EVALUATION OF MOISTURE DAMAGE WITHIN ASPHALT CONCRETE MIXES

A Thesis

by

BRIJ D. SHAH

Submitted to the Office of Graduate Studies of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

August 2003

Major Subject: Civil Engineering

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Approved as to style and content by:

Dallas N. Little (Chair of Committee)	Robert L. Lytton (Member)
Willa Chen (Member)	Paul Roschke (Head of Department)

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ABSTRACT

Evaluation of Moisture Damage
within Asphalt Concrete Mixes. (August 2003)
Brij D. Shah, B.E., Gujarat University
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Pavements are a major part of the infrastructure in the United States. Moisture damage of these pavements is a significant problem. To predict and prevent this kind of moisture damage a great deal of research has been performed on this issue in the past.

This study validates an analytical approach based on surface energy aimed at assessing moisture damage. Two types of bitumen and three aggregates are evaluated in the study. The two types of bitumen represent very different chemical extremes and the three aggregates (a limestone, siliceous gravel, and granite) represent a considerable range in mineralogy. Moisture damage was monitered as a change in dynamic modulus with load cycles. The analysis demonstrates the need to consider mixture compliance as well as bond energy in order to predict moisture damage.

Mixtures with the two types of bitumen and each aggregate with and without hydrated lime were evaluated. The hydrated lime substantially improved the resistance of the mixture to moisture damage.

To my parents.

ACKNOWLEDGMENTS

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TABLE OF CONTENTS

Page

ABSTRACT …… iii

DEDICATION …… v

ACKNOWLEDGMENTS …… vi

TABLE OF CONTENTS …… vii

LIST OF FIGURES …… ix

LIST OF TABLES …… xii

CHAPTER

I INTRODUCTION …… 1

II AGGREGATES AND ASPHALT …… 4

Types and Properties of Aggregates …… 4

Composition and Properties of Asphalt …… 7

III CHEMISTRY OF THE ASPHALT-AGGREGATE BOND …… 11

Chemistry Involved in Stripping Mechanism …… 15

IV BACKGROUND …… 23

Theoretical Explanation of Predicting Moisture Damage Using Surface Energy Concept …… 23

Effects of Hydrated Lime as a Mineral Filler in Asphalt Concrete Mixes …… 28

Behavior of Asphalt Concrete Mixes under Cyclic Loading Condition …… 33

TABLE OF CONTENTS (Cont'd)

CHAPTER Page

V A PERFORMANCE TEST …………………………………….. 38

Materials ……………………………………………………... 38

Mix Design ………………………………………………… 40

Testing Procedure …………………………………………. 49

VI RESULT ANALYSIS ……………………………………. 57

Dry and wet test comparisons of various aggregates with two binder types ……………………………………….. 62

Dry and wet test comparisons of various aggregate mixes with and without Hydrated lime ………………………….. 68

Bar-chart comparisons of various aggregate mixes ……………….. 76

VII CONCLUSIONS AND RECOMMENDATIONS ………………… 78

Conclusions ……………………………………………. 78

Recommendations …………………………………………. 79

REFERENCES…………………………………………………………… 81

APPENDIX I …………………………………………………………….. 84

APPENDIX II ……………………………………………………………. 90

VITA ……………………………………………………………………… 94

LIST OF FIGURES

FIGURE Page

1 Asphalt-aggregate chemistry (stage-1) ………………………………….. 13

2 Asphalt-aggregate chemistry (stage-2) ………………………………….. 13

3 Moisture within asphalt concrete mix due to environmental factors ……………………………………………………………….. 19

4 A model showing the damage to asphalt concrete sample due to water, under loading condition ……………………………….. 21

5 A plot of stress-strain showing elastic behavior of material at low temperatures ………………………………………………….. 34

6 A plot of stress-strain showing viscoelastic behavior of material ………………………………………………………………... 34

7 A permanent microstrain test data showing initiation of tertiary creep ……………………………………………………….. 36

8 A test data showing the loops of dissipated strain energy of a mix under repeated loading ……………………………………… 37

9 A gradation chart for Brazos gravel ………………………………….. 41

10 A gradation chart for Texas Limestone ………………………………. 44

11 A gradation chart for Georgia granite ……………………………….. 47

12 Asphalt concrete mixing equipment used in laboratory ………………… 50

13 Superpave gyratory compactor ……………………………………….. 52

14 Compacted asphalt concrete samples (4” x 6”) ………………………… 53

15 MTS machine used for running permanent deformation test …………… 55

16 Dry and wet test modulus values of Brazos gravel with AAD-1 and AAM-1 ………………………………………………………… 62

LIST OF FIGURES (Cont'd)

FIGURE Page

17 Ratio (K) of wet/dry test modulus vs. no. of loading cycles for Brazos gravel ………………………………………………………. 63

18 Dry and wet test modulus values of Texas Limestone with AAD-1 and AAM-1 ………………………………………………….. 64

19 Ratio (K) of wet/dry test modulus vs. no. of loading cycles for Texas Limestone ………………………………………………….. 65

20 Dry and wet test modulus values of Georgia Granite with AAD-1 and AAM-1 …………………………………………………… 66

21 Ratio (K) of wet/dry test modulus vs. no. of loading cycles for Georgia Granite …………………………………………………… 67

22 Dynamic modulus values of mixes with and without hydrated lime (Brazos Gravel with AAD) ……………………………………… 68

23 Ratios of wet/dry values of dynamic modulus for mixes with and without hydrated lime (Brazos Gravel with AAD) …………………. 69

24 Dynamic modulus values of mixes with and without hydrated lime (Brazos Gravel with AAM) ……………………………………… 70

25 Ratio of wet/dry values of dynamic modulus of mixes with and without hydrated lime (Brazos Gravel with AAM) …………………. 71

26 Dynamic modulus values of mixes with and without hydrated lime (Georgia Granite with AAD) …………………………………….. 72

27 Ratio of wet/dry values of dynamic modulus of mixes with and without hydrated lime (Georgia Granite with AAD) ………………… 73

28 Dynamic modulus values of mixes with and without hydrated lime (Georgia Granite with AAM) …………………………………….. 74

29 Ratio of wet/dry values of dynamic modulus of mixes with and without hydrated lime (Georgia Granite with AAM) ………………… 75

LIST OF FIGURES (Cont'd)

FIGURE Page

30 Comparison of E* ratios of the mixes with AAM and AAD (Bar chart) ………………………………………………………….. 76

31 Comparison of E* ratio of the mixes with and without hydrated lime (Bar chart) . ………………………………………………. 77

32 A test data showing strain values for Brazos gravel with AAD ………… 84

33 A test data showing strain values for Brazos gravel with AAM ………… 85

34 A test data showing strain values for Georgia granite with AAD ………. 86

35 A test data showing strain values for Georgia granite with AAM ………. 87

36 A test data showing strain values for Texas Limestone with AAD ……… 88

37 A test data showing strain values for Texas Limestone with AAM ……… 89

38 A test data showing strain values for Brazos gravel with AAD (With hydrated lime) …………………………………………………… 90

39 A test data showing strain values for Brazos gravel with AAM (With hydrated lime) …………………………………………………… 91

40 A test data showing strain values for Georgia granite with AAD (With hydrated lime) …………………………………………………… 92

41 A test data showing strain values for Georgia granite with AAM (With hydrated lime) …………………………………………………… 93

LIST OF TABLES

TABLE Page

1 Adhesive bond energy values for mixes with and without moisture …… 27

2 Cohesive bond energy values for asphalts …………………………... 27

3 Aggregate gradation for Brazos gravel …………………………………. 40

4 Specific gravity and water absorption for Brazos gravel ……………… 42

5 Aggregate gradation for Texas Limestone ……………………………… 43

6 Specific gravity and water absorption for Texas Limestone …………… 45

7 Aggregate gradation for Georgia Granite ………………………………... 46

8 Specific gravity for Georgia Granite …………………………………… 48

9 Basic criteria for asphalt concrete mix design …………………………… 48

10 Mix with Hydrated lime as a mineral filler ……………………………… 56

11 Mix without Hydrated lime ……………………………………………… 56

CHAPTER I

INTRODUCTION

Since late 1970s and early 1980s it has been recognized that moisture has a detrimental influence on asphalt concrete pavements. Premature rutting, raveling, and wear have been observed in many pavements. Distress and deterioration in large number of pavements as a result of moisture damage is an indication of the significance and the severity of the problem. Moisture damage can be generally classified in two mechanisms: (a) loss of adhesion and (b) loss of cohesion (1). The loss of adhesion is due to water getting between the asphalt and the aggregate and stripping away the asphalt film. The loss of cohesion is due to a softening of asphalt concrete mastic. The two mechanisms being interrelated a moisture damaged pavement may be a combined result of both the mechanisms. Further the moisture damage is a function of several other factors like the changes in asphalt binders, decreases in asphalt binder content to satisfy rutting associated with increases in traffic, changes in aggregate quality, increased widespread use of selected design features, and poor quality control (2).

This thesis follows the style and format of the *Transportation Research Record.*

A number of test procedures have been developed and used to evaluate the moisture damage potential of asphalt-aggregate mixtures in the past. These tests are performed on loose or compacted HMA (Hot Mix Asphalt) to determine water sensitivity of the paving material and they do not couple the effects of moisture on material properties with pavement performance prediction; hence they cannot be used directly to rationally predict performance. Test methods and pavement performance prediction tools need to be developed that couple the effects of moisture on the properties of HMA mixtures with performance prediction to estimate the behavior of the mixture in resisting rutting, fatigue, and thermal cracking when it is subjected to moisture under different traffic levels in various climates. Many public agencies use the test methods listed in AASHTO and ASTM standards. AASHTO T283,"Resistance of compacted bituminous mixture to Moisture Induced Damage" is the standard used for test methods performed to predict moisture damage effect in asphalt concrete mixes. Recently Strategic Highway Research Program (SHRP) has recommended the use of AASHTO T283 to evaluate the water sensitivity of HMA within the Superpave volumetric mixture design system (2).

Methods of treatment to reduce moisture damage, particularly stripping, include use of good aggregate, pretreatment of aggregates, and use of additives. One such additive is hydrated lime. Based on the laboratory and field testing in the last several years, it has been proved that hydrated lime improves the composition of the mastic and produces multifunctional benefits in the mixture. Work done in the United States and several other countries has proved that hydrated lime can substantially improve the resistance of the HMA to permanent deformation damage at high temperatures. Hydrated lime substantially improves low temperature fracture toughness without reducing the ability of the mastic to dissipate energy through relaxation. Hydrated lime acts as filler and reacts with bitumen resulting in some of the beneficial mechanisms in terms of strength. It has been also proved that there are also benefits of reduced susceptibility to age hardening and improved moisture resistance. There is a need for a simple and repeatable test that can evaluate the multifunctional aspects of pavement performance in presence of moisture. Some of such tests and methods have been developed in recent years to evaluate the permanent deformation in Asphalt concrete mix in wet conditions in presence and absence of such fillers. Developing more test methods to predict the performance of asphalt mix would always add to the available knowledge regarding effects of additives in asphalt concrete. One such method has been tried to develop here which shows the effect of Hydrated lime and influence of two different binder types on the dry and wet asphalt concrete mixes in terms of permanent deformation.

CHAPTER II

AGGREGATES AND ASPHALT

TYPES AND PROPERTIES OF AGGREGATES

The main aggregates used in road pavements on their own or in combination with a cementitious material are either natural rock materials, gravels and sands, or slag aggregates.

Natural rock aggregates are classified in to three main groups, based on their origin; igneous, sedimentary, and metamorphic.

Igneous rocks are formed at or below the earth's surface by the cooling of molten material, called magma, which erupted from, or was trapped, beneath the earth's crust. The best igneous roadstones normally contain medium grain sizes. The important igneous rock aggregates belong the basalt, gabbro, granite and porphyry groups (3).

Sedimentary rocks are formed when the products of disintegration or decomposition of any older rock are transported by wind or water, redeposited as sediment, and then consolidated or cemented into a new rock type. Some rocks are also formed as a result of the chemical deposition of organic remains in water, e.g. calcareous rocks (3).

Metamorphic rocks form as a result of great heat or great heat and pressure, are transformed into new rocks by the recrystallization of their constituents. The main metamorphic aggregate groups of importance in road-making are hornfels, quartzite and, to a lesser extent, schists (3).

Gravel and Sands are unconsolidated, natural, coarse-grained rock particles that have been transported by wind, water or glacial ice and deposited when movement slowed or stopped. The individual particles are hard and usually rounded or irregular rather than angular, depending upon the amount of abrasion encountered during the prior movement. Some gravel used in roadworks has to be crushed during processing to make them more angular (3).

Slag Aggregates are the ones produced from smelting of iron ore in a blast furnace. Air-cooled blast furnace slag has very good anti-skid properties and, hence, it is highly regarded as a surface dressing aggregate. The high angularity and irregular shapes of the slag particles mean that pavements incorporating this aggregate have high internal friction. Bituminous surfacing using slag is normally very stable (3).

The main properties of the aggregates used in road pavements are: cleanliness; size and gradation; shape and surface texture; hardness and toughness; durability; and relative density.

A *Clean aggregate* is one that has its individual particles free from adherent silt-size and clay-size material. Aggregate cleanliness is generally ensured by the maximum allowable adherent deleterious materials present in the coarse and fine aggregate fractions, as they reduce the bonding capabilities of cements and bituminous binders in mixes (3).

The *Size and gradation* of an aggregate affect the strength, density and cost of a pavement. They have a major influence upon the strength and stiffness characteristics of a bituminous mix, as well as its permeability, workability, and skid resistance (3).

Aggregate *Shape and surface texture* are used to describe aggregate and to provide information regarding their internal friction properties. There are various aggregate types based on the shape; rounded, irregular, flaky, angular, elongated, flaky and elongated. Also based on the surface texture aggregates can be classified into glassy, smooth, granular, rough, crystalline, and honeycombed and porous. Aggregates with the angular shapes and rough surface texture are considered to be excellent for pavement mixes because of high internal friction. While rounded smooth aggregates have relatively low internal friction as particle interlock and surface friction are poor (3).

Hard aggregates are those which can resist the abrasive effects of traffic over a long time and *Tough* aggregates are those which are better able to resist fracture under applied loads during construction and under traffic (3).

Durable aggregates are those that are able to resist the disintegrating actions of repeated cycles of wetting and drying, freezing and thawing, or changes in temperature. *Relative density* of an aggregate is also one of the important properties affecting the aggregate mix. It is the ratio of mass of aggregate in air to the mass of equal volume of water in air (3).

COMPOSITION AND PROPERTIES OF ASPHALT

Asphalt is basically obtained from refining the crude petroleum formed by nature from plant life. The nature and molecular structure of asphalt varies from source to source due to varying amount of hydrocarbons in it. Elements present within asphalt are carbon, hydrogen, nitrogen, sulfur, oxygen, vanadium and nickel (4). Carbon and hydrogen are the principal elements present in asphalt molecules. There are three types of arrangements of carbon atoms within asphalt structure (4).

- Aliphatic or Paraffinic type which consists of straight or branched chains of carbon.
- Naphthenic type which consists of simple and complex saturated rings of carbon atoms.
- Aromatic type which consists of one or more stable six-carbon condensed, unsaturated ring structures.

Asphalt structure consists of two types of groups: Polar or Functional group and Non polar group (4). Polar groups are formed by the various combinations of heteroatoms within asphalt and have electropositive and electronegative characteristics similar to a magnet which has north and south poles. Even a small amount of variation in polar group availability in asphalt changes the characteristics and behavior of asphalt with aggregate surface. Also, there are non-polar components of asphalt, which act as solvents or dispersions for the polar or functional groups and play a major role in determining the effect that the polar groups will have on the physical and aging properties of the asphalt cement.

According to the simple concept the asphalt structure is composed of three parts: Asphaltenes, resins, and oils (4). Resins and oils fall under Maltenes. The component of asphalt which is insoluble when asphalt is dissolved in nonpolar solvents is called asphaltene and the one which dissolves is called maltene.

Asphaltenes are generally dark brown, friable solids. They are the most complex components with the highest polarity and hence they have a very high tendency to interact and associate. They also play a major role as the viscosity-building component of asphalt cements.

Resins are generally dark in color and semi-solid or solid in character. They are generally in fluid form when heated and become brittle when cooled. On oxidation resins yield asphaltene type of molecules.

Oils are generally colorless and white liquids. They are soluble in most solvents. On oxidation they yield asphaltene and resin molecules.

Asphalt consists of certain rheological properties based on its characteristics and performance. The properties of asphalt based on its nature and characteristics are

- Age hardening
- Penetration
- Ductility
- Viscosity
- Temperature Susceptibility
- Shear Susceptibility
- Stiffness

Age hardening of asphalt occurs with time and change in temperature. Age hardening can also occur at a constant temperature with increase in time. Basically this procedure is divided in to 6 segments: Oxidation, Volatization, Polymerization, Thixotropy, Syneresis and Separation (4).

Penetration is the other measure of estimating the nature of asphalt. It is measured using a penetration test where a needle is allowed to fall freely in asphalt cement and according to penetration of that standard needle the nature of asphalt is decided. Penetration varies with variation in temperature and asphalt type.

Ductility of asphalt is measured by the distance to which it will elongate before breaking when two ends of specimen are pulled apart at a specified speed and temperature.

Viscosity is essentially the ratio of shear stress to shear strain rate at any given temperature and shear rate (4). Viscosity of asphalt is constant at certain high temperatures as 275^{o}F. Viscosity of asphalt is measured using the tests like Rotational viscometer, Brookfield Viscometer and Dynamic shear rheometer in laboratory.

Temperature susceptibility is the rate at which the consistency of asphalt changes with a change in temperature (4). Asphalts with very high temperature susceptibility are not desirable due to the consequent problems during compaction of mix. Temperature susceptibility can be determined with three different approaches: Penetration Index, Pen-vis number, and Viscosity-temperature susceptibility.

Shear susceptibility is basically the rate of change of viscosity with rate of shear (4). It is an intrinsic property of asphalt. Shear susceptibility by its own does not describe the behavior or performance of asphalt. Its value with respect to viscosity or aging gives the exact idea of asphalt performance.

Stiffness is one of the important properties of asphalt. It is basically the relationship between stress and strain as a function of time of loading and temperature. Behavior of asphalt concrete is based on the stiffness of asphalt indirectly since it affects the tensile and compressive behavior of asphalt concrete. Also the bonding of asphalt with aggregate depends on the stiffness of asphalt (4).

CHAPTER III

CHEMISTRY OF THE ASPHALT-AGGREGATE BOND

Asphalt and Aggregates are the two main components of an asphalt concrete mix. Their interaction with each other in a mix plays a major role in the performance of a pavement. Both the components have their separate chemical and physical properties. These properties of asphalt and aggregates also interact with each other when both are in close contact. The weight ratio of asphalt to aggregate in a mix is typically 5 to 6 wt % asphalt and 94 to 95 wt % of aggregate. Aggregates vary widely in terms of composition, surface chemistry, and morphology, including surface area, pore-size distribution, and friability. Dust composed of clays or other minerals frequently coats the surface and is not completely removed during aggregate preparation. This results in different parts of the surface from the same aggregate having different surface chemistries. Aggregates possess various active and inactive sites on its surface which play a major role in the interaction of asphalt molecules with it (5). Aggregates vary in terms of surface texture. There are certain aggregates with larger surface area as well as a favorable pore size for adequate asphalt penetration. At times it is found that air is entrapped in these fine pores on the aggregate surface due to which it becomes difficult for asphalt to penetrate on entire aggregate surface. Based on this there are various active and inactive sites on aggregate surface.

