
1180	6	121000	6	3.4	3.48	pass	
1180	6	131600	7	3.4	3.92	pass	
1180	12	250000	7	3.4	4.76	pass	
1180	12	305000	11	3.4	6.52	pass	
1180	12	277000	9	3.4	5.64	pass	
1180	12	495000	19	3.4	10.04	pass	
1180	12	490000	18	3.4	9.6	pass	
1180	12	500000	20	3.4	10.48	pass	
1180	12	100000	2	3.4	2.56	**fail**	**Treat SG with Cement**
1180	12	121000	6	3.4	4.32	pass	
1180	12	131600	7	3.4	4.76	pass	

Table 4.7. Soil 3 Pavement Design by AASHTO, Untreated Subgrade and Crushed Stone Base

E3:Soil	inch	E=ff(Asp %) psi	inch	SN needed	SN existing	Pass/Fail	Recommendation
	H2	E1, Surface	H1='				
4375	6	250000	7	3.9	3.92	pass	
4375	6	305000	11	3.9	5.68	pass	
4375	6	277000	9	3.9	4.8	pass	
4375	6	495000	19	3.9	9.2	pass	
4375	6	490000	18	3.9	8.76	pass	
4375	6	500000	20	3.9	9.64	pass	
4375	6	100000	2	3.9	1.72	**fail**	**Treat SG with Cement**
4375	6	121000	6	3.9	3.48	**fail**	**Treat SG with Cement**
4375	6	131600	7	3.9	3.92	pass	
4375	12	250000	7	3.9	4.76	pass	
4375	12	305000	11	3.9	6.52	pass	
4375	12	277000	9	3.9	5.64	pass	
4375	12	495000	19	3.9	10.04	pass	
4375	12	490000	18	3.9	9.6	pass	
4375	12	500000	20	3.9	10.48	pass	
4375	12	100000	2	3.9	2.56	**fail**	**Treat SG with Cement**
4375	12	121000	6	3.9	4.32	pass	
4375	12	131600	7	3.9	4.76	pass	

4.3 Pavement Design by AASHTO, Cement Treated Subgrade and Untreated Crushed Stone Base

(1) SN needed is calculated as 2.5 from the monograph as shown in Table 4.8, Table 4.9, and Table 4.10 for Soil 1, Soil 2, and Soil 3, respectively.

   a. for lime treated subgrade E3 = 2,800 psi, and soil support value is S = 9.

   b. Traffic is 10 million standard axles.

   c. SN bar is determined.

   d. Regional factor is taken a 1.0.

   e. SN weighted (SN needed) is calculated.

   f. Existing SN is shown in Table 4.8, Table 4.9, and Table 4.10.

(2) Determination of base thickness is shown in Table 4.8, Table 4.9, and Table 4.10.

H1 is taken as 2 inches for the design of most economical pavement.

4.4 Crushed Aggregate Base

The base thickness (H2) required to meet the SN needed (2.5 and 3.4) for Crushed Aggregated Base is calculated from the AASHTO equation as shown in Tables 4.8, Table 4.9, and Table 4.10.

(1) H1 (2 inches) * 0.44 (plant mix) + H2 * 0.14 (crushed stone) = 2.5 (SN needed)

Soil Support (S) = 9, for E3 = 28,000 psi, 7,570 (psi virgin + cement treated subgrade), traffic 10 million SN bar = 2.5, Regional Factor (R) = 1.0, SN Weighted = SN needed = 2.5.

H2 required = 11.57 inches Table 4.8.

(2) H1 (2 inches) * 0.44 (plant mix) + H2 * 0.14 (crushed stone) = 3.4 (SN needed)

S = 7, for E3 = 14,000 psi, 1,180 psi (virgin + lime treated subgrade), traffic = 10 million, SN bar = 3.4, R = 1.0, SN weighted = SN Needed = 3.4.

H2 required = 18 inches Table 4.9.

(3) H1 (2 inches) * 0.44 (plant mix) + H2 * 0.14 (crushed stone) = 2.5 (SN needed)

S = 9, for E3 = 25,000 psi, 4,375 psi (virgin + cement treated subgrade), traffic 10 million, SN bar = 3.1, R = 1, SN Weighted = SN needed = 3.1.

H2 required = 15.86 inches Table 4.10.

Table 4.8. Soil 1 Pavement Design by AASHTO, Cement Treated Subgrade (E3 = 28 ksi) and Untreated Crushed Stone Base

E3:Soil	inch	E=ff(Asp %) psi	inch	AASHTO design		crushed aggregate base
	H2	E1, Surface	H1='	LIME TREATED SOIL		h2=(p-(2*.44))/0.14
				S=9, SN needed=2.5	SN existing	base thickness inch
7570	6	250000	7	2.5	2.5	11.57
7570	6	305000	11	2.5	2.5	11.57
7570	6	277000	9	2.5	2.5	11.57
7570	6	495000	19	2.5	2.5	11.57
7570	6	490000	18	2.5	2.5	11.57
7570	6	500000	20	2.5	2.5	11.57
7570	6	100000	2	2.5	2.5	11.57
7570	6	121000	6	2.5	2.5	11.57
7570	6	131600	7	2.5	2.5	11.57
7570	12	250000	7	2.5	2.5	11.57
7570	12	305000	11	2.5	2.5	11.57
7570	12	277000	9	2.5	2.5	11.57
7570	12	495000	19	2.5	2.5	11.57
7570	12	490000	18	2.5	2.5	11.57
7570	12	500000	20	2.5	2.5	11.57
7570	12	100000	2	2.5	2.5	11.57
7570	12	121000	6	2.5	2.5	11.57
7570	12	131600	7	2.5	2.5	11.57

Table 4.9. Soil 2 Pavement Design by AASHTO, Lime Treated Subgrade (E3 = 14 ksi) and Untreated Crushed Stone Base

E3:Soil	inch H2	E=ff(Asp %) psi E1, Surface	inch H1='	SN existing	crushed aggregate base h2=(p-(2*.44))/0.14 base thickness inch
1180	6	250000	7	3.4	18.00
1180	6	305000	11	3.4	18.00
1180	6	277000	9	3.4	18.00
1180	6	495000	19	3.4	18.00
1180	6	490000	18	3.4	18.00
1180	6	500000	20	3.4	18.00
1180	6	100000	2	3.4	18.00
1180	6	121000	6	3.4	18.00
1180	6	131600	7	3.4	18.00
1180	12	250000	7	3.4	18.00
1180	12	305000	11	3.4	18.00
1180	12	277000	9	3.4	18.00
1180	12	495000	19	3.4	18.00
1180	12	490000	18	3.4	18.00
1180	12	500000	20	3.4	18.00
1180	12	100000	2	3.4	18.00
1180	12	121000	6	3.4	18.00
1180	12	131600	7	3.4	18.00

Table 4.10. Soil 3 Pavement Design by AASHTO, Cement Treated Subgrade (E3 = 25 ksi) and Untreated Crushed Stone Base

E3:Soil	inch H2	E=ff(Asp %) psi E1, Surface	inch H1='	SN existing	crushed aggregate base h2=(p-(2*.44))/0.14 base thickness inch
4375	6	250000	7	3.1	15.86
4375	6	305000	11	3.1	15.86
4375	6	277000	9	3.1	15.86
4375	6	495000	19	3.1	15.86
4375	6	490000	18	3.1	15.86
4375	6	500000	20	3.1	15.86
4375	6	100000	2	3.1	15.86
4375	6	121000	6	3.1	15.86
4375	6	131600	7	3.1	15.86
4375	12	250000	7	3.1	15.86
4375	12	305000	11	3.1	15.86
4375	12	277000	9	3.1	15.86
4375	12	495000	19	3.1	15.86
4375	12	490000	18	3.1	15.86
4375	12	500000	20	3.1	15.86
4375	12	100000	2	3.1	15.86
4375	12	121000	6	3.1	15.86
4375	12	131600	7	3.1	15.86

4.5 Lime Treated Base

The base thickness (H2) required to meet the SN needed (2.5 and 3.4) for Lime Treated Base is for lime treated base calculated from the AASHTO equation as shown in Table 4.11, Table 4.12 and Table 4.13.

(1) H1 (2 inches) * 0.44 (plant mix) + H2 * 0.14 (crushed stone) = 2.5 (SN needed)

S = 9, for E3 = 28,000 psi, 7,570 psi (virgin + cement treated subgrade), traffic 10 million, SN bar = 2.5, R = 1.0, SN Weighted = SN needed = 2.5.

H2 required = 5.40 inches, shown in Table 4.11.

(2) H1 (2 inches) * 0.44 (plant mix) + H2 * 0.14 (crushed stone) = 3.4 (SN needed) S =7, for E3 = 14000 psi, 1,180 psi (virgin + lime treated subgrade), traffic = 10 million, SN bar = 3.4, R=1.0, SN weighted = SN Needed = 3.4.

H2 required = 8.40 inches, Table 4.12. Plant mix (0.44) and crushed stone (0.14) materials are used.

(3) H1 (2 inches) * 0.44 (plant mix) + H2 * 0.14 (crushed stone) = 3.1 (SN needed)

S = 9, for E3 = 25,000 psi, 4,375 psi (virgin + cement treated subgrade), traffic 10 million, SN bar = 2.5, R = 1.0, SN Weighted = SN needed = 2.5.

H2 required = 7.40 inches, shown in Table 4.13.

Table 4.11. Soil 1 Pavement Design by AASHTO, Cement Treated Subgrade (E3 = 28 ksi) and Lime Treated Base

E3:Soil	inch H2	E=ff(Asp %) psi E1, Surface	inch H1='	lime treated base h2=(p-(2*.44))/0.3 base thickness inch
7570	6	250000	7	5.40
7570	6	305000	11	5.40
7570	6	277000	9	5.40
7570	6	495000	19	5.40
7570	6	490000	18	5.40
7570	6	500000	20	5.40
7570	6	100000	2	5.40
7570	6	121000	6	5.40
7570	6	131600	7	5.40
7570	12	250000	7	5.40
7570	12	305000	11	5.40
7570	12	277000	9	5.40
7570	12	495000	19	5.40
7570	12	490000	18	5.40
7570	12	500000	20	5.40
7570	12	100000	2	5.40
7570	12	121000	6	5.40
7570	12	131600	7	5.40

Table 4.12. Soil 2 Pavement Design by AASHTO, Lime Treated Subgrade (E3 = 14 ksi) and Lime Treated Base

E3:Soil	inch H2	E=ff(Asp %) psi E1, Surface	inch H1='	lime treated base h2=(p-(2*.44))/0.3 base thickness inch
1180	6	250000	7	8.40
1180	6	305000	11	8.40
1180	6	277000	9	8.40
1180	6	495000	19	8.40
1180	6	490000	18	8.40
1180	6	500000	20	8.40
1180	6	100000	2	8.40
1180	6	121000	6	8.40
1180	6	131600	7	8.40
1180	12	250000	7	8.40
1180	12	305000	11	8.40
1180	12	277000	9	8.40
1180	12	495000	19	8.40
1180	12	490000	18	8.40
1180	12	500000	20	8.40
1180	12	100000	2	8.40
1180	12	121000	6	8.40
1180	12	131600	7	8.40

Table 4.13. Soil 3 Pavement Design by AASHTO, Cement Treated Subgrade (E3 = 25 ksi) and Lime Treated Base

E3:Soil	inch H2	E=ff(Asp %) psi E1, Surface	inch H1='	lime treated base h2=(p-(2*.44))/0.3 base thickness inch
4375	6	250000	7	7.40
4375	6	305000	11	7.40
4375	6	277000	9	7.40
4375	6	495000	19	7.40
4375	6	490000	18	7.40
4375	6	500000	20	7.40
4375	6	100000	2	7.40
4375	6	121000	6	7.40
4375	6	131600	7	7.40
4375	12	250000	7	7.40
4375	12	305000	11	7.40
4375	12	277000	9	7.40
4375	12	495000	19	7.40
4375	12	490000	18	7.40
4375	12	500000	20	7.40
4375	12	100000	2	7.40
4375	12	121000	6	7.40
4375	12	131600	7	7.40

Appendix B contains detailed answers to the questions posed by the candidate's proposal. Table B1 and Table B2 show the detailed statistical analysis using ANOVA. Graphs showing the regression analysis are shown in Figures B2, B3, B4, B5, B6, B7, B8, B9, B10, B11, B12, B13, B14, and B15.

