

A NEW TESTING PROTOCOL FOR SEAL COAT (CHIP SEAL)

AGGREGATE-BINDER COMPATIBILITY

by

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TABLE OF CONTENTS

ACKNOWLEDGEMENTS	ii
LIST OF TABLES	vii
LIST OF FIGURES	ix
CHAPTER	
I. INTRODUCTION	1
1.1 Importance of Seal Coats	1
1.1.1 Types of Pavement Maintenance	1
1.1.2 The Concept of Preventive Maintenance	2
1.1.3 Comparison of Seal Coat with Other Preventive Maintenance Techniques	3
1.2 Functions of a Seal Coat	5
1.3 Statement of the Problem	7
1.3.1 Aggregate-Binder Bond	8
1.4 Scope of the Research	11
II. LITERATURE REVIEW	13
2.1 Aggregate	13
2.1.1 General Properties	13
2.1.2 Surface Chemistry	16
2.2 Precoated Aggregate	21
2.3 Hot Applied Asphalt	24
2.3.1 Rheological Properties	25

2.3.2 Polymer Modified Binders	27
2.4 Asphalt Emulsions	28
2.4.1 Factors Affecting the Characteristics of Emulsion	30
2.4.2 Breaking and Curing	31
2.5 Rubberized Asphalt	38
2.6 Construction Factors	42
2.7 Stripping	44
2.7.1 Mechanism of Stripping	45
2.8 Antistripping Additives	50
2.9 Test Methods	52
2.9.1 Moisture Sensitivity Tests	52
2.9.2 Aggregate Retention Tests	63
III. CONSTRUCTABILITY REVIEW	68
3.1 Construction Site Visits	68
3.1.1 Data Collection	68
3.1.2 Sample and Material Collection	71
3.2 Follow-up Monitoring Study	73
3.3 Conclusions	78
IV. DEVELOPMENT OF NEW TESTING PROTOCOL	80
4.1 Specimen Preparation	80
4.2 Impact Testing	82
4.3 Specimen Conditioning	84

4.4 Effect of Freeze-Soak/Thaw	86
4.5 Sensitivity of the Protocol to Aggregate Type	88
4.6 Verification of Testing Protocol	89
4.7 Conclusions	92
V. LABORATORY TESTING	94
5.1 Specimen Preparation	94
5.2 Hot Applied Asphalt	97
5.2.1 Embedment Depth	97
5.2.2 Dust Content	102
5.2.3 Surface Temperature and Time Elapsed between Aggregate Spreading and Rolling	104
5.2.4 Results	104
5.3 Asphalt Emulsions	111
5.4 Precoated Aggregate	113
5.5 Suggested Evaluation Criteria for the Testing Protocol	116
5.6 Conclusions	117
VI. INTERFACIAL BONDING (PULL-OUT) TEST	120
6.1 Test Procedure and Experimental Plan	120
6.2 Test Results	121
6.2.1 Hot Applied Asphalt	121
6.2.2 Asphalt Emulsions	124
6.3 Conclusions	125
VII. CONCLUSIONS AND RECOMMENDATIONS	128

REFERENCES	134
APPENDIX	
A. DATA COLLECTED FROM THE CONSTRUCTABILITY REVIEW	138
B. STRESS-DISPLACEMENT CURVES OF INTERFACIAL BONDING TEST	144
C. DIMENSIONS AND SPECIFIC GRAVITIES OF THE AGGREGATES USED IN INTERFACIAL BONDING TEST	161

LIST OF TABLES

1.1	Typical Unit Costs and Expected Life of Typical Pavement Maintenance Treatments	4
1.2	Example of Cost-Effectiveness of Various Pavement Maintenance Treatments	5
2.1	Aggregate Gradation used in NRA Net Adsorption Test	61
2.2	Criteria Suggested (SHRP) for Aggregate-Binder Adhesion Performance	62
3.1	Results from Field Monitoring Follow-Up Study	77
4.1	Effect of Soaking Duration	85
4.2	Effect of Freeze-Thaw Cycles and Soaking Duration	87
4.3	Sensitivity of Testing Protocol to Aggregate Type	89
4.4	Performance of Field Specimens under Proposed Test Protocol	90
5.1	Percent Loss for AC-15 5TR + Limestone	107
5.2	Percent Loss for AC-5 w/2% Latex + Siliceous Gravel	110
5.3	Percent Loss for CRS-2P + Siliceous Gravel, Limestone, and Lightweight	112
5.4	Percent Loss for Precoated Aggregate Specimens	115
5.5	The Occurrence of Data for the Corresponding Ranges based on Percent Loss for AC-15 5TR + Limestone	117
5.6	Suggested Evaluation Criteria	117
6.1	Peak Stress Pull-out for AC-5 w/2% Latex + Rhyolite Gravel	122
6.2	Interfacial Bonding Test Peak Stress Results for AC-15 5TR + Limestone	124
6.3	Interfacial Bonding Test Peak Stress Results for CRS-2P with Limestone and Siliceous Gravel	126

A-1	Data Collected from the Constructability Review	139
C-1	Dimensions of the Aggregates used in IBT	162
C-2	Specific Gravities of the Aggregates	163

LIST OF FIGURES

1.1	Typical Pavement Life Cycle	3
3.1	Districts Visited	69
3.2	Data Collection Sheet for Construction Site Visits	70
3.3	Collection of Field Specimen at Test Projects	72
3.4	Typical Seal Coat Specimen collected from the field	73
3.5	An Illustration of Aggregate-Binder Incompatibility	76
4.1	Specimens being compacted by the roller	81
4.2	Impact Test Apparatus	83
5.1	Specimen Preparation Set up	95
5.2	Hot Plate	96
5.3	Factors Affecting Embedment Depth	99
5.4	Amount of Dust generated and retained with Micro Deval Test for Limestone	103
5.5	Percent Loss due to Dust Content and Percent Embedment Depth at 70° F and 3 minutes roller time delay for AC-15 5TR + Limestone	107
5.6	Percent Loss due to Time Elapsed between Aggregate Application and Rolling and Embedment Depth at 140° F and 2 min MD for AC-15 5TR + Limestone	108
5.7	Percent Loss due to Surface Temperature and Embedment Depth for 2 min MD and 3 minutes roller time delay for AC-15 5TR + Limestone	109
5.8	Percent Loss due to Surface Temperature and Embedment Depth with 3 minutes roller time delay for AC-5 w/2% Latex + Siliceous Gravel	110
6.1	Peak Pull-out Stress versus Dust Content and Temperature of Binder at the time of Aggregate Application for AC-5 w/2% Latex + Rhyolite Gravel	123

B-1	Stress-Displacement Curve of Rhyolite + AC-5 @ 100° F (Fully Embedded)	145
B-2	Stress Displacement Curve of Rhyolite + AC-5 @ 100° F (0 Embedment)	145
B-3	Stress Displacement Curve of Rhyolite + AC-5 @ 140° F (Fully Embedded)	146
B-4	Stress Displacement Curve of Rhyolite + AC-5 @ 140° F (0 Embedment)	146
B-5	Stress-Displacement Curve of Rhyolite + AC-5 @ 140° F (Fully Embedded, Dusty)	147
B-6	Stress-Displacement Curve of Rhyolite + AC-5 @ 100° F (Fully Embedded, Dusty)	147
B-7	Stress-Displacement Curve of Rhyolite + AC-5 @ 140° F (0 Embedment, Dusty)	148
B-8	Stress-Displacement Curve of Rhyolite + AC-5 @ 100° F (0 Embedment, Dusty)	148
B-9	Stress-Displacement Curve of Rhyolite + AC-5 @ 140° F (0 Embedment, Dusty)	149
B-10	Stress-Displacement Curve of Limestone + AC-15 @ 100° F (Fully Embedded)	149
B-11	Stress-Displacement Curve of Limestone + AC-15 @ 140° F (Fully Embedded)	150
B-12	Stress-Displacement Curve of Limestone + AC-15 @ 140° F (Fully Embedded, Dusty)	150
B-13	Stress-Displacement Curve of Limestone + AC-15 @ 100° F (Fully Embedded, Dusty)	151
B-14	Stress-Displacement Curve of Limestone + AC-15 @ 140° F (0 Embedment)	151
B-15	Stress-Displacement Curve of Limestone + AC-15 @ 100° F (0 Embedment)	152
B-16	Stress-Displacement Curve of Limestone + CRS-2P (3 Hours Curing + Air Dry + Before Break)	152

B-17 Stress-Displacement Curve of Limestone + CRS-2P (10 Hours Curing + Air Dry + Before Break)	153
B-18 Stress-Displacement Curve of Limestone + CRS-2P (3 Hours Curing + Air Dry + After Break)	153
B-19 Stress-Displacement Curve of Limestone + CRS-2P (10 Hours Curing + Air Dry + After Break)	154
B-20 Stress-Displacement Curve of Limestone + CRS-2P (3 Hours Curing +SSD + Before Break)	154
B-21 Stress-Displacement Curve of Limestone + CRS-2P (3 Hours Curing + SSD + After Break)	155
B-22 Stress-Displacement Curve of Limestone + CRS-2P (10 Hours Curing + SSD + Before Break)	155
B-23 Stress-Displacement Curve of Limestone + CRS-2P (10 Hours Curing + SSD + After Break)	156
B-24 Stress-Displacement Curve of Fordyce + CRS-2P (3 Hours Curing + SSD + Before Break)	156
B-25 Stress-Displacement Curve of Fordyce + CRS-2P (10 Hours Curing + Air Dry + Before Break)	157
B-26 Stress-Displacement Curve of Fordyce + CRS-2P (3 Hours Curing + Air Dry + After Break)	157
B-27 Stress-Displacement Curve of Fordyce + CRS-2P (10 Hours Curing + Air Dry + After Break)	158
B-28 Stress-Displacement Curve of Fordyce + CRS-2P (3 Hours Curing + SSD + Before Break)	158
B-29 Stress-Displacement Curve of Fordyce + CRS-2P (10 Hours Curing + SSD + Before Break)	159
B-30 Stress-Displacement Curve of Fordyce + CRS-2P (3 Hours Curing, SSD + After Break)	159
B-31 Stress-Displacement Curve of Fordyce + CRS-2P (10 Hours Curing + SSD + After Break)	160

CHAPTER I

INTRODUCTION

A seal coat, also known as a chip seal or a surface dressing, is a thin surface treatment that consists of a single application of binder and a single layer of uniform size aggregate. Seal coats are constructed by spraying the bituminous material with a distributor truck followed by the chip spreader that spreads a single layer of aggregate to form a sealing surface. After application of aggregate, rollers are used to embed chips into the binder. The thickness of the seal coat layer is governed by the nominal maximum size of the aggregate used.

1.1 Importance of Seal Coats

1.1.1 Types of Pavement Maintenance

Generally, three types of pavement maintenance are recognized: preventive, corrective (or reactive) and emergency maintenance. Preventive maintenance (PM) is used to arrest minor deterioration, retard progressive failures, and reduce the need for corrective maintenance. Corrective maintenance is performed after one, or a combination of, deficiencies such as moderate rutting, lack of skid resistance or cracking occur in the pavement. Emergency maintenance is applied when the pavement is no longer able to perform its intended function. The same treatment may be used for each maintenance type. A seal coat can be applied to a pavement in relatively good condition to extend its service life by sealing the surface. Such a seal reduces the oxidation of asphaltic material

on the existing surface and also seals the cracks to prevent moisture migration to the pavement foundation. This type of treatment is classified as PM. Same treatment can also be used to a moderately distressed pavement, but since the pavement is damaged the full benefits of the treatment is not realized (corrective maintenance). Finally under emergency conditions, seal coats may only provide a temporary improvement (emergency maintenance). The differences in all these maintenance levels are the pavement conditions, not the treatment type (Zaniewski and Mamlouk, 1999).

1.1.2 The Concept of Preventive Maintenance

After the conceptual frame work for a Pavement Management Systems has been established, highway agencies tend to use PM in the early stages of the pavement in order to extend the service life. AASHTO (1999) defines PM as, "... the planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional conditions of the system without increasing structural capacity" (p. 2). Therefore PM often involves early application of maintenance treatments even when the pavement is in good structural condition. Commonly used PM treatments include seal coats, crack seals, fog seal, thin hot-mix overlays, slurry seals and microsurfacing.

In Figure 1.1, the typical pavement life cycle is illustrated. It shows that speed of deterioration increases rapidly in the late life of the pavement. Moreover, the cost of a possible rehabilitation at the late stages is 3 to 4 times more expensive than a PM at the

early life of the pavement. Therefore, PM is a cost-effective tool to increase the life of the pavement provided it is used at the appropriate time.

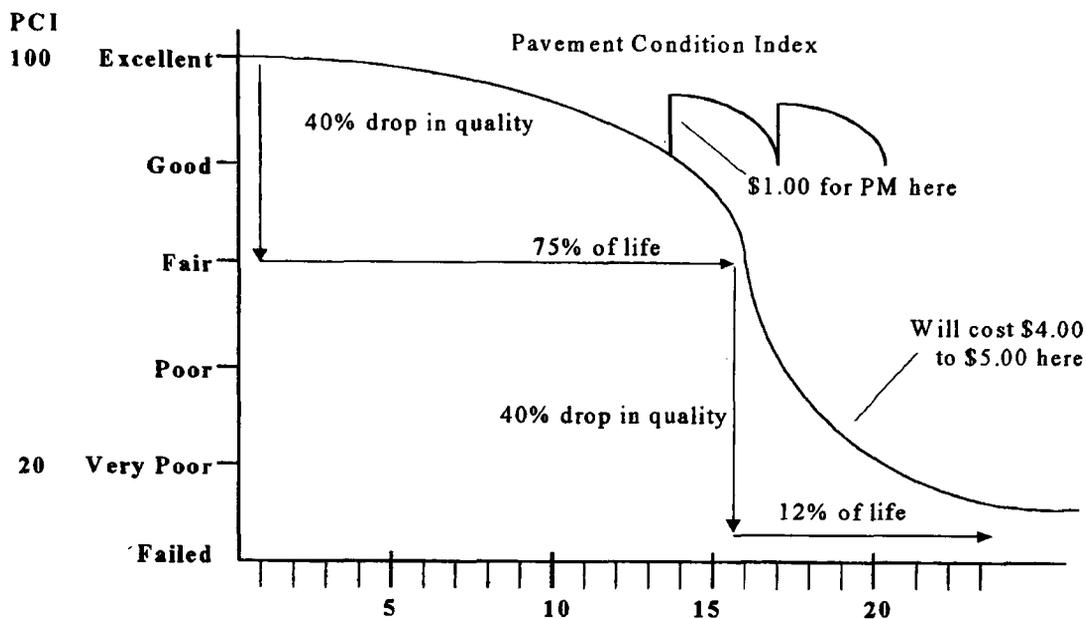


Figure 1.1: Typical Pavement Life Cycle

Source: Excerpted from Hicks et al., “Selecting Preventive Maintenance Treatment for Flexible Pavements,” 1999.