On other hand asphalt is composed of a mixture of hydrocarbons that contain some polar functionalities, as well as organometallic constituents that contain metals such as nickel, vanadium, and iron (5). A study by Scott (1978) has shown that when asphalts were contacted with aggregates, oxygen-containing groups from asphaltenes were preferentially adsorbed on the aggregate surface. Also a similar kind of behavior was observed by Fritschy and Papirer (1978) from the polar asphaltenes. Later on with further research it is believed for sure that chemistry at the interface between the asphalt and aggregate leads to bonding interactions that influence the ultimate adhesive strength. The aggregate provides a surface that is heterogeneous and has a variety of sites of different composition and levels of activity (figure-1). These active sites are frequently charged or contain partial charges that attract and orient the polar constituents of asphalt. The chemical components with the strongest affinity for a particular site compete most effectively and win the position on the site. Autoradiographic experiments by Ross (1991) have confirmed the presence of active sites on the aggregate surface. The polar functionalities present at the point of contact between the asphalt film and the aggregate surface stick to the surface due to electrostatic force, hydrogen bonding, or Van der Waals interactions (5). If the polar surface character of the aggregate is completely covered with a nonpolar surface coating, then the adsorption characteristics of the aggregate change radically (Liu, 1992). The polar functionalities then find it more favorable to remain in the asphalt phase since no driving force or electrostatic potential exists for adsorption or adhesion at the aggregate surface. The more polar functionalities,

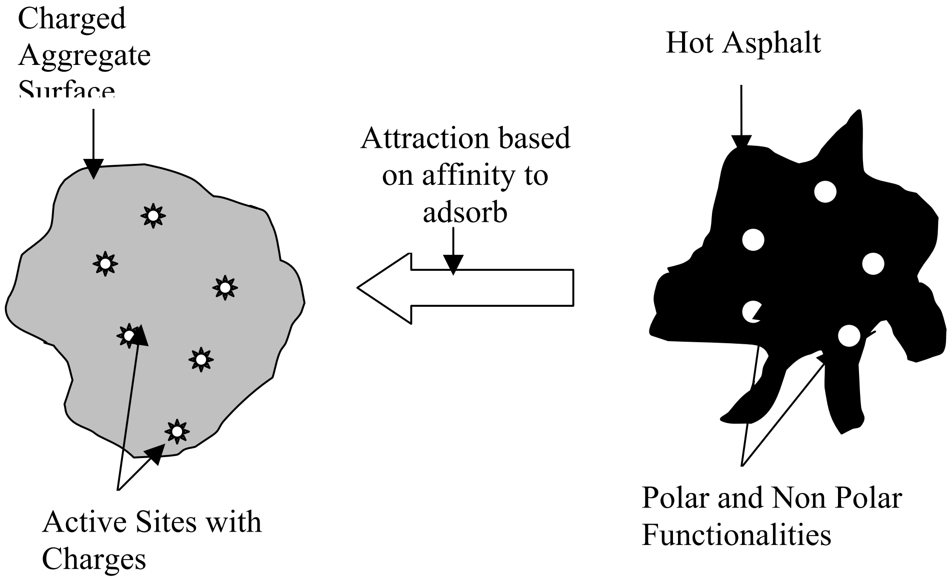


FIGURE-1 Asphalt-aggregate chemistry (Stage-1)

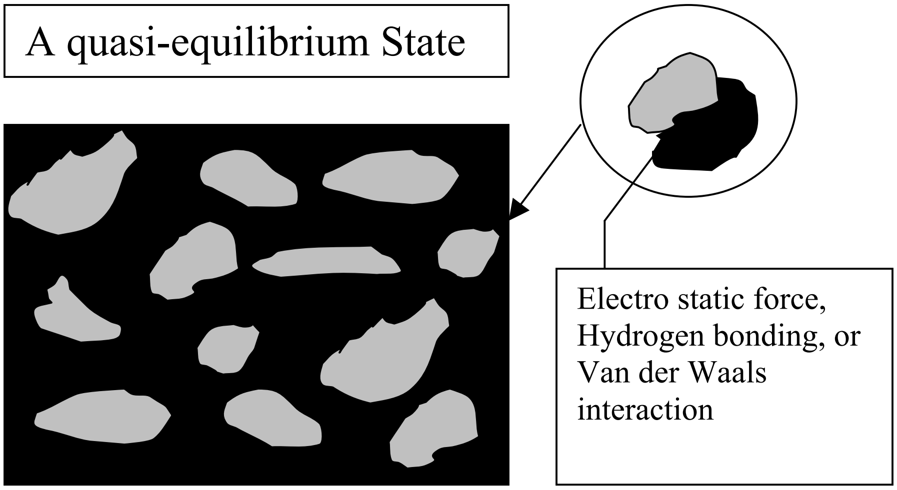


FIGURE-2 Asphalt-aggregate chemistry (Stage-2)

such as sulfoxides, carboxylic acids, and nitrogen bases, are adsorbed more strongly (5). While the less polar asphaltic groups, including the aromatic hydrocarbons, have much less affinity for the surface and tend not to adhere strongly to the aggregate (Curtis, Ensley and Epps.J, 1991).

Chemical reactions between asphalt and aggregates occur at the time of mixing. A longer range chemical effects last because of these chemical reactions and affect long-term durability between the asphalt and aggregate. When hot asphalt coats the aggregate particle, it tends to enter any available crevice or pore. A charged aggregate surface attracts an oppositely charged or partially charged functional group contained in the asphalt. The part of the attracted molecule that is available for interaction with other asphalt molecules would then be the charge of the aggregate and hence would have electrostatic interaction with other oppositely charged or partially charged asphalt molecules. A quasi-equilibrium state at the asphalt-aggregate bond may remain for some time (figure-2). The disruption caused by attriting forces changes the equilibrium state either into a new quasi-equilibrium or into a state of steady, though perhaps slow, decay of the asphalt-aggregate bond (5). Failure of this bond between asphalt and aggregate can fail at the interface, either in asphalt as a cohesive failure, or within the aggregate as a structural failure (5).

One of the reasons for the deterioration of asphalt concrete mix with time is aging (5). Hardening of the asphalt in service may be expected to influence the asphalt aggregate bond because of the changes in chemical composition that occur during aging. The changes caused by oxidative aging can change the nature of the chemistry of the interface. The compounds typically produced during aging are sulfoxides, carboxylic acids, and ketones (5). Both of them have a high affinity for the aggregate surface. Aging studies show that carboxylic acids, ketones, and sulfoxides increase with oxidative aging, both at the interface and in asphalt at distances of 25 um to 100 um from the aggregate surface (McKay, 1990). The adhesion of the asphalt to the surface is dependant on the types of functional groups at the interface and on their ability to bond strongly to the surface. The resistivity of that bond to environmental factors, particularly the intrusion of water, is essential for maintaining a long life of mixes.

CHEMISTRY INVOLVED IN STRIPPING MECHANISM

Stripping is one of the major distresses within asphalt concrete pavements caused due to penetration of water within the interface of asphalt-aggregate matrix. The force of water intrusion within asphalt and aggregate can destroy the pavements. There are various mechanisms of stripping. Water may be present in aggregate pores used for making a mix, or it may invade by seeping through cracks in the asphalt. Water can destroy asphalt-aggregate bond by diffusing through the asphalt film and then reaching the surface and competing for the active sites present on the aggregate surface. Based on the literature (Taylor and khosla, 1983; kiggundu and Roberts, 1988; and Terrel and Alswalilmi, 1994) there are about seven different mechanisms of stripping: detachment, displacement, spontaneous emulsification, pore pressure, hydraulic scour, pH instability, and the effects of the environment on asphalt-aggregate material systems (6).

Detachment is the separation of an asphalt film from an aggregate surface by a thin film of water without an obvious break in the film (Majidzadra and Brovold, 1968). It is necessary to develop a good bond between the asphalt and aggregate. Such a bond is initially dependant on the ability of the asphalt to wet the aggregate. Wettability of aggregate increases as surface tension or free energy of adhesion decreases (Majidzadra and Brovold, 1968). Surface energy measurements at Texas A&M University have established that when the free energy at the asphalt-aggregate interface is calculated in the presence of water, energy is released meaning that the aggregate surface has a strong preference for water over asphalt. More negative the value of bond energy, the stronger is the preference for detachment of asphalt from aggregate in the presence of water (6).

Displacement involves displacement of asphalt at the aggregate surface through a break in the asphalt film (Tarrer and Wagh, 1991; and Fromm, 1974). The source of the break or disruption may be incomplete coating of the aggregate surface, film rupture at sharp aggregate corners or edges, pin holes originating in the asphalt film due to aggregate coatings, etc. The process of displacement can proceed through changes in the pH of the water at the aggregate surface that enters through the point of disruption. The changes alter the type of polar groups adsorbed leading to the build-up of opposing, negatively-charged, electrical double layers on the aggregate and asphalt surfaces. The drive to reach equilibrium attracts more water and leads to physical separation of the asphalt from the aggregate (Scott, 1978; and Tarrer and Wagh, 1991) (6).

Spontaneous emulsification is an inverted emulsion of water droplets in asphalt cement. Fromm (1974) demonstrated how an emulsion forms and that once the emulsion formation penetrates the substrata, the adhesive bond is broken. He observed that spontaneous emulsification occurs when asphalt films are immersed in water and that the rate of emulsification depends on the nature of the asphalt and the presence of additives (6).

The water entrapped within asphalt concrete mix can lead to pressure and in turn can cause distress. Stresses imparted to the entrapped water from repeated traffic load applications will worsen the damage as the continued build up in pore pressure disrupts the asphalt film from the aggregate surface or can cause the growth of microcracks in the asphalt mastic. Bhairampally et al. (2000) used a tertiary damage model developed by Tseng and Lytton (1987) to demonstrate that well-designed asphalt mixtures tend to “strain-harden” upon repeated loading. The rate of this accelerated or tertiary damage is exacerbated in the presence of water as the pore pressure developed in the microcrack voids increases the rate of crack growth and damage through the developments of higher pressures at the crack tip and through a weakening of the mastic and of the adhesive bond between the mastic and the aggregate (6).

Hydraulic scour is likely to happen on the pavement surface. Here stripping results from the action of tires on a saturated surface. Water is sucked under the tire into the pavement by the tire action. Osmosis and pull back have been suggested as possible mechanisms of scour (Fromm, 1974). Osmosis occurs in the presence of salts or salt solutions in aggregate pores and creates an osmotic pressure gradient that actually sucks water through the asphalt film. Still there exists a doubt amongst researchers on this process (6).

Scott (1978) and Yoon (1987) demonstrated that asphalt-aggregate adhesion is strongly influenced by the pH of the contact water. Kennedy (1984) investigated the effect of various sources of water on the level of damage that occurred in a boiling test. Fehsendfeld and Kriech (undated) observed that pH of contact water affects the value of the contact angle and the wetting characteristics at the aggregate-asphalt interface region. Kiggundu and Roberts (1988) point out that these results indicate that stabilization of the pH sensitivity at the asphalt-aggregate interface can minimize the potential for bond breakage, provide strong, durable bonds, and reduce stripping. Tarrer (1996) concluded that the bond between asphalt and aggregate depends on surface chemical activity, water at the aggregate surface (in the field) is at a high pH, some liquids used as antistrips require a long curing period (in excess of about 3 hours) to achieve resistance to loss of bond at higher pH levels, and it is possible to achieve a strong chemical bond between aggregate and asphalt cement that is resistant to pH shifts and/or a high pH environment. This strong chemical bond can be achieved by the formation of insoluble organic salts (such as calcium based salts) which form rapidly and are not affected by high pH levels or pH shifts (6).

Terrel and Shute (1989) reported that factors such as temperature, air, and water have a profound effect on the durability of asphalt concrete mixtures. In mild climates where good quality aggregates and good quality asphalt cements are available, the major contribution to deterioration is traffic loading and the resulting distress manifestations.

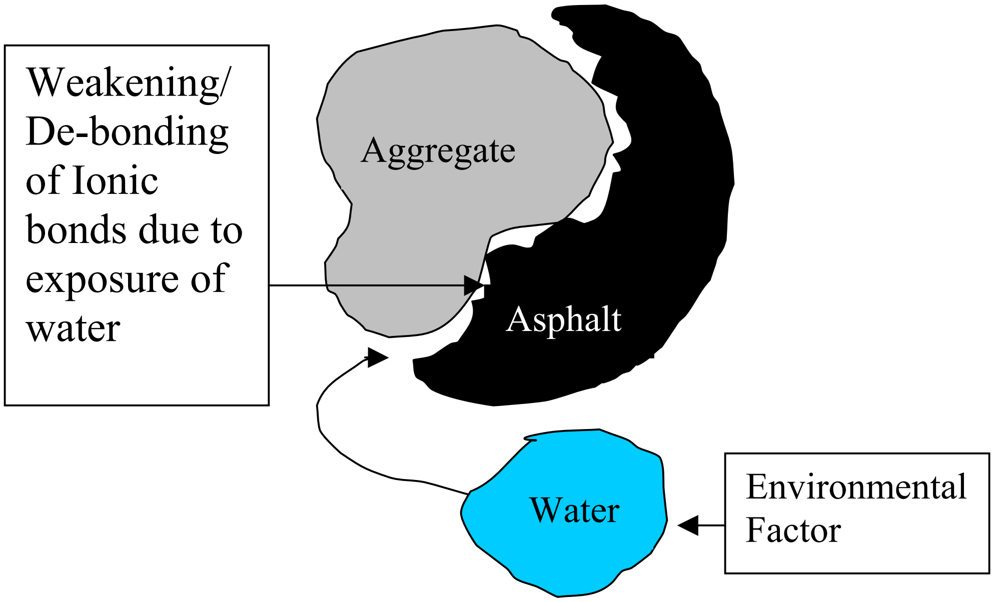


FIGURE-3 Moisture within asphalt concrete mix due to environmental factors

Premature failure may result when poor materials and traffic are coupled with severe weather. Terrel and Al-swailmi (1994) identified a number of environmental factors of concern: water from precipitation of groundwater sources (figure-3), temperature fluctuations, and aging of the asphalt. They identified traffic and construction techniques as important factors, which are external to the environment (6).

Adhesive failure in aggregates and asphalt occurs at an interface, while cohesive failure occurs directly within asphalt or aggregate surface. Under harsh water treatment the outer surface of the aggregate breaks away from the main body, carrying asphalt with it (Podoll et al, 1991). From the tests performed in past on various aggregates like granite, Limestone and gravel, it has been observed that the ranking for adsorption affinity of the functional groups present on asphalt is:

Sulfoxide > carboxylic acid > nitrogen base > phenol > ketone > pyrrole > 4-ring aromatic > 2-ring aromatic (5)

It is also evident that the components that have the largest affinity for the aggregates also have the most sensitivity to water (5). Adsorption of asphalt from solution and subsequent desorption of the asphalt by water are dependent on both the asphalt composition and on the aggregate chemistry and morphology (Brannan et al, 1991). Aggregates can be mainly divided as siliceous and calcareous types. Both siliceous and calcareous aggregates can be strippers as well as nonstrippers. There are many other characteristics of aggregates other than bulk composition which affect their behavior. It is evident that the different aggregates have different surface textures,

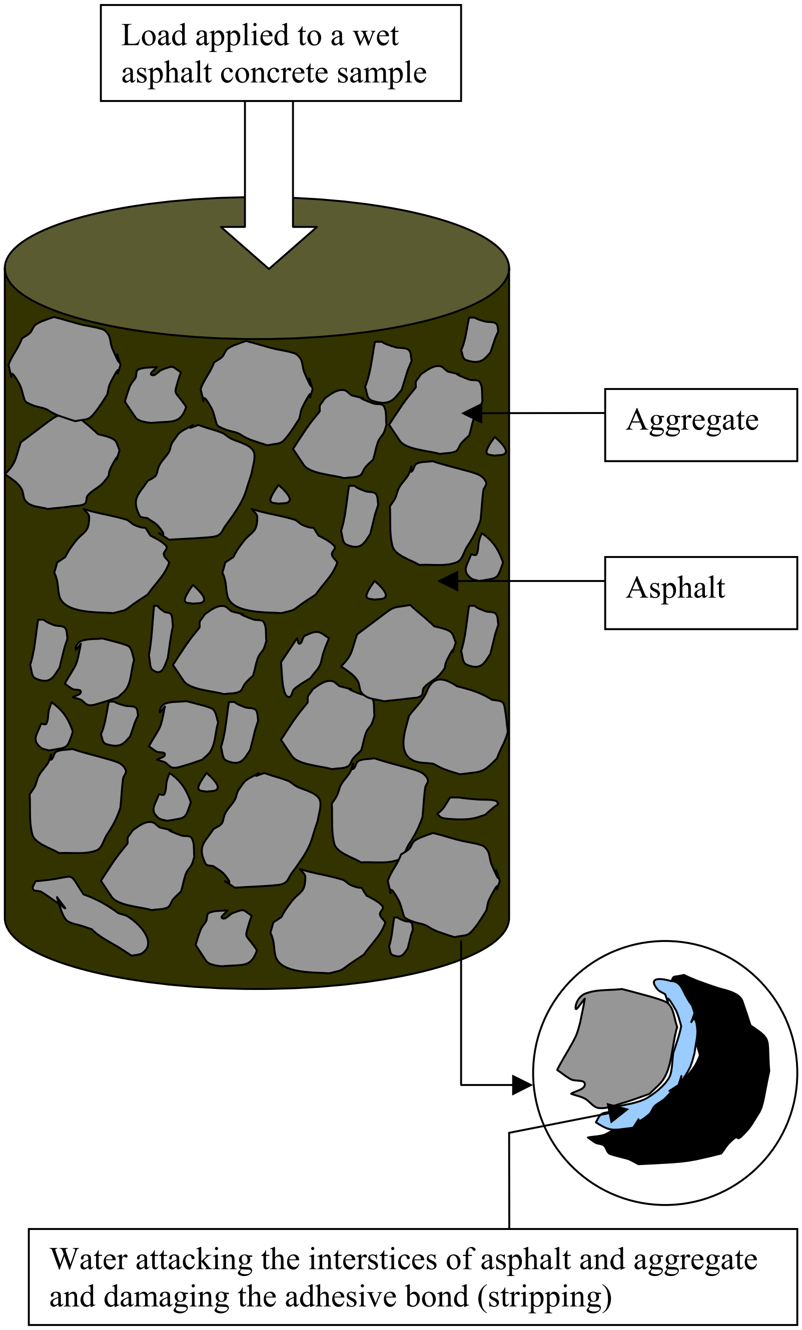


FIGURE-4 A model showing the damage to asphalt concrete sample due to water, under loading condition

ranking from smooth to rough. The siliceous aggregates are generally slick and smooth at the surface, while calcareous aggregates are little bit rough and hence it might promote bonding between aggregate and asphalt.

Once the asphalt has wetted the aggregate surface, some of its organic chemical functionalities enter into bond formation with the aggregate constituents. The functional groups frequently combine with alkali metals present on the aggregate surface to form water-soluble salts (5). These asphalt-aggregate bonds being ionic in nature weaken or solubilize over time with exposure to moisture (figure-4).

There are various methods of preventing stripping within asphalt concrete mixing. Typically this is done by adding antistripping agent within asphalt-aggregate mixture. One such agent is hydrated lime. It has been observed that some lime treated aggregates tend to form stronger, more robust, and durable bonds with asphalt (5). This is believed to be caused by the insensitivity of these bonds to the action of water. The bonds formed in this case are strong, insoluble bonds. Also the effect of fines within asphalt-aggregate mixture plays a critical role. The presence and amount of fines determine the extent of stiffening of the asphalt near the aggregate surface by having a bridging effect between the bulk asphalt and aggregate surface (Wagh and Tarrer, 1990). Also the number of active sites on aggregate surfaces can be increased by the addition of agents enriched in cations of iron, magnesium, and calcium (5). Specific asphalt-aggregate pairs can be promoted for either adsorption or increased resistivity to water.

CHAPTER IV

BACKGROUND

Permanent deformation and moisture damage are the serious problems in pavement industry existing since many years. Many theories have been studied and developed explaining the behavior of asphalt concrete mixtures showing permanent deformation and moisture damage. But as mentioned earlier the basic theory explaining the moisture damage within asphalt concrete mixtures is based on adhesion and cohesion of materials used to prepare such mixes. This thesis explains the theoretical and experimental concept of predicting moisture damage in asphalt concrete mixes using surface energy concept and actual laboratory testing analysis respectively. It also shows the effects and importance of hydrated lime, mineral filler used to prevent permanent deformation and moisture damage within the mixes.

THEORETICAL EXPLANATION OF PREDICTING MOISTURE DAMAGE USING SURFACE ENERGY CONCEPT

Asphalt pavement performance is related to cohesive and adhesive bonding within the asphalt-aggregate system and the cohesive and adhesive bonding are related to the surface free energy characteristics of the system (6). Stripping as an adhesion failure is most likely to occur either at the pavement surface or internally within the mixture. Stripping starts occurring at the weak points such as pavements joints and areas of high air void content due to improper compaction. When such patches of pavements pass through various loading conditions due to heavy traffic, in presence of water and temperature, the mastic bond with the surface of the coarse aggregates weakens. The asphalt film separates from the aggregate by emulsification. Theory of adhesion uses surface free energy to reflect the physical and chemical interaction of the surface. The surface energies present on the surface of aggregates and binder; the two main components of asphalt concrete mix, play an important role in predicting the behavior of pavement after load application. The surface energies of different aggregates and binder types can be measured using different methods. One such method of measuring individual surface energies of aggregates is using USD (Universal Sorption Device). Also the surface free energies of asphalt can be measured using principle of Wilhelmy plate method (6).