Use of the AASHTO method cannot ensure that there exists tensile stress not only in the subbase layers but also in the base layers. The materials commonly used in these layers cannot take tensile stress because they do not contain asphalt or any other binding materials. The research method in this study ensures that tensile stress is not present in these layers. Tensile stress is eliminated by properly modifying the thickness of the corresponding immediate top layer. The AASHTO method cannot specifically address the stiffness values of materials such as Low Density Polyethylene (LDPE) plastic. This method not only determines the stiffness of Low Density Polyethylene (LDPE) plastic by conducting laboratory tests as per the standards of the American Society for Testing and Materials but also uses the stiffness values in the design process for determining the structural design of surface layers for 63 pavement sections representing various temperatures and material conditions.

4.6 Mechanics of the Asphalt Mixtures

The normal and shear stress transmission between particles through deformable interparticle cement binders improves by adding performance graded asphalt and plastic to the aggregates (Dvorkin & Yin, 1995). Conventional performance graded asphalt aggregate mixtures contain microcracks. As a result of external loading in the form of repeated heavy axle loadings, these microcracks increase in size and number (Zhong & Chang, 1999). The gap grading present in the porous asphalt mixture exponentially aggravates this problem causing binders to weaken and fail. Since plastics have higher tensile strengths than traditional performance graded asphalts, plastics can increase the normal and shear strength of the mixture, thus resisting the normal and shear stress transmission between particles through interparticle cement binders. Long range binder-performance is significantly improved in rutting and fatigue modes of field pavements. It is important to note that the conventional performance graded asphalts present in rigidly adsorbed asphalt layers can exhibit only relatively smaller inter-particle attraction forces and particle-to-particle interactions (Buttlar & Roque, 1996). These forces and interactions decrease exponentially with respect to the degree of gap (or pores) grading of the mixture. When plastic is added to traditional performance graded asphalt, the inter-particle attraction forces and particle-to-particle interactions significantly increase and performance of gap graded asphalt mixtures improves to a great extent.

Increased aggregate angularity leads to a stiffer response of asphalt-aggregate mixtures in pavements (Zhu & Nodes, 2000). This is due to improved interlocking interaction for coarse aggregates such as those used in this study.

In summary, adding plastic to traditional asphalt binders improves mixture properties such as normal stiffness, shear stiffness, and elastic bulk modulus due to improvements in aggregate-binder interaction and intergranular interactions in the presence of the binder (Rothenburg et al., 1992).

CHAPTER 5

CONCLUSIONS

The minimum surface course thickness, 2 inches, corresponded to the lowest surface course stiffness which is 100,000 psi. The highest surface course thickness, 20 inches, corresponded to 500,000 psi of surface course stiffness. The average surface course thickness was 11 inches and the average surface course stiffness was 296,600 psi.

(1) For pavement sections with untreated subgrade and crushed stone base, 5 sections failed for soil with 7,570 psi stiffness and two failed for soil with 1,180 psi stiffness. The failure was attributed to a lack of minimum Structural Number (SN) value. More specifically, the two sections with 3.92 SN needed only one inch increase in the thickness of the base. For other sections with significantly smaller SN strengthening the Subgrade (SG) with lime is recommended. For soil with 7,570 psi stiffness, lime treatment is recommended rather than cement treatment because of cost considerations. For soil with 1,180 psi stiffness cement treatment is recommended as the first choice and lime treatment as the second choice.

(2) When sections with soil of 7,570 psi stiffness are treated with cement their stiffness increased to 28,000 psi. 11.6 inches of untreated crushed stone base was needed to handle 10 million standard axles of traffic.

(3) When sections with soil of 1,180 psi stiffness were treated with lime, their stiffness increased to 14,000 psi. 18 inches of untreated crushed stone base was needed to handle 10 million standard axles of traffic.

(4) When sections with soil of 7,570 psi stiffness are treated with cement their stiffness increased to 28,000 psi. 5.4 inches of lime treated crushed stone base was needed to handle 10 million standard axles of traffic.

(5) When sections with soil of 1,180 psi stiffness were treated with lime, their stiffness increased to 14,000 psi. 8.4 inches of lime treated crushed stone base was needed to handle 10 million standard axles of traffic.

In all these calculations, the general principle of pavement design was validated as the subgrade stiffness decreased, the base thickness increased for the same surface course thickness and traffic.

REFERENCES CITED

AASHO Road Test. (1961). The American Association of State Highway Officials (AASHO). Report 1, History and Description of the Project. *Highway Research Board Special Report Nos. 61A*, Highway Research Board, National Academy of Sciences, National Research Council, Washington D.C.

AASHO Road Test. (1962). The American Association of State Highway Officials (AASHO). Report 5, Pavement Research. *Highway Research Board Special Report Nos. 61E*, Highway Research Board, National Academy of Sciences, National Research Council, Washington D.C.

Acum, W. E. A. & Fox, L. (1950). Computation of load stresses in a three layer elastic system. *Geotechnique*, vol.2. no. 4, pp. 293-300.

Ahlborn, G. (1972). *ELSYM 5, computer program for determining stresses and deformations in a five-layer elastic system*. University of California, Berkeley, California.

Ahlvin, R. G. & Ulery, H. H. (1962). Tabulated values for determining the complete pattern of stresses, strains and deflections beneath uniform circular load on a homogeneous half space. *Highway Research Board Bulletin*, 342, pp. 1-13.

Ashton, J. E. & Moavenzadeh, F. (1968). Analysis of stresses and displacements in a three layered viscoelastic system. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements*. University of Michigan, pp. 209-219.

Asphalt. (2012). Asphalt 64-22. Cas no 8052-42-4 PG-2010-321, NuStar Asphalt Refining, LLC, 4Paradise Rd. Paulsboro, NJ, 08066, phone 856 224 7405.

Barksdale, R. D. & Leonards, G. A. (1967). Predicting performance of bituminous surfaced pavements. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements.* University of Michigan, pp. 321-340.

Barksdale, R. D. (1969). *Analysis of layered systems*. *Final Report, Project B-607*. School of Civil Engineering, Georgia Institute of Technology, Atlanta, Georgia.

Burmister, D. M. (1943). The theory of stresses and displacements in layered systems and applications to the design of airport runways. *Proceedings, Highway Research Board*. vol. 23, pp. 126-144.

Burmister, D. M. (1945). The general theory of stresses and displacements in layered soil systems. *Journal of Applied Physics*, vol.16, No.2, pp. 89-96; No. 3, pp. 126-127; No. 5, pp. 296-302.

Burmister, D. M. (1958). Evaluation of pavement systems of the WASHO road test by layered systems methods. *Highway Research Board Bulletin*. 177.

Buttlar, W. G. & Roque, R. (1996). Evaluation of empirical and theoretical models to determine asphalt concrete stiffnesses at low temperatures. *Journal of the Association of Asphalt Paving Technologists,* 66: 99-141.

Canadian Good Roads Association. (1962). Pavement evaluation studies in Canada. *Proceedings, International Conference on the Structural Design of Asphalt Pavements*. Michigan, pp. 137-218.

Cetin, M., Brooks, R. M. & Udo-Inyang, P. (2012a). An innovative design methodology of pavement design by limiting surface deflection. *International Journal of Research and Reviews in Applied Sciences*, ISSN: 2076-734X, EISSN: 2076-7366, volume 13, Issue 2, S. No: 26, November, 2012, pp. 607-610.

Cetin, M., Brooks, R. M. & Udo-Inyang, P. (2012b). A comparative study between the results of an innovative design methodology by limiting surface deflection and AASHTO design method. *International Journal of Research and Reviews in Applied Sciences,* ISSN: 2076-734X, EISSN: 2076-7366, volume 13, Issue 2, S. No: 27, November, 2012, pp. 611-616.

Coffman, B. S., Kraft, D. C. & Tamayo, J. (1964). A comparison of calculated and measured deflection for the AASHO test road. *Proceedings of the Association of Asphalt Paving Technologists*. vol.33, February, pp. 54-90.

Crawford, J. E. (1971). An analytical model for airfield pavement analysis. *AFPAVE, Air Force Weapons Laboratory Technical Report No*. *AFWL-TR-71-70*, 1 Nove 69-1 Jun 1971, Kirtland Air Force Base, New Mexico.

Croney, D. & Loe, J. A. (1965). Full-scale pavement design experiment on A1 at Alconbury Hill, Huntingdonshire. *Proceedings, Institution of Civil Engineers.* Paper No. 6848, vol. 30, London, February, pp. 255-270.

DeJong, D. L., Peutz, M. G. F. & Korswagen, A. R. (1973). *Computer programs, BISAR. Layered systems under normal and tangential surface loads*. *Koninklijke/shell-laboratorium*, Amsterdam, External Report AMSR 0006,73.

Doddihal, S. R. (1982). *Finite element solution of stress in granular materials and rutting in a flexible pavement*. Dissertation for the Degree of Philosophy. Indian Institute of Technology, Kharagpur.

Doddihal, S. & Pandey, B. B. (1984). Stresses in full depth granular pavements. *Transportation Research Board, Transportation Research Record* 954, Washington D.C., pp. 94-100.

Duncan, J. M., Monosmith, C. L. & Wilson, E. C. (1968). Finite element analysis of pavements. *Highway Research Board,* Highway Research Record, 228, pp. 18-33.

Dvorkin, J. & Yin, H. (1995). Contact laws for cemented grains: implications for grain and cement failure. *International Journal of Solids and Structures*, Volume 32, Issues 17-18, September 1995, pp. 2497-2510.

Foster, C. R. & Ahlvin, R. G. (1954). Stresses and deflections induced by a uniform circular load. *Proceedings, Highway Research Board*. vol. 33, pp. 467-470.

Fox, L. (1948). Computation of traffic stresses in a simple road structure. Department of Scientific and Industrial Research, Road Research Laboratory, *Road Research Technical Paper 9*, Crowthorne, U.K., pp. 1-27.

Freudenthal, A. M. & Lorsh, H. G. (1957). The infinite elastic beam on a linear viscoelastic foundation. *Journal of the Engineering Mechanics Division*. Proceedings of the ASCE, vol.83, no. Engineering Mechanics Divisons, No.1, January, pp. 1158-1158, 22.

Frocht, M. M. (1957). *Photoelasticit*, vol. 2, John Wiley and Sons, New York.

Hadley, W. O., Hudson W. R. & Kennedy T. W. (1971). Evaluation and prediction of the tensile properties of asphalt-treated materials. *Highway Research Board Annual Meeting*. Reseacrh Report No 98-9, Washington, D.C.

Haldar, A. & Mahadevan, S. (2000). *Probability, reliability and statistical methods in engineering design*. John Wiley and Sons, New York.

Hank, R. J. & Scrivner, F. H. (1948). Some numerical solutions of stresses in two and three layered systems. *Proceedings, Highway Research Board,* vol.28, pp. 457-468.

Heukelom, W. & Klomp, A. J. G. (1967). Consideration of calculated strains at various depths in connection with the stability of asphalt pavements. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements*. University of Michigan, Ann Arbor, pp. 155-168.

Hoskin, B. C. & Lee. E. H. (1959). Flexible surfaces on viscoelastic subgrades. *Journal of the Engineering Mechanics Division*, Proceedings of the ASCE., vol. 85, no. EM 4, October, pp. 11-30.

Huang, Y. H. (1967). Stresses and displacements in viscoelastic layered system under circular loaded area. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements*. University of Michigan, pp. 225-244.