1.1.3 Comparison of Seal Coat with Other Preventive Maintenance Techniques

1.1.3.1 Typical Unit Costs and Expected life

Table 1.1 summarizes typical unit cost and expected life of different maintenance treatments. These values are based on empirical observations depending on local conditions such as project location, material availability and climate.

Table 1.1: Typical Unit Costs and Expected Life of Typical Pavement Maintenance Treatments

Treatment	Cost/m ² (\$)†	Cost/yd ² (\$)†	Expected Life
Crack Seals	3.29	1.00	2 to 3 years
Fog Seals ^a	0.54	0.45	3 to 4 years
Slurry Seals ^b	1.08	0.90	4 to 6 years
Microsurfacing ^c	1.50	1.25	5 to 7 years
Seal Coats ^d	1.02	0.85	4 to 6 years
Thin-Hot Mix Overlays ^e	2.09	1.75	2 to 10 years

Notes:

^a0.21/m² (0.05 g/yd²) of a 1:1 dilution of CSS emulsion and water

^b7 kg/m² of ISSA Type II slurry

^c14 kg/m² of ISSA Type II microsurfacing

^d15 kg/m²

^e30 to 44 mm/m²

†Cost per meter/linear foot

Source: Excerpted from Hicks et al., “Selecting Preventive Maintenance Treatment for Flexible Pavements,” 1999.

1.1.3.2 Cost-Effectiveness Evaluation

There are several approaches available to evaluate the effectiveness of different PM techniques. Hicks et al. (1999) used the Equivalent Annual Cost (EAC) method. The equation for EAC is given as:

$$\text{EAC} = \text{Unit Cost} / \text{Expected life of treatment in years.}$$

For instance, the EAC value for seal coat has been calculated as:

$$\text{EAC for seal coat} = \$0.85 / 5 = \$0.17 \text{ yd}^2/\text{year.}$$

Table 1.2: Example of Cost-Effectiveness of Various Pavement Maintenance Treatments

Treatment	Life of Treatment (years)	Equivalent Annual Cost (\$/yd ²)
Crack Seals	2.5	0.40
Fog Seals	3.5	0.13
Slurry Seals	5	0.18
Microsurfacing	6	0.21
Seal Coats	5	0.17
Thin-Hot Mix Overlays	6	0.29

Source: Excerpted from Hicks et al., “Selecting Preventive Maintenance Treatment for Flexible Pavements,” 1999.

Fog seal has the lowest EAC value followed by the seal coat, but these two treatments are virtually exclusive from each other in terms of reason for application. Fog seal is the application of diluted emulsion to the existing pavement surface to reduce aggregate loss in existing pavement. It helps to avoid asphalt oxidation, raveling, and is considered as a short term treatment. Also, its effectiveness as a PM treatment is often questioned. Crack seals and fog seals can be classified on the same categorical level while the rest of the treatments are often classified in another. Based on EAC evaluation, seal coats and fog seals are the most cost-effective treatments for each of these two categories.

1.2 Functions of a Seal Coat

Seal coats can provide several functions to a pavement. They are primarily applied as a maintenance treatment to existing pavements for the sole purpose of extending the useful life of the pavement structure. They do not provide additional

structural capacity to the pavement, and they are typically expected to provide useful service for about five to eight years. The service life of a seal coat varies, however, depending upon volume of traffic, weather, and numerous other variables” (p.)

The following are a list of surface deficiencies that can be corrected using seal coats:

1. Cracks: some cracks can be sealed, if they are not very wide. Hence a seal coat will effectively prevent water from seeping through narrow surface cracks and that ruins the pavement foundation layers.

2. Raveling (Shelling): aggregate particles in the old pavement surface often break due to oxidative hardening of asphalt. A seal coat will cement loose aggregate material in place and prevent additional raveling.

3. Bleeding: the binder of the existing seal coat pavement may rise to the surface resulting in bleeding. A pavement surface with bleeding appears as a black patch, and has no appreciable skid resistance. A seal coat will often cover these spots with fresh aggregate, thus rejuvenating the surface.

4. Lack of skid resistance: a seal coat will bring fresh, irregularly shaped aggregate particles into contact with vehicle tires to improve skid resistance. This is particularly important where bleeding has occurred or where the aggregate in the old pavement has worn smooth, exposing polished surfaces.

Seal coats are not intended to correct structural deficiencies in pavements. A list of pavement distresses that cannot be corrected using seal coats is given below.

1. Strengthen the existing pavement: a seal coat is only a thin surface treatment, and therefore cannot be expected to make a pavement any stronger than when it was constructed.

2. Increase the load-bearing capacity: the seal coat does not penetrate the surface of old pavement. It seals the surface and holds fresh aggregate in place.

3. Smooth out rough pavement: if the old pavement is rutted, shoved, or washboarded, a seal coat is not able to correct them effectively.

4. Bridge major cracks: cracks wider than about $\frac{1}{4}$ in will not be totally bridged by the sprayed binder. Large cracks must be repaired well before a seal coat is applied.

5. Eliminate the need for maintenance or reconstruction: if any existing pavement is not in an acceptable structural condition, applying a seal coat will be a waste of money. With the possible exception of skid resistance, a seal coat cannot improve an existing pavement. It can only help to prevent it from getting worse.

Having expressed the function of seal coats, it can be understood that seal coats are capable of solving some types of waterproofing, raveling, bleeding, and skid resistance problems. However, they are not convenient to address structural problems in the pavement.

1.3 Statement of the Problem

State agencies such as Texas Department of Transportation (TxDOT) are responsible for the effective management of large pavement networks. Therefore, preventive maintenance treatments such as seal coats (chip seals) are an effective

pavement management tool used to extend the useful life of pavement structures. Consequently, at any given moment, the majority of highway lane miles managed by TxDOT has seal coat as the wearing surface. Thus, good performance of seal coats is vital for TxDOT and other state transportation agencies to satisfy the needs of the traveling public by keeping the highway system in serviceable condition.

1.3.1 Aggregate-Binder Bond

The performance of seal coats mainly depends on the effectiveness of aggregate-binder bond. Unlike hot-mix asphalt, where the aggregate particles fully covered with binder, the seal coat system, which is one aggregate thick, has to count on the bonding established by a partial binder coating, which is approximately 30-40% of the aggregate surface area. Increasing the strength of this bond will result in better seal coats, decreasing the occurrence of problems observed in seal coats such as loss of aggregate (raveling). In addition, better aggregate-binder bond will enable designers to optimize the binder and aggregate quantities so that the use of excess binder can be avoided. This will save substantial money on binder costs and reduce the likelihood of bleeding/flushing problems.

1.3.1.1 Factors Affecting Aggregate-Binder Bond

The bond between aggregate and binder is a function of mechanical, chemical, electrostatic and adhesive bonding mechanisms. Both physical and chemical properties of the aggregate and binder play an important role in this regard. Aggregate

characteristics that influence bonding include porosity, surface texture, mineralogy and surface chemistry. Binder characteristics include chemical composition, surface tension, and viscosity at the time of aggregate application. In the case of emulsified asphalts, the type of emulsion (cationic/anionic) and its breaking and curing processes affect bonding.

Change of the active metal species at the surface of the aggregate is dependent upon electrokinetic properties. The adsorption of asphaltic components is promoted by the active sites on the surface. The covering of these active sites by nonpolar hydrocarbons would block their activity. For instance, natural dust on aggregate surfaces can change the chemistry of adhesion as well as result in weak bonding between the dust and the aggregate surface (Curtis et al., 1993). In addition, the dust generated by specific types of aggregates could act as a separate surfactant, preventing the aggregate from forming an effective bond with the binder. On the other hand, the moisture state of the aggregate can also affect the performance of bond. Since seal coat aggregate is stored in stockpiles at the construction site, they are exposed to ambient conditions and various weather conditions can cause different moisture states within the aggregate.

In addition to the varying states the seal coat aggregate may be in, different conditions that may exist at the construction site during material application and rolling makes the seal coat construction process play a very critical role in the development of aggregate-binder bond compared to hot-mix, which is prepared in plants under more controlled conditions. This often leads to the notion among practitioners that seal coating is more of an “art” that leaves the construction personnel and inspectors make critical decisions at the site.

The viscosity of asphaltic materials is a function of its temperature. It is known that when the binder is less viscous, it will wet the aggregate better and penetrate into the pores more effectively resulting in a better aggregate-binder bond. Moreover, the ambient temperature and the temperature of existing pavement surface affect the cooling rate of the binder. Therefore, the time lag between the application of binder and the aggregate spreading becomes a critical factor that determines aggregate-binder bonding effectiveness. As more time elapses, more heat will be lost from the binder and the resulting higher viscosity will cause a weaker bond between aggregate and binder.

Another construction-related factor is the time elapsed between spreading of aggregate and rolling. This is of particular significance when hot asphalts are being used. Rollers apply the energy needed to embed aggregates into the binder and it also seats each aggregate particle such that its center of gravity is at the lowest possible position, giving it more stability. The longer it takes for the rolling operation to begin, the stiffer the binder will get and as a result, the aggregate may end up having a lower embedment depth than the design value.

In summarizing the preceding discussion, the following facts can be identified as having a significant influence on aggregate-binder bond in seal coats.

- Aggregate dust
- Moisture state of aggregate
- Ambient and pavement surface temperature during construction
- Time elapsed between the spraying the binder and spreading the aggregate
- Time elapsed between the spreading of aggregate and rolling

- Condition of the existing pavement
- Weather conditions immediately after the construction of seal coat.

1.4 Scope of the Research

Highway agencies such as TxDOT are in the process of implementing performance-based materials specifications. These specifications are developed based on test methods that better simulate conditions of the materials being exposed, during construction and service. Due to the many factors affecting performance of seal coats, a performance-based test appears to be the most appropriate way to evaluate the compatibility of seal coat material combinations. TxDOT uses various tests to determine the suitability of seal coat aggregate and binder separately. On the issue of compatibility, the agency has a test method (Tex-216-F) to determine aggregate retention of seal coats. This procedure is not currently used and one reason could be its inability to simulate the field conditions in the lab environment. Therefore, a need exists for a performance based testing protocol that simulates the field conditions in the lab, has to be developed in order to assess seal coat aggregate binder compatibility. In addition, this new test method has to identify the appropriateness of the material combinations and also estimate design parameter values.

This thesis is an integral part of the research project entitled *Developing a Testing and Evaluation Protocol to Assess Seal Coat Aggregate-Binder Compatibility* sponsored by TxDOT. The goal of this research project was to develop a new test method and an evaluation protocol to identify compatibility problems between the aggregate and the

binder in seal coats. At the end of this research, the following three products will be developed for implementation.

1. Draft test procedure for aggregate-binder compatibility and acceptance criteria;
2. Draft specification for seal coats and surface treatments including new acceptance criteria;
3. Evaluation protocol for seal coat aggregate and binder selection.

This thesis is organized into 7 chapters. This introductory first chapter is followed by a comprehensive literature review in Chapter II. In Chapter III, the constructability review and field studies including performance monitoring of field test sections is presented. The work leading to the development of new testing protocol is presented in Chapter IV. The new testing protocol was then used to conduct the test program involving many experimental factors. The test program and its results are discussed in Chapter V. In addition, the investigation of aggregate-binder bonding has been done in laboratory. The findings of this work are given in Chapter VI. Finally, the conclusions and recommendations are presented in Chapter VII.

CHAPTER II

LITERATURE REVIEW

2.1 Aggregate

Aggregates provide the skid resistance in a seal coat, while the bitumen seals the pavement surface and holds the aggregate in place. Aggregates have a major influence on the success or the failure of a seal coat. Both physical and mechanical properties such as surface texture, porosity, surface chemistry, mineralogy, and surface charges of the aggregate influence the aggregate-binder bonding.

2.1.1 General Properties

A study conducted by Nevitt (1951) outlined the desirable aggregate properties for successful seal coating. For seal coat aggregates, irregular shape is preferred but not essential. The aggregates that have cubical shape is the most desirable ones and the aggregate particles should be neither flaky nor splintered. Rounded particles are not appropriate for seal coats, primarily because of texture rather than shape. Crushed aggregates generally give better results due to better skid resistance but roundness is acceptable if the aggregate has other desirable characteristics. The sharp corners and edges imparted by crushing can improve the gripping action by their tendency to dig into the tire, but this effect does not last long enough to be critical.

Surface texture is the most important single property of aggregates that influence bonding. A surface that exposes a continuous layer of particles of a rough, gritty texture

free of bitumen and resistant to wear is a good one. Not all aggregates improve in surface texture by crushing. The majority of aggregates do improve in both surface texture and shape characteristics from the operation, and crushed aggregates are therefore commonly and desirably used for seal coating; but their selection should be on the basis of the properties imparted, rather than because they are crushed. Seal coat aggregate must have desirable skid resistant characteristics. They should be either resistant to wear, or the exposed surface not be smooth (Nevitt, 1951).

Chemical characteristics of aggregate also have an influence on successful seal coating. Since seal coat aggregates get maximum exposure to extreme climatic elements and are not completely covered with a protective bituminous film, any tendency to break down through leaching, slacking, or other effects of air, water and traffic exposure must not be present (Nevitt, 1951).

The aggregates should not be excessively hydrophilic. The high tendency of such aggregate to attract water would result in the loss of asphalt coating (stripping) resulting in loss of aggregate (raveling). The aggregate should have good weight bearing qualities. A surface treatment course is expected to bear the greatest share of the traffic loads and to transmit it evenly to the rest of the pavement. This dictates an aggregate of high bearing strength and resilience. Generally, the greater weight per unit volume, the higher is the bearing strength. Resistance to abrasion is a must. As a matter of technique this surface or aggregate must be embedded into binder course to the maximum depth. The traveling surface left exposed, not being very deep, must be resistant to wear as its powdering away would of course result in its final submergence and loss. The aggregate should be

durable and give high resistance to the erosive and corrosive action of elements. The first geometric characteristic desired in mineral aggregate is uniformity in size. The submergence of any considerable fraction of the cover aggregate not only results in a fat condition on the surface; it also brings a keying action within the bonding layer, which will rob the surface course of flexibility. The brittleness will take place within the course, which will cause the surface to fail under adverse conditions. Any considerable portion of dust will destroy the bonding action of the protective film for the aggregate with the obvious result of cover loss and weak bonding. The second desired geometric characteristic is the aggregate shape. The more nearly spherical any particle is, the less surface area it presents for any given volume. Certainly the greater surface in contact with the bonding layer, the more effective grip between the binder and the aggregate will be obtained (Herbert, 1955).