The surface free energies of asphalt and aggregate mainly comprise of a non polar component and an acid-base component. Ding Xin Cheng (2002) used the equations developed by Good and Van Oss (1991) to find combined surface free energy of aggregates and asphalt in presence and absence of water. Total surface free energy of aggregate and asphalt can be individually described in form of following equation:

$$\Gamma = \Gamma^{LW} + \Gamma^{AB} \tag{1}$$

Where, Γ = surface free energy of asphalt or aggregate,

Γ^{LW} = Lifshitz-van der waals component of the surface free energy, and

Γ^{AB} = acid-base component of the surface free energy.

From the thermodynamic point of view, the free energy of cohesion, ΔG_i^c , is the energy needed to create a “cohesive” unit area of a crack within a material under vacuum. For an individual component it can be shown as:

$$\Delta G_i^C = 2\Gamma_i \tag{2}$$

Hence equation (1) can be written as:

$$\Delta G_i^C = \Delta G_i^{cLW} + \Delta G_i^{cAB} \tag{3}$$

The free energy of adhesion corresponds to the creation of an “adhesive” unit fracture of two unlike bodies in a vacuum. The equation showing combined surface energy of two components i.e. asphalt and aggregate is as follows:

$$\Delta G_{ij}^a = \Gamma_i + \Gamma_j - \Gamma_{ij} \tag{4}$$

$$\Delta G_{ij}^a = \Delta G_{ij}^{aLW} + \Delta G_{ij}^{aAB} \tag{5}$$

This can be written as:

$$\Delta G_{ij}^a = 2\sqrt{\Gamma_i^{LW}\Gamma_j^{LW}} + 2\sqrt{\Gamma_i^{+}\Gamma_j^{-}} + 2\sqrt{\Gamma_i^{-}\Gamma_j^{+}} \tag{6}$$

If the value of the free energy of cohesion or adhesion is positive, it means the two phases of the material tend to bind together and the higher magnitude of free energy of cohesion or adhesion gives the higher bonding strength. The free energy of adhesion of two different materials in contact within a third medium, such as water is given by:

$$\begin{aligned}\Delta G_{132}^a &= \Gamma_{13} + \Gamma_{23} - \Gamma_{12}\\ &= 2\Gamma_3^{LW} + 2\sqrt{\Gamma_1^{LW}\Gamma_2^{LW}} - 2\sqrt{\Gamma_1^{LW}\Gamma_3^{LW}} - 2\sqrt{\Gamma_2^{LW}\Gamma_3^{LW}}\\ &+4\sqrt{\Gamma_3^{+}\Gamma_{3.}^{-}} - 2\sqrt{\Gamma_3^{+}}\left(\sqrt{\Gamma_1^{-}} + \sqrt{\Gamma_2^{-}}\right) - 2\sqrt{\Gamma_3^{-}}\left(\sqrt{\Gamma_1^{+}} + \sqrt{\Gamma_2^{+}}\right)\\ &+2\sqrt{\Gamma_1^{+}\Gamma_{2.}^{-}} + 2\sqrt{\Gamma_1^{-}\Gamma_{2.}^{+}}\end{aligned} \tag{7}$$

Where, 1= Asphalt; 2 = Aggregate; 3 = Water.

Above equation can be used to calculate the adhesive bond strength of asphalt concrete mix in presence of water. Hence using above equations it is possible to calculate the surface free energy of individual components of asphalt concrete mix i.e aggregates and asphalt as well as combined surface energy values for entire mix. Surface energy values for individual component are defined as cohesive bond energy values while surface energy values for combined components are defined as adhesive bond energy values.

Based on this concept certain laboratory tests were performed where the individual surface energy values for aggregates and asphalt were measured (table-2) (6). The aggregates used were Georgia granite and Texas Limestone. Also the asphalt types used were AAD-1 and AAM-1. Later on the adhesive bond strength of each mix was calculated in presence and absence of water (table-1). After performing these tests and calculating adhesive bond values it was observed that the mixes with AAM-1 asphalt type were less susceptible to moisture damage compared to the mixes with AAD-1 asphalt. Also it was observed that the bond strength calculated in presence of water turned out to be less than the bond strength in absence of water. Same results were obtained in case of each aggregate mix. So it was concluded that based on these values the quality of asphalt concrete mix in terms of bond strength can be characterized and the best possible combinations of aggregate and asphalt can be used while preparing the mixes for pavements. In order to validate the tests and calculations based on surface energy concept a performance test was performed, which is shown later in the thesis.

TABLE-1 Adhesive bond energy values for mixes with and without moisture (6)

AGGREGATE MIX	ΔG_{12} (ergs/cm^2)	ΔG_{123} (ergs/cm^2)
Texas Limestone + AAD	141.5	-66.8
Texas Limestone + AAM	204.9	-30.9
Georgia granite + AAD	152.8	-48.3
Georgia granite + AAM	198.6	-30

TABLE-2 Cohesive bond energy values for asphalts (6)

ASPHALT TYPE	COHESIVE BOND ENERGY (ΔG_{total}) (ergs/cm^2)
AAD-1	54.3
AAM-1	96.7

EFFECTS OF HYDRATED LIME AS A MINERAL FILLER IN ASPHALT CONCRETE MIXES

Composition of Asphalt concrete has played a significant role in the performance of flexible pavements. Aggregate and Binder type have been an important aspect of asphalt concrete performance. Along with these aspects the importance of additives in the asphalt concrete mix has been also realized based on its use in past years. These additives have been very affective in the reduction of distresses like permanent deformation in asphalt concrete mix. Some of the additives like hydrated lime, fly ash, polymer rubber are example of it. These additives act like a filler in the voids and result in increased strength of mix.

Based on the laboratory and field testing in last several years, it has been proved that hydrated lime improves the composition of the mastic and produces multifunctional benefits in the mixture. Work done in the United States and several other countries has proved that hydrated lime can substantially improve the resistance of the HMA to permanent deformation damage at high temperatures (7). Hydrated lime substantially improves low temperature fracture toughness without reducing the ability of the mastic to dissipate energy through relaxation. Hydrated lime acts as filler and reacts with bitumen resulting in some of the beneficial mechanisms in terms of strength. It has been also proved that there are also benefits of reduced susceptibility to age hardening and improved moisture resistance (7).

1) Benefits of adding Hydrated Lime

There are various benefits of adding hydrated lime in asphalt concrete mix (7, 8).

- It acts as mineral filler and stiffens the asphalt binder and HMA.
- It improves fracture toughness at low temperatures.
- It favorably alters the oxidation mechanism and interacts with products of oxidation to reduce their deleterious effects.
- It changes the plastic properties of clay fines and improves moisture stability and durability.
- Above all it also has the ability to control water sensitivity and is well accepted as an antistrip to inhibit moisture damage.

Study on the effects of hydrated lime on asphalt concrete mix has been performed by various state DOTS and research institutes using difference performance tests.

Tarrer(1996), based on his investigation concluded that water at the surface of the aggregate has a high pH and therefore most liquid antistrip agents remain at the surface because they are water soluble at high pH levels. To overcome being washed away these liquid antistripping agents must be given some time to cure. While on other hand, hydrated lime cures very fast and forms water insoluble compounds. Hydrated lime creates a very strong bond between the bitumen and the aggregate, preventing stripping at all pH levels. He further found that hydrated lime reacted with silica and alumina aggregates in a pozzolanic manner that added considerable strength to the mixture.

Research performed by J.Epps (1992) show that the addition of hydrated lime to HMA increases stiffness. This helps to distribute and reduce the stresses and strains within the pavement structure created by traffic loads and generally reduce permanent deformation potential.

Some of the Creep Tests performed at Texas by Little (1994) also show that hydrated lime promotes high temperature stability, thereby increasing resistance to permanent deformation.

Studies by Little (1996) and Lesueur, Little, and Epps (1998) evaluated the changes in rheology, aging kinetics, oxidative hardening created by adding hydrated lime to HMA. It showed the improvements in resistance to permanent deformation, fatigue cracking and low temperature fracture.

Johannson(1998) conducted extensive research on bitumen ageing and adding of hydrated lime to bitumen. His findings showed that although the filler effect of lime increases low temperature stiffness, fracture toughness is also increased substantially. He also saw that hydrated lime reduces the effects of age-hardening more so at high temperatures than at low temperatures.

Hopman (1998) did some of the most powerful research work demonstrating a lime-bitumen interaction which showed similar results as reported by Lesueur, Little, and Epps (1998). Hopman used light absorption measurements and gel permeation chromatography to show a significant change in generic composition of the bitumen after the addition of lime which indicated the lime is active filler.

2) Methods of adding Lime to Asphalt concrete

There are various methods adopted to add hydrated lime in the production of asphalt concrete (7). Some of these methods are listed below:

- Adding lime in proportion to the drum or batch mixer.
- Adding dry lime to dry aggregate and to wet aggregate.
- Adding lime slurry to dry aggregate prior to making a mix.
- Marinating or stockpiling of lime treated aggregates prior to mixing.

All these methods have their own advantages and disadvantages. Various DOTS are using different methods to add lime during production of HMA. Use of lime slurries improve resistance of the treated hot mix to stripping, reduces dusting associated with the addition of dry lime and improved distribution of the lime on the aggregate. However the disadvantage of using lime slurry is that it adds more water than is typically used for conventional lime applications and can substantially increase the water content of the aggregate prior to entering the drying and mixing portions of the HMA facility. The advantage of marinating and stockpiling is reduction in moisture content and improvement in the resistance to moisture. It is also followed by disadvantages like carbonation of lime in stockpiles and washing of lime from the aggregate. The other method which can be followed prior to marinating is adding dry lime to dry aggregate and to wet aggregate. Moisture on the aggregate surface ionizes the lime and helps distribute it on the aggregate surface. Instead of marinating these aggregates a faster rate of drying can be achieved by heating them for few hours prior to production of asphalt concrete.

Many tests done to evaluate such additives are based solely on short term retained strengths and do not represent long-term performance of asphalt. There is a need for a simple and repeatable test that can evaluate the multifunctional aspects of pavement performance. Some of such tests and methods have been developed in recent years to evaluate the permanent deformation in asphalt concrete mix due to addition of such fillers. Developing more test methods to predict the performance of asphalt mix would always add to the available knowledge regarding effects of additives in asphalt concrete. One such method has been tried to develop here which shows the effect of hydrated lime on the asphalt concrete mix in terms of permanent deformation.

BEHAVIOR OF ASPHALT CONCRETE MIXES UNDER CYCLIC LOADING CONDITION

The basic components used in asphalt concrete mixes are aggregates and asphalt. These components exhibit different types of behavior when subjected to loading. Their behavior can be classified as elastic, viscous or viscoelastic. Elastic materials regain their original form after being released from stress. Asphalt under very low temperature becomes very stiff and shows this kind of behavior. Some asphalts act differently based on their composition. Viscous materials are the one which cross the limit of elasticity and enter fully in to plastic stage from where they cannot return to their original form after a stress application. Aggregate and asphalt together act as a composite with intermediate properties. They demonstrate a viscoelastic behavior. Upon loading the material deforms first and later rebounds by some amount. It shows some amount of deformation. Viscoelastic materials exhibit dependence on the entire history of loading or deformation as well as the current state of load or deformation (9). Viscoelastic material behavior is categorized in two forms: linear and nonlinear. Linear viscoelastic materials are dependent on the time history of the loading or deformation while nonlinear viscoelastic materials are dependent on the stress or strain history (9).

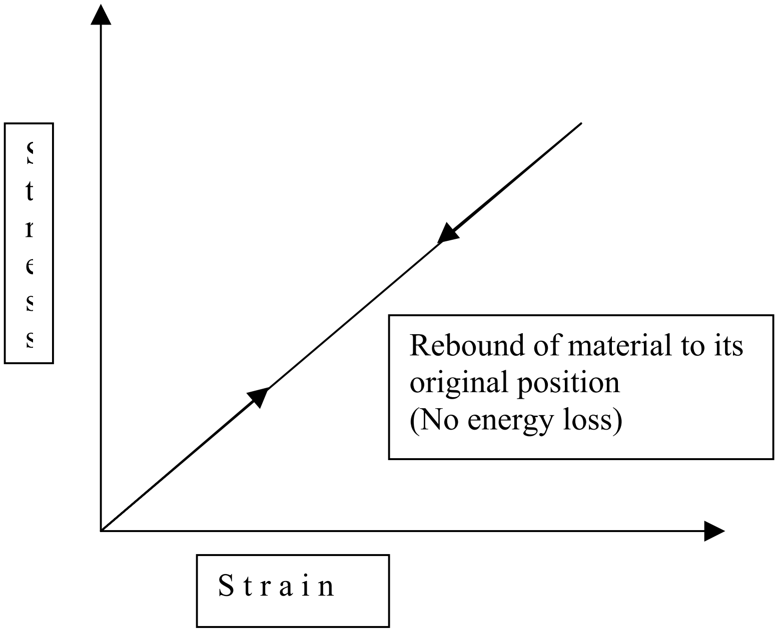


FIGURE-5 A plot of stress-strain showing elastic behavior of material at low temperatures

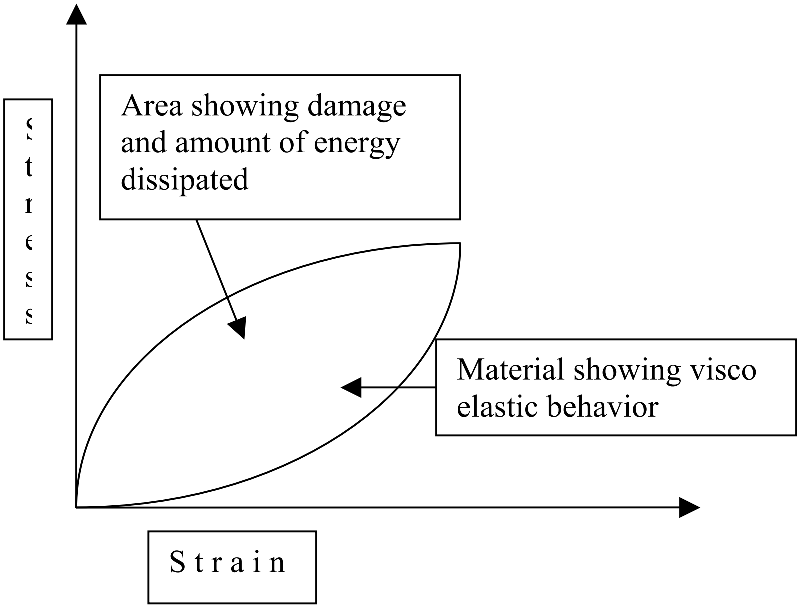


FIGURE-6 A plot of stress-strain showing viscoelastic behavior of material

On plotting a stress-strain curve for one cycle of load applied to a viscoelastic material a loop like pattern is observed (figure-5). The area of this loop exhibits the amount of energy dissipated from the material while loading during that particular cycle (figure-6). The area of loop increases with increasing damage. A certain amount of energy is lost during a cycle of load when the material deforms and returns back to certain position. This energy is known as dissipated strain energy (figure-8).

Asphalt mixes exhibit microcracking during repeated loading. During the crack initiation process, microcracks initiate from microscopic size and grow to macrocracking size, where crack propogation begins. Recent studies (9) at Texas Transportation Institute (TTI) of Texas A&M University indicate that the development of microcracks in the asphalt layer under repeated loading applications accelerates permanent deformation. Work hardening occurs in well-designed asphalt mixtures, while work hardening may not be present in rut-susceptible mixtures. If asphalt concrete work hardens under repeated loading with accumulating plastic deformation, but neither contains microcrack arresters nor heals rapidly, it may reach a point where it is stiff enough for microcracks to initiate and grow (10). On initiation of these microcracks more permanent deformation is accumulated in asphalt mixes which is commonly called “tertiary creep” (figure-7).

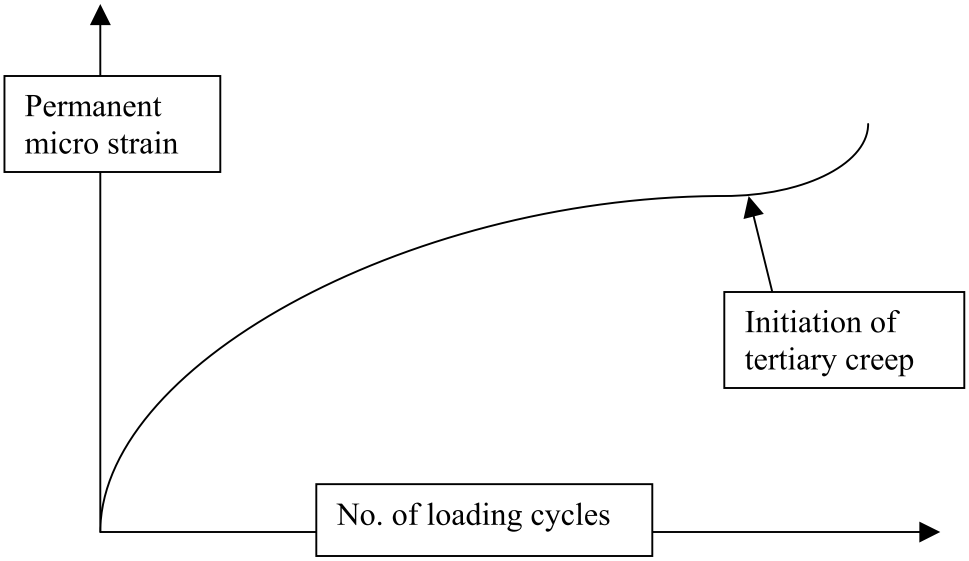


FIGURE-7 A permanent microstrain test data showing initiation of tertiary creep (10)

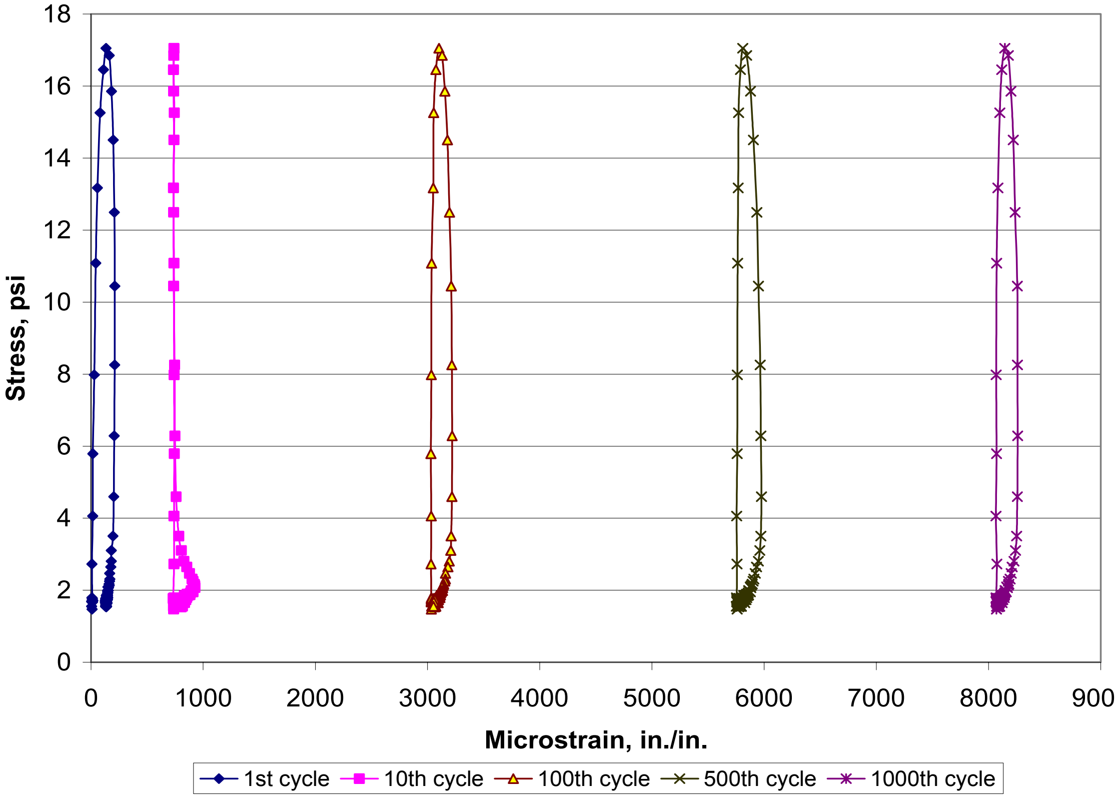


FIGURE-8 A test data showing the loops of dissipated strain energy of a mix under repeated loading

CHAPTER V

A PERFORMANCE TEST

MATERIALS

Results obtained from any kind of testing depend on the type and quality of materials used for testing. The basic materials used for this type of testing are asphalt cement, aggregates and additives/Fillers.