Huang, Y. H. (2004). *KENPAVE. A Computer Package for Pavement Analysis and Design*. Professor Emeritus of Civil Engineering, University of Kentucky.

Ishihara, K. & Kimura, T. (1967). The theory of viscoelastic two-layer systems and conception of ITS application to the pavement design. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements.* University of Michigan, pp. 245-254.

Jones, A. (1962). Tables of stresses in three-layer elastic systems. *Highway Reach Board Bulletin 342,* pp. 176-214.

Kennedy, T. W. & Hudson W. R. (1968). Application of the indirect tensile test to stabilized materials. *Highway Research Board Annual Meeting*, Washington, D.C.

Ku, A. B. (1967). Stress strain law for viscoelastic flexible pavements under temperature variations. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements.* University of Michigan, pp. 255-257.

Kuebler, R. R. & Smith, H. (1976). *Statistics: a beginning approach*. John Wiley and Sons, New York.

LDPE. (2012). Husky, 2008. Polly America, 2000. West Marshall Drive, Grand Prairie, Texas, 75051, phone 800 527 3322, press 7654, www.poly-america.com (accessed 2 July, 2012).

Lee, A. R. & Croney, D. (1962). British full-scale pavement design experiments. *Proceedings, International Conference on the Structural Design of Asphalt Pavements.* Michigan, pp. 114-136.

Lipsey, M. W. (1990). *Design sensitivity: statistical power for experimental research*. Sage, Newbury Park, California.

Liu, Q., Schlangen E., Garcia, A. & Ven M. V. D. (2010). Induction heating of electrically conductive porous asphalt concrete. *Construction and Building Materials*. vol. 24, Issue 7, July 2010, pp. 1207-1213.

Matthews, J. M. & Pandey, B. B. (1991). Performance of flexible pavements. *Transportation Research Record.* Issue Number: 1307, Transportation Research Board, ISSN: 0361-1981, pp. 51-62.

Michelow, J. (1963). *Analysis of stresses and displacements in an N-layered elastic system under a load uniformly distributed on a circular area*. CHEVRON Computer Program, Sep., California Research Corporation, Richmond, California.

Monismith, C. L. & Secor, K. E. (1963). Viscoelastic behavior of asphalt concrete pavements. P*roceedings, Second International Conference on the Structural Design of Asphalt Pavements*. University of Michigan, pp. 476-498.

Monismith, C. L., Epps, J. A. & Finn, F. N. (1985). Improved asphalt mix design. *Proceedings, Association of Asphalt Paving Technologists*, vol.54, 1985.

Monismith, C. L., Finn, F. N. & Vallerga, B. A. (1987). A comprehensive asphalt concrete mixture design system. Paper prepared for presentation at *American Society for testing and Materials* (ASTM) symposium on development of more rational approaches to asphalt concrete mix design procedure, Bal Harbor, Florida, 9 December, 1987, pp. 39-71.

Muki, R. (1960). Asymmetric problems of the theory of elasticity for a semi-infinite solid and a thick plate. *Progress in Solid Mechanics*, vol. 1, North Holland Publishing Co., Amsterdam, pp. 400-439.

Newmark, N. M. (1947). Influence charts for computation of vertical displacements in elastic foundations. University of Illinois, *Experiment Station*, *Bulletin 367*.

NPDES. (2012). National Pollutant Discharge Elimination System (NPDES). http://cfpub.epa.gov/npdes/stormwater/menuofbmps/index.cfm?action=browse&Rbutton=detail&bmp=137 (accessed 5 August, 2012).

Peattie, K. R. (1962). Stress and strain factors for three layer elastic systems. *Highway Reach Board Bulletin 342*, pp. 215-253.

Perloff, W. H. & Moavenzadeh, F. (1967). Deflection of viscoelastic medium due to a moving load. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements*. University of Michigan, pp. 269-276.

Pister, K. S. & Monismith, C. L. (1960). Analysis of viscoelastic flexible pavements. Flexible Pavement Design Studies 1960, *Highway Research Bulletin 269*, pp. 1-15.

Pister, K. S. & Westmann, R. A. (1963). Analysis of viscoelastic pavements subjected to moving loads. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements.* University of Michigan, pp. 522-529.

Punith, V. S. & Veeraragavan, A. (2011). Characterization of OGFC mixtures containing reclaimed polyethylene fibers. *Journal of Materials in Civil Engineering*, ASCE, 341, March.

Rothenburg, L., Bogobowicz, A., Haas, R., Jung, F. W. & Kennepohl, G. (1992). Micromechanical modeling of asphalt concrete in connection with pavement rutting problems. *In Proceedings of the seventh International Conference on Asphalt Pavements*, University of Nottingham, Nottingham, U.K. vol. 1, pp. 230-245.

Sanborn, J. L. & Yoder, E. J. (1968). Stress and displacement in an elastic mass under semi-ellipsoidal loads. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements,* University of Michigan, pp. 309-319.

Schiffman, R. L. (1963). General analysis of stresses and displacements in layered elastic systems. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements*. University of Michigan, pp. 365-375.

Seed, H. B., Mitry, F. G., Monismith, C. L. & Chan, C. K. (1961). *Prediction of pavement deflections from laboratory repeated load test*. Department of Civil Engineering, University of California, Berkeley.

Setiawan, A. (2005). Design and properties of hot mixture porous asphalt for semi-flexible pavement applications. *Media Technic Sipil*, Vol 5, No.2, July 2005, pp. 41-46.

Shen, D., Wu, C. & Du J. (2009). Laboratory investigation of basic oxygen furnace slag for substitution of aggregate in porous asphalt mixture. *Construction and Building Materials 23*, vol 23, Issue 1, ISSN: 0950-0618, January 2009, pp. 453-461.

SPI. (2011). Society of Plastics Industry Trade Association. (SPI). SPI Resin Identification Code - Guide to Correct Use. Washington. D.C. http://www.plasticsindustry.org/AboutPlastics/content.cfm?ItemNumber=823&navItemNumber=1125 (accessed 2 November, 2011).

Subagio, B., Kosasih, D., Busnial, & Tenrilangi, D. (2005). Development of stiffness modulus and plastic deformation characteristics of porous asphalt mixture using Tafpack Super™. *Proceedings of the Eastern Asia Society for Transportation Studies*, vol. 5, pp. 803 - 812.

Suresha, S. N., George, V. & Shankar, A. U. R. (2010). Effect of aggregate gradations on properties of porous friction course mixes. *Material and Structure*, vol 43, Issue 6, July 2010, pp. 789-801.

Ullidtz, P. & Peattie, K. R. (1980). Pavement analysis by programmable calculators. *Journal of the Transportation Engineering Division*, Proceedings of the ASCE. vol.106, no.TE5, September, pp. 581-597.

US EPA. (2012). United States Environmental Protection Agency (US EPA). Resource Conservation - Common Wastes & Materials http://www.epa.gov/osw/conserve/materials/plastics.htm (accessed 10 September, 2012).

Verstraeten, J. (1968). Stresses and displacements in elastic layered systems. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements.* University of Michigan, pp. 227-290.

WASHO Road Test (1954). The Western Association of State highway officials (WASHO). The WASHO Road Test-part II: test data, analysis, findings, *Highway Research Board Special Report Ns. 18*, Highway Research Board, National Research Council, Washington D.C.

WASHO Road Test (1955). The Western Association of State highway officials (WASHO). The WASHO Road Test-part II: test data, analysis, findings, *Highway Research Board Special Report Ns. 22*, Highway Research Board, National Research Council, Washington D.C.

Waterhouse, A. (1967). Stresses in layered systems under static and dynamic loading. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements.* University of Michigan, pp. 291-308.

Westman, R. A. (1962). *Viscoelastic and thermoplastic analysis of layered systems.* Dissertation for the Degree of Doctor of Philosophy, University of California.

Xiao, F., Zhao, W., Gandhi, T. & Amirkhanian, S. N. (2010). Influence of antistripping additives on moisture susceptibility of warm mix asphalt mixture. *Journal of Materials in Civil Engineering*, ASCE, October, 2010, pp. 1047-1055.

Yettram, A. L., O'Flaherty, C. A. & Fleming, M. E. (1968). The analysis of pavements under arbitrary loading. *Proceedings, Second International Conference on the Structural Design of Asphalt Pavements.* University of Michigan, Ann Arbor, pp. 259-268.

Yoder, E. J. & Witczak, M. W. (1975). *Principles of pavement design*. John Wily and Sons, Inc., Second Edition, New York.

Zhong, X. & Chang, C. S. (1999). Micromechanical modeling for behavior of cementitious granular materials. ASCE *Journal of Engineering Mechanics,* 125(11): 1280-1285.

Zhu, H. & Nodes, J. E. (2000). Contact based analysis of asphalt pavement with the effect of aggregate angularity. *Mechanics of Materials*, 32(3): 193-202.

APPENDICES

APPENDIX A

SAMPLE PREPARATION PICTURES

Figure A1 shows the preparation of the Porous plastic asphalt samples. Figure A2 shows the plastic asphalt compactor. Figure A3 shows the mold used for making sample. Figure A4 is taken after compaction, extracting material from the mold. Figure A5 shows the Marshall Specimen Extractor. Figure A6 shows the sample in the water bath. Figure A7 shows the sample after being removed from the water bath. Figure A8 shows the sample prior o testing. Figure A9 shows the sample prior to ITS testing. Figure A10 shows the sample after the ITS test was completed. Figure A11 shows the computer used for testing with MTS machine.

Soil samples weight for determining moisture content was taken as shown in Figure A12. Figure A13 shows soil samples in the oven for taking moisture content. Figure A14 shows preparation of the soil sample. Water was added for preparing the soil sample as shown Figure A15. Figure A16 shows the soil sample after adding the water. Figure A17 shows the preparation and materials for soil compaction. Figure A18 shows the preparations for weighing soil samples before compaction. Figure A19 shows the weighing before compaction of soil samples. The first layer of compaction is shown Figure A20. The first layer height was measured as shown in Figure A21. Figure A22 shows the soil compaction of the first layer. The second layer height was measured as shown in Figure A23.

Figure A24 is taken after all four layers were compacted. Figure A25 shows the preparations for taking out the samples. Figure A26 shows the sample with the membrane to obtain height and weight before the MTS test. Figure A27 shows the soil sample after removing the membrane. Figure A28 shows the procedure for obtaining the soil specimen weight. Figure A29 shows the preparing to put the new membrane. Figure A30 shows the arrangement of the new membrane for MTS testing. Figure A31 shows the sample after the MTS test. Figure A32 shows the samples after they are removed from the oven, before determining moisture content.

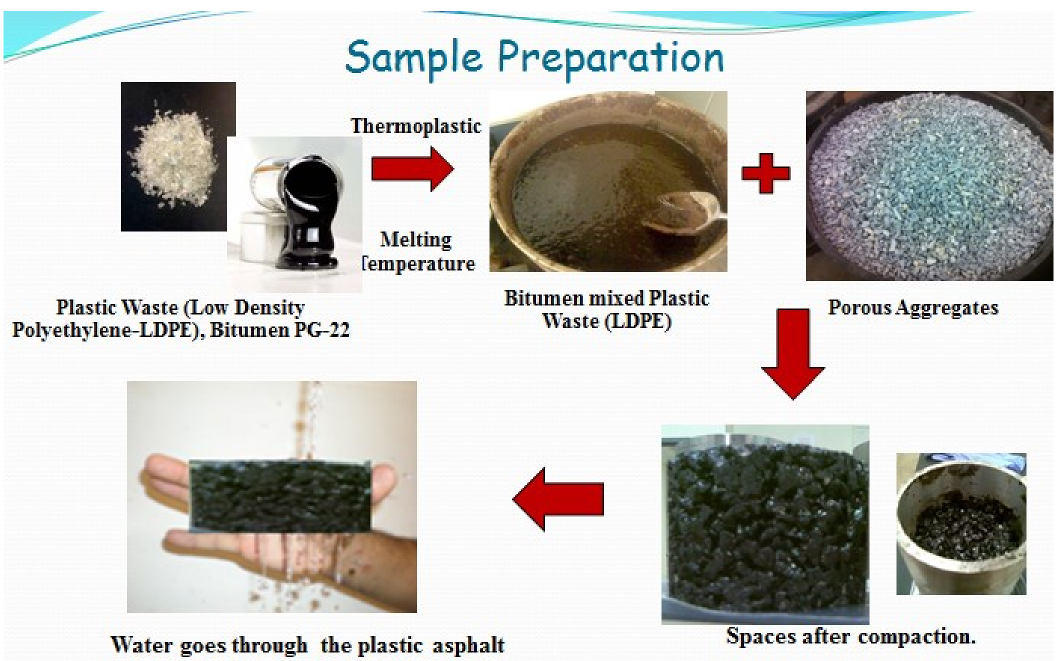

Figure A1. Sample Preparations for Permeable Plastic Asphalt that Mixing of LDPE, Porous Aggregates, and Binder to Create.