According to Aggregate handbook, the aggregates for surface treatments should meet the following requirements:

Surface treatment cover aggregate is nominally a one size material. If a gradation is used having different aggregate sizes, smaller aggregate sizes can be embedded in the binder first, keeping the coarse aggregate from being embedded. The use of large maximum size aggregate for a surface treatment creates substantial tire noise, which affects the ride quality of the road. Also, large aggregate, if loosened, may cause significant windshield breakage, as traffic is applied. Surface treatment aggregate should generally be crushed. Uncrushed, rounded aggregate particles are not stable and often exhibit inadequate skid resistance. Cubical aggregate particle shape is the best, whereas flat and elongated particles are not desirable shapes for surface treatment aggregate. Good short-term, as well as long-term, bond of asphalt to the aggregate is important. Dusty aggregate as well as excess moisture inhibits asphalt-aggregate bond. Precoating the aggregate is one technique of enhancing aggregate retention on the roadway. (p. 13-62)

2.1.2 Surface Chemistry

Surface chemistry of the aggregate is one of the important factors that affect the bond between the aggregate and the binder. It also plays a key role in adhesion. Every aggregate of a specific mineralogical type has a unique surface chemistry. Curtis et al. (1993) claimed:

The interactions between asphalt and aggregate are dominated by aggregate chemistry. Asphalt chemistry also has an influence, though much smaller than that of the aggregate, on asphalt-aggregate interactions. Evaluation of asphalt-aggregate interactions shows that the aggregate chemistry is much more influential than the asphalt composition for both adhesion and sensitivity to water, except where cohesive asphalt failures occur. In addition to that, asphalt can penetrate into the pores, crevices, and interstices of the aggregate. The active sites on the particle attract the most polar and bondable asphaltic species on initial contact. The asphalt directly contacting the aggregate is important because this asphalt must adhere and remain adhered under different environmental conditions. But the interstitial asphalt, the asphalt that is adjacent to or between these various sized particles, also serves a major cohesive role in binding the particles together and maintaining the integrity of the mix. (p. 2)

The aggregate provides a surface that is heterogeneous and has a variety of sites of different composition and levels of activity. These active sites are frequently charged or contain partial charges that attract and orient the polar constituents of asphalt. The polar functionalities present at the point of contact between the asphalt film and the aggregate surface adhere to the surface through electrostatic forces, hydrogen bonding, or Van der Waals interactions. Any given asphalt molecule may not be too far distant from an inorganic surface either by contact with that surface or by contact with another asphalt molecule in contact with or influenced by the aggregate. Short-range chemical interactions in asphalt are more feasible because of electrostatic interactions that occur between the charged surface and the molecules attracted to the surface. A charged

aggregate surface attracts an oppositely charged or partially charged species or functional group contained in the species. 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. If this buildup of interlinked molecules caused by induced polarization occurred, the buildup would taper off rather quickly with distance from the aggregate surface, because the influence of the polar surface decreases with increasing distance, and the partially charged asphalt species could become neutral by a variety of interactions with surrounding asphalt molecules (Curtis et al., 1993).

Compounds with polar functional groups were more competitive and adsorbed much more strongly than nonpolar compound types. Among these species that were most competitive, the ones with functional groups, such as sulfoxide, carboxylic acid, nitrogen base, and phenol were most strongly adsorbed on aggregates, such as granite, limestone, greywacke and gravel. The less polar species, such as pyroles and ketones, and the nonpolar species, such as polynuclear aromatic ring compounds, were much less adsorbed and were not competitive for the aggregate surface. Since most aggregates have polar sites on their surface that actively attract polar species, any polar species may be attracted, such as organic species in asphalt or water that comes from rainfall or seepage. The two most polar species containing sulfoxide and carboxylic acid functional groups were readily removed from the aggregate surfaces in the presence of substantial amounts of water. The less polar species gave intermediate adsorption, while the compounds with nitrogen base and phenol were most resistive to moisture. The aggregates' susceptibility

to desorption of the organics by water ranked as gravels > limestones > greywacke > granites (Curtis et al., 1993).

In another study made by Curtis et al. (1992), they compared the aged and unaged asphalt adsorption properties of different asphalt-aggregate combinations and concluded that there is a little difference between these two materials. They concluded, “The small changes in chemistry as reflected by a relatively small increase in viscosity of the asphalt aged in the thin-film oven had little effect on the adsorptive interactions between asphalt and aggregate” (p. 5).

Gorman et al. (1998) studied the influence of aggregate surface chemistry in emulsion-aggregate interactions. In that study, researchers pointed out the importance of contact angle measurement when surface and interfacial tension data are unable to be obtained and it allows information to be acquired regarding the surface hydrophobicity^a-hydrophilicity^b of an aggregate surface. Gorman et al. (1998) explained the term “contact angle” with this definition:

A drop of liquid, when placed on the surface of a substrate, will form a contact angle. A contact angle of 0° indicates complete wetting, whereas an angle of 180° indicates non-wetting. For aggregate samples the higher the water contact angle, the more hydrophobic is the sample, and hence the more likely a bitumen phase would spread over its surface. Thus, contact angles can be used to rank the reactivity of each aggregate. (p. 4)

^aRepelling, tending not to combine with, or incapable of dissolving in water.

^bHaving an affinity for water, readily absorbing or dissolving in water.

Gorman et al. (1998) suggested the Washburn method^c for measuring advancing water contact angles as the most appropriate method to determine the surface properties of aggregates, as it provides a contact angle that is averaged over all of the particulate material in a given sample. An aggregate with a Washburn advancing water contact angle of greater than 65° is an acceptable aggregate for slurry surfacing applications.

Zeta potential is another important property of the aggregate surface. *Zeta potential* can provide critical information regarding the surface charge present on the aggregate surface. The aggregate surface has an electrostatic interaction with the surfactant molecules. In this particular study, Gorman et al. (1998) found that:

It can be seen that the *Zeta potential* for all aggregate samples, under natural pH conditions, is negative. This indicates that the natural pH for all aggregates is above the isoelectric point.^d In all cases this probably reflects the dominance of SiO₂ (from 45 to 72% composition in each aggregate). It may be the case that the sign of the charge of the aggregate could influence the interaction with the cationic emulsion surfactant. The fact that the *Zeta potential* for most of the aggregates is negative would suggest that the sign of the aggregate charge has little role to play in the breaking of a bitumen emulsion over the surface of the aggregate. (p. 8)

Most aggregates absorb asphalt to some degree. Asphalt absorption is directly related to the porosity of an aggregate. The total porosity is a good indicator for absorption. Pore-size distribution of an aggregate has a direct effect on the nature and degree of asphalt absorption. In general, asphalt absorption increases as these coarse-pore porosities increase. Chemical composition of a carbonate rock has an effect on porosity and therefore the asphalt absorption of the rock (Lee, 1969).

^c The Washburn technique for measuring the advancing contact angle of a liquid on a solid is based upon a measurement of the liquid intrusion rate into a packed bed of the powder (Washburn 1921, Gorman et al., 1998).

^d The point at which there is zero surface charge (Gorman et al., 1998).

Scott (1978) studied the de-bonding mechanisms and concluded that:

The presence of adhesion additives in the asphalt causes changes in the composition of the adsorbate. However, the results demonstrate that cationic adhesion additives are not exclusively adsorbed from hot asphalt to mineral, but adsorb together with other polar asphalt components. This suggests that while the surface activity of most additives will cause rapid initial adsorption and thus promote wetting of the surface, this may not be the ultimate equilibrium situation. After some time additive may no longer be the major component of the adsorbate. Both mineral surfaces and the nature of the asphalt appear to influence the effectiveness of an additive and the extent of its adsorption. If we turn to the nature of the interactions, which are likely to take place between the asphalt and the mineral, we find evidence for a spectrum of bonding, from hydrogen bonds to salt formation. Apart from the hydrolysis of covalent linkages, if these exist at the interface, the hydrogen bonds and salt links are likely to be susceptible to changes in their interaction equilibrium if the adsorption environment changes from nonpolar to polar (aqueous) or if the pH of a contacting aqueous phase is altered. At the asphalt/aggregate interface, the recovered adsorbates display basically the same type of behavior as either asphalt or asphaltene solutions since they become ionized and dissociated, particularly at high pH. Minor differences in the point at which dissociation causes a fall in the interfacial tension, show that there are some variations in adsorbate properties with different asphalt-mineral combinations, and at different temperatures. The highly anionic surface activity at high pH confirms the likelihood of carbonyl compound predominance. (p. 37)

Oxygen containing groups from asphaltene fraction molecules predominate in the asphalt fractions adsorbed on the mineral surface. The exact composition is based on the asphalt and the minerals along with the adsorption conditions. Cationic adhesion additives based on amines adsorb from the asphalt onto the mineral surface in combination with polar asphalt molecules. The extent of the role of additives will depend on the asphalt, minerals and contact conditions. The interaction between many types of minerals and water is frequently extensive enough to alter the pH of the surrounding water layer by several units. Both the nature of the adsorbate and its chemical state can be altered by changes in the adsorption environment. pH changes in microscopic water accumulations at the mineral surface can alter the type of polar groups adsorbed from the

surrounding asphalt as well as their state of ionization-dissociation. De-bonding of the asphalt from mineral aggregate under the action of water is probably due to a combination of these effects. Changes in the adsorption environment caused by the presence of water create changes in the adsorbate, leading to desorption and the build-up of opposing negatively charged electrical double layers on the mineral and asphalt surfaces. The drive to reach equilibrium leads to the uptake of more water and the physical separation of asphalt from the mineral. With many minerals, cationic adhesion additives in the asphalt can delay but not prevent loss of adhesion on water sensitive surfaces, due to the influences of changing adsorption environment. Anionic additives are only likely to be successful where there is opportunity for water-insoluble-salt formation with tightly bound surface metal ions. Modification of the mineral surface with inorganic salt is bound by the same restrictions. The best practical protection from adhesion loss with sensitive minerals remains the use of a good wetting agent under proper mixing conditions, or the addition of a material (e.g., hydrated lime) to control the adsorption of oxidized asphalt components on the water sensitive aggregate (Scott, 1978).

2.2 Precoated Aggregate

Existence of dust on the cover aggregate is one of the common reasons of seal coat failure due to aggregate loss. Dust avoids the good adhesion between the aggregate and binder. The most common solution adopted to prevent the negative effects of dust is to use precoated aggregate in seal coat applications. Precoated aggregate is the aggregate that is covered with a very thin film of bituminous binder (Khandhal and Motter, 1997).

The current TxDOT specifications require that “precoated aggregate shall be aggregate of type and grade specified, coated with 0.5 to 1.5%, by weight, of residual bitumen from a precoating material” (p. 279).

Precoated Aggregate Types, according to TxDOT Standard Specifications TxDOT (1993) are given as;

Type PA: type PA shall be precoated aggregate consisting of gravel, crushed slag, crushed stone or natural limestone rock asphalt.

Type PB: type PB shall be precoated aggregate consisting of crushed gravel, crushed slag, crushed stone or natural limestone rock asphalt.

Type PC: type PC shall be precoated aggregate consisting of gravel, crushed slag or crushed stone.

Type PD: type PD shall be precoated aggregate consisting of crushed gravel, crushed slag or crushed stone.

Type PE: type PE shall be precoated aggregate as shown on the plans.

Khandhal and Motter (1997) conducted research on precoated aggregates for seal coats and surface treatments. They found that the rate of increase in knock-off loss^e for uncoated aggregate increased significantly after about 3% dust content (by weight) in most cases. Secondly, they affirmed that “although no relationship was observed between the percent knock-off loss and percent water absorption or particle index of the

^eThe researchers used knock-off loss and initial loss terms. Initial loss presents the aggregate loss of the pan containing the seal coat, which is inverted to allow the aggregate particles to fall, after 24 hours of curing. After determination of initial loss, the pan is placed upside down at the top of five ½ in sieves and this assembly is placed in the Mary Ann Sieve Shaker and subjected to the shaking and tapping action for 5 minutes. After this the aggregate loss is again measured and this loss is called knock-off loss.

aggregate, a correlation between the flakiness indices of the aggregates and the corresponding aggregate retention was observed” and the researchers deduced that “when the percentage of the flat particles in the sample (or flakiness index) increases, the corresponding retention loss also increases” (p. 14)

In that study, five aggregates (AASHTO No. 8 size) with different mineral compositions containing 3.0% dust content were precoated to get five different degrees of precoating. All of the materials were subjected to Pennsylvania Aggregate Retention Test.^f The first observation Khandhal and Motter (1997) made is that the more than 90% precoating is by far the best when the percent initial loss is considered. In other words, the immediate adhesion of cover aggregate with the bituminous binder is best acquired with more than 90% precoating. The second observation was increasing the percentage of precoating decreased the initial aggregate retention loss. The final observation in this phase was, as stated to the test results of more than 90% precoating, initial aggregate retention loss was reduced by as much as 80%. In addition, the researchers claimed that in spite of more than 90% precoating giving the best results in the percent initial loss, it gave poor results in the knock-off test. On the other hand, the researchers noted that use of precoated aggregate with emulsified asphalt would slow the breaking duration of applied emulsified asphalts so that the reopening time of the road to traffic will be delayed.

^f Look to test methods part for further information.

Janssen made a different approach on this subject in 1941, by investigating precoating seal coat aggregate with furfural.⁸ Janssen (1941) concluded that the furfural treatment was effective in reducing the tendency of aggregate to strip but overall performance of treatment is not satisfactory.

2.3 Hot Applied Asphalt

At ambient temperatures, asphalt is either solid or semi-solid. To be used in seal coat applications, it must be made liquid enough to coat the aggregate. This is accomplished in three different ways, by heating in a heating kettle or a storage tank (hot asphalt), by dissolving in a petroleum solvent (cutback asphalt), by combination with water (asphalt emulsion). Today, due to environmental concerns, cutback asphalts are used very rarely.

Any binder that is used for seal coating, whether cutback asphalt, hot asphalt or asphalt emulsion, has to be suitable for sticking chips to the road surface. For seal coat construction, a binder should have good adhesion to the existing road surface and to aggregate. It should develop the necessary bonding to hold the aggregate in place and should keep this bonding in order to prevent loss of aggregate under adverse conditions.

The binder should be able to resist deterioration under conditions of outdoor exposure and not become undesirably hard or brittle for a substantial period of time. In addition, the binder should have the proper fluidity or consistency to permit easy and

⁸ Furfural: (C₄H₃O-CHO) is a by-product of the manufacture of breakfast oats, used in making plastics and as a selective solvent for refining crude resin and lubricating oils.

accurate application. As conditions vary from project to project it is obvious that no one grade or type of binder will satisfy all of the requirements for every project, considering the wide variety of conditions of the existing road surface, types of aggregate, equipment available, and climatic conditions which may be encountered.

Asphalt is a product of crude oil, which in turn is a product formed by nature primarily from plant life. The process occurs over millions of years under varied conditions of temperature and pressure. Thus, even though all petroleums are hydrocarbons, i.e., chemical combinations of carbon and hydrogen, crudes vary widely in the amount and nature of the hydrocarbons they contain. Similarly, asphalts, which are residues left after the more volatile fractions of the petroleum have been distilled off, vary significantly from one crude oil to another, both in their characteristic and in the volume percentages present. The principal elements in asphalt are carbon and hydrogen. Sulfur is the next most abundant element, followed by small amounts of nitrogen and oxygen, which are referred as heteroatoms (Halstead, 1985).

2.3.1 Rheological Properties

Rheology is the study of time-temperature response of a material to an applied force or stress. The resistance to flow of asphalt pavements has an important bearing on their ability to resist the effects of oxidation and moisture. When the asphalt content of a compacted asphalt mixture reaches the point where the air voids are no longer connected, there is a marked reduction in the bearing capacity of the pavement. There is also a marked reduction in the effect of oxidation and moisture since it is more difficult for

water and air to reach the interface between the aggregate and the asphalt. If the bearing capacity of a pavement can be increased sufficiently to support traffic with reduced air voids, the subsequent improvement in resistance to oxidation and moisture can greatly increase the life expectancy of the pavement (Davis, 1987).