1) Asphalt Cement

   Two types of asphalt cements are used in this testing. They are AAM-1 and AAD-1. These binders are classified by Strategic Highway Research Program (SHRP).

2) Aggregates

   Aggregates used in this testing are Brazos valley river gravel, Texas Limestone and Georgia crushed granite. All the three materials are widely used in construction of asphalt concrete pavements.

   - Brazos valley river gravel is siliceous, sub- rounded, smooth surface textured river gravel with more susceptibility to permanent deformation. It can make more homogeneous material mix.
   - Texas crushed Limestone is characterized as a very hard, low-porosity, low absorption, and somewhat dolomitic limestone. It has much more angularity and more rough texture compared to the Brazos river gravel.
   - Georgia Granite is rougher in terms of surface texture. It is more Angular and is highly used in the construction of HMA.

3) Filler/Additive

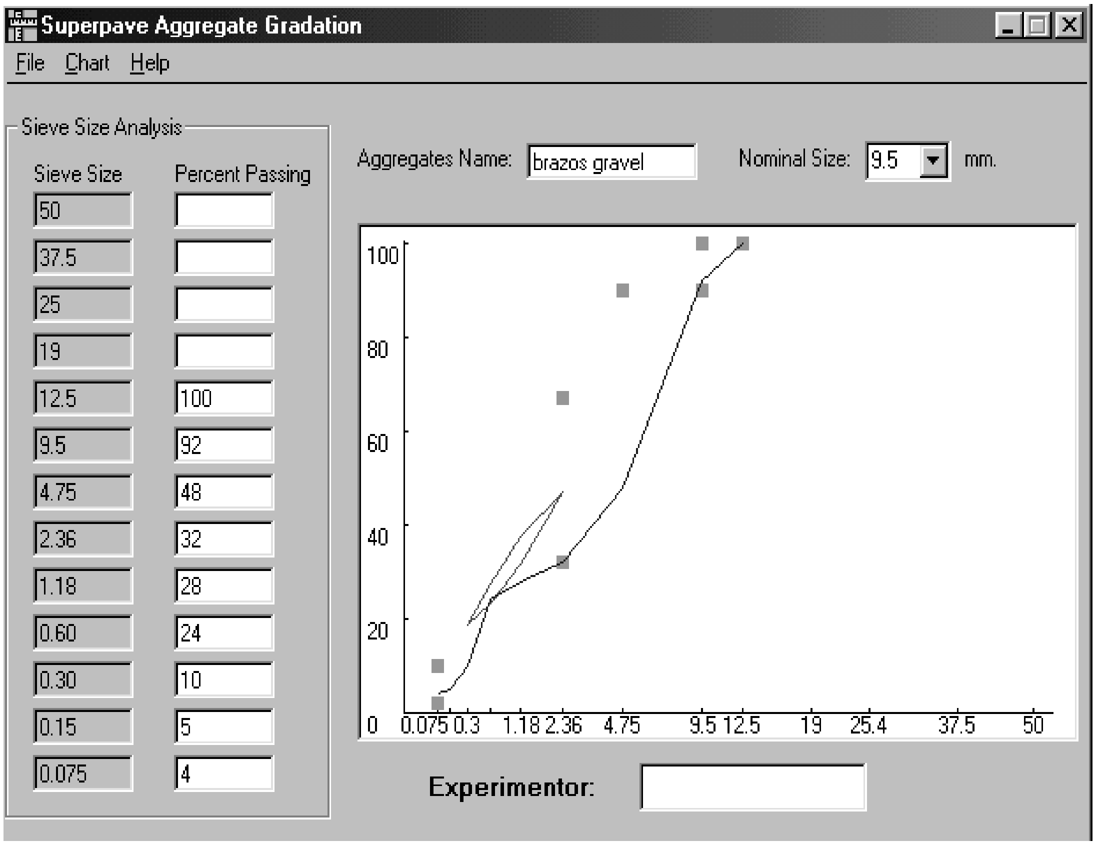
In this testing two types of fillers were used. Testing was carried out taking material retained in #-200 sieve as filler in one case. In second case hydrated lime was used as an additive. Both types of fillers were tested with binders AAM and AAD. Results obtained with both fillers are compared. Hydrated lime is an interactive additive. It has a potential ability to interact with different asphalts. It has been used as an anti-stripping agent for a longtime. According to various postulates, lime interacts with acids in the asphalt cement that are readily absorbed on the aggregate surface; lime provides calcium ions which can replace hydrogen, sodium, potassium and other cations on the aggregate surface; and lime reacts with most silicate aggregates to form a calcium silicate crust which has a strong bond to the aggregate and has sufficient porosity to allow penetration of the asphalt cement to form another strong bond. Testing has been carried out in order to verify how well this characteristic of lime works with different type of asphalt concrete mixes.

MIX DESIGN

1) Mix Design of Asphalt concrete using Brazos Gravel (figure-9, table-3 and 4)

TABLE-3 Aggregate gradation for Brazos gravel

Aggregate size (mm)	Large Gravel	Small Gravel	River Sand	Field Sand
Sieve Size(mm)	Percent Passing (%)			
12.5	100	100	100	100
9.5	77.35	99.65	100	100
4.75	2.86	41.49	98.58	100
2.36	0.68	3.19	85.29	100
1.18	0.39	0.69	72.08	100
0.6	0.36	0.45	53.36	100
0.3	0.34	0.35	11.93	98.94
0.15	0.31	0.32	2.41	33.75
0.075	0.31	0.31	1.42	18.42



Sieve Size	Percent Passing
50	
37.5	
25	
19	
12.5	100
9.5	92
4.75	48
2.36	32
1.18	28
0.60	24
0.30	10
0.15	5
0.075	4

Aggregates Name: brazos gravel

Nominal Size: 9.5 mm.

Experimentor:

FIGURE-9 A gradation chart for Brazos gravel

- Optimum Asphalt content used is 3.6%
- Rice Specific Gravity (Gmm) of mix is 2.484; Bulk Specific Gravity (Gmb) of mix is 2.385

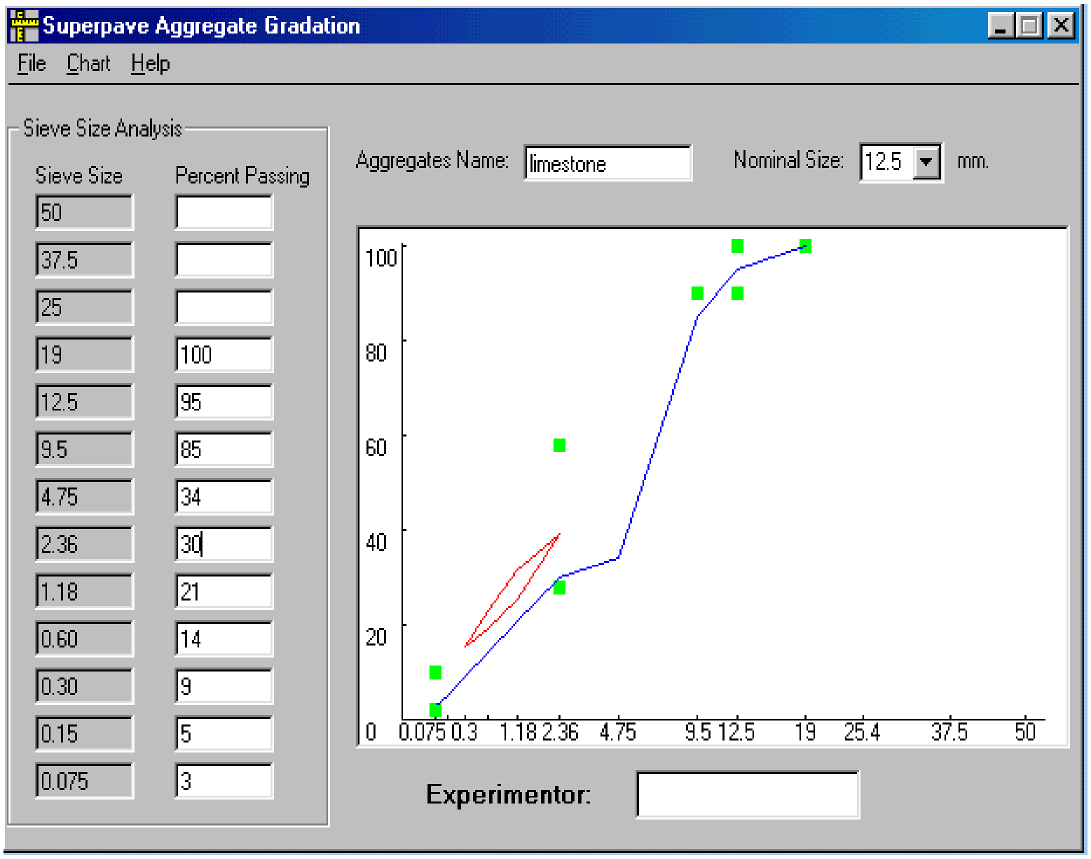
TABLE-4 Specific gravity and water absorption for Brazos gravel

Aggregate Size	Bulk Specific Gravity	Bulk Specific Gravity(SSD)	Apparent Specific Gravity	Absorption %
Large Gravel	2.625	2.644	2.676	0.72
Small Gravel	2.620	2.646	2.690	0.99
River Sand (Coarse)	2.587	2.606	2.637	0.74
River Sand (Fine)	2.575	2.633	2.733	2.25
Field Sand	2.623	2.632	2.646	0.34

2) Mix Design of Asphalt Concrete using Texas Crushed Limestone (figure-10, table-5 and 6)

TABLE-5 Aggregate gradation for Texas Limestone

Aggregate size (mm)	12.5	9.5	4.75	Washed Screen
Sieve Size(mm)	Percent Passing (%)			
12.5	100	100	100	100
9.5	72.50	100	100	100
4.75	1.59	5.57	64.83	99.35
2.36	0.88	3.19	8.75	88.28
1.18	0.80	2.35	5.26	62.29
0.6	0.78	1.91	4.44	40.51
0.3	0.77	1.67	4.00	24.55
0.15	0.74	1.50	3.65	12.49
0.075	0.71	1.38	3.36	6.80



Sieve Size Analysis

Sieve Size	Percent Passing
50	
37.5	
25	
19	100
12.5	95
9.5	85
4.75	34
2.36	30
1.18	21
0.60	14
0.30	9
0.15	5
0.075	3

Aggregates Name: limestone

Nominal Size: 12.5 mm.

Experimentor:

FIGURE-10 A gradation chart for Texas Limestone

- Optimum Asphalt content used is 4.3% for mix without hydrated lime.
- Rice Specific Gravity (Gmm) of mix without hydrated lime is 2.512; Bulk Specific Gravity (Gmb) of mix without hydrated lime is 2.412

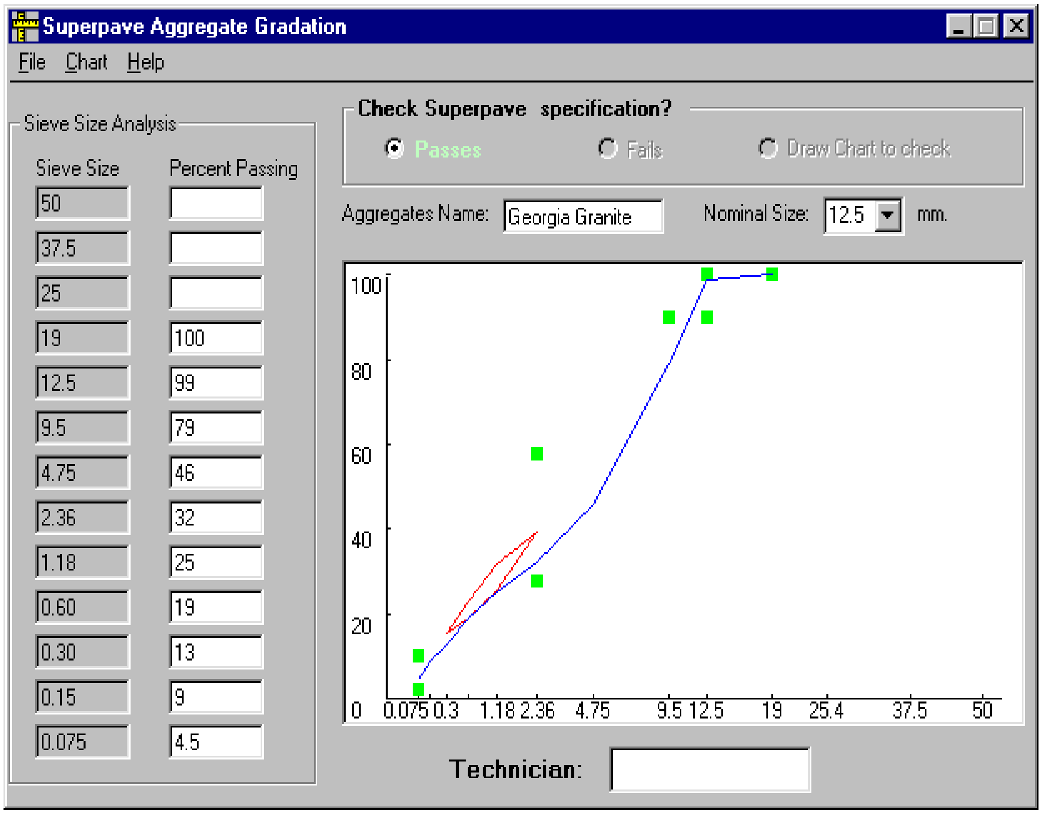
TABLE-6 Specific gravity and water absorption for Texas Limestone

Aggregate Size	Bulk Specific Gravity	Bulk Specific Gravity(SSD)	Apparent Specific Gravity	Absorption %
12.5	2.675	2.695	2.729	0.73
9.5	2.660	2.685	2.725	0.89
4.75(Coarse)	2.655	2.676	2.712	0.80
4.75(Fine)	2.586	2.631	2.698	1.75
Washed Screen (Coarse)	2.645	2.673	2.719	1.03
Washed Screen(Fine)	2.676	2.654	2.720	1.46

3) Mix Design of asphalt concrete using Georgia Granite (figure-11, table-7and 8)

TABLE-7 Aggregate gradation for Georgia Granite

Aggregate size (mm)	007 ID	810 ID	W10 ID	089 ID
Sieve Size(mm)	Percent Passing (%)			
19.00	100	100	100	100
12.5	97	100	100	100
9.5	51	100	100	99
4.75	4	86	99	30
2.36	3	63	77	2
1.18	2	53	57	1
0.6	1	43	40	0
0.3	0	31	25	0
0.15	0	24	10	0
0.075	0	12	3	0



Sieve Size Analysis

Sieve Size	Percent Passing
50	
37.5	
25	
19	100
12.5	99
9.5	79
4.75	46
2.36	32
1.18	25
0.60	19
0.30	13
0.15	9
0.075	4.5

Check Superpave specification?

● Passes ○ Fails ○ Draw Chart to check

Aggregates Name: Georgia Granite

Nominal Size: 12.5 mm.

Technician:

FIGURE-11 A gradation chart for Georgia granite

- Optimum Asphalt content used is 4.5%
- Rice specific Gravity (Gmm) of mix is 2.451

  Bulk specific Gravity (Gmb) of mix is 2.358

TABLE-8 Specific Gravity for Georgia Granite

Aggregate Size (mm)	Bulk Specific Gravity (Oven Dry)	Bulk Specific Gravity (Sat.Surface dry)	Apparent Specific Gravity
+19mm (coarse)	2.7059	2.7186	2.7432
+12.5mm (coarse)	2.7044	2.7181	2.7426
+9.5mm(coarse)	2.7043	2.7174	2.7430
+4.75mm(coarse)	2.7045	2.7177	2.7416
Fine Aggregates	2.6721	2.7012	2.7517

Following are the basic criteria which should be fulfilled for an asphalt mix to be proper (table-9).

TABLE-9 Basic criteria for asphalt concrete mix design

Mix Property	**Criteria**
% Air Voids	4.0%
% VMA	13.0% min
% VFA	65%-75%
Dust Proportion	0.6-1.2
%Gmm @ N_{in}	Less than 89%
%Gmm @ N_{max}	Less than 98%

These criteria satisfy for both the type of materials with their respective mix design.

TESTING PROCEDURE

Testing procedure has been broken in step wise method explained as follows:

1) Preheating

- Aggregates are heated overnight or minimum for 6 hours in oven to dry them to maximum extent.
- Binder is kept in oven for 2 hours at an appropriate mixing temperature.
- Mixing temperature for Binder while using Brazos valley river gravel is 149°C.
- Mixing temperature for Binder while using Texas Limestone without Hydrated lime is 149 °C.
- Mixing temperature for Binder while using Georgia crushed granite is 159°C.

2) Mixing

A mix is prepared in a mixing bucket (figure-12). This bucket is kept in oven before half an hour of mixing so that it achieves the required mixing temperature. After that the heated aggregates and binder are mixed as per the mix design in bucket. Mixing is done according to field conditions using a rotator in laboratory. Aggregate binder composite is mixed properly by rotating at least for 4 minutes in order to achieve proper mixing. The mix is removed in a tray which is again preheated up to certain temperature. This mix is kept in oven at appropriate compaction temperature for 2 hours.

Compaction Temperature for mix with Brazos valley river gravel is 135°C.

Compaction Temperature for mix with Georgia crushed granite is 145°C.

Compaction Temperature for mix with Texas Limestone is 135°C.

FIGURE-12 Asphalt concrete mixing equipment used in laboratory

3) Compaction

Compaction of mix is done using Superpave Gyratory compactor (figure-13). This compactor is an advanced gyratory compactor after Texas gyratory compactor. This compactor is designed in order to achieve the compaction similar to that achieved in field. This has proved very successful to achieve the compaction up to desired density. The mix is compacted to form the samples of size having 100mm diameter and about 150 mm height considering required density and maintaining 4% air void.

4) Preconditioning

- Samples are preconditioned before testing them on loading machine. Dry samples are kept in oven at 40^{o}C and wet samples are soaked in water at 40^{o}C, till they are fully saturated.
- For Brazos gravel mix the preconditioning of dry samples is done for 2hrs and wet samples are saturated and vacuumed for 3 hours in all.
- For Texas Limestone mix the preconditioning of dry samples is done for 3 hrs and wet samples are saturated and vacuumed for 4 hours in all.
- For Georgia granite mix the preconditioning of dry samples is done for 4 hrs and wet samples are saturated and vacuumed for 5 hours in all.

FIGURE-13 Superpave gyratory compactor

FIGURE-14   Compacted asphalt concrete samples (4" x 6")

5) Permanent Deformation Test

After preconditioning of samples, the samples are loaded on MTS (Materials Testing System) machine (figure-15). This machine is used to apply a repeated unconfined compressive load to the sample in a controlled stress mode. The test continues with 1 Hz haversine wave loading for 50,000 cycles. A preset program is used to record the selected data at 1, 10, 100, 1000, 10000, 20000, 30000, 40000, and 50000 cycles. The permanent deformation in a sample is obtained by measuring the micro strain obtained through 2 LVDT's (linear variable differential transducers) on the periphery of sample. Dry samples are tested in dry condition while wet samples are tested in wet conditions by keeping the sample in water at the time of loading.

6) Sample Replicates

Three samples were made for each mixture type and each testing condition. Three samples were compacted out of one batch of asphalt concrete mix (figure-14). All the three samples were subjected to dynamic loading under loading equipment and the consistency of data result was checked for all of them. An average result was considered out of three test results obtained. A statistical analysis of these test results was not possible due to only 3 samples for each mix type and testing condition.

FIGURE-15 MTS machine used for running permanent deformation test

The testing is performed considering a matrix (table-10 and 11).