Figure A2. Plastic Asphalt Compactor.

Figure A3. Mold Used for Making Sample.

Figure A4. After Compaction, Extracting Material from Mold.

Figure A5. Marshall Specimen Extractor.

Figure A6. Sample's Bath.

Figure A7. After Water Bath.

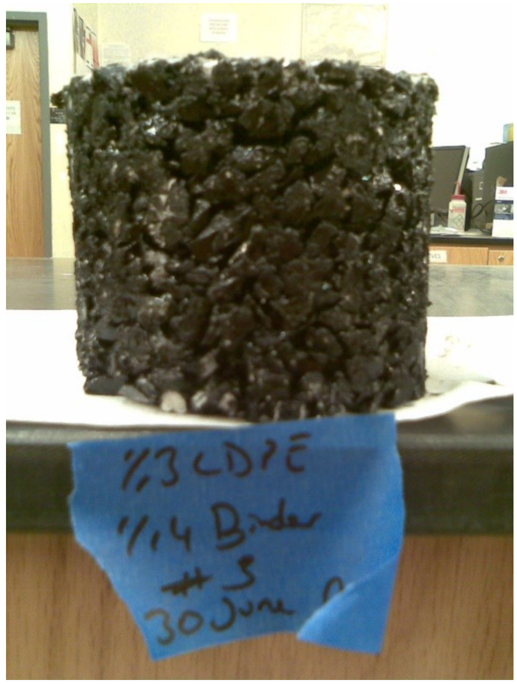

Figure A8. After Extracting, Material is Ready for Testing.

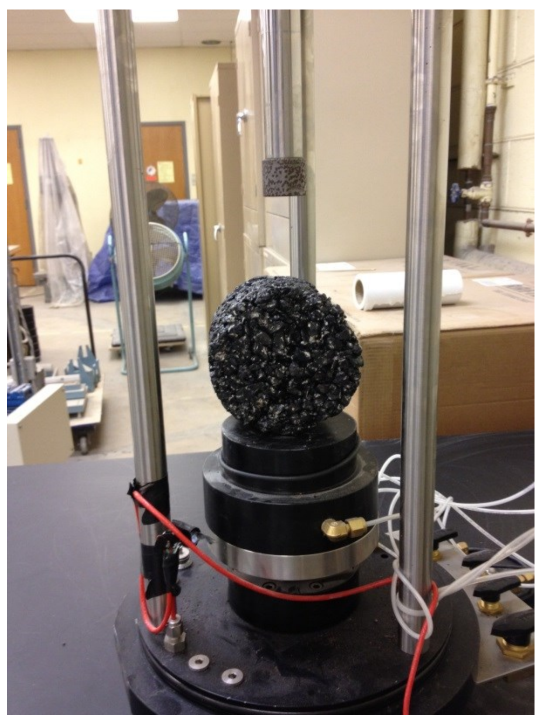

Figure A9. Plastic Asphalt Sample Ready for ITS Testing.

Figure A10. After ITS Test is Completed.

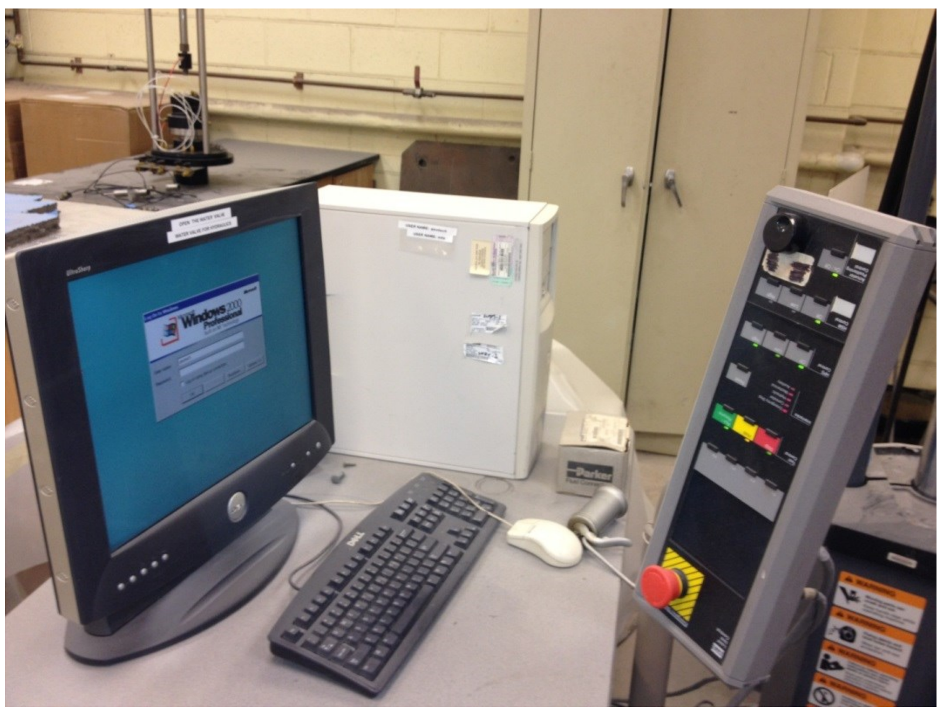

Figure A11. Computer Used for Testing with MTS Machine.

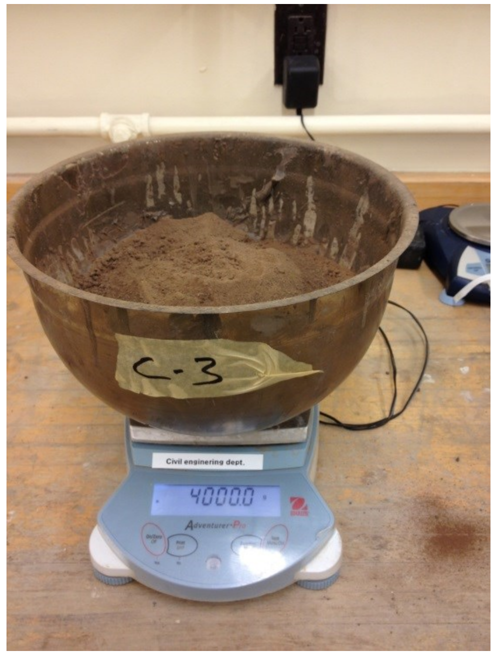

Figure A12. Weighing Soil Samples for Moisture Content Test.

Figure A13. Soil Samples in Oven for Taking Moisture Content.

Figure A14. Preparing Soil Sample.

Figure A15. Water Added to the Soil Samples.

Figure A16. After Adding Water for Making Soil Samples.

Figure A17. Preparing for Soil Compaction of Materials.

Figure A18. Preparing for Weighing Soil Samples before Compaction.

Figure A19. Weighing before Compaction of Soil Samples.

Figure A20. First Layer Soil Compaction.

Figure A21 Scaling for First Layer Height after First Layer Compaction.

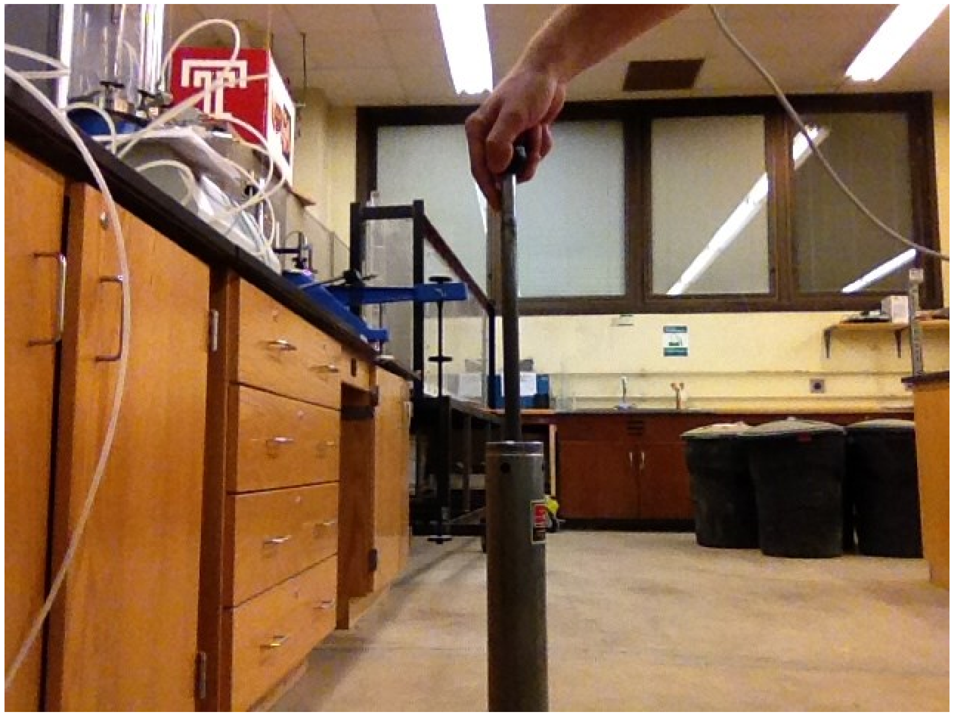

Figure A22. Soil Compaction.

Figure A23. Scaling after Second Layer Compaction.

Figure A24. After the Fourth Layer Compaction.

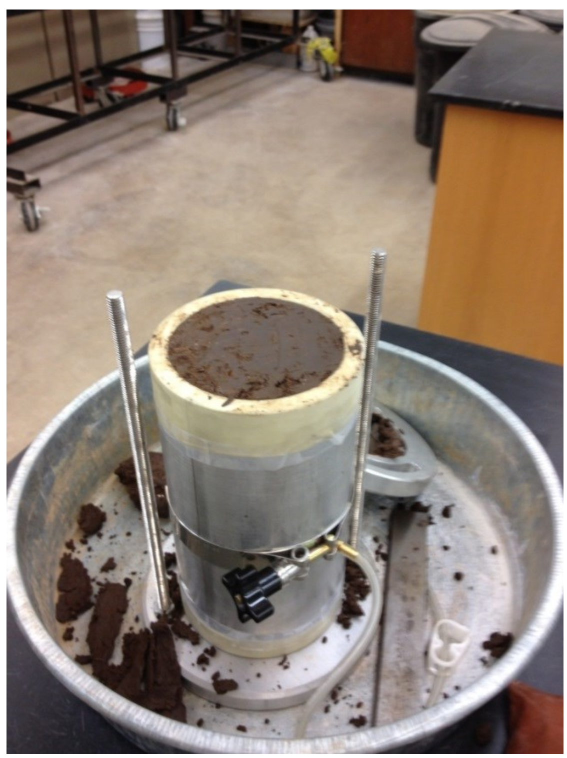

Figure A25. Preparing to Take It Out after Fourth Layer Compaction.

Figure A26. Covering with Membrane to Obtain Height and Weight before MTS Test.