The primary rheological requirement for asphalt pavements, is that bearing capacity not be exceeded under the highest temperatures reached during the design life of pavement. Asphalt pavements should be designed with bearing capacity at the highest temperatures expected in the roadway in excess of the expected highest truck tire pressure. The best approach to reduce problems with oxidation and moisture in asphalt pavements is a sufficient level of bearing capacity so that air voids can be kept below 3%. An important factor increasing the bearing capacity of pavements is the selection of the top size for the aggregate that is greater than two-thirds of the thickness of the pavement layer (Davis, 1987).

For pavements in hot climates with no likelihood of frost, to sustain the stability of a pavement, as the temperature susceptibility of a paving asphalt increases, its penetration at 25°C (77° F) must be decreased. In cold climates, to eliminate low-temperature transverse pavement cracking in winter, and to provide adequate pavement stability for summer traffic, the penetration at 25°C (77° F) and temperature susceptibility must be coordinated for heavy, medium and light traffic. The natural temperature susceptibilities of paving asphalts can be reduced dramatically by the addition of small percentages of certain polymers (McLeod, 1987).

2.3.2 Polymer Modified Binders

Although polymers appear to stiffen the original binder as measured by conventional specification tests, the shear-thinning nature of modified binders appear to indicate that no special handling is required during normal construction operations. Rheology of modified binders indicates that elastic modulus is reduced at low test temperatures and increased at high temperatures compared to the control materials (Shuler et al., 1987).

The mixing conditions of an asphalt-polymer blend have a significant impact on the structure and properties of the modified binder, the longer the agitation time, the finer the microstructure. There is, however, a threshold beyond which polymer degradation leads to a decrease in mechanical properties, more precisely in low-temperature deformation scenarios. The higher the temperature, the more rapidly this threshold is reached. There is a relationship between microstructure and physical properties. Binders with a coarse microstructure are less deformable at low temperatures. The finer the microstructure, the greater the deformability will be for binders with a polymer network. The amount of polymer required for asphalt matrix inversion and for obtaining highly modified practical properties depends to a great extent on the asphalt itself, but chemical analysis by generic groups does not allow a prediction of the value of this inversion threshold. The extent of swelling in asphalt-polymer blends is practically independent of temperature over the 80 to 160 °C range. Swelling is not highly dependent on polymer content for high polymer proportions, but increases substantially as the amount of polymer decreases. For low polymer contents, the extent of swelling becomes a function

of the asphalt itself, but chemical analysis by generic groups does not allow prediction of the extent swelling. The saturated oil fraction of the asphalt may play an important role in the modification of the properties of asphalt-polymer blends. An increase in the ring and ball softening point and in the plasticity interval, and a decrease in the thermal sensitivity of asphalt-polymer blends is not directly proportional to polymer content and depends significantly on the asphalt itself. There is a colloidal instability index value beyond which the polymer is no longer swollen in the modified binder, but dissolved by the asphalt (Brule et al., 1988).

2.4 Asphalt Emulsions

Asphalt Emulsions are widely used as a binder in seal coat operations. Asphalt emulsion is basically a mix of asphalt with water and an emulsifying agent. The separation of water from emulsion is called breaking. If the asphalt emulsion is to perform its ultimate function as a binder, the water must separate from the asphalt phase and evaporate. Curing, which is the removal of water from the emulsion, involves the development of the mechanical properties of the asphalt cement. The end result is a continuous cohesive film that holds the aggregate in place with a strong adhesive bond.

According to the Asphalt Emulsion Manual, some of the factors affecting breaking and curing rates of asphalt emulsions include:

- Water Absorption: a rough-textured, porous aggregate speeds-up the setting time by absorbing water from the emulsion.

- **Aggregate Moisture Content:** while wet aggregate may facilitate coating, it tends to slow curing process by increasing the amount of time needed for evaporation.
- **Weather Conditions:** temperature, humidity, and wind velocity all have a bearing on water evaporation rate, emulsifier migration and water release characteristics. While breaking usually occurs more quickly at warmer temperatures, that is not always the case. Hot weather can cause skin formation on seal coats, trapping water and delaying curing. Some chemical formulations have also recently been developed to expedite breaking at lower temperatures.
- **Mechanical Forces:** roller pressure and, to a limited extent slow moving traffic, forces the water from the emulsified asphalt and help attain mix cohesion, curing and stability.
- **Aggregate Surface Area:** greater the surface area, particularly due to excessive fines or dirty aggregate, quicker the breaking of emulsion.
- **Surface Chemistry:** intensity of the aggregate surface charge, in combination with the intensity of emulsifier charge, can impact setting rate, particularly for cationic emulsions. Calcium and magnesium ions on the aggregate surface can react with, and destabilize, certain anionic emulsifiers, thus accelerating setting.

- Emulsion and Aggregate Temperature: breaking is retarded when emulsion and aggregate temperatures are low. This is particularly evident in microsurfacing.
- Type and Amount of Emulsifier: the surfactant used in the manufacture of emulsion determines the breaking characteristics of seal coat and mixing grade emulsions.

2.4.1 Factors Affecting the Characteristics of Emulsion

Generally, the properties affecting the characteristic of emulsions are the type and content of binder, asphalt particle size distribution, and composition of the aqueous phase. Deneuvillers and Samanos (1999) studied the correlation between characteristics and properties of cationic bitumen emulsions in order to improve their behavior in the field. They correlated the breaking index of emulsions with the median diameter^h of their particles and found that, for slow-breaking emulsions, the breaking index decreased with the increasing median diameter. This was explained by the fact that, as the median diameter decreases the specific surface increases, as does the number of bitumen particles. Deneuvillers and Samanos (1999) concluded that it takes more filler to make contact with all the particles and break all of the emulsion. For a given emulsion, the breaking index increase as the median diameter of the filler used increases, for which the explanation is the same (it takes more filler with a smaller specific surface to break a given quantity of

^h The median diameter is the particle diameter at which half of the distribution (by volume) is larger and half is smaller (Deneuvillers and Samanos, 1999).

emulsion). The researchers also found that the relationship between breaking index and specific surface is not linear. They affirmed that the relationship between the developed surface area of the emulsion at the time of breaking and the breaking index is not constant. The specific surface is not the only parameter involved. Steric particle overcrowding is probably the cause of this phenomenon.

This systematic study investigated the effect of constituent parameters on emulsion properties. It also quantified the influence of median droplet size and standard deviation on three important properties, viscosity, breaking rate, and rate of cohesion building. By controlling the standard deviation, it is possible to control viscosity and rate of cohesion build up. These two properties are particularly important for surface treatments. High viscosity prevents runs, and a fast rise in cohesion means the newly-dressed road can be opened to traffic sooner, with a reduced risk of aggregate loss (Deneuvillers and Samanos, 1999).

2.4.2 Breaking and Curing

The basis of emulsification is the creation of small (1-5 micron) particles of asphalt that are coated with a chemical that allows the particles to stay apart. The emulsion must, however, break back to asphalt films in order to perform its functions such as coating, water-proofing and adhesion. The asphalt particle has emulsifier ions that are dissolved in the asphalt and also in the water phase. Thus two particles that approach each other are repelled due to like electrostatic charge. However, particles cannot exist in a charged state and this must be balanced. The charge density at the

interface, as measured by the *Zeta potential*, determines the stability of the emulsion. The thickness of double layer is determined by the molecular size of the emulsifier, and its ability to dissolve in the water phase, which is Hydrophile/Lipophile Balance (Hooleran, 1999).

The success of a emulsified asphalt relies heavily on its ability to form films on aggregates. Film formation involves coalescence without entrapment of water into a continuous film. It can be described in terms of Young's equation:

$$\text{Wetting (s/l)} = \text{interfacial energy a/l} - \text{interfacial energy s/l} \times \cos(\text{contact angle}).$$

As shown in Young's equation, film formation is a function of kinetic factors such as temperature, viscosity and internal stability and also thermodynamic factors. Films form more slowly at low temperatures, with larger particles and at higher viscosities. Addition of solvents or coalescing agents will assist film formation, especially at low temperatures. Adhesion agents, doped into the asphalt, can enhance adhesion (Hooleran, 1999).

The mechanism of chemical interaction between emulsion and aggregate is not fully understood, but the following general description appears to represent agreement amongst most researchers. The stable emulsion particles are attracted to the surface of the aggregate, where emulsifier from the bulk solution interacts with the aggregate surface charges. This changes the equilibrium of the emulsified asphalt system, and it is destabilized. This leads to deposition of asphalt on to the aggregate surface. This is where the wetting and film formation become critical. The rate of film formation will determine the adhesion, and the emulsifier/formulation will determine the breaking rate.

The internal flocculation and coalescence will also have an effect. Within the forming film, the particles are in closer proximity and these mechanisms will accelerate. However, if this is too fast, water will be trapped and the binder film forming capability will be diminished. The bulk part of the emulsion remote from the aggregate surface will break by flocculation and coalescence. In seal coating, emulsions with high binder contents will accelerate the break in the bulk emulsion (Hooleran, 1999).

The “curing” of an emulsion film is often confused with its breaking. Curing is simply the loss of water from the film and bulk emulsion. Therefore, curing rates depend on water content, rate of evaporation and diffusion of water through the curing binder. In systems with strong energy differences between the aggregate surface and the emulsified binder, an extra driving force is present to push water away from the aggregate-binder interface. Emulsifier choice relative to aggregate is thus a very key issue. Cement is often used to enhance this (Hooleran, 1999).

The properties of base asphalt used in an emulsion can have a significant effect on properties of the emulsified asphalt. Therefore, a sound knowledge of the asphalt cement composition and its effect on emulsions is important in producing good emulsified asphalts. Emulsion properties and stability are largely bulk properties, but the emulsion can be improved by creating finer particle size distributions. This makes interfacial properties more important and provides a chemical solution to stability problems. Polymer addition improves rheological properties, and processing and compatibility also play important roles in emulsion morphology and performance (Hooleran, 1999).

The desorption process of a surfactant molecule adsorbed on the bitumen surface influences not only the electrokinetic properties of bitumen particles under diluted conditions, but also the reactivity of emulsion during the breaking process. During mixing with aggregates, the desorption rate of surfactant from bitumen and the adsorption of bitumen on aggregate, could be the kinetic control of the process. Moreover, surfactants with different reactivity with aggregates may desorb at different rates to different equilibrium states. The surfactant that leads to rapid setting (RS) emulsions desorbs from bitumen surface rapidly while the less reactive surfactant is attached to the stronger surface (Paez et al., 1999).

The *Zeta potential* of bitumen particles decreases with increasing pH and presents a different isoelectric point for each emulsion. At higher pH, the *Zeta potential* is negative, consequence of negative surface charge density. These results indicate that the surfactant used to manufacture cationic slow setting (CSS) emulsions has a stronger basic character than at used in cationic rapid setting (CRS) emulsions. In other words, it keeps positive charge at higher pH. The negative surface charge density at high pH is due to the acid groups present in bitumen from natural oxidation processes. Siliceous aggregates have acid character. The *Zeta potential* of siliceous filler suspended in pure water is negative and close to -10mV . Decreasing the pH, the potential decreases slightly but remains negative in all the pH range. In fact, this decrease could be attributed to the increase of ionic strength that the decrease of pH leads to, and not to any specific adsorption. It may be concluded that pH changes do not affect significantly the surface properties of siliceous aggregates (Paez et al., 1999).

The adsorption of surfactants leads to a neutralization of initial negative charge and, after that, adsorption continues up to a *Zeta potential* close to 50 mV. In spite of the sharper increase on *Zeta potential* with slow setting (SS) surfactants, the saturation potential is the same for both. These results indicate that cationic surfactant adsorption on aggregate surface is not only led by electrostatic interaction between opposite charges, but also by thermodynamically favored adsorption on positively charged surface. The formation of insoluble salts between siliceous and amino groups agrees with the electrokinetic behavior (Paez et al., 1999).

The desorption rate of surfactant used to manufacture CRS bituminous emulsions is higher than the one used to manufacture CSS emulsions. The adsorption of RS-type surfactants on pen 110-130 bitumen particles leads to higher *Zeta potentials* than the SS one. In fact, the qualitative adsorption isotherms are quite similar for both. The *Zeta potential* of bituminous emulsions decreases with pH. The isoelectric point of CSS emulsion is higher than CRS emulsion. The surface charge of bitumen particles at high pH is negative due to natural acid groups in bitumen. The adsorption of cationic surfactants on siliceous aggregates changes the surface charge from low negative potentials to positive potentials of 50 mV. Electrophoretic mobility measurement is a rapid and useful method to analyze qualitatively the interfacial equilibrium dynamics in bituminous emulsion-aggregate system (Paez et al., 1999).

The adsorption of cationic surfactants at the mineral aqueous solution interface modifies the wetting behavior of mineral surfaces. One of the key steps of mineral coating by bitumen emulsion is the transformation of mineral surfaces from hydrophilic

to hydrophobic, through the addition of surfactants (which results in the heterocoalescence of bitumen droplets upon the mineral). This operation is based on the selective adsorption of surfactants at the mineral aqueous solution interface, the nature of which will govern adsorption mechanisms. In mineral-cationic surfactant systems, the adsorption phenomena are resulting from physical mechanisms such as electrostatic attraction and hydrocarbon chain association. Electrostatic forces are created by attraction of the positively charged polar groups of the surfactant ion by a negatively charged site of the mineral surface. When the adsorption density of the surfactant ions exceeds a critical concentration (which depends on mineral nature and pH), the hydrocarbon chains of adsorbed surfactants begin to associate themselves at the interface. This association can be in different types. The electrostatic characteristics of mineral surface and the cationic surfactant solution composition (nature and concentration) are two important parameters that will control the adsorption processes of surfactant in the system (Deneuvillers, 1999).

Irrespective of the specific surface of materials, the nature of their surface determines the mechanism of surfactants adsorption. It governs the breaking of emulsion globules when they come in contact with the mineral. In the ideal scenario, when the mineral surface is covered by surfactant molecules and has becomes hydrophobic, the minerals are penetrated by bitumen globules which are also hydrophobic, and the heterocoalescence begins. For acid type materials, the quantity of surfactant necessary in aqueous phase to make the surface hydrophobic is lower. The coating of this kind of mineral should be done with emulsion with less surfactant(s). However, it is usually

common to use emulsions with high concentration of surface-active agent, and it assumes that mineral particles are free, and their surface can be wetted by water. The globule charge is an additional parameter, which affects the breakdown mechanism. With basic-type materials, the pH increases rapidly, the surfactant, less hydrophilic, will be less soluble in water and will then facilitate heterocoalescence (mineral will be almost as hydrophobic as if it was covered by only one layer of surfactant). For maximum effectiveness, the bitumen emulsion formulation should be optimized with regard to the type of mineral to treat, while its stability must be maintained (Deneuvillers, 1999).