Table-10 Mix with Hydrated lime as a mineral filler

Aggregate type	Binder type	Type of test
Brazos River gravel	AAD	Dry and Wet
	AAM	Dry and Wet
Georgia Granite	AAD	Dry and Wet
	AAM	Dry and Wet

Table-11 Mix without Hydrated lime

Aggregate type	Binder type	Type of test
Brazos River gravel	AAD	Dry and Wet
	AAM	Dry and Wet
Georgia Granite	AAD	Dry and Wet
	AAM	Dry and Wet
Texas Limestone	AAD	Dry and Wet
	AAM	Dry and Wet

CHAPTER VI

RESULT ANALYSIS

As mentioned above, asphalt concrete samples are subjected to repeated dynamic compressive loading on MTS (Materials testing system) machine. Output obtained from this machine is in terms of permanent microstrain due to deformation occurring in the sample at intermediate loading cycles. The stress level is kept constant throughout the test. The strain values are measured up to 20,000 loading cycles, each cycle with loading and rest period in it. A graphical display of increasing strain values with each loading cycle is done after obtaining data for each sample. On performing a graphical comparison of dry and wet testing samples a significant difference is observed in the permanent microstrain values of each. Using the data obtained from MTS (Materials testing system) machine and known stress level applied during each test, dynamic modulus values are calculated with the help of peak stress and peak strain at each load cycle. These dynamic modulus values are plotted against number of loading cycles for each sample. The dynamic modulus values for each dry and wet sample of a mix are compared. It has been observed that the dynamic modulus values of wet mixes are low compared to the one for dry mixes at the end of loading test. After that the dynamic modulus values for the same aggregate with two different asphalt types have been compared, where it has been observed that the dynamic modulus of mix with AAM-1 asphalt type is higher compared to AAD-1. Later the ratios of dynamic modulus values of wet test by dry test are calculated and are compared for both the asphalt types with same aggregate used. Similar kind of analysis approach has been used to show the effect of adding hydrated lime as mineral filler in the asphalt concrete mixes. Here the dynamic modulus values of mixes with hydrated lime in it are higher than the one without lime in it. Also the ratios of dynamic modulus values for wet/dry tests with and without lime are compared graphically for each mix. Dynamic modulus value (**E***) for any mix at a particular loading cycle is given by:

$$E^{*} = \frac{\sigma_{\max}}{\varepsilon_{\max}} \tag{8}$$

Where, σ_{max} = Peak stress at a particular load cycle, and

ε_{max} = Peak strain at a particular load cycle.

The ratio of dynamic modulus for wet and dry tests is shown as:

$$K = \frac{E^{*}_{wet}}{E^{*}_{dry}} \tag{9}$$

Higher the value of K better the mix is in terms of resistance to moisture damage.

Similarly in the case of hydrated lime it is shown that the K value is higher for the mix with lime in it as mineral filler. Further the moisture damage of the mixture can be better understood by calculating the percentage of surface area of the aggregate that is replaced by water in the mixture, P (7). The wet-to-dry compression stiffness ratio (ratio of stiffness under wet conditions to stiffness under dry conditions) can be approximated by the work of adhesion ratio between asphalt and aggregate in wet and dry conditions shown as equation (10).

$$\frac{E^*_{wet}}{E^*_{dry}} = \frac{\Delta G_{12} \times (1 - P) + \Delta G_{132} P}{\Delta G_{12}} \tag{10}$$

For the cyclic loaded control stress permanent deformation testing, equation (11) is derived.

$$\frac{E^*_{wet}}{E^*_{dry}} = \frac{(\sigma / \varepsilon)_{wet}}{(\sigma / \varepsilon)_{dry}} = \frac{\varepsilon_{dry}}{\varepsilon_{wet}} = \frac{\Delta G_{12} \times (1 - P) + \Delta G_{132} P}{\Delta G_{12}} \tag{11}$$

Where ε_{dry} and ε_{wet} represent the strain induced in the mixture in the wet and dry testing condition, respectively. All of the variables in equation (11) are obtainable from permanent deformation testing except P, the percent of the aggregate surface area that has been exposed to water due to each cycle. Thus P can be calculated using Equation (11). From the test results it has been observed that the percent of area displaced by water, P, is higher for the AAD-aggregate mixtures than the AAM-aggregate mixtures.

Other way of developing a correlation of surface energy measurements with laboratory testing is by comparing surface energy values with pseudo-strain energy values. Pseudo-strain energy can be obtained by calculating pseudo-strain. Pseudo strain can be calculated from the relaxation modulus function and input strain function using linear viscoelastic constitutive convolution integral (10). Relaxation modulus of a mix can be obtained by applying a low stress level to a sample in its initial testing condition taking care that sample is not damaged. This modulus of mix obtained in its undamaged condition is known as relaxation modulus. Using the relaxation modulus with the input haversine strain wave function, a linear viscoelastic stress under uniaxial loading can be calculated using the following linear viscoelastic constitutive equation:

$$\sigma(t) = \int_0^t E(t-\tau)\frac{d\varepsilon(\tau)}{d\tau}d\tau \tag{12}$$

Where,

σ (t) = time dependent linear viscoelastic stress

t = present time

τ = is the time history at which strains were measured

E (t-τ) = relaxation modulus of the material at loading time, t-τ, under the undamaged condition

ε (τ) = measured strain at the previous time, τ

Once the linear viscoelastic stress is calculated, the uniaxial pseudo strain can then be calculated by dividing the calculated linear viscoelastic stress by a reference modulus, E_R (11).

$$\varepsilon_R = \frac{1}{E_R}\int_0^t E(t-\tau)\frac{d\varepsilon(\tau)}{d\tau}d\tau \tag{13}$$

The equation (13) can be rewritten as follows:

$$\varepsilon_R(t) = \frac{\sigma(t)}{E_R} \tag{14}$$

Using above equation, pseudo-strain values are obtained. These pseudo-strain values when plotted against stress on a graphical plot the data points develop a hysteresis loop. The area within this loop exhibit pseudo-strain energy. With the increase in load cycles the area of the loop change. It is the real dissipated strain energy which describes the real damage during the fatigue test or permanent deformation test, because both the time dependent viscoelastic behavior and the nonlinear behavior have been eliminated by using nonlinear pseudo strain concept. This dissipated pseudo strain energy can also be used to predict the microcrack fatigue life.

The graphical presentation for each mix is shown next page onwards.

The presentation is divided in two parts:

1) Comparison of dynamic modulus values of mixes without hydrated lime with different asphalt content within same aggregate type (figures 16-21).
2) Comparison of dynamic modulus values of mixes with and without hydrated lime with both asphalt types for each aggregate type (figures 17-29).

DRY AND WET TEST COMPARISONS OF VARIOUS AGGREGATES WITH TWO BINDER TYPES

a) Brazos Gravel

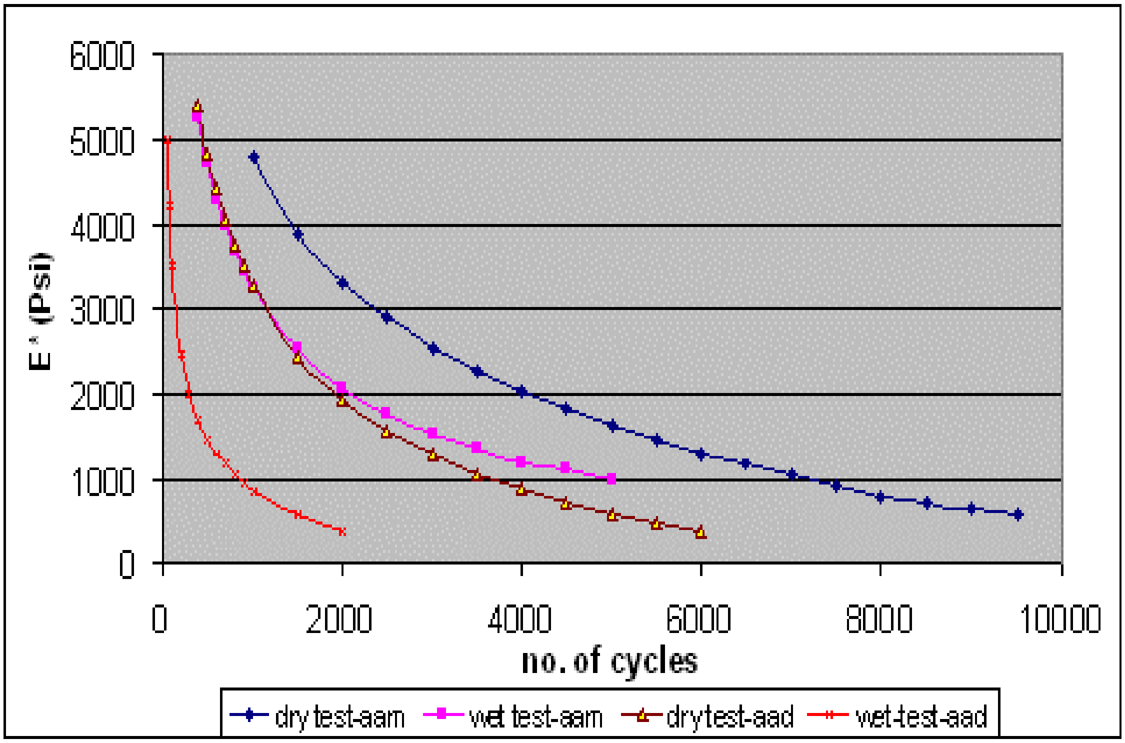


FIGURE-16 Dry and wet test modulus values of Brazos gravel with AAD-1 and AAM-1

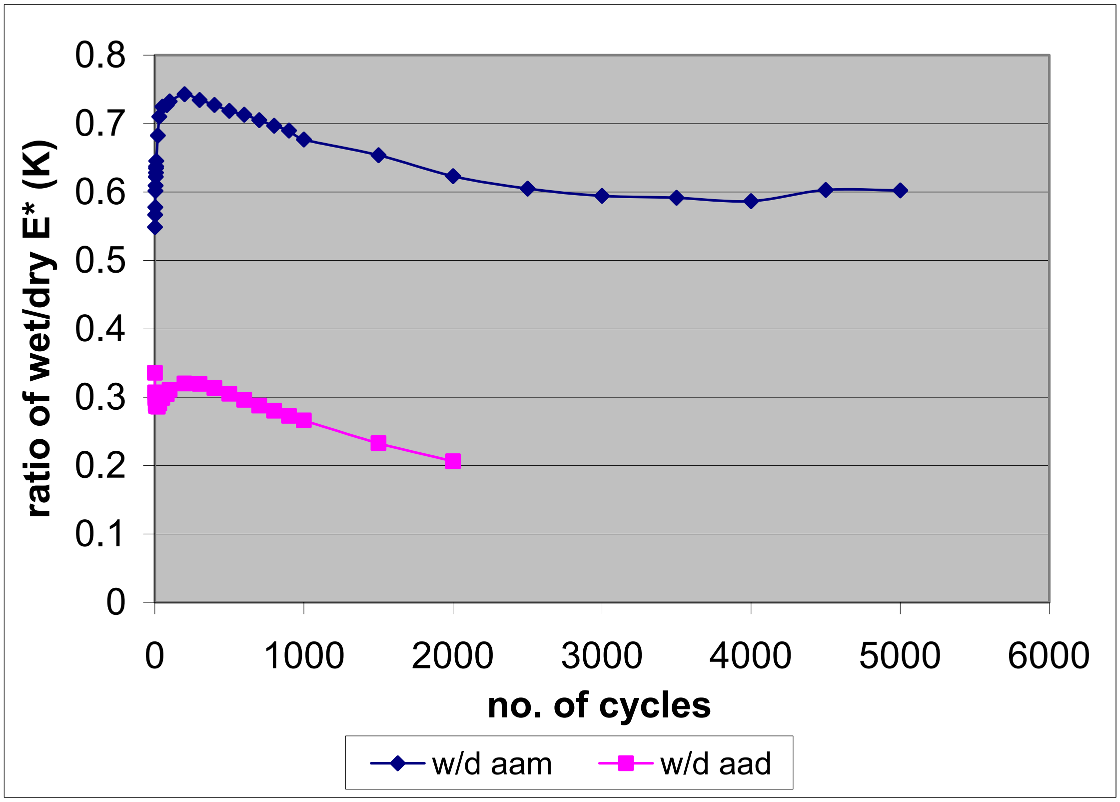


FIGURE-17 Ratio (K) of wet/dry test modulus vs. no. of loading cycles for Brazos gravel

b) Texas Limestone

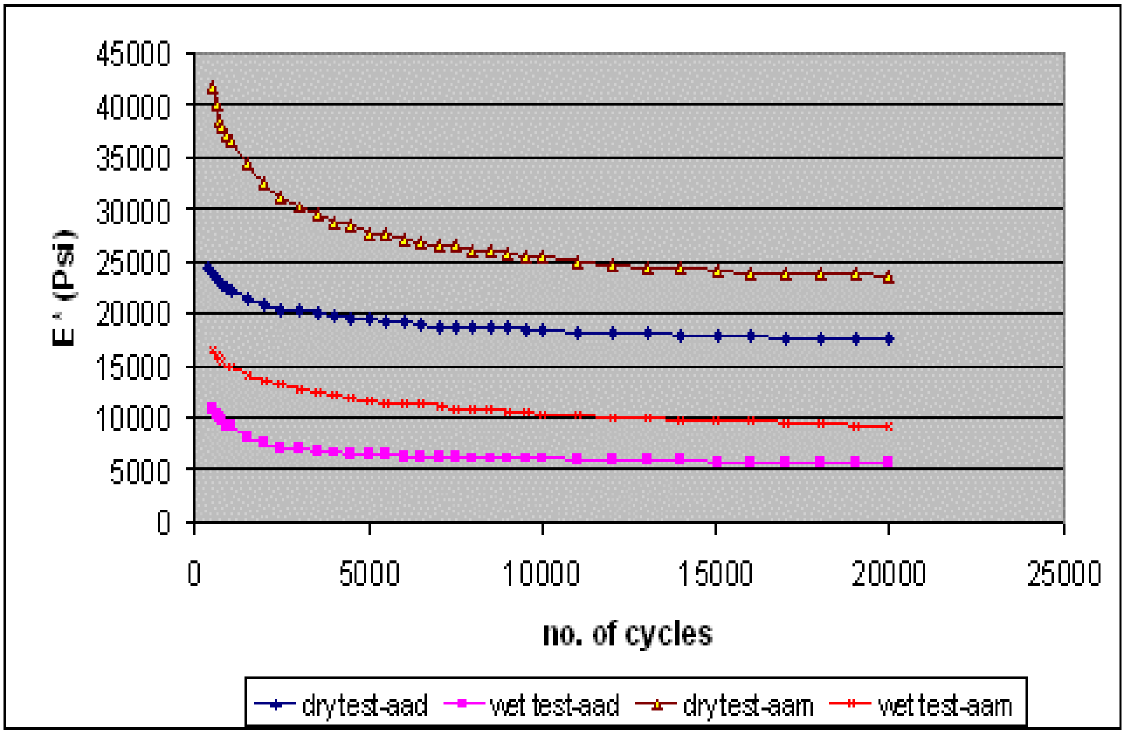


FIGURE-18 Dry and wet test modulus values of Texas Limestone with AAD-1 and AAM-1

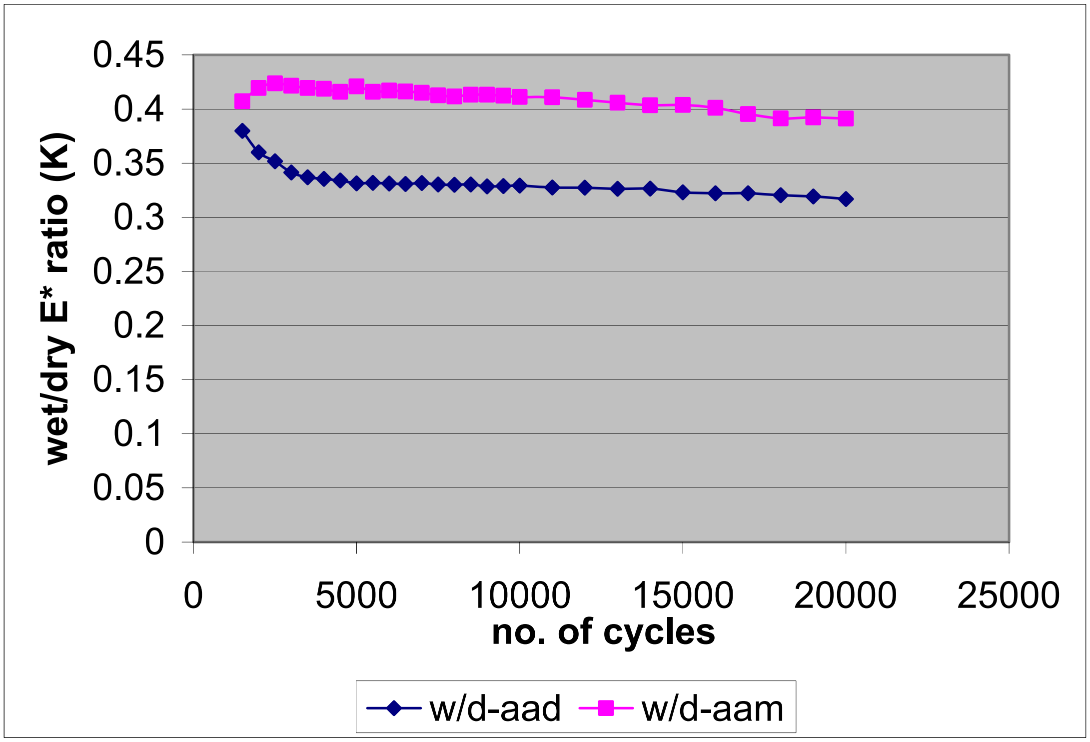


FIGURE-19 **Ratio (K) of wet/dry test modulus vs. no. of loading cycles for Texas Limestone**

c) Georgia granite

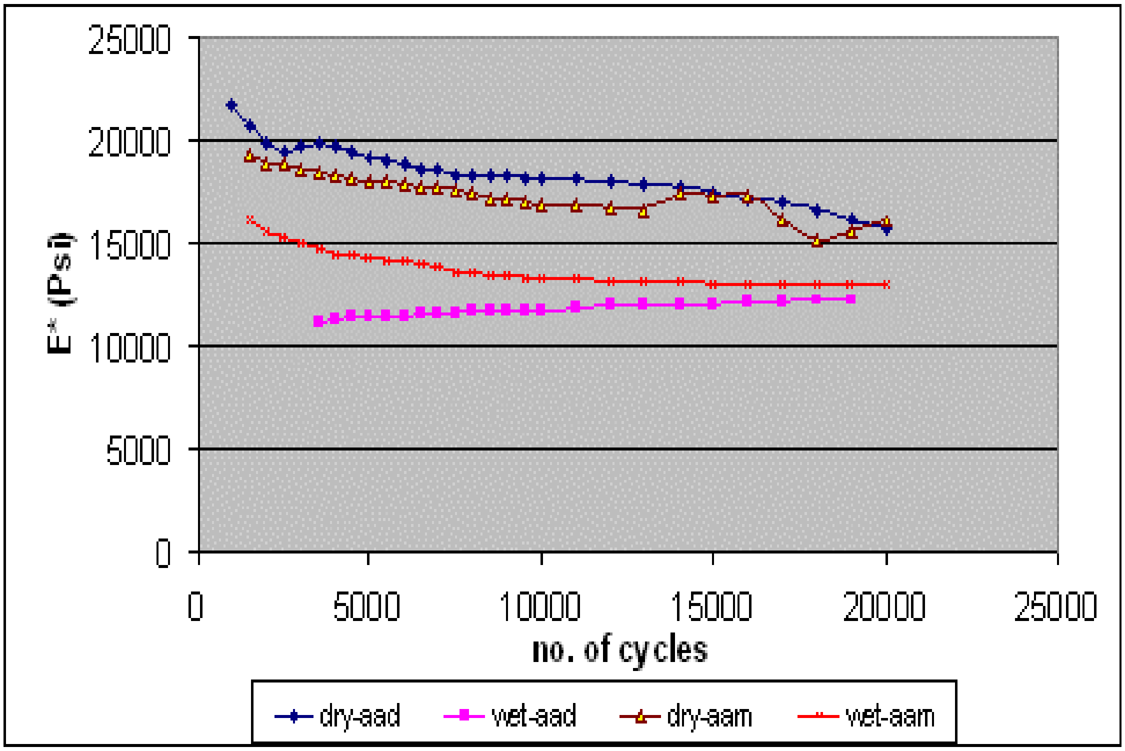


FIGURE-20 Dry and wet test modulus values of Georgia Granite with AAD-1 and AAM-1