Figure A27. After Removing the Membrane to Obtain Height and Weight before Testing.

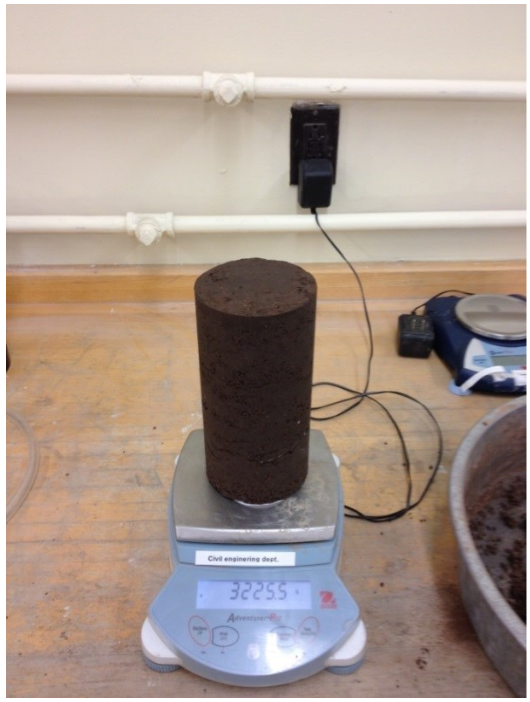

Figure A28. Weighing before MTS Test.

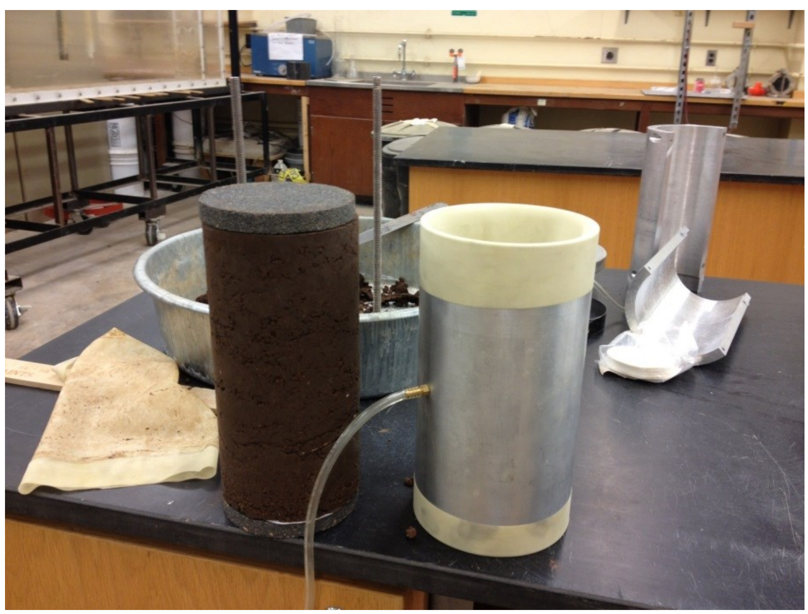

Figure A29. Preparing to Put New Membrane.

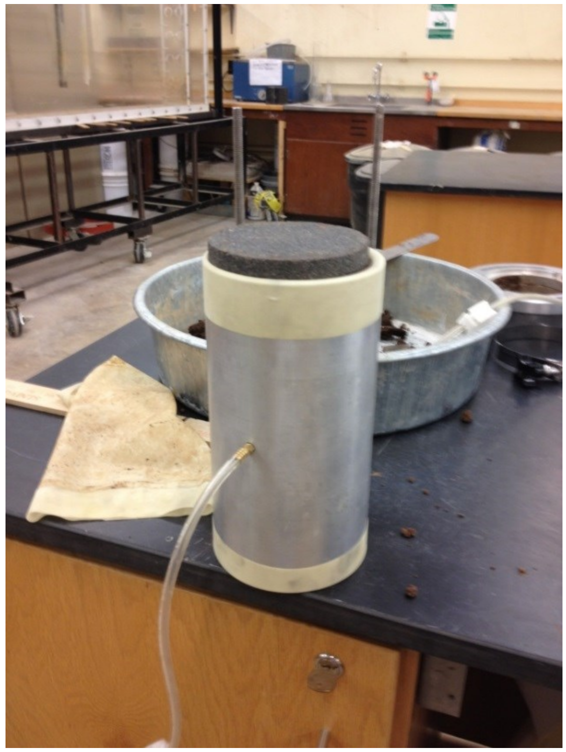

Figure A30. Putting New Membrane for MTS Testing.

Figure A31. After MTS Test, Taking Weight and Height and Moisture Content.

Figure A32. Removed Sample from Oven after Moisture Content.

APPENDIX B

DIAMETRAL TEST PROCEDURE, ENVIRONMENTAL CONDITIONS, AND STATISTICAL ANALYSIS

M_R and ITS Diametral Test Equations

- The resilient modulus (M_R) is a valuable constant for using in the mechanistic pavement design as it is being worked as an input to the multilayer elastic theories or finite elements models for calculating pavement reaction during traffic loading.
- These reactions should be worked through transfer capacities for computing the minimum thickness plan for new pavement or to approximately decide the resting life of a current pavement.
- The resilient modulus is identified as the scale of the deviator stress to the recoverable strain that is shown Equation B1.
- This research was usually implemented for checking into thoroughly the result of various factors on the resilient modulus of an asphalt mix subjected to resilient modulus testing by the indirect tensile strength (Yoder & Witczak, 1975).

- $E = \frac{P(v+0.27)}{Hh}$ [B1]

  Where;

  E = resilient modulus (MPa or 145 psi)

  P = peak load (N or 0.225 pounds)

  V = poison ratio (0.35 for commonly using asphalt)

  H = recovered horizontal deformation sample (mm or 0.0394 inches)

  h = height of sample (mm or 0.0394 inches)

- The elastic modulus of asphalt-treated materials is used as resilient modulus.
- It is fundamentally a repetitive load test by use of the stress allocation principles of the indirect tensile strength.
- This research the corresponding of the maximum tensile strain side to side the horizontal diametral of a specimen either computed or calculated whereby equations.
- For the horizontal diametral level, the mode of being of stress is shown by Equations B2, B3, and B4. For the vertical diametral level onward the load axis, the stresses are demonstrated by Equations B5, B6, and B7 (Yoder & Witczak, 1975).

Horizontal diametral:

- $\sigma_x = \frac{2P}{\pi td}\left[\frac{d^2-4x^2}{d^2+4x^2}\right]^2$ [B2]

- $\sigma_y = -\frac{2P}{\pi td}\left[\frac{4d^2}{(d^2+4x^2)} - 1\right]$ [B3]

- $\tau_{xy} = 0$ [B4]

Vertical diametral:

- $\sigma_x = \frac{2P}{\pi td} = Constant$ [B5]

- $\sigma_y = -\frac{2P}{\pi t}\left[\frac{2}{d-2y} + \frac{2}{d+2y} - \frac{1}{d}\right]$ [B6]

- $\tau_{xy} = 0$ [B7]

Where P = total applied load (lb)

t = specimen thickness (in)

d = specimen diameter (in)

x, y = coordinate values from center of specimen

- Frocht (1957) is demonstrated the showing all of equations to an idealized elastic dimensional. Beginning failure happens by tensile splitting in accordance with Equation B8 for usually engineering materials. As a result, the tensile strength S_t of the material is shown by:

- $S_t = \frac{2Pmax}{\pi td}$ [B8]

Where; S_t = Tensile strength

P = total applied load (lb)

t = specimen thickness (in.)

d = specimen diameter (in)

- In the process a repetitive pulsating load of 0.1 second duration 2.9 second dwell time is implemented diametrically to the specimen.
- The dynamic load outcomes in dynamic deformations side to side the horizontal diametral level, respectively. These deformations are documented by transducers appending on each side of the horizontal sample axis.
- assess to approximately calculation the capacity for rutting or cracking.
- worked for balance materials.
- Knowledge of the dynamic load and deformation permits the resilient modulus M_R results for being computing.
- The outcome is worked to identify the capacity for area pavement moisture damage when outcomes are acquired on both moisture-conditioned and unconditioned samples.

Tensile Strain at the bottom of asphalt layer is shown Figure B1.

- $\sigma z1$ = vertical stress at interface 1
- $\sigma z2$ = vertical stress at interface 2
- $\sigma r1$ = horizontal stress at bottom of layer 1
- $\sigma r2$ = horizontal stress at bottom of layer 2
- $\sigma r3$ = horizontal stress at the top of layer 3

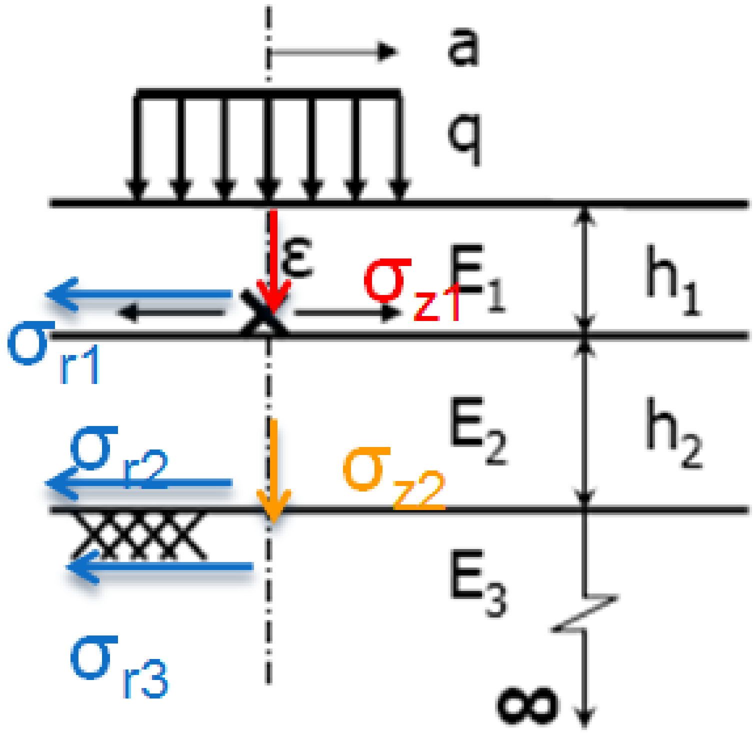

Figure B1. Tensile Strain at the Bottom of Asphalt Layer.

Environmental and load effects through stiffness values

- Effect of temperature on rutting
- Effect of temperature on fatigue
- Effects of subgrade-moisture on rutting
- Loss of serviceability due to traffic load
- Effect of swell and frost on rutting
- Effect of swell and frost on fatigue

All the above are shown by the Regional Factor in the AASHTO Design method

Statistical Analysis

Table B1 shows the results of ITS for unconditioned and conditioned asphalt. After using ANOVA test for statistical analysis, the results are shown Table B2.

Table B1. The Results of ITS for Unconditioned and Conditioned Asphalt for Statistical Analysis

Binder	LDPE	Unconditioned	Conditioned
	0%	60	38
	0.5%	58	42
	1%	56	46
	2%	48	48
%3.5	3%	46	52
	4%	43	53
	5%	41	55
	6%	36	62
	7%	31	64
	0%	67	45
	0.5%	63	49
	1%	58	52
	2%	55	55
%4	3%	46	56
	4%	45	58
	5%	44	61
	6%	41	63
	7%	39	65
	0%	69	47
	0.5%	66	49
	1%	63	51
	2%	62	53
%4.5	3%	60	57
	4%	57	59
	5%	53	61
	6%	50	65
	7%	46	67

Table B1. (continued)

Binder	LDPE	Unconditioned	Conditioned
	0%	71	54
	0.5%	65	57
	1%	62	59
	2%	60	60
%5	3%	57	62
	4%	56	64
	5%	55	66
	6%	54	68
	7%	51	70
	0%	70	55
	0.5%	69	59
	1%	67	62
	2%	63	66
%5.5	3%	60	68
	4%	58	70
	5%	56	73
	6%	54	76
	7%	51	78
	0%	73	63
	0.5%	71	65
	1%	69	67
	2%	67	69
%6	3%	66	71
	4%	64	72
	5%	62	74
	6%	61	75
	7%	57	79
	0%	77	68
	0.5%	75	72
	1%	71	74
	2%	68	77
%6.5	3%	63	79
	4%	61	81
	5%	59	85
	6%	56	88
	7%	54	91

Using ANOVA for statistical results in shown Table B2.