For the case of RS emulsion, the time of setting decreases with increase of the temperature, within the interval 5° C (41° F)–50° C (122° F). At low temperatures, 5° C (41° F), siliceous materials result in setting times higher than for calcareous ones. While the temperature increases, differences in the times of setting decrease for the different aggregates. The time of setting for different aggregates is about the same when the temperature is at 50° C (122° F). The influences of the water vapor pressure that augments its influence with the increment of temperature could be a reason for this. As the amount of emulsifier in the emulsion is increased, the time of setting also increases within the entire considered temperatures interval, whatever the type of aggregate. On the other hand, aggregates of siliceous type result in the greatest differences. The residual adherence of bitumen on the aggregates is not being influenced by temperature, nor by the amount of emulsifier. For the case of the SS emulsion with fine aggregates similar to those used in slurry seal, the variation of the rate of setting with the temperature is more perceptible than in the case of mixtures with the RS emulsion.

Incorporation of different concentrations of stabilizer agents within the prewetting water increases in an appreciable way the time of setting, with respect to the mixture having non-pretreated aggregates. But, as the temperature increases, the differences decrease, until reaching values of the same order when getting close to 30° C (86° F) (Pinilla and Agnusdei, 1975).

2.5 Rubberized Asphalt

The use of rubberized asphalt to seal pavements was initially developed by the city of Phoenix in the 1960s, and was being used in seal coat applications to this day (Morrison and Hesp, 1995). This type of seal is often referred as a hot rubber seal. The low cost, inherent cohesive nature, weather resistant properties, and ease of processing in the molten state are the main reasons for the extensive use of asphalt cement based materials as waterproof coatings, sealants and binders (Nadkarni et al., 1985). Over hundred millions of tires are discarded every year in United States. Therefore, the use of waste rubber as an asphalt cement modifier is beneficial from the engineering, economic and environmental points of view (Zaman et al., 1995).

TxDOT specifications require that in a mixture of Type I rubberⁱ and asphalt cement, the proportions by weight in the mixture shall be $75 \pm 2\%$ and $25 \pm 2\%$, respectively. The temperature of asphalt cement and rubber shall be between 350° F and 425° F during the addition of the rubber. The reaction period shall be at least 30 minutes after all rubber has been added. Due to viscosity concerns, a diluent up to a maximum

ⁱ Type I: Ground tire rubber.

amount of 7.5% by volume of the hot asphalt-rubber mixture may be added to obtain optimum viscosity for spray application and better “wetting” of cover aggregate. In a mixture of Type II rubber^j and asphalt cement, the proportions by weight of the asphalt cement (including extender oil, if needed) and rubber shall be $78 \pm 2\%$, and $22 \pm 2\%$, respectively. The asphalt cement and extender oil (if needed) shall be combined and heated to a temperature of not less than 400° F. The reaction period shall be at least 30 minutes after all the rubber has been added prior to mixing rubber. Temperature of the material during the reaction period shall be between 375° F and 425° F.

Nicholson (1998) identified the following seven advantages of rubberized asphalt seals:

- A long lasting, durable pavement surface
- Resistance to reflective cracking, rutting and shoving
- Skid resistant surface
- Reduced tire noise
- Color contrast for stripping and marking
- Environmentally friendly
- Effective choice for chip and slurry seals.

Morrison and Hesp (1995) discussed two rubberized asphalt processes; wet process and dry process. Wet process, as developed by McDonald, 14-20% by weight of ground tire rubber in the sieve range between #8 and #20 sieves is swollen in the asphalt at high temperatures to form a gel-like material after mixing for approximately 45

^j Type II: Mixture of ground tire rubber and high natural reclaimed scrap rubber.

minutes at 347-428° F. To this gel, some aromatic kerosene fraction is added to increase its workability. This so-called “asphalt rubber” is mixed with aggregate to form the pavement. Morrison and Hesp (1995) affirmed that, “The thicker films of asphalt at the aggregate interface results in greater durability. These asphalt rubber binders also have higher softening points which can result in less bleeding and permanent deformation” (p. 2584).

The dry process was originally developed in Sweden and it differs from the wet process in that the rubber crumb is slightly larger, i.e., 1/16-1/4 in (1.56-6.34 mm) and is directly mixed with the aggregate before asphalt binder is added. Moreover, the amount of rubber added is of 3-4% by weight of the aggregate and this process requires a special aggregate gradation in order to avoid any interference of the rubber crumb with the aggregate which can lead to premature stripping. In cold climates, this technology has been used quite successfully to reduce the harmful effects of ice formation on roads (Morrison and Hesp, 1995).

Another study made by Robert E. Olsen in 1973 also points out the advantages of using rubberized asphalt as a binder. In that study, the McDonald method has been used and the proportion of rubber used was 25 to 30%. Olsen (1973) claimed that using rubber in larger proportions gave good results in controlling fatigue cracking. In addition to that, he mentioned, “This type of construction prevents reflection cracking from the substrate pavement because of its flexibility and the interlaced particles of rubber discourage the propagation of cracks” (p.21). However, Olsen (1973) also stated that this type of seal coat construction is not a solution to treat all types of bituminous pavements.

Zaman et al. (1995) studied rheological properties of rubberized asphalt. The researchers deduced that, “The viscosity of asphalt cement increases with addition of rubber, and rubber-modified asphalt cement samples show a more uniform and higher resistance against loading as the amount of rubber is increased” (p. 466). In addition to this statement, the results of that study showed that the elastic behavior of asphalt cement modified with 10% rubber is approximately two times larger than that of asphalt cement modified with 7.5% rubber. However there was only a small difference in the elasticity of asphalt cement modified with 5% and 7.5% rubber. From the findings of that study, it can be concluded that increasing the amount of rubber in asphalt cement will increase both dynamic viscosity and storage modulus. Also Zaman et al. (1995) claimed that, “There is a great improvement in the creep resistance and the fracture time of asphalt cement modified with the rubber. Both the fracture time and creep resistance of asphalt cement samples increase as the amount of rubber increased” (p. 466).

An early study conducted by Itakura and Sugawara (1959), analyzed the effects of rubber on binder properties. They pointed out that:

Heating and stirring time influence several physical characteristics of asphalt cement especially penetration, float test value, softening point, ductility and toughness. Especially, in case of the natural rubber powder, this tendency was remarkable except float test value and softening point, where 2 hours heating gave the maximum value ductility at 0° C (32° F) was increased the more as heating time increased. Inclusive, it seems that heating time of 5 hours is necessary and sufficient to change characteristics of asphalt and toughness of mixture at low temperature for all kinds of rubber powder investigated. Amount of rubber powder to be added mixtures and construction costs prevailing. But from the standpoint of improving the quality minimum of 3 to 5% of the natural rubber powder and 5 to 10% (based on the asphalt) of reclaimed rubber will be used for a given pavement, as wearing course. (p. 407)

2.6 Construction Factors

Construction factors are as important as material factors that affect a seal coat performance. These factors include equipment related factors like rolling or sweeping and also environmental factors like temperature at time of the seal, or traffic control after the construction of seal coat.

TxDOT specifications require that surface treatments such as seal coats shall not be applied when the air temperature is at or below 60° F and is falling, but may be applied when the air temperature is above 50° F and rising. The air temperature shall be in the shade and away from artificial heat. Surface treatments shall not be applied when the temperature of the surface on which the surface treatment is to be applied is below 60° F. The area to be sealed shall be cleaned of dirt, dust or other deleterious matter by sweeping or other approved methods. If it is found necessary by the engineer, the surface may be lightly sprinkled with water just prior to the first application of asphaltic material. Once the sealing is completed, the finished surface shall be cleared of any surplus aggregate by sweeping or other approved methods.

Jackson et al. (1990) recommended that rolling must occur immediately after application of aggregate when emulsions are used, and also the maximum allowable time between the placement of emulsion and aggregate should be limited to 1 minute. They also suggested a minimum of 3 pneumatic rollers traveling at a speed of 5 mph. Shuler (1998) concluded that use of steel roller should be avoided, particularly on high traffic volume facilities due to lack of compaction and potential for crushing.

Jackson et al. (1990) concluded that brooming should be accomplished as soon as possible after the emulsion has set and brooms must be motorized control vertical pressure. In addition, the researchers also recommended an additional brooming before 10 a.m. the following morning. A study by Wood (1999) also recommended the complete final sweeping of roadway as soon as possible and no later than the next morning. Shuler (1998) stated that:

Cleaning the surface after chip application using push or vacuum sweepers is highly recommended. Light sweeping should be done as soon after rolling as practical such that the fresh seal is not damaged but so any loose, or excess chips are removed before trafficking. Additional light sweeping after approximately 12 hours was also found highly beneficial. (p. 113)

Traffic control is of the utmost importance and pilot cars should be operated to maintain slow speeds during the period immediately after the seal coat is applied. This patrol should maintain these speeds until sundown or nearly dark or until the asphalt has cooled sufficiently to reduce fluidity. It may be necessary to continue the piloting of traffic during the next day. Any time that traffic is permitted to operate at high speeds, damage will be done to the new seal coat in direct proportion to the amount of the traffic using the roadway. Shuler (1998) deduced that, "Traffic control using pilot vehicles at 25 mph to keep traffic on the new chip seal surface is recommended for one hour following final rolling. This trafficking appears to benefit the early performance of the seal by improving initial embedment" (p. 113).

Another factor that affects seal coating is weather conditions at the time of seal. Erickson (1973) concluded that, "If pavement temperature is on the cool side, the asphalt and aggregate may not bond and the chips will begin to be whipped off almost

immediately” (p. 12). The timing of the work also plays a critical role due to the weather conditions. Jackson et al. (1990) stated that, “late season work does not provide for adequate cure and increased embedment of aggregate under the traffic” (p. 6).

2.7 Stripping

The bonding between aggregate and binder has to be strong and durable enough to satisfactorily carry the high traffic loads during the whole service life. Any weakness or loss of bonding will produce problems through out the life of pavement. Water or moisture is known to be a major cause of the adhesion loss. Stripping in asphalt pavements is defined as the loss of adhesive bond between the aggregate surface and asphalt binder due to the adverse action of water or moisture, resulting in the displacement of asphalt cement films from aggregate surfaces by water (Tarrer, 1986).

Stripping is a complex phenomenon and has been related to many factors including the physical and chemical properties of both the aggregate and the asphalt cement. Several mechanisms have been proposed to explain stripping, and they can be classified into three types; mechanical, thermodynamic, and chemical. The mechanical concept suggests that the surface texture of the aggregate surface is the main factor affecting mechanical adhesion. The thermodynamic or surface energy concept involves the wetting behavior of asphalt at the asphalt-aggregate-water-air interface. The chemical concept suggests that when aggregate is coated by asphalt, selective adsorption occurs at the interface, followed by a chemical reaction (Tarrer, 1986).

Curtis et al. (1993) identified the following modes of failure:

- Separation of the bond at the interface
- Failure within the asphalt where soluble components are removed
- Cohesive failure within the aggregate
- Phase separation of components when the presence of water increases the solubility of polar compounds through hydrogen bonding.

2.7.1 Mechanism of Stripping

Fromm (1974) studied the mechanism of stripping and concluded that:

Stripping is caused by water penetrating between an asphalt film and the aggregate surface and preferentially wetting the aggregate surface. Water may enter and penetrate asphalt films by spontaneous emulsion formation. Asphalt films may be breached at an air-water-asphalt interface by interfacial tension. Asphalts vary in their rate of emulsification depending on the nature of the asphalt. The rate and extent of emulsification may be increased or decreased with use of different additives. The emulsified films were tender and were easily injured or broken. Emulsion formation is a mechanism by which asphalt stripping occurs. If it can be retarded or prevented, pavements will last longer. (p. 217)

Firstly, Tarrer (1986) stated that:

“An asphalt, or the oils from it, normally penetrate into the pores or cracks in the surface of an aggregate particle. The interlocking of the asphalt cement with these pores makes its adhesive joint to the aggregate surface stronger or less readily stripped by water. In addition to aggregate porosity and irregularity, the mechanical bond also depends upon factors such as the size of the aggregate’s individual crystal faces and the absorption capability of the aggregate” to explain the mechanical concept of stripping. (p. 35)

Secondly, he affirmed that:

“Adhesion results from the interfacial energy relationships at the aggregate-asphalt-water-air interface. When asphalt spreads over and wets the aggregate surface, a change in energy takes place called “adhesion tension.” Adhesion tension depends on closeness of contact, mutual affinity of the two

materials, and time of contact. An aggregate tends to become coated by the liquid with which it has the greatest adhesion tension. The adhesion tension for water to aggregate is higher than for asphalt to aggregate in most cases; therefore, water tends to strip an asphalt coating at the interface” to explain the thermodynamic concept of stripping. (p. 35)

Thirdly, he claimed that:

“The interaction between an aggregate and water may cause a change in the adsorption environment, resulting in changes in the nature and chemical state of the adsorbate and the interfacial electrical properties between asphalt and the aggregate. The quality of bond that develops between the aggregate and the asphalt depends on factors such as acidic and alkaline surface adsorption site characteristics, the surface charge of the aggregate, and composition of the asphalt” to explain the chemical concept of stripping. (p. 36)

Tarrer (1986) tested five different aggregates (granite, limestone, dolomite, chert gravel, and quartz gravel) for their pore volumes and surface areas, pH values of contacting water, and surface charges in contact with water. Tarrer (1986) concluded that, “Because of low pore volume and surface area, granite had a relatively high moisture sensitivity. Low pore volume or surface area suggests a smooth, crystalline surface with low surface roughness” (p. 42). He also concluded that, “Large areas of interfacial contact and surface roughness are required for good adhesion and interlock. This low pore volume and surface area should reduce adhesive bond strength, resulting in high moisture susceptibility” (p. 42). Therefore, the pore volume and the surface area of an aggregate are two primary physical properties on which its moisture susceptibility is expected to be most dependent.

Tarrer (1986) also made an important comparison between dolomite and crushed quartz gravel in that study. He stated that,

Although dolomite had a higher surface area, it also had more stripping propensity because of its smaller pore size. While dolomite had nearly the same

amount of pore volume, it had about ten times the surface area, which meant that dolomite had smaller pore size than crushed quartz gravel. This result suggests that asphalt cement coating over a rough surface with fine pores traps air and can hardly penetrate into the fine pores, so that only a fraction of the apparent surface area might be involved in good interfacial contact. (p. 42)

The researcher also deduced that pH value of the system has a significant effect on the stripping. Tatter claimed that:

During the aggregate coating with asphalt, the aggregate is likely to adsorb some components of asphalt, probably the more polar species of the asphalt, forming hydrogen bonds or salt links. These links between the adsorbed asphalt component and the aggregate surface are likely to be susceptible to the presence of water and the increase in pH of contacting water. Based on this discussion, one is led to conclude that stripping damage may be expected for an aggregate, which changes the pH of contacting water to a relatively high value. (p. 52)

The interfacial activity occurring between charged surfaces of mineral aggregates and asphalt cements is fundamental to the nature of adhesion and stripping of asphalt from aggregate. The surface charge of the aggregate is as important as are specific chemical interactions, and in fact mineral aggregates possess distinctive polarities or electrochemical properties. The functional group types of an asphalt adsorbed on an aggregate surface consist mainly of the acid fraction of the asphalt. The acid molecule is represented by carboxylic acid (R-COOH). In the presence of water, the acid molecules are separated into two ions; the carboxylate anion (R-COO⁻) and the proton (H⁺), resulting in a negative polarity of the asphalt at the interface. The increase in pH due to present water at the aggregate surface increases the extent of dissociation of the acid molecules. Since mineral surfaces are also negatively charged in varying intensities, repulsion develops between the two negatively charged surfaces of the asphalt and

aggregate. This result suggests that aggregates, which have a relatively high surface potential in water could be more susceptible to stripping (Tarrer, 1986).