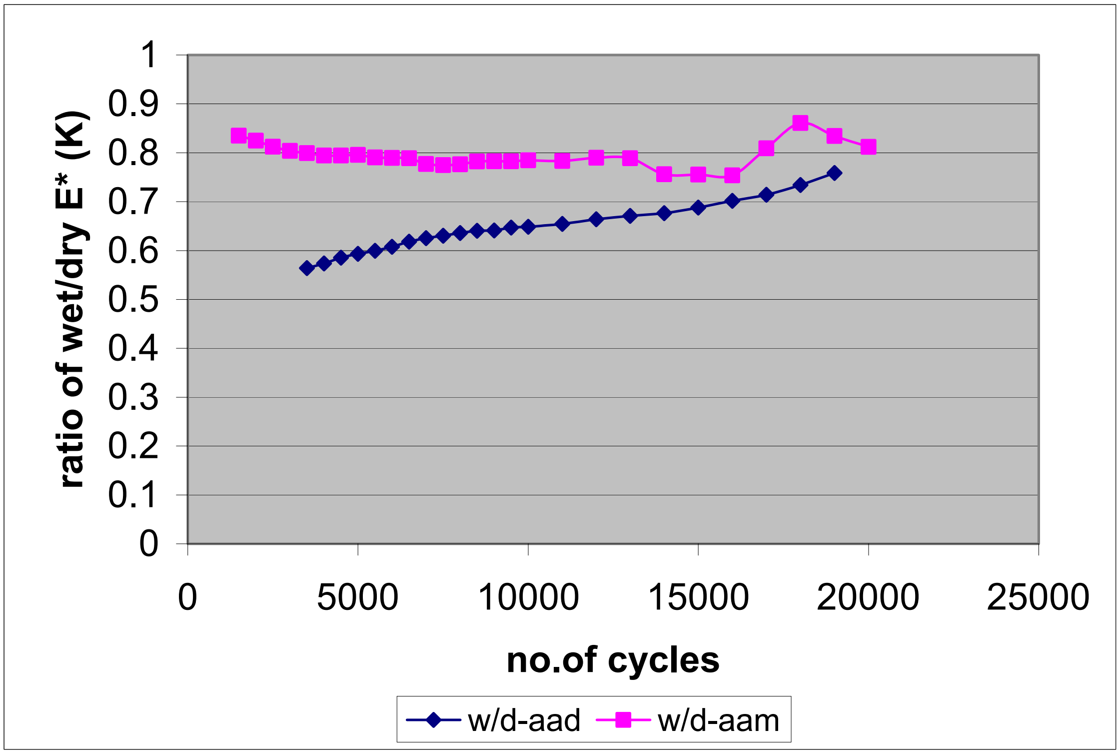


FIGURE-21 **Ratio (K) of wet/dry test modulus vs. no. of loading cycles for Georgia Granite**

DRY AND WET TEST COMPARISONS OF VARIOUS AGGREGATE MIXES WITH AND WITHOUT HYDRATED LIME

a) Brazos gravel with AAD

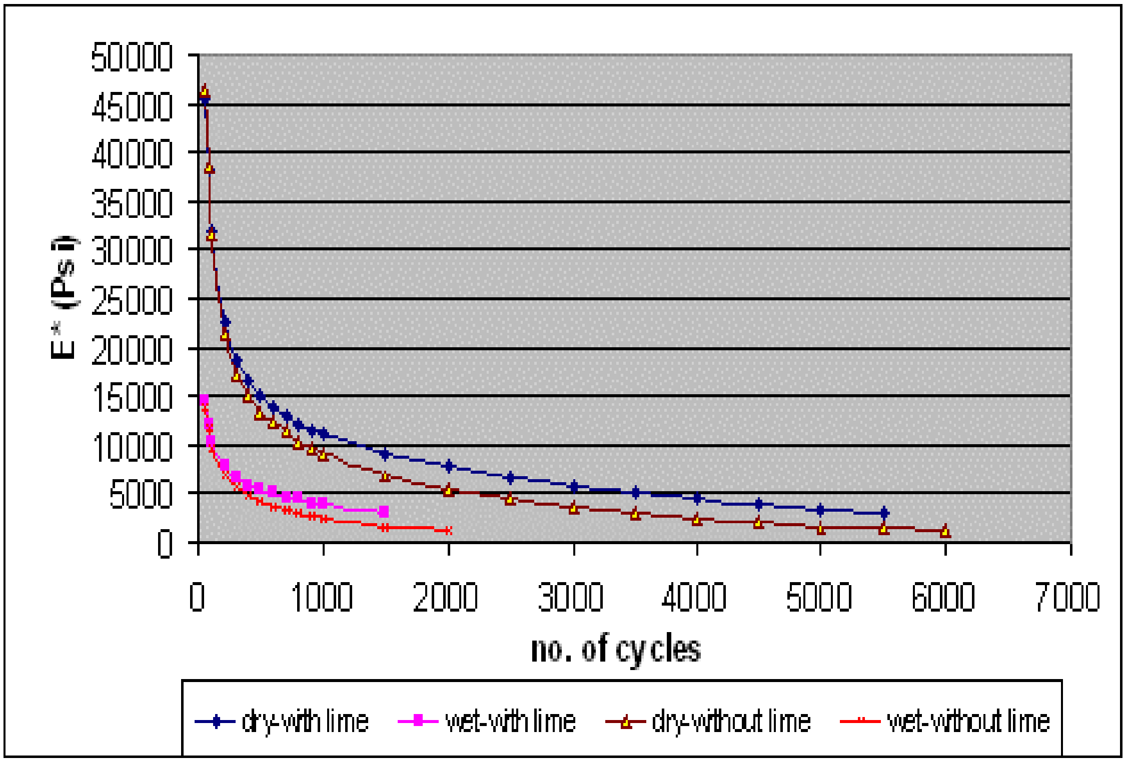


FIGURE-22 **Dynamic modulus values of mixes with and without hydrated lime (Brazos Gravel with AAD)**

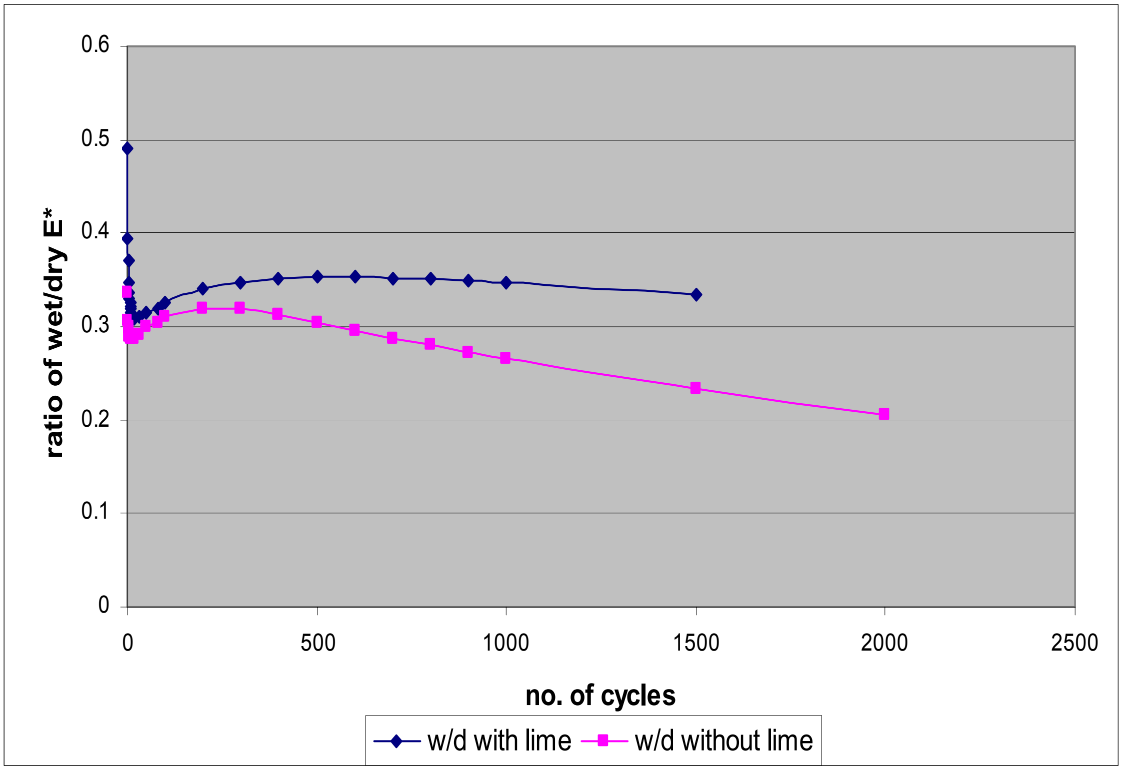


FIGURE-23 **Ratios of wet/dry values of dynamic modulus for mixes with and without hydrated lime (Brazos Gravel with AAD)**

b) Brazos gravel with AAM

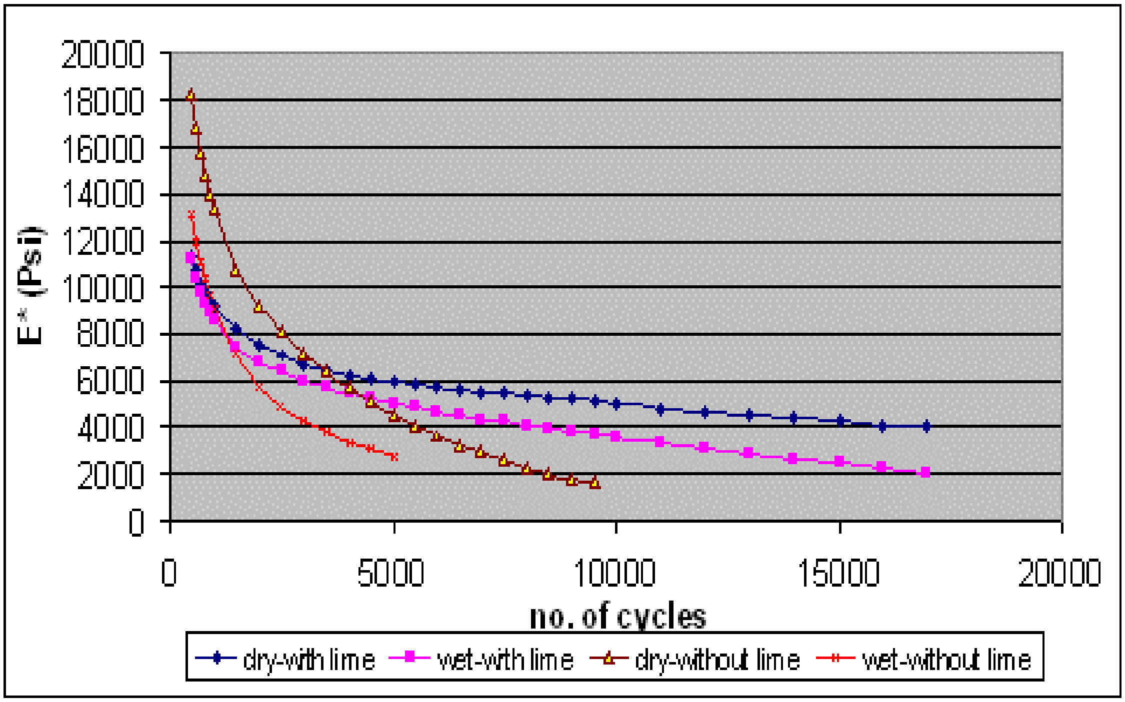


FIGURE-24 **Dynamic modulus values of mixes with and without hydrated lime (Brazos Gravel with AAM)**

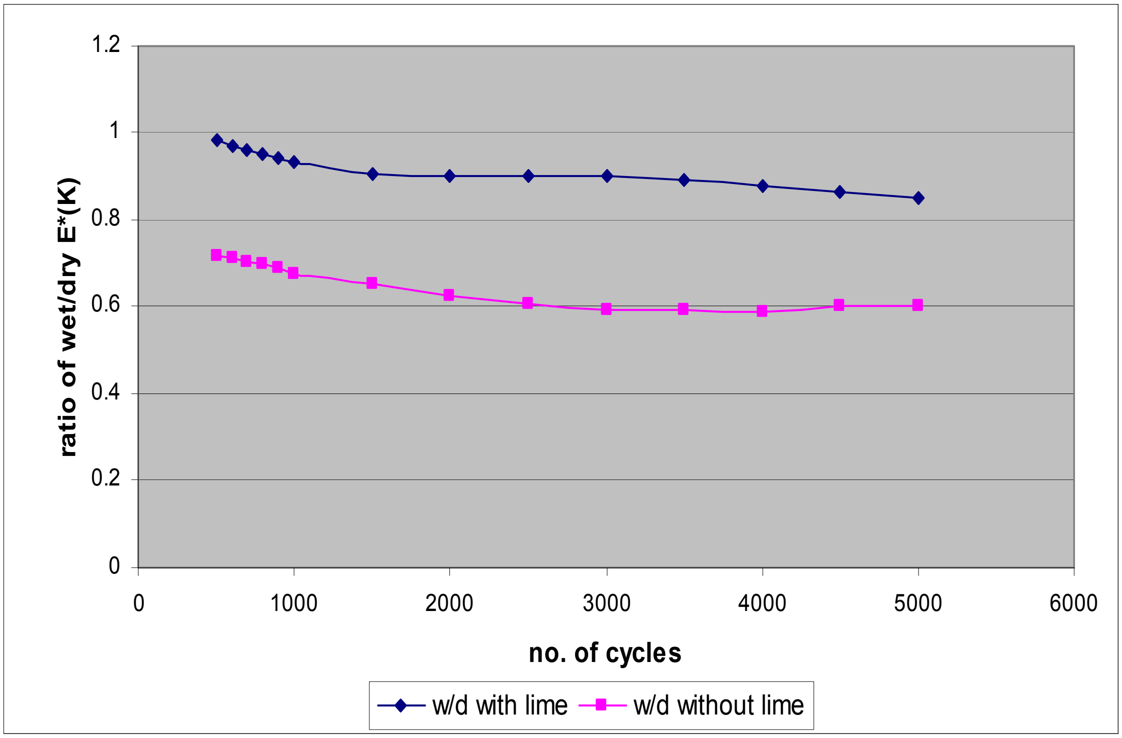


FIGURE-25 **Ratio of wet/dry values of dynamic modulus of mixes with and without hydrated lime (Brazos Gravel with AAM)**

c) Georgia Granite with AAD

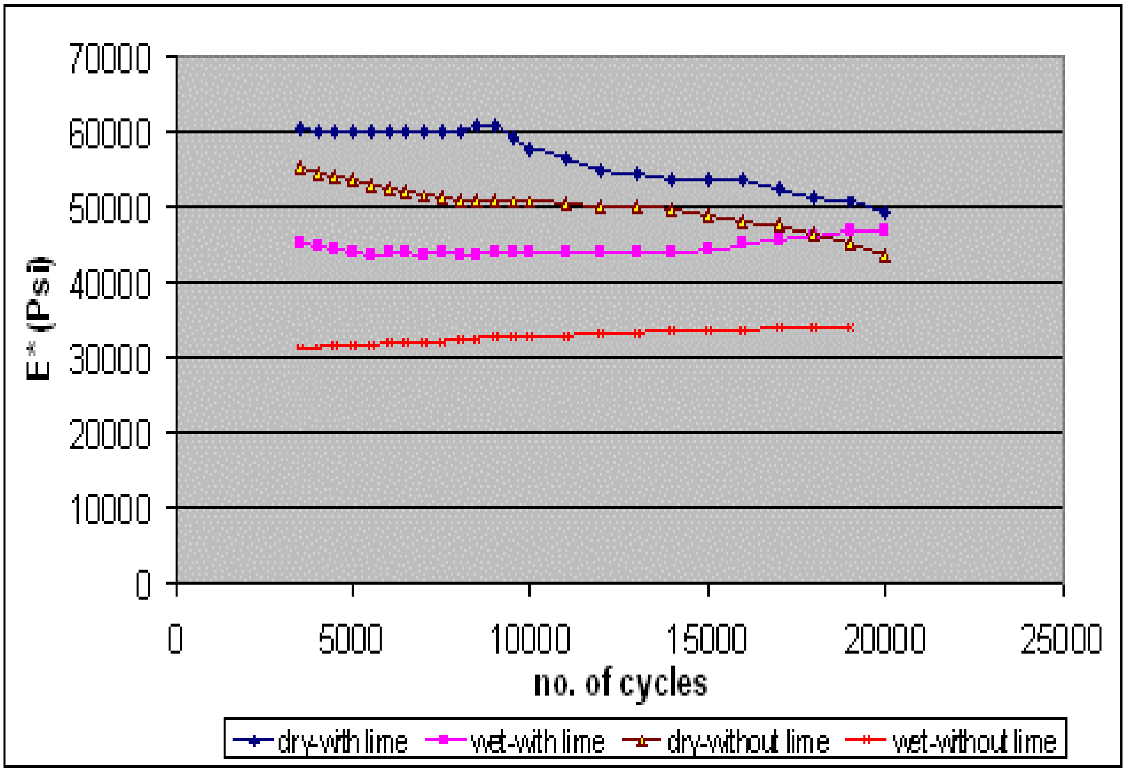


FIGURE-26 Dynamic modulus values of mixes with and without hydrated lime (Georgia Granite with AAD)

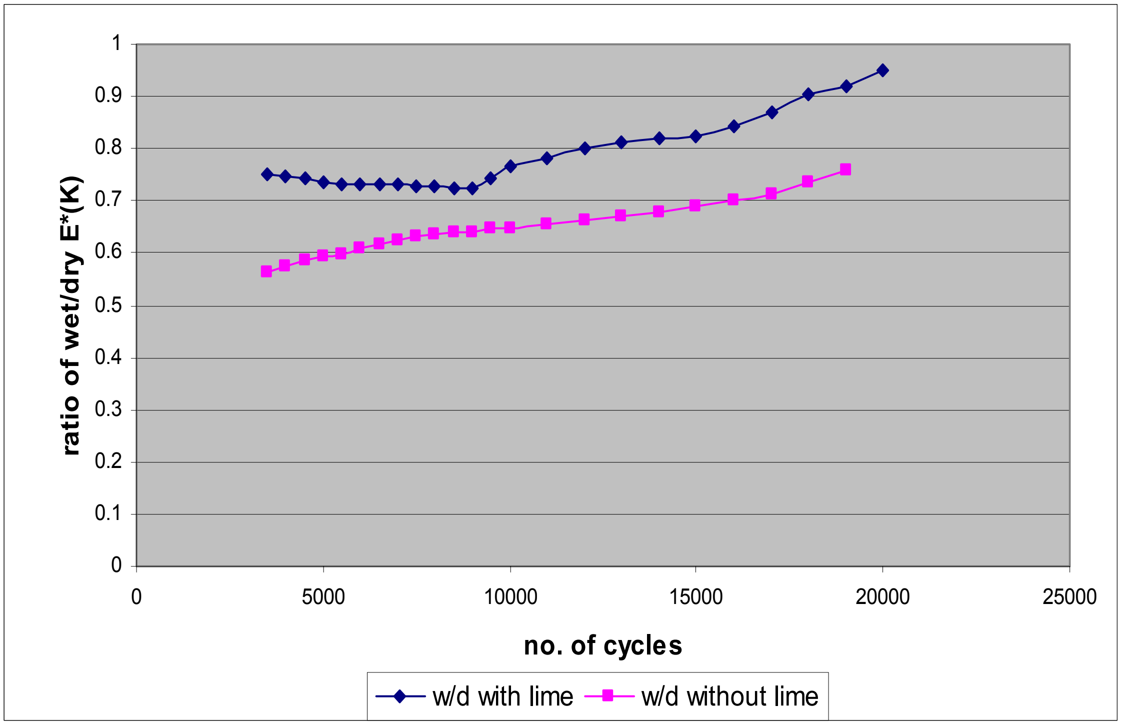


FIGURE-27 Ratio of wet/dry values of dynamic modulus of mixes with and without hydrated lime (Georgia Granite with AAD)

d) Georgia Granite with AAM

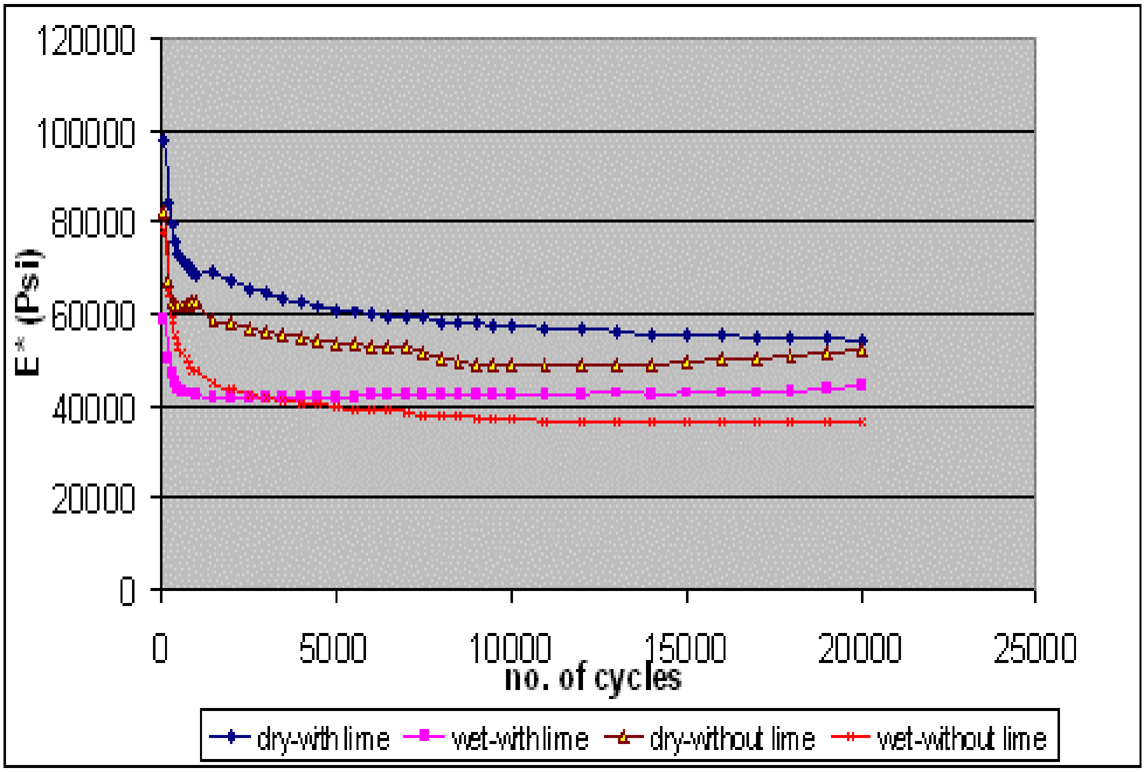


FIGURE-28 Dynamic modulus values of mixes with and without hydrated lime (Georgia Granite with AAM)