Table B2. ANOVA Results

Anova: Single Factor		alfa=0.01				
SUMMARY						
Groups	*Count*	*Sum*	*Average*	*Variance*		
LDPE	63	1.995	0.031667	0.000564516		
Unconditioned	63	3656	58.03175	99.5796211		
Conditioned	63	3990	63.33333	127.7741935		
ANOVA						
Source of Variation	*SS*	*df*	*MS*	*F*	*P-value*	*F crit*
Between Groups	155383.5578	2	77691.78	1025.163173	3.6135E-101	4.721095
Within Groups	14095.97151	186	75.78479			
Total	169479.5293	188				

ANOVA Conclusion: **The difference between unconditioned and continued results is significant at alfa=0.01.**

Regression analysis

For regression analyses for the results of ITS 3.5%, 4%, 4.5% 5%, 5.5%, 6%, and 6.5% asphalt mix LDPE unconditioned and conditioned are shown in Figures B2, B3, B4, B5, B6, B7, B8, B9, B10, B11, B12, B13, B14, B15.

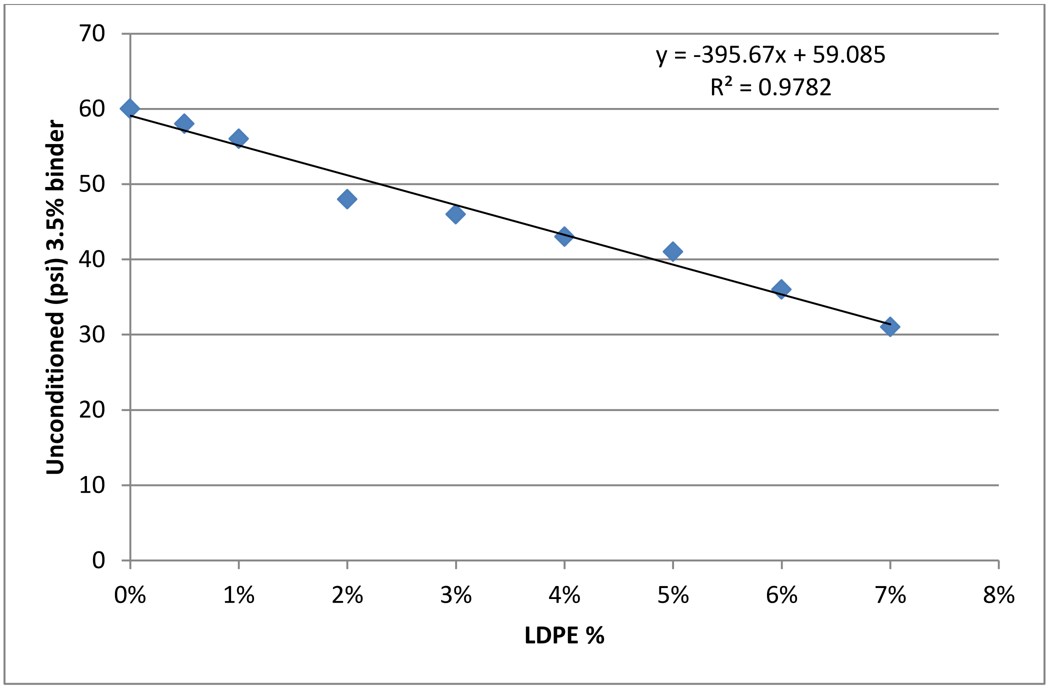

Figure B2. LDPE % vs. Stiffness (Unconditioned) 3.5% Binder.

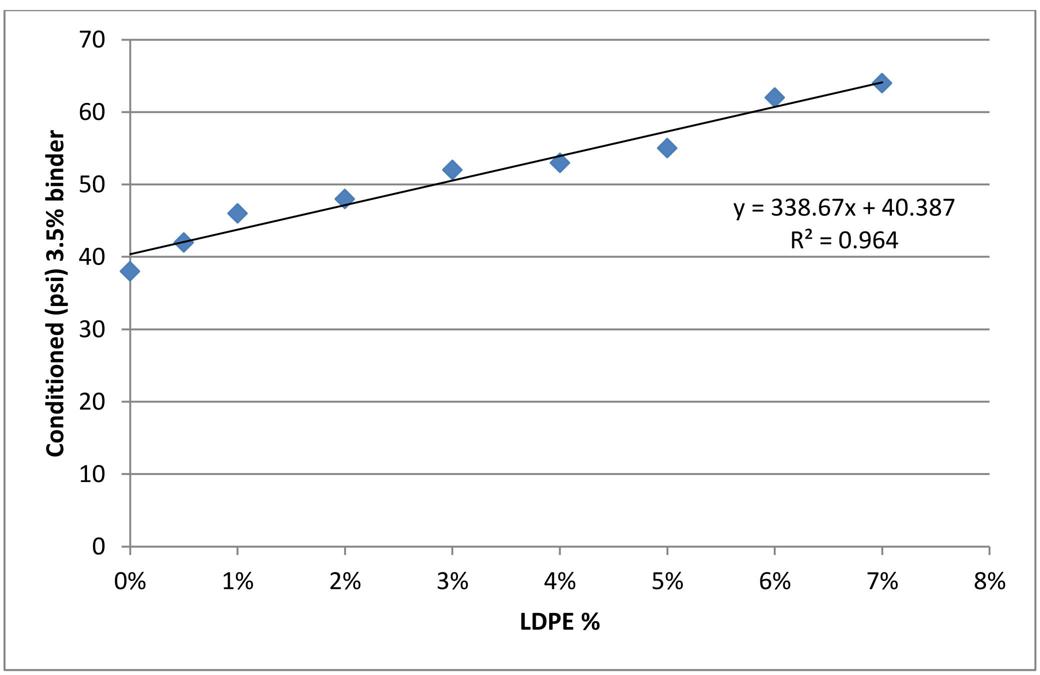

Figure B3. LDPE % vs. Stiffness (Conditioned) 3.5% Binder.

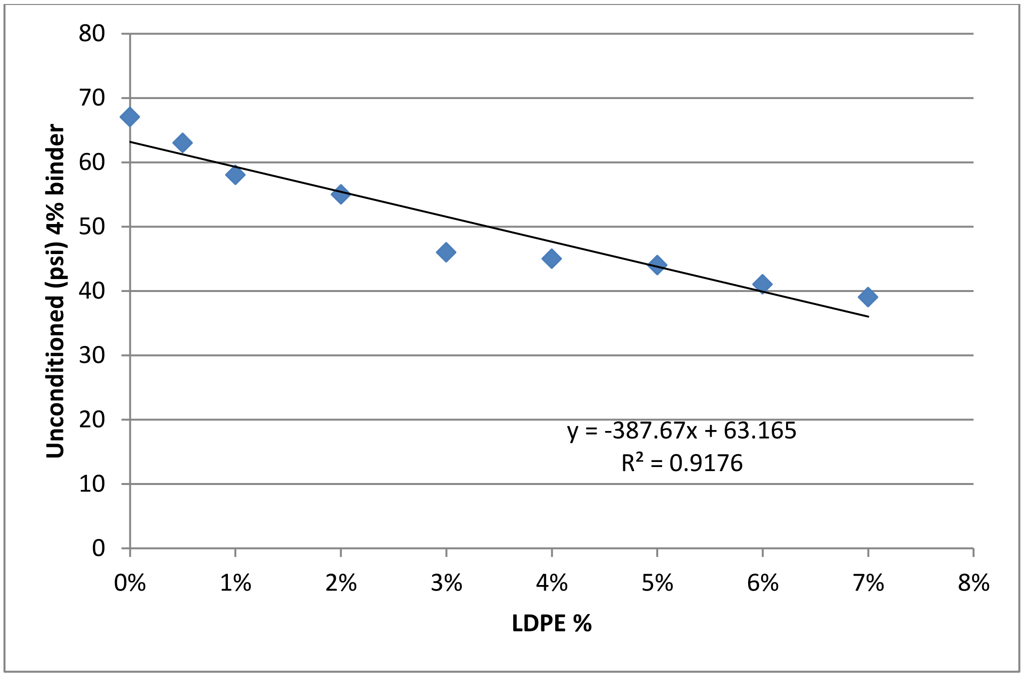

Figure B4. LDPE % vs. Stiffness (Unconditioned) 4 % Binder.

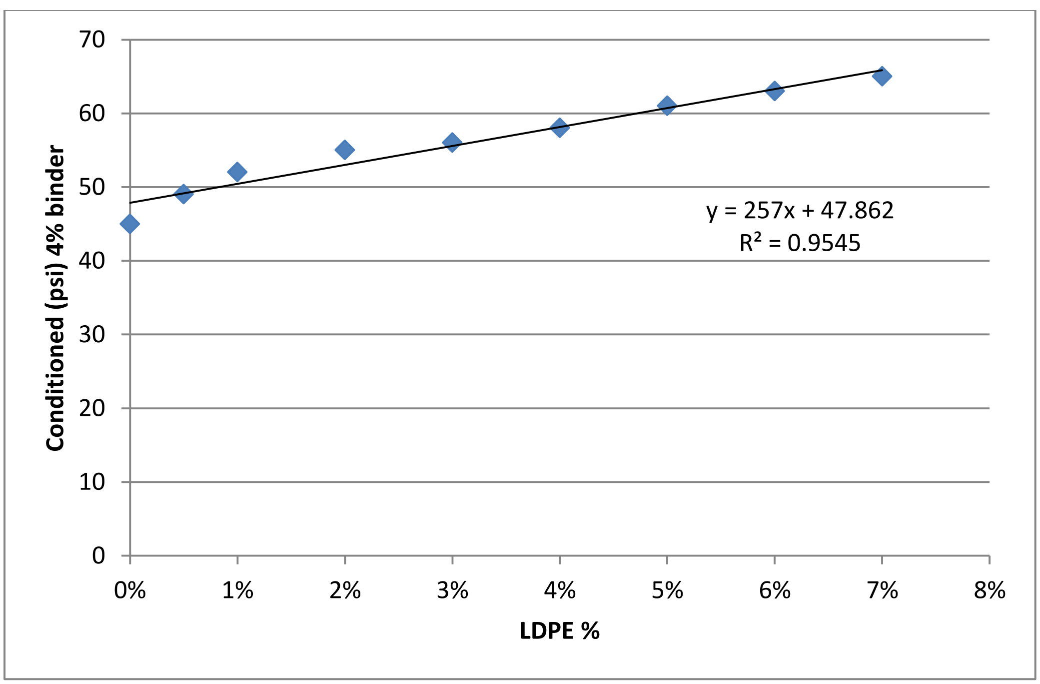

Figure B5. LDPE % vs. Stiffness (Conditioned) 4 % Binder.