Although physical properties of aggregate affect stripping, there was no strong correlation between the physical properties of aggregate, such as pore volume and surface area, and the stripping propensity of the aggregate. Chemical and electrochemical properties of aggregate surface in the presence of water were most important factors for stripping (Tarrer, 1986).

A study made by Curtis et al. (1992), which was focused on influence of aggregate chemistry on the adsorption and desorption of asphalt, suggested two mechanisms to explain stripping. The researchers suggested:

In both cases, water is drawn from the air voids through the asphalt surrounding the aggregate and onto the asphalt-aggregate interface, decreasing the Gibbs' free energy^k and diminishing the strength of the bond between the asphalt and the aggregate. In one mechanism the asphalt is pulled free of the aggregate surface by tensile stresses in the asphalt generated by traffic or environmental stresses. In the other mechanism repeated tensile stresses in the asphalt initiate and propagate a de-bonding crack along the asphalt-aggregate interface. (p. 4)

In that study, researchers found that the accurate measurements of enthalpy, ΔH , can give reasonable estimates of the minimum tensile strength of the asphalt-aggregate interface, which is useful in estimating the potential of an aggregate of stripping. On the other hand, Curtis et al. (1992) concluded that the role of Gibbs' free energy of the asphalt-aggregate interface is to increase tensile strength and resistance to de-bonding fracture and stated, "As water is drawn by diffusion through the asphalt and to the

^k Gibbs free energy of adsorption is part of the total energy with which the asphalt is attached to the aggregate surface and carries a negative sign (Curtis et al., 1992).

interfaces, it reduces magnitude of Gibbs' free energy and thus the tensile strength and the resistance to de-bonding fracture. Thus, Gibbs' free energy represents the safety factor that an asphalt-aggregate interface has against stripping.”

The bonding between asphalt, an organic material, and a siliceous aggregate is governed mostly by weak secondary forces, which are generally less than 25kJ/mole. On the other hand, the magnitudes of the bonds, which water form with oxide surfaces, are substantially higher, typically in the 40-65 kJ/mole. Consequently, the affinity of water for a siliceous aggregate is greater than that of asphalt to the same aggregate.

Furthermore, reversible work of adhesion between an organic film and an oxide, including SiO₂, is highly negative in the presence of water, implying that the asphalt-siliceous substrate bonds are not stable in water. This means that water is likely to enter the interface and displace the asphalt from a siliceous aggregate when an asphalt/siliceous mixture is exposed to water or to high relative humidities. The asphalt, with its low basicity and high acidity, interacts weakly with acidic SiO₂ but adheres strongly with basic aggregates, such as limestone. The strong asphalt-basic aggregate bonds can resist the water-SiO₂ interactions, thus preventing stripping of asphalt from the aggregate, consistent with practical observation. Water probably enters the asphalt-siliceous interface by breaking the water-silanol (SiOH) bonds and building up the water layer in the silanol-terminated surface. This occurs because the hydrogen bonds between the first water layer and the silanol groups on a silica surface are substantially weaker (about 25 kJ/mol) than the bonds between the first and the second layer of water (>40 kJ/mol). Water-soluble materials migrated from the environment as well as from the asphalt film,

and those present at the interface (from both asphalt and aggregate) constitute a water-sensitive layer at the asphalt-aggregate interface. This water sensitive layer is the main reason for the formation of a water layer many monolayers thick at the interface, responsible for the stripping of asphalt from a siliceous aggregate. Modifying siliceous surface with a monolayer of a base, such as amine, will render the surface basic. This surface would form strong bonds with the acidic asphalts and thus can resist the displacement (Nguyen et al., 1996).

2.8 Antistripping Additives

Stripping results in weakness and loss of adhesive bond between the aggregate and binder, requiring early and costly maintenance. In order to eliminate this problem, a great number of antistripping additives, which are actually surfactants, have been developed and used. Many of these chemical antistripping additives, which are mostly proprietary chemical compounds, have good records in decreasing stripping damage (Tarrer, 1986).

Antistripping additives are substances added to asphalt cement to amend adhesion of asphalt cement to the aggregate surface and thus to improve the resistance to stripping damage in asphalt pavements. Most of these additives are amines or chemical compounds derived from ammonia. They are also cationic, designed to promote adhesion between acidic aggregate surfaces and acidic asphalt cements (Tarrer, 1986).

TxDOT specifications require that the liquid antistripping additives shall be a uniform liquid with no evidence of crystallization, settling or separation of components.

When a liquid antistripping additive is used, the selected dosage shall be from 0.3 to 1.0% by weight of the asphalt in the mixture, but shall not exceed the amount recommended by the manufacturer. When lime is used, the selected amount of additive shall be in the range of 0.5 to 2.0% by weight of the individual aggregate or aggregates being treated.

Tarrer (1986) tested eight different antistripping additives, and the following conclusions were made:

The functionalities contained in the antistripping additives tested were found to be primary and secondary amines and other organic nitrogen compounds. All the additives studied in that research, were capable of reaction with asphalt components at temperatures typical of asphalt handling (325° F). This reaction causes the additive concentration in the asphalt to decrease while the asphalt is stored at a high temperature. Based on the interfacial energy concept, the contact angle of an asphalt drop on an aggregate surface immersed in water was related to the stripping resistance of the asphalt. The contact angle and the stripping propensity of asphalt were observed to be reduced markedly with the addition of an antistripping additive to the asphalt. In general, all the commercial antistripping additives tested were found to improve stripping resistance to some extent, depending on their concentration in the asphalts. It was found that with each additive a specified resistance could be attained by sufficiently increasing its dosage. The optimum dosage of an additive varied with different asphalts, as well as with different aggregates. Direct application of additive to the aggregate surface was observed to require less additive than that required when the additive was added indirectly to the asphalt: with direct addition the required dosage was reduced to only about 5% of the amount of the additive with its indirect addition to the asphalt. All antistripping additives tested lost their effectiveness and failed to function to some extent after being held for a period of hours in a hot asphalt cement. The performance of an asphalt-aggregate mixture was improved remarkably by storing the mixture, that is curing, at about 300° F for several hours. The apparent strength of the bond between asphalt and an aggregate surface increased with the curing of the asphalt-aggregate mixture. It was found by measuring the diffusivity of an antistripping additive in asphalts that the additive migration rate is relatively fast, and the additive concentration in the asphalt-aggregate surface mixture reaches equilibrium quickly. The improved adhesion of the asphalt to the aggregate surface during the mixture curing is most likely then not due to diffusion effects but rather some type of coupling reactions between the asphalt and the aggregate surface. These coupling reactions appear

to be promoted by the presence of an antistripping additive. The addition of most antistripping additives softened the asphalt. The changes in viscosity due to the addition of an additive varied for different asphalt-additive combinations. These viscosity changes could be sufficient to cause the properties of the asphalt to exceed specification limits. Age hardening of asphalt can be reduced by the addition of amine type antistripping additives. (p. 158)

Halstead (1985) concluded that:

Antistripping additives can affect the physical properties (consistency) of asphalt cement. In general, the addition of the additives tends to soften the asphalt. Aging characteristics and temperature susceptibility can also be affected; the limited results indicate that a reduction in temperature susceptibility and an improvement in aging characteristics. The effect of antistripping additive is asphalt specific, that is, different asphalts affected differently by different antistripping additives. It appears that the additives affect properties of asphalts primarily by disruption or altering the molecular interactions of polar species in the asphalts. The addition of an antistripping additive to an asphalt that meets specifications may be sufficient to cause noncompliance. Each asphalt has a certain “demand” for antistripping additive. This suggests that a minimum or threshold percentage of antistripping additives must be added to an asphalt before it is available to the aggregate surface, and explains why different asphalts require different minimum additive dosages. (p. 103)

2.9 Test Methods

2.9.1 Moisture Sensitivity Tests

2.9.1.1 Boil Test

Boil test is one of the common tests used for evaluating the stripping potential of aggregates. Due to its simple procedure and relatively accurate results, it is widely used.

After soaking in distilled water for 24 hours and towel drying, 100 g of saturated aggregate in a stainless steel bowl is kept in an oven at 300° F for 1 hour of preheating time. Next, 5.5 g of asphalt cement, with or without an antistripping additive, heated at 275° F for 10 minutes, is poured onto the preheated aggregate. The mixture of the asphalt

and aggregate is then mixed with a hot spatula for 2 minutes and kept in an oven maintained at a specified temperature (reheating temperature) for a specified period of time (reheating time). After it is cooled down to room temperature, the mixture is put in boiling water (250 ml in a 400 ml beaker) on a hot plate. The water is maintained at a slow boil for 10 minutes, and stirred with a glass rod at 4 minutes and 8 minutes for 10 seconds each time. The mixture is then allowed to cool down to room temperature while still in the water. After cooling, the water is drained from the beaker and the mixture is placed on a paper towel and allowed to dry.

The amount of stripping is determined by visual observation, expressed in terms of the percentage of asphalt coating retained on the aggregate after boiling. A rating board developed with 10 intervals from 0 to 100 percent coating retained is available, to standardize the visual evaluation.

Tarrer (1983) concluded that,

The boiling water test procedure is relatively simple and rapid, and also has reasonable correlation with field performance. The boiling water test is most sensitive to preheating of the aggregate and reheating of the mixture. As preheating time and/or reheating time increased, the percentage of asphalt coating retained on the aggregate surface after increased considerably. (p. 161)

2.9.1.2 Comparison of Boil Test, Stress Pedestal Test and Indirect Tensile Test

Parker and Gharaybeh (1988) compared three stripping tests on five different aggregates whose stripping potentials were known from field performances. The researchers concluded the following:

1. The tests accurately categorized three of the five aggregates correctly.

2. The tests may not be valid indicators of stripping, or the subjective reported field performance may not be valid for specific mixes.

3. The tests can measure the effectiveness of antistripping additives. The stress pedestal test appears to assess the effect of lime favorably and the boil test appears to assess its effects unfavorably.

4. Two aggregates with high tensile strengths had different stripping potential although they possess high tensile strength.

2.9.1.3 Comparison of Louisiana Ten Minute Boil Test, Freeze-Thaw Pedestal Test and Indirect Tensile Test

Paul (1995) compared the three tests based on their ability to measure stripping potential and made the following conclusions:

1. The Boil test was discriminating with respect to aggregate source, antistripping source and asphalt cement source. Even though this is a subjective test, no significant differences were found between raters, for the same asphalt-aggregate combinations.

2. The Indirect Tensile Test (ITT) was able to distinguish performance between different aggregate-asphalt cement combinations. It was not able to distinguish differences between antistripping additives. Hydrated lime slurry provided significantly better ITT results than antistripping additives. Hydrated lime, when added dry, performed similarly to the low efficiency antistripping additives and to when no additives were present.

3. As a diagnostic test, the freeze-thaw pedestal test identified potential moisture problems for 8 of 12 mixtures or aggregate components as determined from field

experience. The ITT using a 75 percent retained strength criterion, also identified these same mixtures as being susceptible to moisture damage but incorrectly identified two mixtures that did not demonstrate stripping with field. The boil test had similar success identifying potential moisture problems.

4. The pedestal test was capable of discriminating between all antistripping additives including hydrated lime but did not demonstrate differences in performance between lime slurry and lime added dry as the other methods did. Also, most mixes tested with the pedestal test did not indicate poor performance, which may not make this test useful for establishing job mix performance.

Finally, Paul (1995) stated that:

“Each of the test methods evaluated can be used to predict the potential moisture susceptibility problems. However, certain shortcomings are associated with each test method as the boil test only examines the coarse aggregate and the pedestal test can take up to 20 days to complete. The boil test, therefore, would not determine problems associated with sands and the pedestal test would not be too time consuming for job mix formula approval or quality control testing. While the ITT examines the entire mixture, it is not discriminating with respect to antistripping additive type or possibly quantity. The boil test, because it is a quick, easily conducted test, should be used to establish the quantity of antistripping additive to be used to prevent moisture damage of the coarse aggregate.”

2.9.1.4 Strategic Highway Research Program Net Adsorption Test

Net Adsorption Test (NAT) is a product of Strategic Highway Research Program (SHRP). The test method is developed for evaluating moisture susceptibility of different aggregate-binder combinations.

The test is composed of three parts. First, an asphalt-toluene solution is allowed to flow over and adsorb onto aggregate using a constant temperature recirculating column. The experiment begins by charging the asphalt-toluene solution into the column, then adding the aggregate. The initial volume of the solution is 27 ml. As the asphalt in toluene solution flows over the aggregate, some of the asphalt is adsorbed onto the aggregate, reducing the asphalt concentration in the toluene solution. The initial concentration of asphalt is determined by measuring the visible absorbance at 410 nm using a UV-visible spectrophotometer. The concentration of asphalt after adsorption is determined by measuring the decrease in the visible absorbance of the asphalt at 410 nm after 7 hr. of adsorption. Second, a small amount of water is introduced to the asphalt-toluene solution to check the desorption potential of adsorbed asphalt due to moisture induced stripping. Third, the amount of asphalt remaining on the aggregate after the introduction of water is determined. This amount is termed net adsorption and gives a measure of the affinity of the asphalt for the aggregate. The difference between the amount of asphalt adsorbed before and after desorption by serves as an indicator of the water sensitivity of the asphalt-aggregate combination (Curtis et al., 1992).

The concentration of the asphalt in solution was determined using Beer's law as shown below:

$$A = abc$$

Where:

A = absorbance at 410 nm;

a = absorptivity (L/g cm) of the asphalt at 410 nm;

$b =$ cell path length (cm);

$c =$ concentration (g/L).

The equations for calculating the amount adsorbed are:

$$C/C_0 = A/A_0$$

$$A_{sp} = V_0 C_0 (A_0 - A) / W A_0$$

Where:

C_0 and $C =$ concentrations of the asphalt solution before and after adsorption, respectively;

A_0 and $A =$ visible absorbances at 410 nm of the asphalt solution before and after adsorption, respectively;

$V_0 =$ solution volume before adsorption;

$A_{sp} =$ amount of asphalt adsorbed per gram of aggregate (g/g);

$W =$ mass of aggregate used (g).