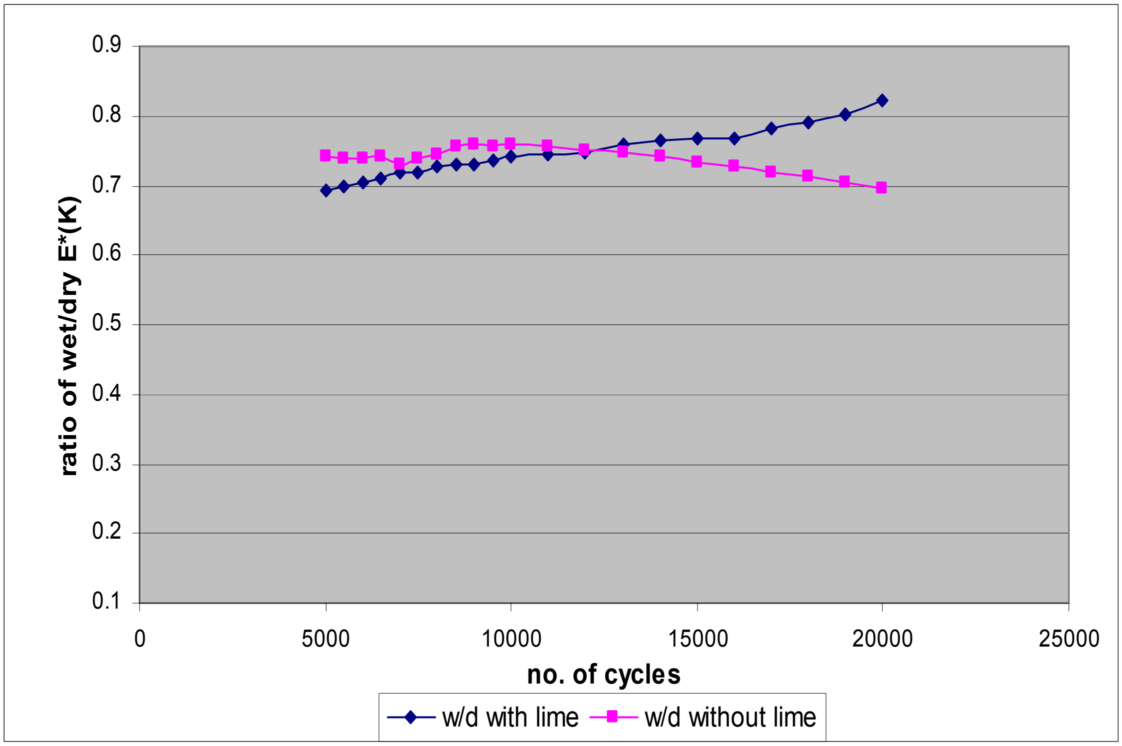


FIGURE-29 Ratio of wet/dry values of dynamic modulus of mixes with and without hydrated lime (Georgia Granite with AAM)

BAR-CHART COMPARISONS OF VARIOUS AGGREGATE MIXES

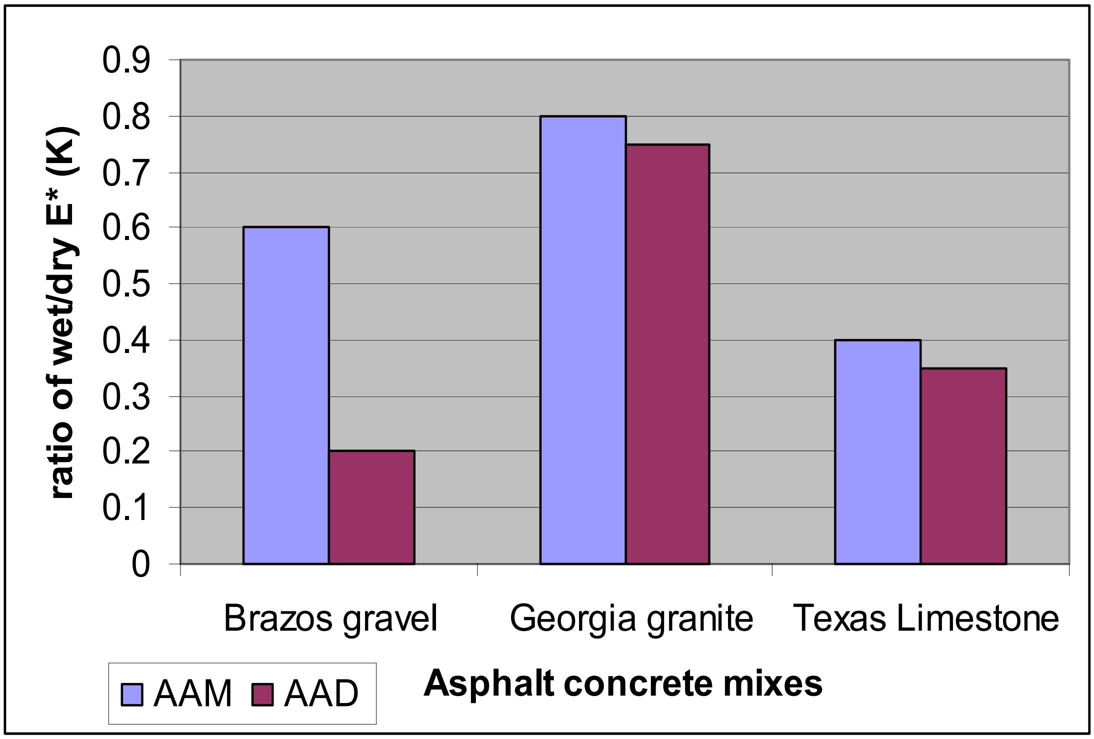


FIGURE-30 Comparison of E* ratios of the mixes with AAM and AAD (Bar chart)

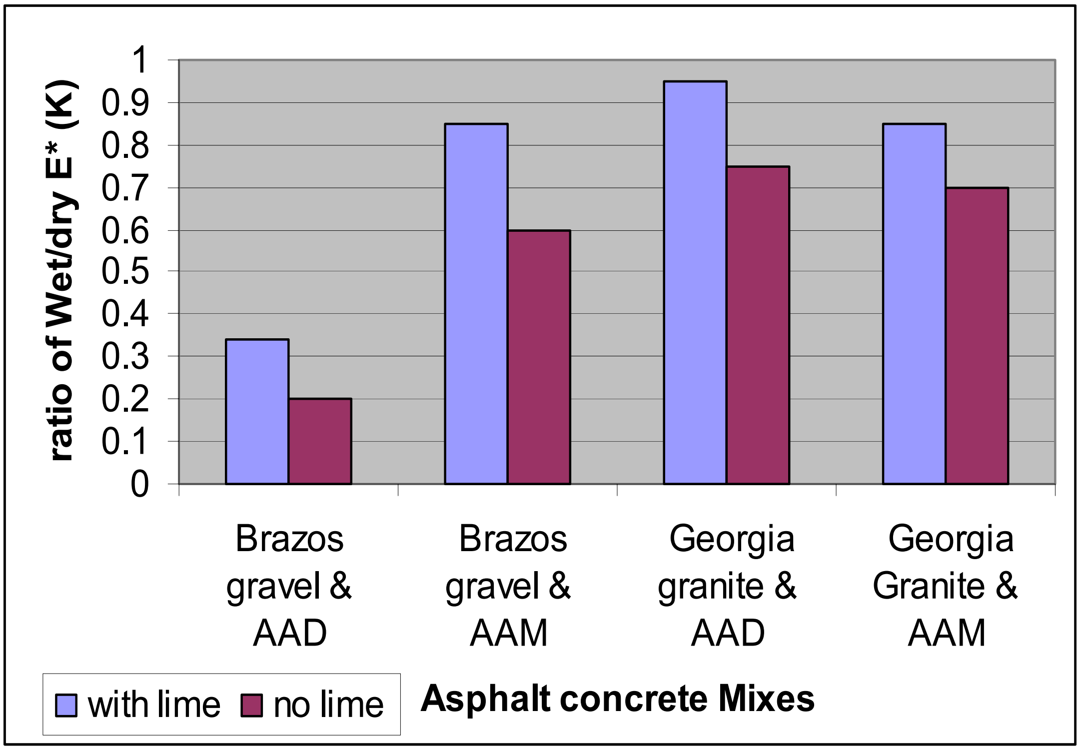


FIGURE-31 **Comparison of E* ratio of the mixes with and without hydrated lime (Bar chart)**

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

As shown before (figures 16-31), based on the results of compressive dynamic repeated loading the data was analyzed for three different aggregate types; Brazos gravel, Texas limestone, and Georgia granite respectively. Also two different binder types were considered; AAD-1, and AAM-1. Tests were performed by making two types of samples. One with adding hydrated lime as filler in it and other without adding lime. Samples for each type of mix were subjected to dry and wet testing under controlled temperature condition on MTS machine. Following observations were made out of the analyzed data:

- Asphalt concrete mix using AAM-1 asphalt showed less moisture induced damage compared to the one with AAD-1 in case of all the aggregate types. Similar kind of results is obtained from the values obtained by calculating combined surface free energies of asphalt concrete mix.

- Dynamic modulus values for mixes with AAM-1 asphalt were higher than the one with AAD-1.

- From the data obtained the percentage of aggregate surface exposed to water, P, is higher in case of wet tests compared to dry ones.

- Mixes with Georgia granite as aggregate and both the types of asphalts showed highest wet/dry ratio in terms of dynamic modulus compared to other two aggregate mixes. Even surface energy values exhibit similar result.

- Addition of hydrated lime to the asphalt concrete mixtures increased the dynamic modulus values for mixes and wet/dry ratio K was higher for such mixes compared to the one without hydrated lime. This shows that addition of lime as mineral filler reduced the moisture susceptibility of the mix and in a way reduced the moisture induced damage within the mix.

- Addition of lime in case of Brazos gravel showed a significant difference between two types of asphalts. While there was not a major difference in the results of Georgia granite with two different asphalts types.

RECOMMENDATIONS

This kind of result analysis indeed was able to validate many of the results obtained based on surface energy concept. But still in order to obtain more precision and better correlation on this topic certain other methods of analyzing data can be adopted.

- One such method of analyzing data is to measure relaxation modulus of the asphalt concrete samples in laboratory in undamaged condition before they are subjected to compressive loading as shown here in this test method.

- Using this relaxation modulus values and permanent microstrain values obtained from compressive loading, one can calculate dissipated pseudo-strain energy of a mix (10, 11). Pseudo-strain energy is the energy dissipated by the mix when it is subjected to loading and it transits from undamaged to damaged phase.

- Once these pseudo-strain energy values are known for various mixes, they can be compared with the surface energy values obtained by calculation as shown before in the report.

This approach seems to be more precise and promising in terms of providing a correlation between surface energy values and laboratory testing.