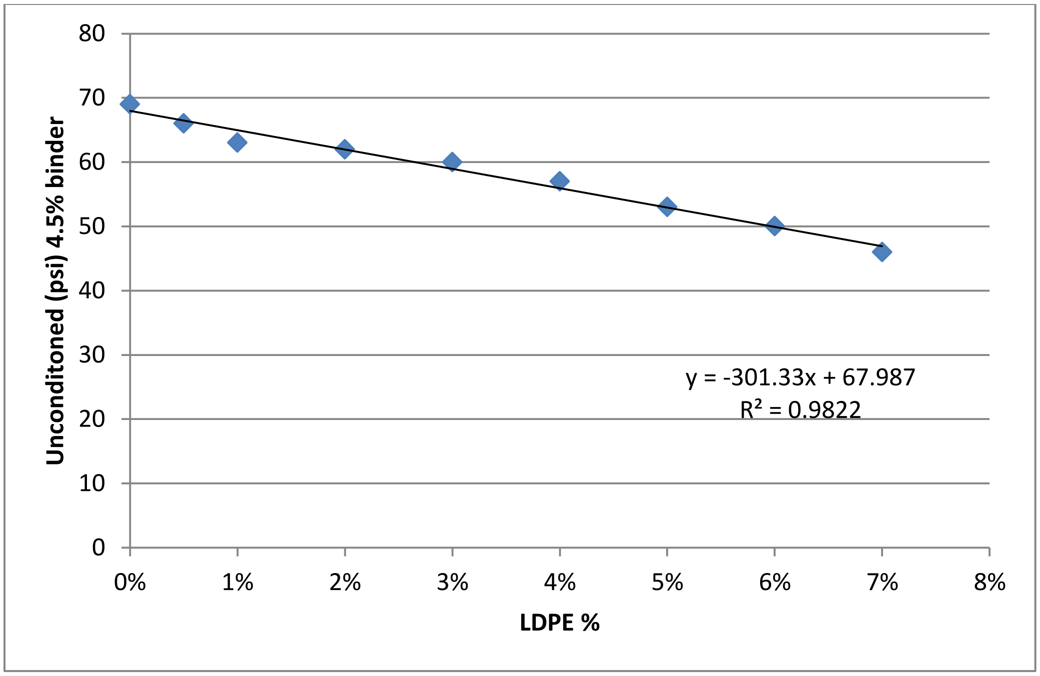

Figure B6. LDPE % vs. Stiffness (Unconditioned) 4.5 % Binder.

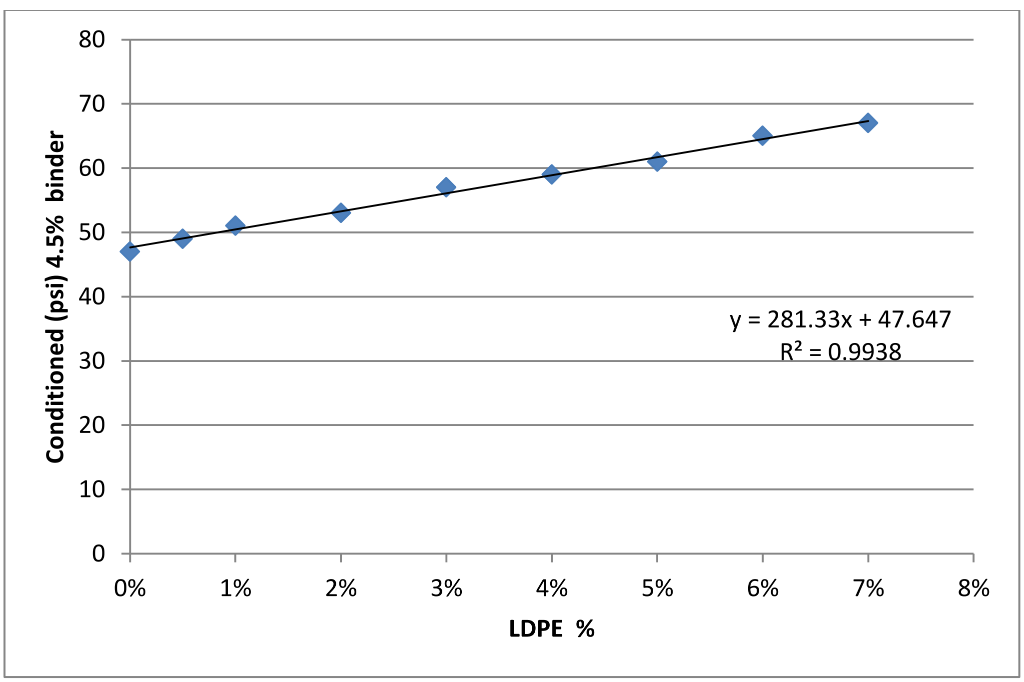

Figure B7. LDPE % vs. Stiffness (Conditioned) 4.5 % Binder.

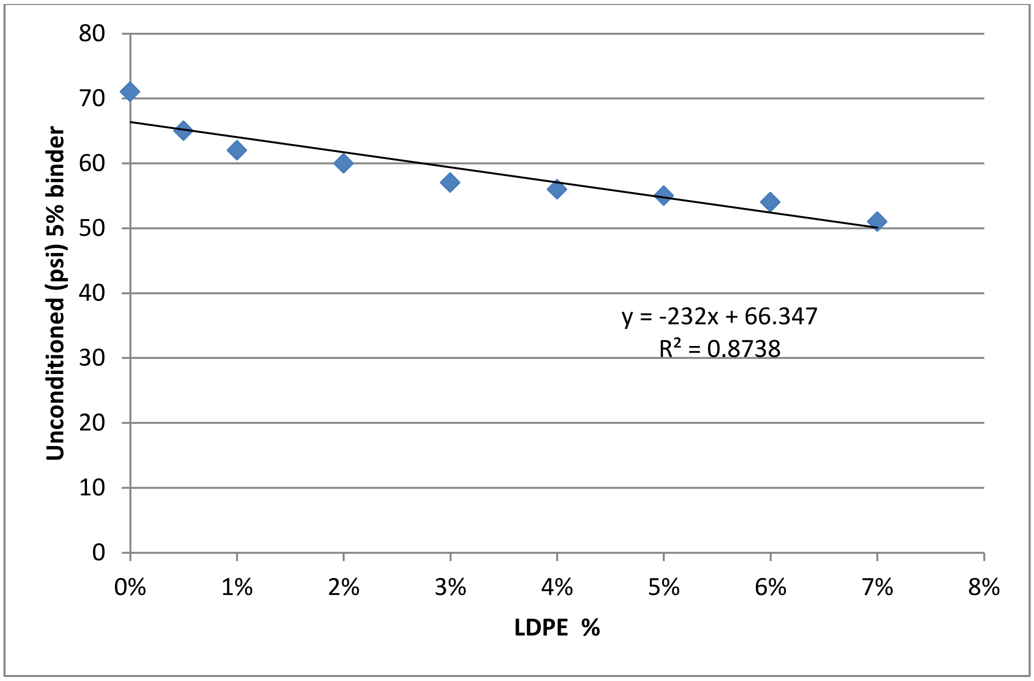

Figure B8. LDPE % vs. Stiffness (Unconditioned) 5 % Binder.

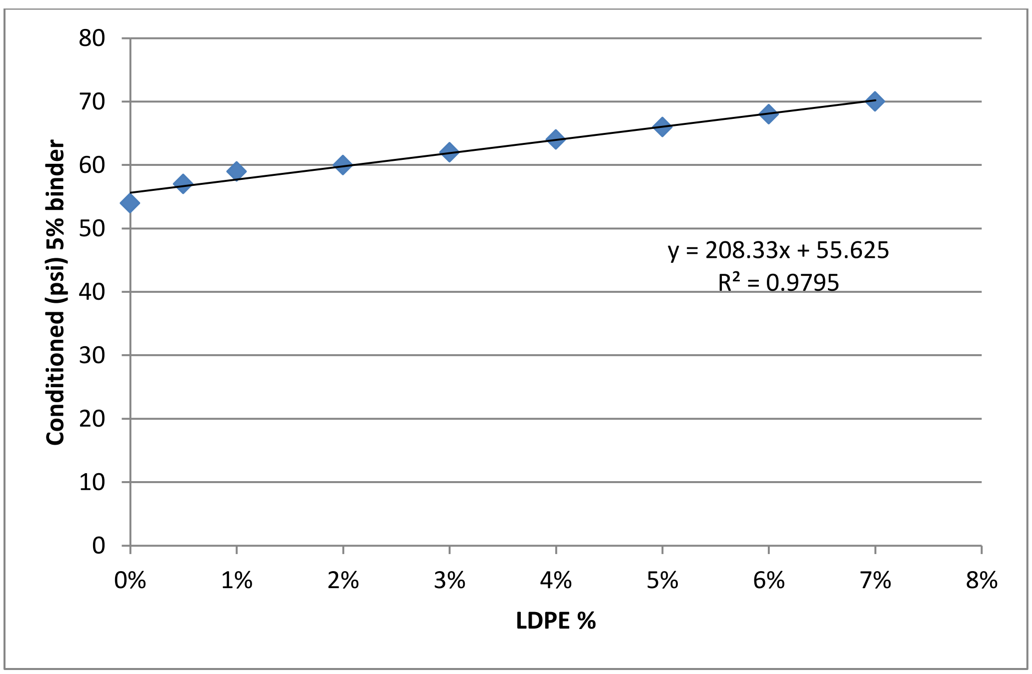

Figure B9. LDPE % vs. Stiffness (Conditioned) 5 % Binder.

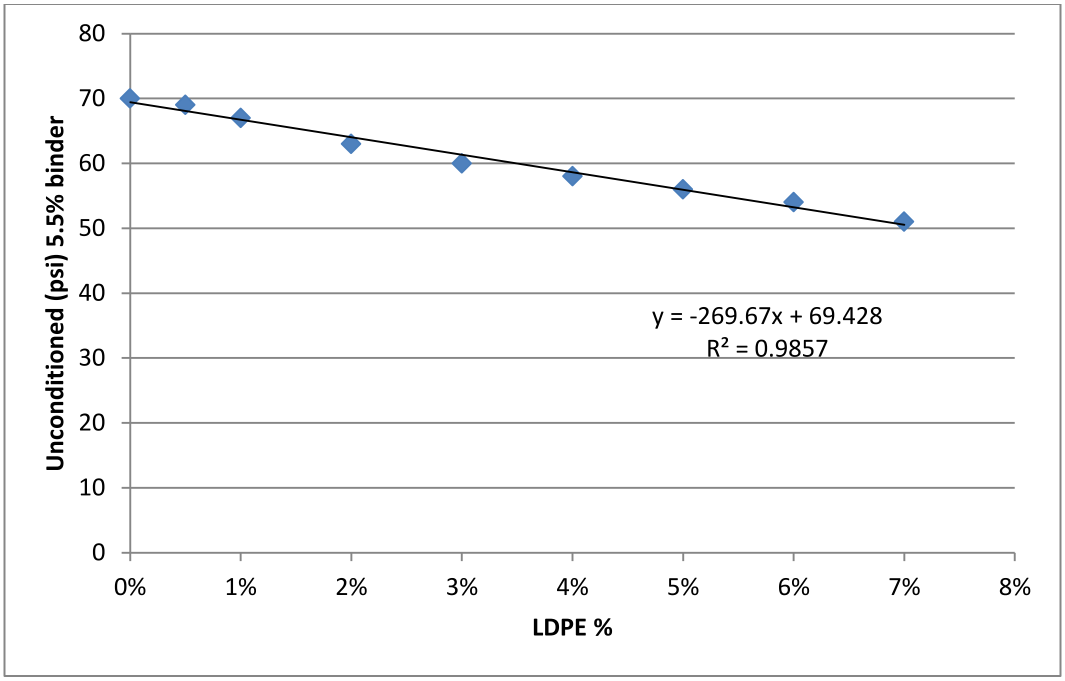

Figure B10. LDPE % vs. Stiffness (Unconditioned) 5.5 % Binder.