The desorption step involved introducing 136 μ l of water into approximately 27 ml of asphalt- toluene solution and allowing the solution plus water to recirculate over the aggregate bed. After this 2-hour desorption step was completed, the absorbance of the asphalt solution was monitored at 410 nm, and the asphalt concentration was determined using the following equations:

$$D_w W = -V_w (C - C_w)$$

$$D_w = -V_w C (A - A_w) / W A$$

Where:

$C_w =$ concentration of the asphalt solution after desorption;

A_w = visible absorbance at 410 nm of the asphalt solution after desorption;

V_w = toluene solution volume during desorption;

D_w = amount of asphalt desorbed per gram of aggregate.

The net adsorption is obtained by subtracting the amount of asphalt desorbed by water from the amount initially adsorbed, where net adsorption = $A_{sp} - D_w$.

This test procedure used an initial concentration of 0.6 grams per liter of asphalt per toluene solution. The adsorption step of the test procedure typically takes 7 hours. However, the time required is dependent on the equipment used; the time selected should be the time required to reach equilibrium in each step of the experiment. It is not always possible to reach equilibrium in the adsorption step. Sometimes this step can be shortened if satisfactory results are obtained sooner. Reaching the equilibrium is mandatory in the desorption step to achieve satisfactory results.

The use of three replicate specimens is recommended for the test, along with a control specimen. Several sets of experiments can be run simultaneously using a multiple-head peristaltic pump.

2.9.1.5 Scaled-up Net Adsorption Test Method

After NAT is developed, a study made by Curtis et al. (1992) continued to research study to make the test more economical and efficient.

The goals of the Scaled-up NAT test are as follows:

- Maximize the quantity of aggregate used in the test while minimizing the amount of the solvent

- Use aggregate gradations as close as practical to those used in producing pavements
- Keep the relationship between materials quantities the same as the initial study
- Adjust the timing of the procedure so that the test can be completed within a typical working day
- Use of commercially available equipment as much as possible.

The resulting compromise between the desirable maximum aggregate quantity and minimum solvent quantity yielded a final aggregate sample size of 50 g. In order to further reduce the amount of solvent used, the volume of solvent is halved compared to quantities used in the initial screening stage of this work, while the concentration of asphalt dissolved in the solvent was doubled. This resulted in no net change in the ratio of asphalt to aggregate.

The duration of the initial adsorption phase was reduced to 6.5 hours and a period of 3 hours was maintained for desorption phase. With 45 minutes of preparation time in the morning and 30 minutes allowed for cleanup, this procedure could be completed during normal laboratory operating hours.

The scaled-up method showed higher initial adsorption than the initial method did. One possible explanation is that flow rate in the scaled-up version was nearly three times faster than the initial method. The increased flow rate would allow the asphalt more opportunities to have contact with the aggregate, since the solution is recirculated.

The net adsorption values were also correspondingly higher for the scaled-up test than for the initial test. However, although the absolute values for the net adsorption were quite different, the resulting rankings of the asphalt-aggregate pairs were nearly identical.

Curtis et al. (1992) concluded that:

The net adsorption test offers an effective means of evaluating the affinity and water sensitivity of an asphalt-aggregate pair. Although the actual performance of the SHRP aggregates not known, the perceived performance and the results of the net adsorption test seem to agree quite well, particularly with siliceous materials. Since the net adsorption test gives a relative measure of the affinity that asphalt has for an aggregate and the sensitivity of the asphalt-aggregate pair to water, this test can best be used as a screening test for materials. (p. 19)

2.9.1.6 National Roads Authority Net Adsorption Test Procedure

The National Roads Authority (NRA) of Ireland developed a further modified version of the NAT. It consists of the following 4 steps and the total test duration is 24 hours.

1. Three test flasks are prepared for each aggregate-binder combination and a fourth flask is prepared for control. The control is carried through the entire procedure to ensure that there is no material on the surface of the aggregate that will interfere with the test.

2. A bitumen-toluene solution of known concentration is prepared;
(approximately 1 g of bitumen to 1 L of toluene).

3. Fifty grams of graded aggregate is added to each flask including the control flask. Walsh et al. (1995) indicated that the precision of this test method was excellent

when specific gradation given in Table 2.1 is used. 140 ml of toluene is added to the control flask and 140 ml of the bitumen-toluene solution is added to each of the other three test flasks. The flasks are placed on a mechanical shaker and shaken for 6 hours.

4. After step 3, 2 ml of water is added to each flask and shaking is continued for a further period of 16 to 17 hours.

Table 2.1: Aggregate Gradation used in NRA Net Adsorption Test (Walsh et al., 1995)

Sieve Size	% Retained	Weight Retained (g)
2.36 mm	8.0	4.3
1.18 mm	25.0	13.5
600 μm	17.0	9.1
300 μm	23.0	12.4
150 μm	14.0	7.5
75 μm	6.0	3.2
		Total 50

In order to calculate both the initial and net adsorption, the following concentration of the bitumen-solvent is measured:

1. Initial concentration of bitumen-toluene solution A_1 .
2. Concentration after 6 hour in contact with the aggregate, A_2 .
3. Concentration after addition of water to the aggregate-bitumen solution, A_3 .

The solution concentrations are determined by a spectrophotometer at 410 nm.

The initial adsorption is given by;

$$A_i = \frac{VC(A_1 - A_2)}{WA_1}$$

Where:

V = volume of solution = 140 ml.

C = concentration of bitumen-toluene solution.

A_1, A_2 = solution concentration measurements.

W = weight of aggregate sample to nearest 0.001 g.

The net adsorption is given by;

$$A_n = \frac{VC(A_1 - A_3)}{WA_1}$$

Where:

Volume at this stage is 136 ml.

$$\text{Percentage net adsorption} = \frac{A_n}{A_i} \times 100$$

Based on net adsorption results of aggregate-binder combinations, SHRP suggests of the following criteria to evaluate aggregate-binder adhesion performance (Table 2.2).

Table 2.2: Criteria Suggested (SHRP) for Aggregate-Binder Adhesion Performance

% Net Adsorption	Aggregate/Binder Bond Performance
>70	Good
55-70	Marginal
<55	Poor

Although the Emulsified Asphalts already contains water, Walsh et al. (1995)

tested bitumen emulsions using NAT and concluded the following:

In testing bitumen emulsions, it is first necessary to remove the water from the solid residue (bitumen containing emulsifying agent). The water was recovered by controlled evaporation in a stream of air under constant pressure and constant temperature for 18 hours. It must be recognized that the real situation may be that the pressure of moisture in the emulsion is likely to aid in the spreading of the bitumen over the aggregate and as such it may promote better initial adhesion than bitumen. For this reason it may be incorrect to compare test results obtained with bitumen emulsions with results obtained paving-grade bitumens. However, there is no reason why the test should not be used to rank various bitumen emulsions with aggregate. (p. 5)

2.9.2 Aggregate Retention Tests

2.9.2.1 Pennsylvania Aggregate Retention Test

During a study made by Khandhal and Motter (1997) to investigate the effect of precoating on the aggregate retention, a test method is developed. Test equipment mainly consists of 8 in sieves, 8 in pans, a sieve shaker, rubber pads, a compression machine and a balance. The test procedure, type of binder, and aggregate-binder rates used in that study is given below.

1. Application of Bituminous Material: the emulsified (CRS-2) is poured on the backside of an 8 in separator pan to obtain an application rate of 0.25 gal/yd². The emulsion is applied at 140° F ± 5.

2. Application of Cover Aggregate: a Mary Ann laboratory sieve shaker was used to apply aggregate at a rate of 17.4 lb/yd². An unclamped stack of 8 inches diameter and 2 inches deep standard laboratory sieves are laid on a pair of 45° inclined rubber-covered power-driven rollers, which revolve the stack. The pan bottom rests on a free-turning turntable. The aggregate is tumbled, mixed and passed as it is carried up on the revolving inclined screen wire. To encourage the clearing of openings, the sieve frames are tapped laterally (from below) using hardwood faced aluminum hammers. These cam-cocked and spring thrown hammers are pivoted on nylon sleeve bearing. The sieve shaker is inclined at an angle of 60° rather than 45°. The pan containing applied emulsion was placed at the bottom of five, inverted ½ in sieves. A retainer or collar (sieve with no screen) was placed on the top. The screen mesh in each ½ in sieve was rotated 45° from the adjacent top or bottom sieve so that two consecutive sieve meshes did not have the

same orientation. After the sieve assembly is placed on the shaker and it is turned on, 300 grams of aggregate is poured into the retainer at the top. After 1 minute, the pan containing the emulsion and the applied aggregate is removed and tapped to spread the aggregate evenly on the emulsion film.

3. Compaction and Curing: within 15 minutes, this pan is covered with a 7-1/2 in diameter x 3/4 in thick Neoprene bearing pad (of 50 durometer hardness) and placed under a compression machine to apply a load of 2000 lbs. for 5 seconds. This is equivalent to a pressure of 40-50 psi, which is normally used in pneumatic tired rollers for seal coats. After compaction, the bearing pad is removed and the pan containing emulsion and aggregate is cured under ambient temperatures for 24 hours. The weight of pan and the seal obtained after curing.

4. Initial Retention Loss: after the 24 hours of curing, the pan containing seal coat is inverted to allow aggregate particles (which did not develop initial adhesion to the binder) to fall. These aggregate particles are weighed to determine the initial loss in grams. The percentage of initial loss is determined as follows:

$$\% \text{ Initial loss} = \frac{B}{A} \times 100$$

where: A = Weight of total aggregate (300 grams)

B = Initial loss in grams.

5. Knock-Off loss: after the initial loss is determined, the pan containing emulsion and aggregate is placed upside down at the top of five 1/2 in sieves (these are used for filling only) and a pan is placed at the bottom of the assembly to collect the knock-off loss. This complete assembly is placed in the Mary Ann Sieve Shaker as

described earlier and subjected to the shaking and tapping action for 5 minutes. The knocked-off aggregate collected in the bottom pan is weighed (C). The percent knock-off loss is determined as follows:

$$\% \text{ Knock-off loss} = \frac{C}{A - B} \times 100$$

Where: A = weight of total aggregate (300 grams)

B = initial loss in grams

C = knock-off loss in grams.

6. Total Loss: the total loss (initial loss + knock-off loss) is calculated as follows:

$$\% \text{ Total loss} = \frac{B + C}{A} \times 100.$$

2.9.2.2 Violet Test

The Violet Drop Test was developed by the LCPC in Paris in the early 1960's. Use of this test method intended to provide information on appropriateness of aggregate design quantities, resistance of aggregate to an impact force over several time intervals and rate of set of binder.

This test uses a 0.62-cm (0.25 in.) steel plate 17.5 by 17.5 cm (7 by 7in.) square as a sample preparation medium. A 0.2-cm (0.25-in) rim prevents binder runoff. A force was imparted to an inverted chip seal sample by means of a dropping steel ball 5 cm (2 in.) in diameter from a height of 45 cm (18 in.)

Both the plates and emulsions are preheated to 60°C (140° F). A quantity of binder equal to field application rates is applied to the steel plate. The plate is rotated

until the binder was evenly distributed over the surface. Aggregates, again duplicating field section quantities, are immediately applied to the plate. Rubber tired rolling was simulated with a weighed tire 396 to 440 lbs (180 to 200 kg) and a tire pressure of 14.8 kg/cm² (40 psi). Three passes (one pass = forward and backward cycle) of the tire rolling is applied. Then, the specimen is rotated 90° and three more passes are applied.

A total of 15 samples are prepared and three replicate samples tested at 10 minutes, 30 minutes, 2 hours, 5 hours, and 24 hours intervals. All samples are stored at 25° C (77° F) and a relative humidity of less than 30%.

An initial weight of the sample and plate are obtained, then the specimen is inverted in the test apparatus for 10 seconds and a second weight is taken and the percent of material retained on the sample plate is calculated by;

$$\% \text{ Retained} = 100 - \frac{(\text{originalwt} - \text{wtafterinvert})}{(\text{originalwt})} \times 100.$$

The plate is immediately re-inverted in the apparatus and a steel ball is dropped in the center of the plate three times within a 10-s period. A final weight is taken, and the percent material retained after impact is calculated.

The data can be used to indicate the presence if excess aggregate and the rate of set of the binder by monitoring the increase in material retained with time. The material lost during the initial invert testing at the 5 hours and 24 hour test times represents excess aggregates. Plotting the percent material retained after impact versus time indicates the curing characteristics of the binder (Stroup-Gardiner, 1990).

Stroup-Gardiner (1990) made the following conclusions for the Violet test.

1. Problems with excess aggregate can be identified with the 5 hour or 24 hour initial invert Violet test.
2. Problems with aggregate pick up on rollers during construction are related to the 10-min impact violet test results showing less than 30% material retained.
3. Problems of surface damage due to early brooming are indicated by the 30-min impact Violet test results of less than 60% material retained.
4. Field sections showing good performance after one month under low-volume traffic conditions generally have 24 hours impact Violet test results greater than 80% material retained.

The test has been also used to measure the adhesion of binder and aggregate. Coyne (1980) made several modifications to the Violet Test. Gallon can lids were used instead of 200-mm square steel plates. The can lids, with the aid of $\frac{1}{4}$ in plywood templates, make it easy to conduct rate-of-set measurement on field jobs. Another modification was the use of a gallon can to hold the gallon can lid in the Violet test device. This shortened the height of drop to approximately 400 mm from the original 500 mm. Limited testing was conducted to determine the effect of these changes on the stone loss. Agreement between the “standard” and “modified” Violet test was found to be quite good.

CHAPTER III

CONSTRUCTABILITY REVIEW

The success of a new testing protocol depends on its ability to accurately predict the field performance. Therefore, a field study program was made an integral part of this research. Visits were made to 15 of the 25 TxDOT districts and test sections were selected from 14 of those districts. The districts visited are illustrated in Figure 3.1.

3.1 Construction Site Visits

3.1.1 Data Collection

During the summer of 2002, a data collection team was sent out to fifteen of the twenty-five TxDOT Districts stretching from Childress to Pharr and Paris to El Paso. All test sections were part of the 2002 summer contract seal coat program. The selection of test sections was based on various factors including climate, materials used, traffic level and history of incompatibility problems. The main purpose of the data collection effort was to come up with different scenarios of incompatibility, and a database of field information was constructed (Appendix A).

The site visits were coordinated with the seal coat construction schedule for each district. The data collection team was in direct contact with both the District Construction Engineer, the head of the district's seal coat program, and the chief Seal Coat Inspector.