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APPENDIX I

1) Comparison of dry and wet test results of the mixes without hydrated lime

a) Brazos gravel with AAD

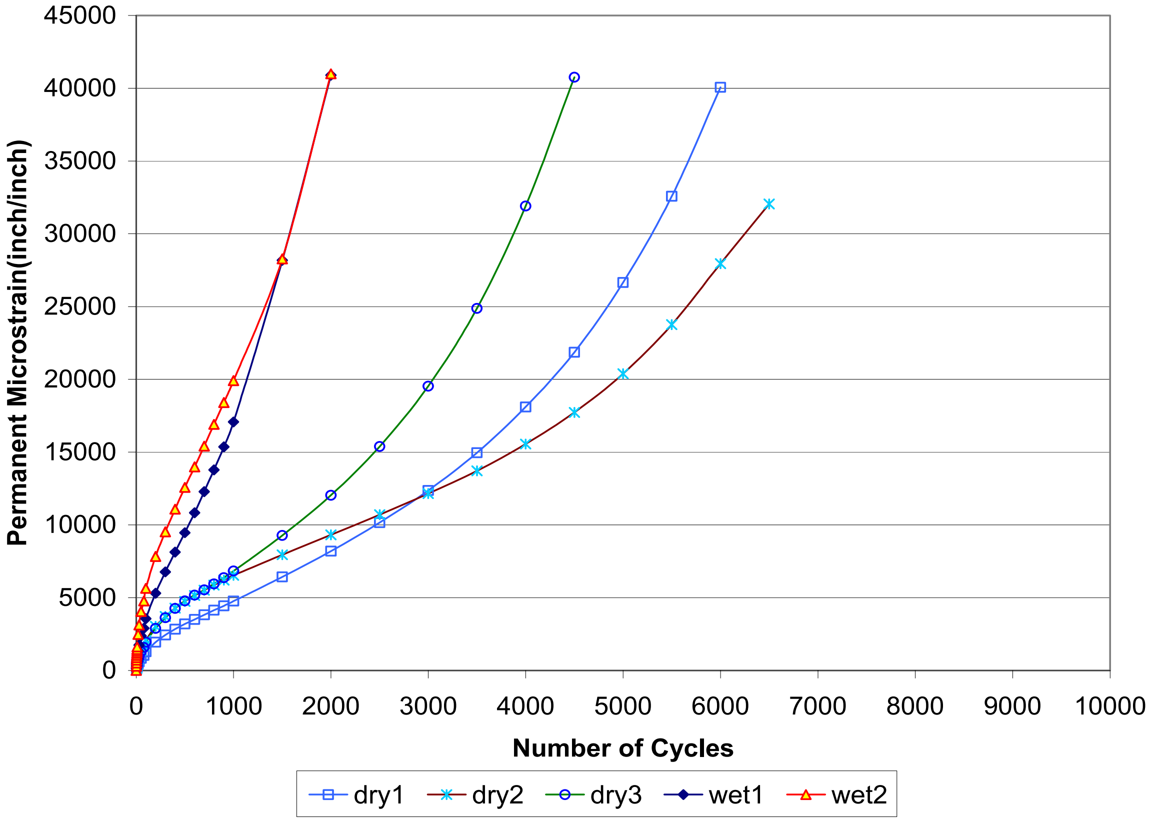

FIGURE-32 A test data showing strain values for Brazos gravel with AAD

b) Brazos gravel with AAM

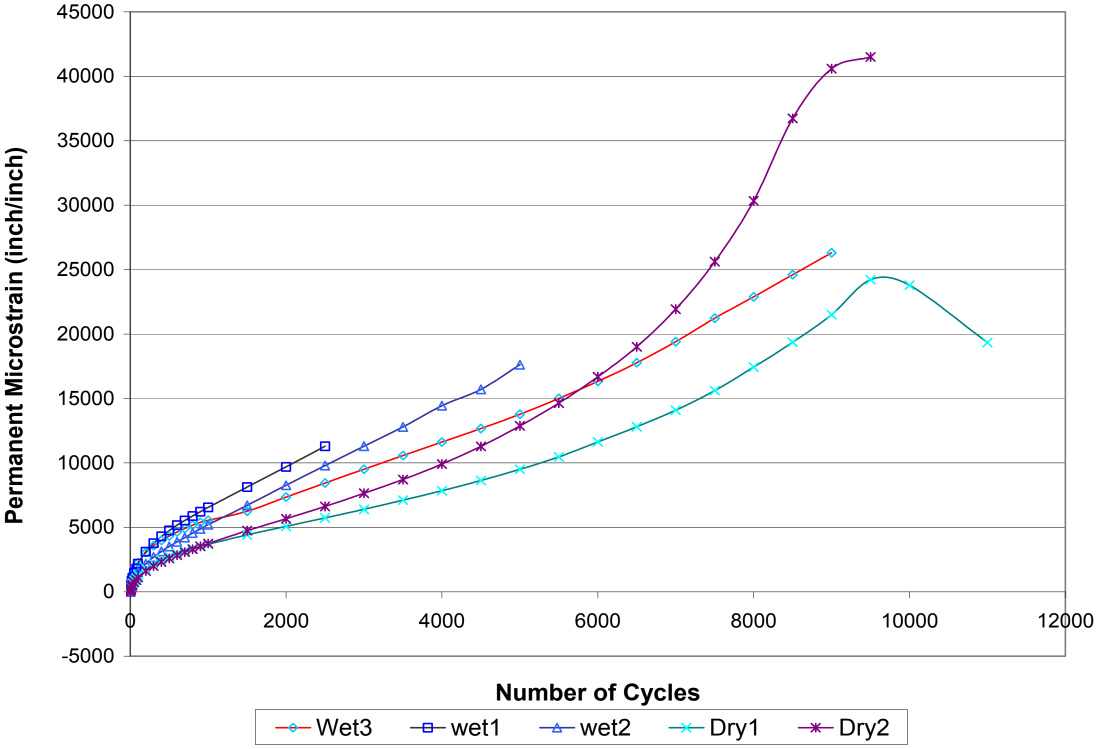

FIGURE-33 A test data showing strain values for Brazos gravel with AAM

c) Georgia Granite with AAD

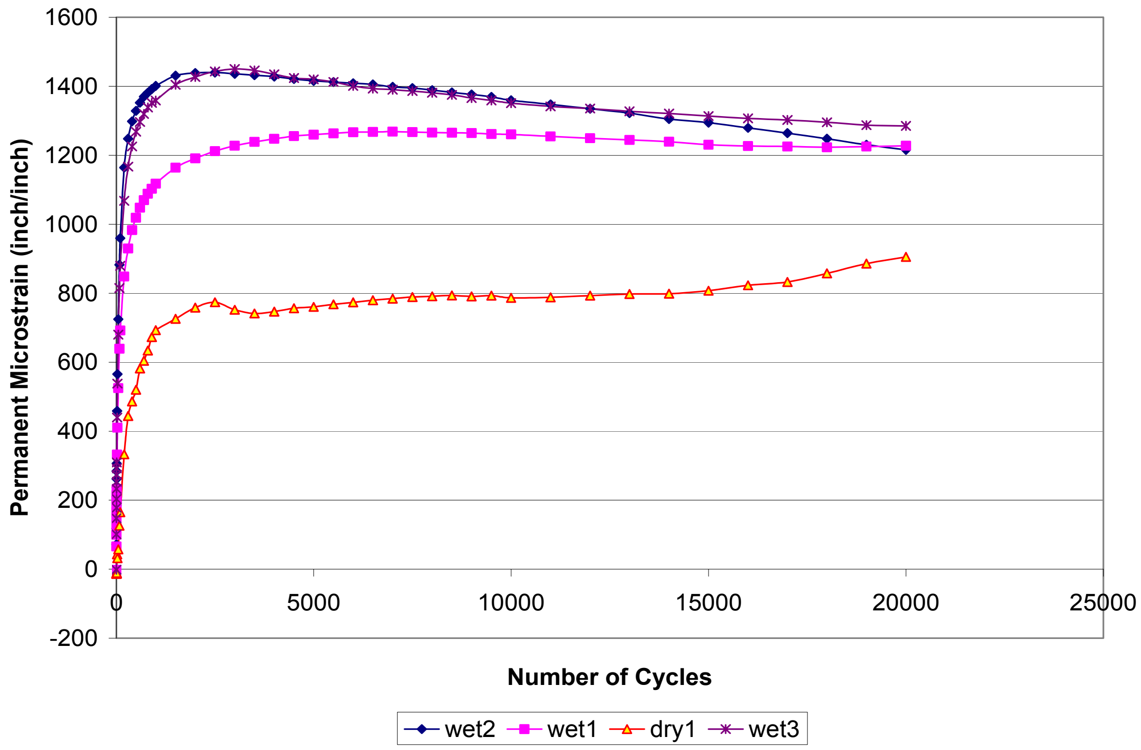

FIGURE-34 A test data showing strain values for Georgia granite with AAD

d) Georgia granite with AAM

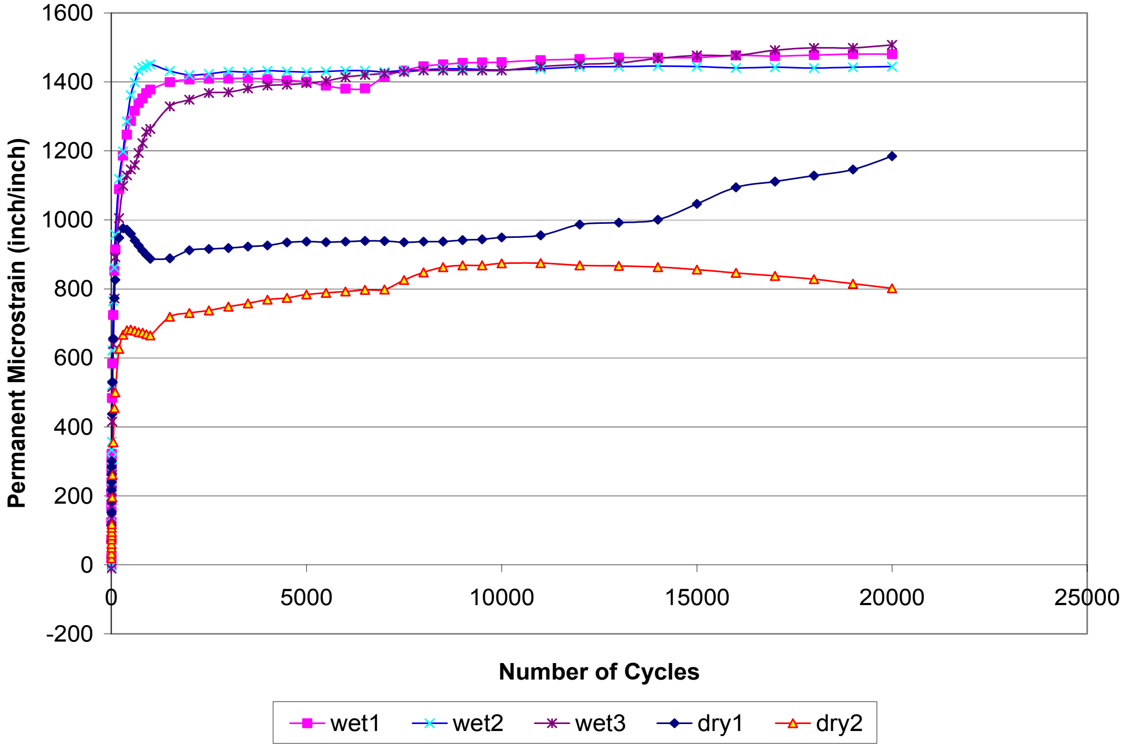

FIGURE-35 A test data showing strain values for Georgia granite with AAM

e) Texas Limestone with AAD

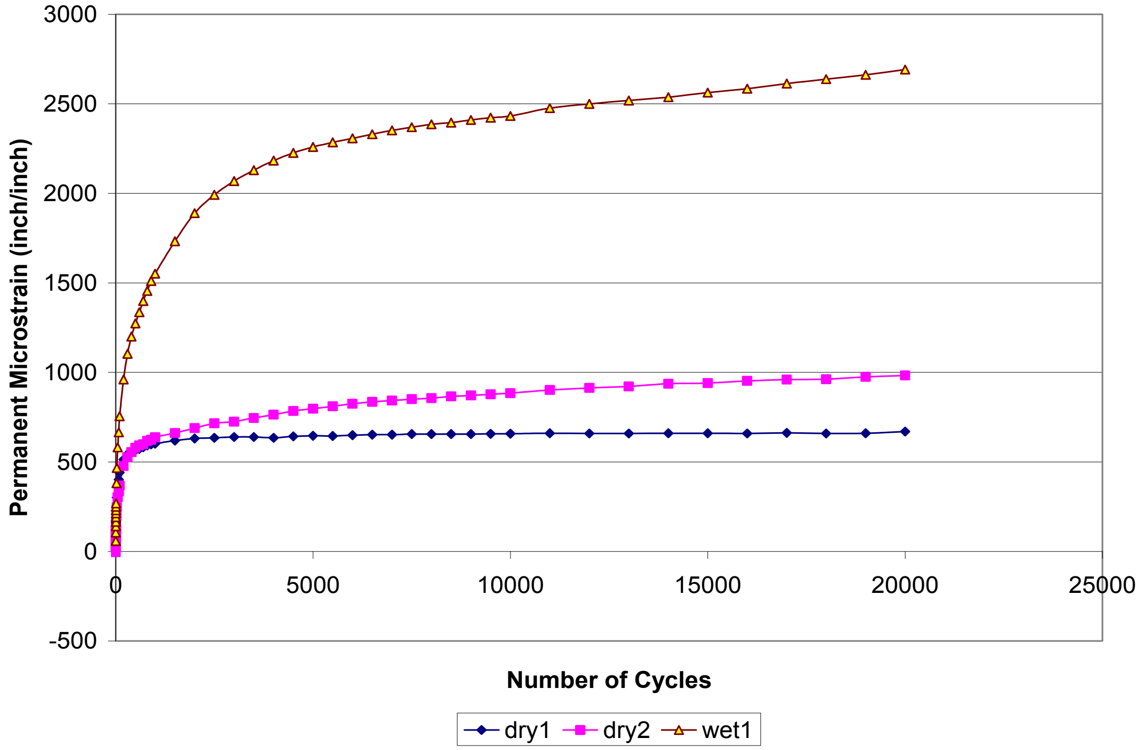

FIGURE-36 A test data showing strain values for Texas Limestone with AAD

f) Texas Limestone with AAM

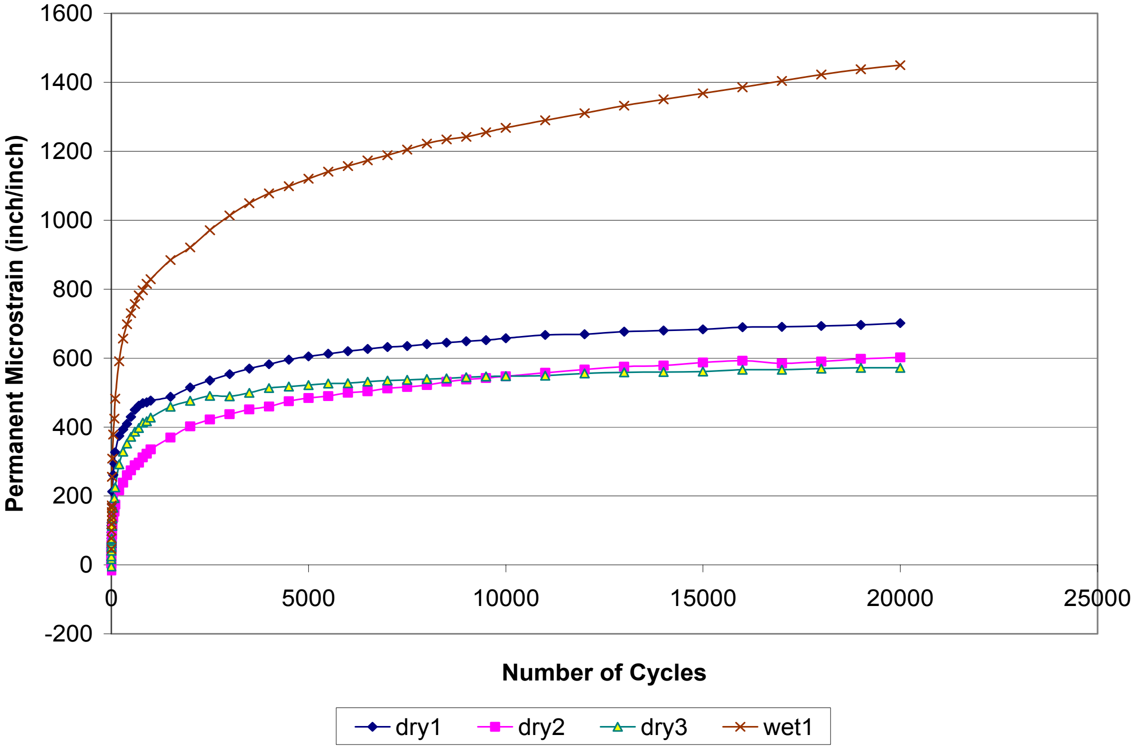

FIGURE-37 A test result showing strain values for Texas Limestone with AAM

APPENDIX II

1) Comparison of dry and wet test results of mixes with and without hydrated lime

a) Brazos gravel with AAD

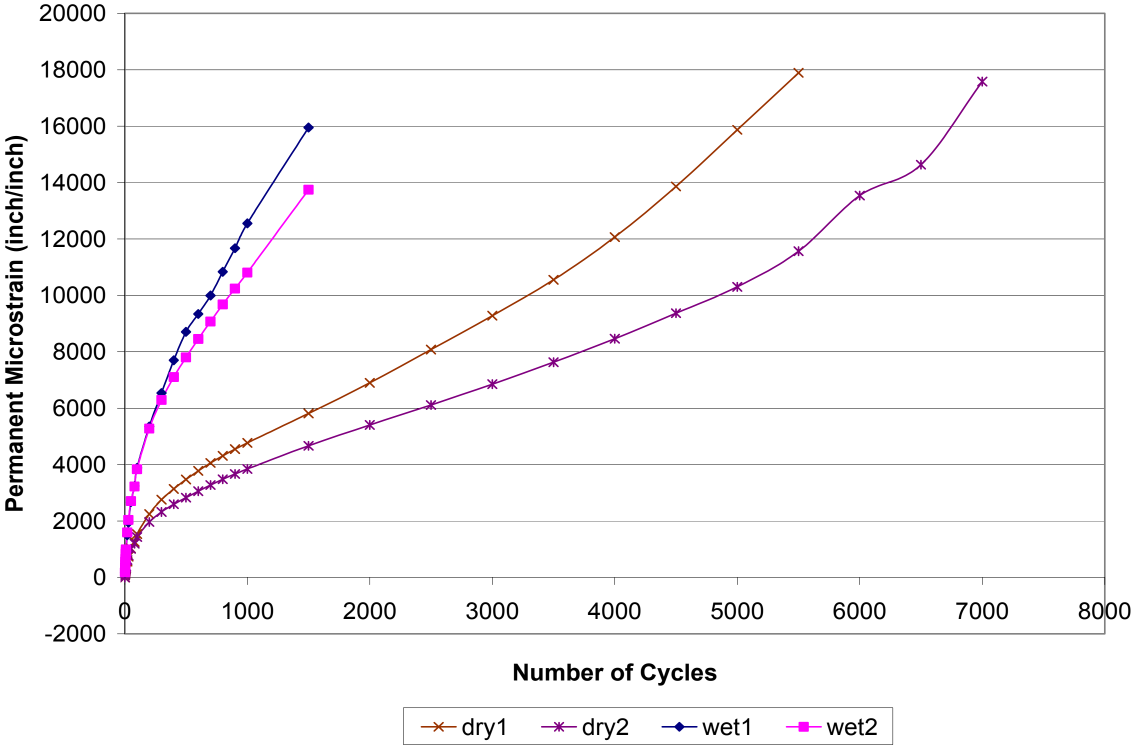

FIGURE-38 A test data showing strain values for Brazos gravel with AAD (With hydrated lime)

b) Brazos gravel with AAM

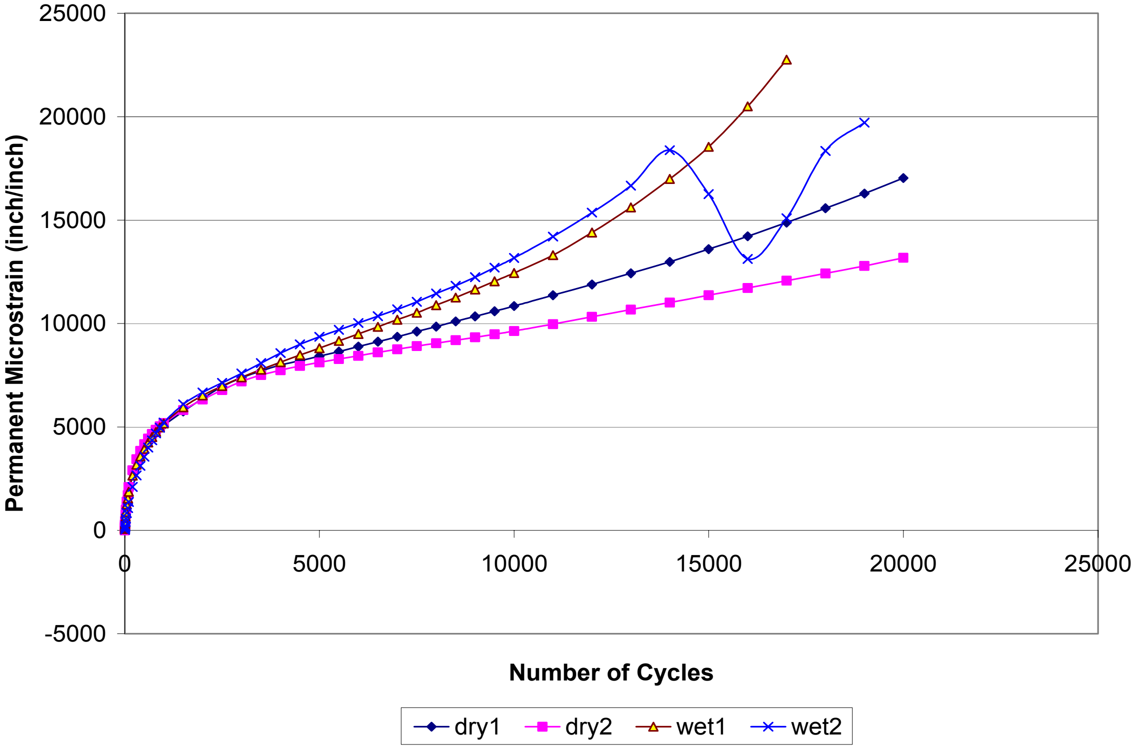

FIGURE-39 A test data showing strain values for Brazos gravel with AAM (With hydrated lime)

c) Georgia Granite with AAD

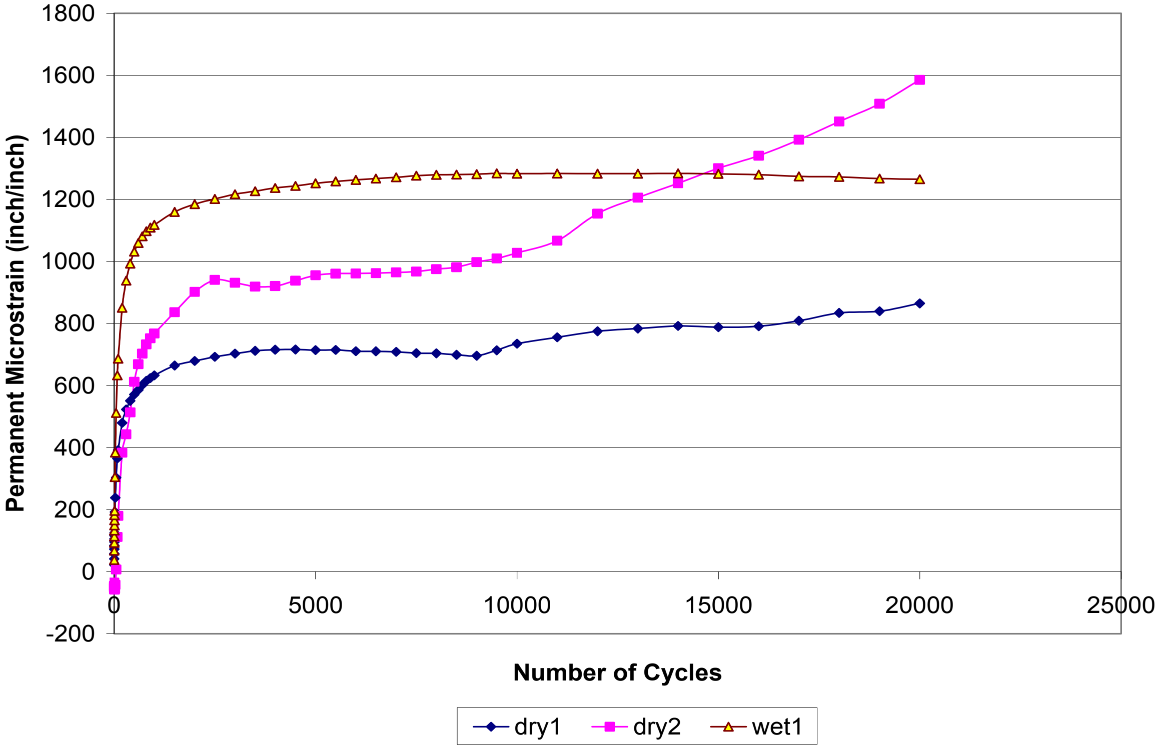

FIGURE-40 A test data showing strain values for Georgia granite with AAD (With hydrated lime)

d) Georgia Granite with AAM

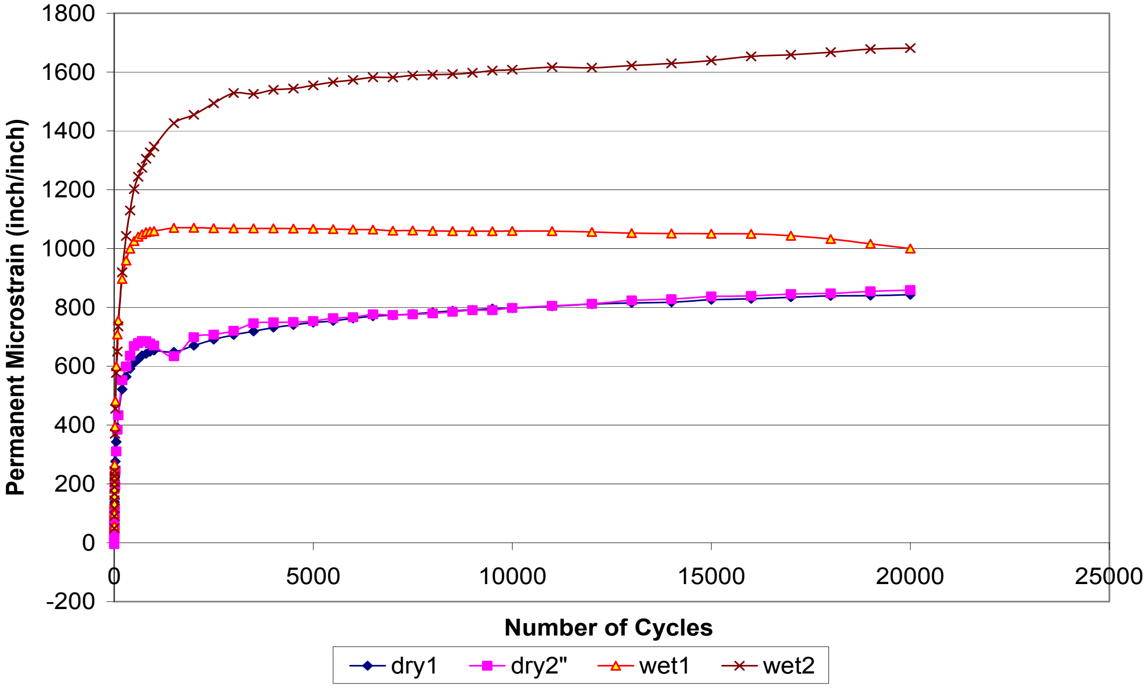

FIGURE-41 **A test data showing strain values for Georgia granite with AAM (With hydrated lime)**

VITA

Brij Shah was born at Mumbai, India in 1980. He grew up in Ahmedabad, India where he graduated from Gujarat Higher Secondary Board in May 1997. He received his Bachelor of Engineering degree in Civil Engineering from Gujarat University in June 2001. In the fall of 2001, he started his master's degree at Texas A&M University, and he graduated in August 2003 with a Master of Science in Civil Engineering. He intends to gain some work experience at any professional Civil Engineering firm within U.S. He can be contacted at following address in India:

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