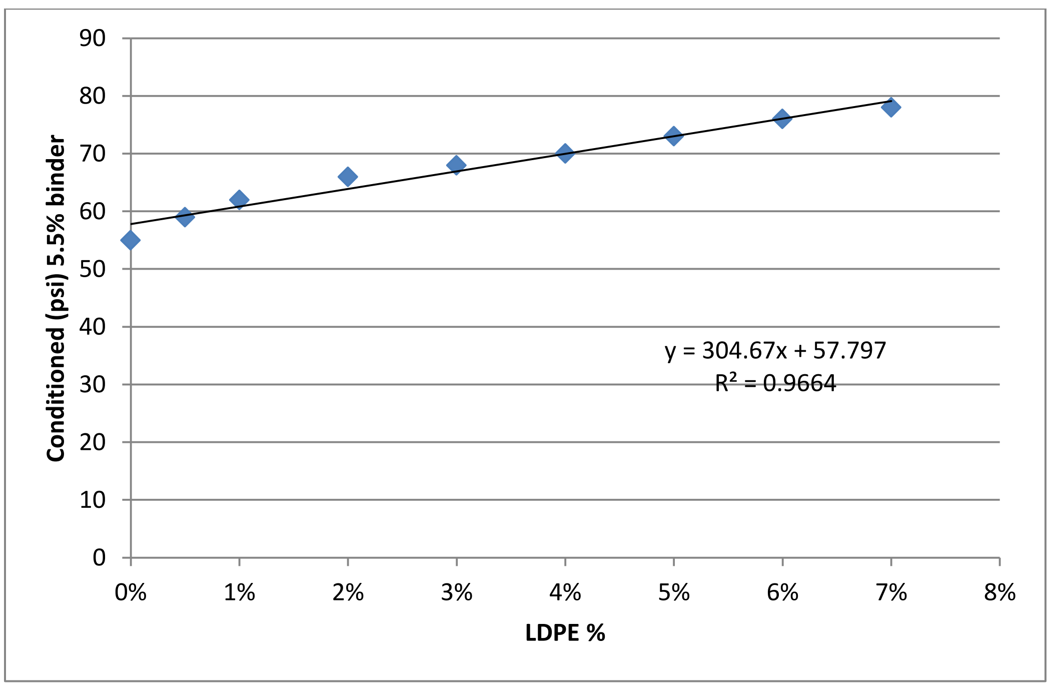

Figure B11. LDPE % vs. Stiffness (Conditioned) 5.5 % Binder.

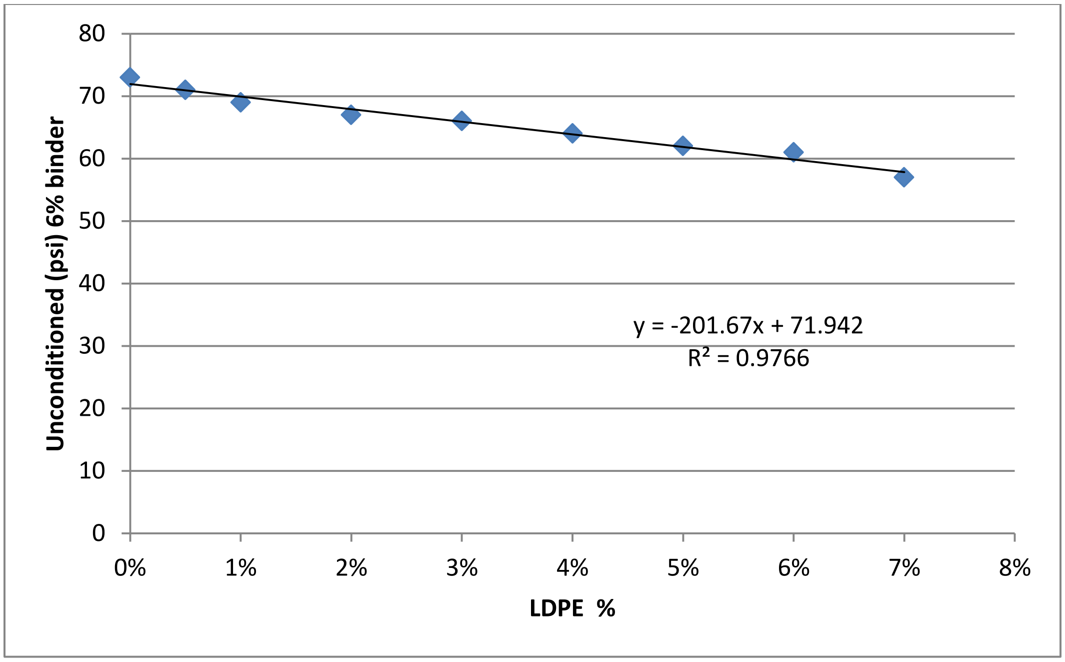

Figure B12. LDPE % vs. Stiffness (Unconditioned) 6 % Binder.

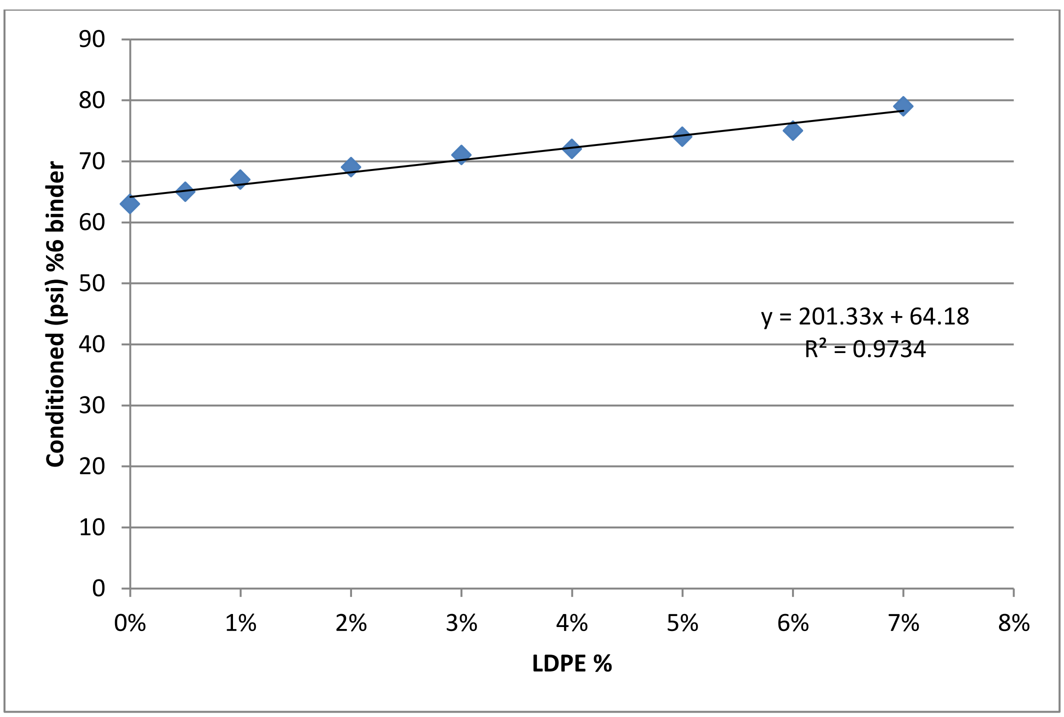

Figure B13. LDPE % vs. Stiffness (Conditioned) 6 % Binder.

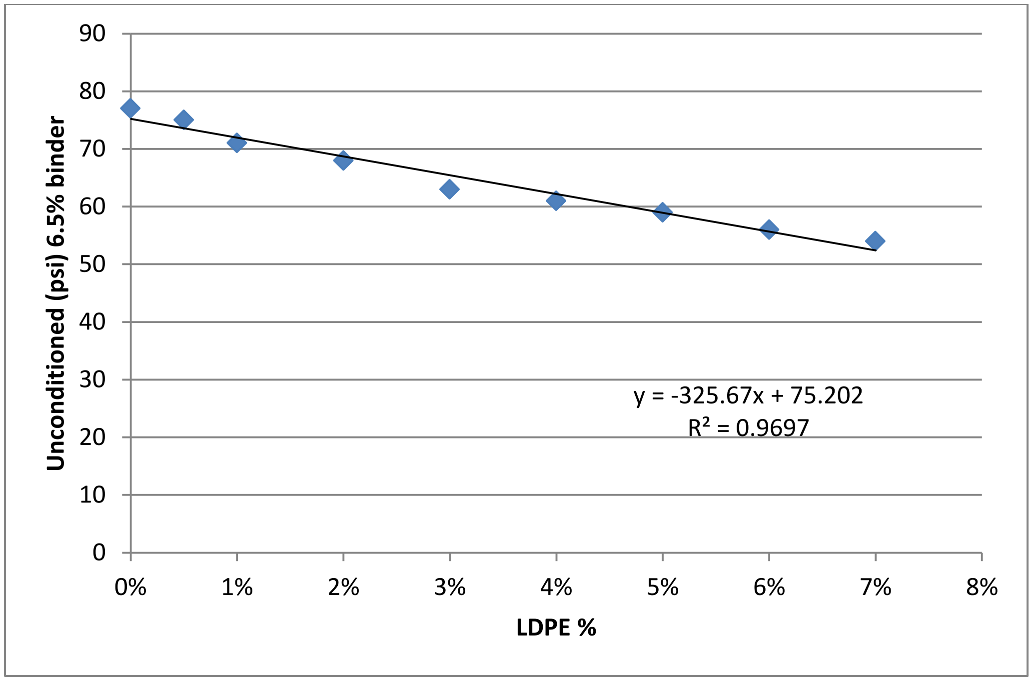

Figure B14. LDPE % vs. Stiffness (Unconditioned) 6.5 % Binder.

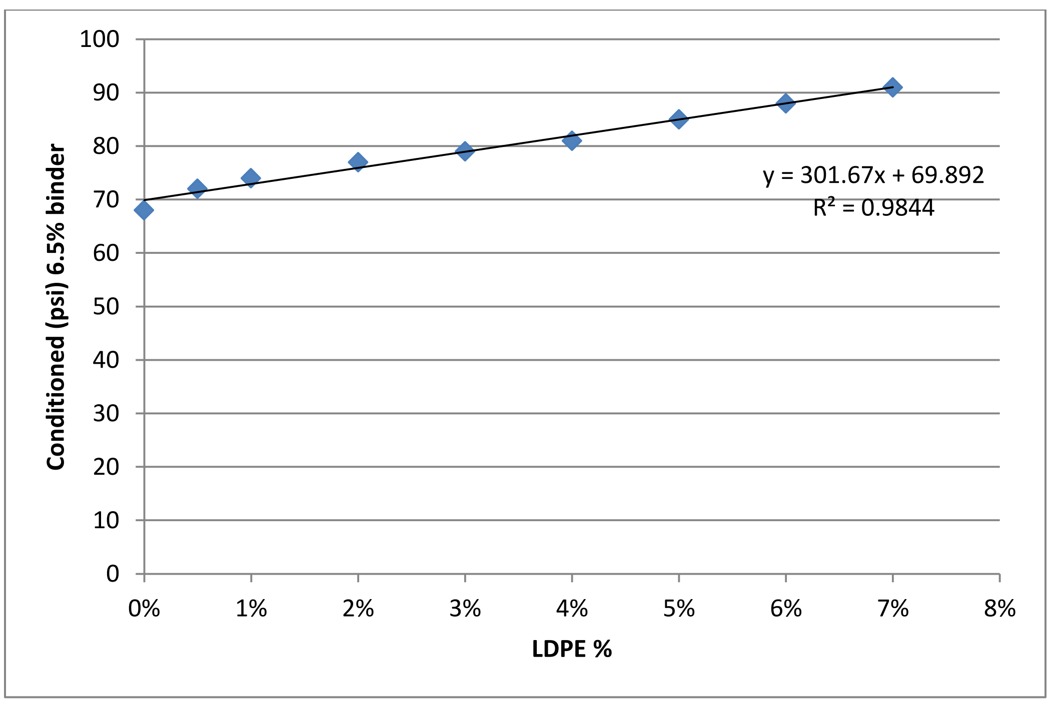

Figure B15. LDPE % vs. Stiffness (Conditioned) 6.5 % Binder.

Conclusion: Excellent relationships were found between LDPE (Low Density Polyethylene) % and Stiffness for various asphalt contents for both conditioned and unconditioned diametral tests.