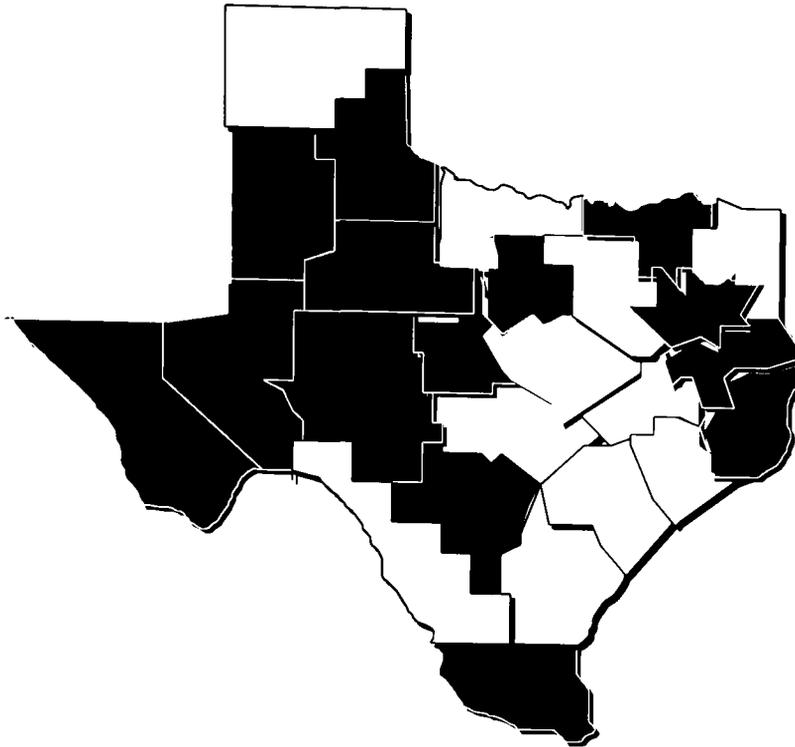


Figure 3.1 Districts Visited (shaded gray)

For each district visited, a data collection sheet was filled out. An example of this sheet is given in Figure 3.2. This data sheet included information such as climatic data, location of the test section, existing surface condition, and construction process data. A note was made of any cloud cover, wind, or humidity. Information regarding the aggregate was recorded including the source, grade, application rate, and the condition of aggregate used for that particular seal coat. The binder source and type was recorded along with the application rate, noting whether or not the binder rate distribution across the lane was uniform or if variable nozzles were used. The temperature of the binder was taken in the distributor. The binder temperature just after the shooting, when the aggregate was spread, and when the new seal coat was rolled, were also measured. The

time difference between the application of the binder and spreading of the aggregate was recorded along with the time difference between the spreading of the aggregate and compaction. The roller type and number of passes was recorded. The data collection team also recorded information on traffic control including whether pilot cars were used or not and if so, for how long. A note was made regarding whether or not there were any shaded areas over the test section. In addition to the data collection sheet, visual data including narrated video footage of the construction and still images taken by a digital camera were also collected.

District	Country	Highway	Location	Date
Surface Before Construction <input type="checkbox"/> New Patch <input type="checkbox"/> Old Patch <input type="checkbox"/> Flushing <input type="checkbox"/> Bleeding <input type="checkbox"/> Dusty <input type="checkbox"/> Level-Up <input type="checkbox"/> Other				
Time of Construction:				
Ambient Temp.	Average:	High:	Low:	Humidity:
Weather Conditions During Construction: <input type="checkbox"/> Sunny <input type="checkbox"/> Cloudy <input type="checkbox"/> Partly Cloudy <input type="checkbox"/> Windy Comments:				
Existing Surface: <input type="checkbox"/> HMA/C Gr <input type="checkbox"/> Seal Coat <input type="checkbox"/> Slurry Seal <input type="checkbox"/> Other Color: <input type="checkbox"/> Oxidized <input type="checkbox"/> Slightly Oxidized <input type="checkbox"/> Not Oxidized				
Condition of the existing surface: <input type="checkbox"/> Bleeding <input type="checkbox"/> Flushed <input type="checkbox"/> Smooth <input type="checkbox"/> Slightly porous <input type="checkbox"/> Slightly pocked, porous <input type="checkbox"/> Badly pocked				
Aggregate Source:		Grade:	Condition: <input type="checkbox"/> Dusty <input type="checkbox"/> Air Dry <input type="checkbox"/> Moist <input type="checkbox"/> Precoated	
Actual Aggregate Rate:				
If the actual aggregate rate is different than the design rate, why?				
Binder Source and Type:				
Actual Binder Rate:				
If the actual binder rate is different than the design rate, why?				
Binder rate distribution across lane:		<input type="checkbox"/> Uniform <input type="checkbox"/> Variable	W/P rate: OWP rate:	
Binder Temperature (a) In Distributor		(b) When Aggregate Spread	(c) When Rolled	
Time difference between binder application and spreading of aggregate:				
When did emulsion break? (Color change etc.) <input type="checkbox"/> Before spread <input type="checkbox"/> After spread			Time to break (min):	
Time difference between the spreading of aggregate and rolling:				
Roller Type: <input type="checkbox"/> Lightweight <input type="checkbox"/> Mediumweight		Brand/Model:	No. of Passes:	
Time difference between the construction and opening the pavement to traffic:				
Pilot Cars Used?		For How Long?		
Actual Embedment Depth:				
Does the field section have shaded areas?				
Additional Remarks:				

Figure 3.2: Data Collection Sheet for Construction Site Visits

3.1.2 Sample and Material Collection

For every district visited, the team identified at least one test section from which actual seal coat samples were retrieved using 15 cm x 15 cm x 0.3 cm (6 in x 6 in x 1/8 in) steel plates. These sample plates were placed on the existing pavement in the intended path of the seal coat train. The layout of the plates varied somewhat depending on the road, but there was one plate in the inside wheel path, one plate in the outside wheel path, and one plate between the wheel paths. Figure 3.3 shows the manner in which specimens were collected from the field. If the shoulder was included in the shot, there were two more plates placed on the shoulder. This alignment of three to five plates was duplicated about every fifty feet down the road for a distance of 100–200 feet. The location of the plates was marked with orange paint on the side of the road for quick retrieval of the sample plates. The plates were not retrieved until the distributor, chip spreader, rollers, and sweepers had traveled over the test section. The plates were then extracted from the road, giving an exact 6 in x 6 in sample of the finished seal coat. These samples were transported back to the Texas Tech University Asphalt Laboratory and stored as a library of seal coat samples.

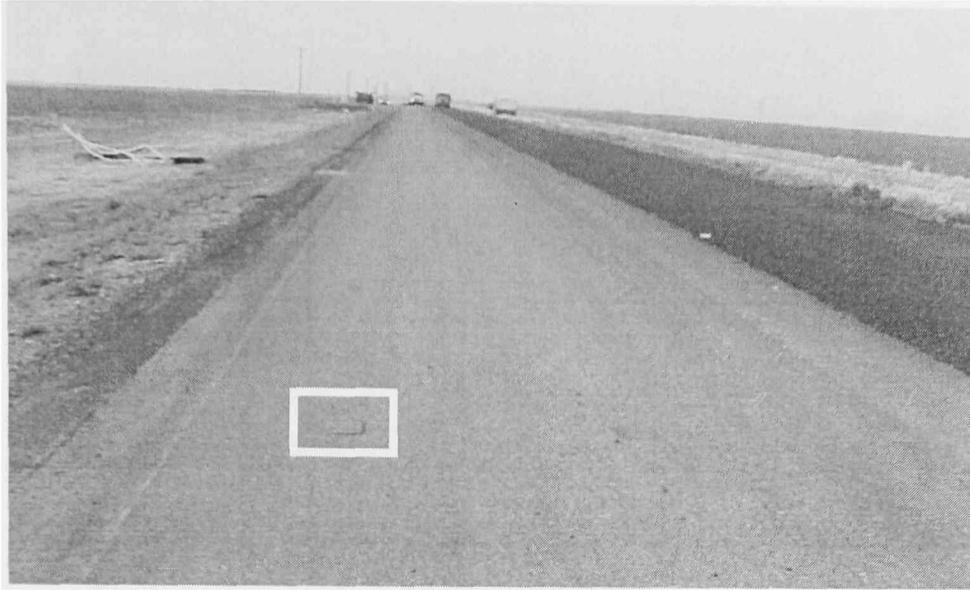


Figure 3.3: Collection of Field Specimen at Test Projects (Insert Shows Specimen Plates Placed on Existing Pavement)

During the laboratory part of the study, seal coat samples brought back from field visits were tested using the new testing protocol developed. These results were compared with actual field performance of seal coats to verify and calibrate the testing protocol.

Along with the seal coat specimens, the data collection team brought back four to five bags of the aggregate and four to five gallons of binder from the site. These materials were used to make additional laboratory specimens for testing. The performance of these specimens and the field observations were used to show the ability of the new test protocol to recognize problems of material incompatibility.

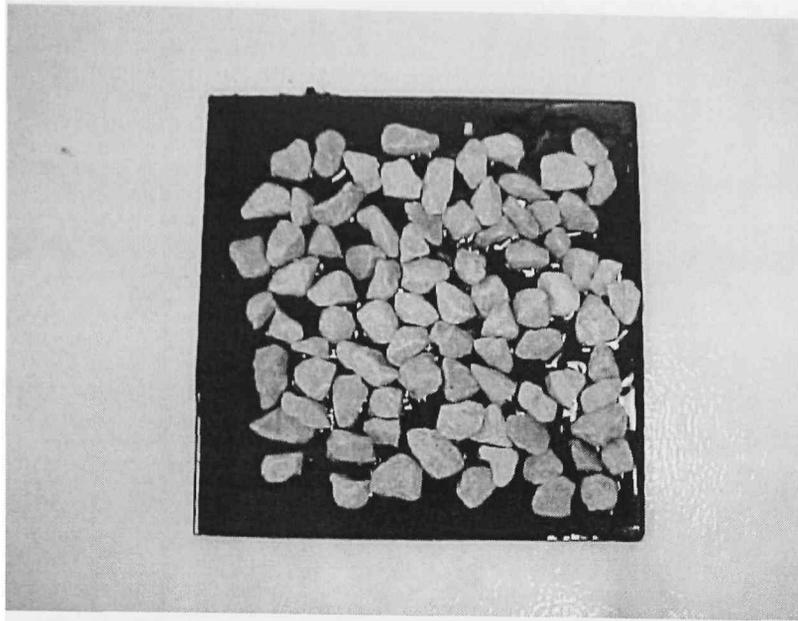


Figure 3.4: Typical Seal Coat Specimen collected from the field

3.2 Follow-up Monitoring Study

It is generally accepted that any problems associated with aggregate-binder incompatibility show up during the first year of service. Therefore, approximately one year after the initial site visits, a research team made a follow-up visit to evaluate the performance of each test section. The purpose of these follow-up visits was to check the integrity of the seal coat one year after construction, with particular emphasis on problems related to aggregate-binder incompatibility. During each follow-up visit, researchers met with TxDOT representatives, including the maintenance supervisor for the county where the test section was located. Information including climate during the previous year, problems associated with the seal coat test sections and any maintenance

done on the test sections were collected. These observations were used as field verification of the test protocol developed during this study. The specimens that were brought from previous year's site visits were tested with the new test method. The comparison of the results of the new test method and the actual field performances of the sections should indicate the capability of the test method of recognizing the problems of incompatibility.

More than half of the test sections visited, including the districts of Childress, Paris, Tyler, Fort Worth, San Angelo, Pharr, Brownwood, El Paso and Odessa, had nearly excellent performance over the past year. According to the TxDOT representatives at each of these sections, there were no known problems related to material incompatibility in the 2002 seal coat program. In the Fort Worth District, there were no signs of aggregate loss on the shoulders of State Highway 114 test section, despite a severe ice storm during the winter of 2002-2003. However, there was some aggregate loss on the IH-30 access road, but only at the edge of lanes. Fort Worth District was included in the "excellent performance" list because what little aggregate that was lost could be attributed to other causes other than aggregate-binder compatibility. The test sections in San Angelo District performed well in general, although the district construction engineer had indicated that there was some aggregate loss in the west end of the district. Aggregate loss was apparent at turning points, intersection approach areas (where vehicles slow down before turning) and low spots in the road. In Brownwood District there were three test sections: U.S. 84 (Brownwood I), FM 1148 (Brownwood II), and FM 503 (Brownwood III). All three had excellent performance ratings although there

were a few areas on FM 1148 that were damaged due to a combination of tight turns, steep slopes, and a motor grader that was blading off some sleet and snow after a winter storm. Odessa District has performed well since the summer of 2002. In the El Paso district, there was raveling at the intersections but there were no signs of incompatibility, due to that reason this section is included in excellent category. Pharr district has not experienced any problem at all. The seal coat was in excellent condition after one year.

Test sections in the remaining districts (Abilene, Lubbock, San Antonio and Lufkin) showed evidence of failure due to aggregate-binder incompatibility. Abilene District test section showed signs of aggregate loss at places where the seal was applied over patches. A “Fog Seal” was applied shortly after construction to retain as much aggregate as possible. In Lubbock District’s Floyd County there were sporadic cases of aggregate loss from as little as 10-15% up to 100% in a few locations. More rock loss was seen near turning areas (i.e., in front of driveways and curves in the road). Lufkin District had some incompatibility problems with their seal coat in San Jacinto County. Approximately one mile south of the southern city limit of Shepherd, there was severe rock loss in the inside wheel path of the outside lane in the northbound direction. The previous road surface could easily be seen (Figure 3.5). The district maintenance engineer commented that a possible cause for this severe loss could be a thunderstorm soon after the construction of the seal coat. San Antonio district had lost some aggregate. The aggregate loss was apparent between the wheel paths. In addition, edge cracking and alligator cracking on the edges have been observed. The section had minor bleeding

problems on the wheel paths at some points. Table 3.1 summarizes the evaluation data from the follow-up monitoring study.

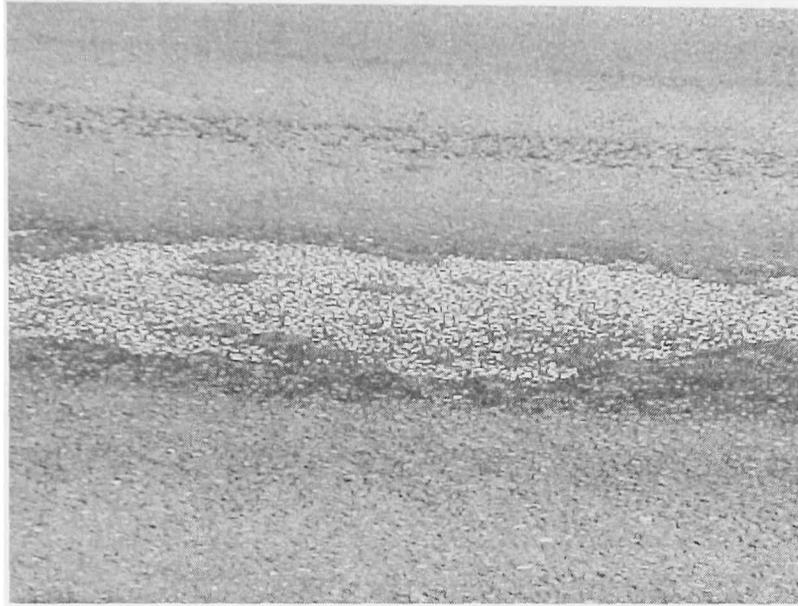


Figure 3.5: An Illustration of Aggregate-Binder Incompatibility (Lufkin test section)

Table 3.1: Results from Field Monitoring Follow-Up Study

District/Test Section	Rating* (1-5 scale)	Comments On Aggregate Loss (where applicable)
Childress	5	N/A
Tyler	5	N/A
Ft. Worth	5	N/A
San Angelo	5	N/A
Brownwood I	5	no aggregate loss, bleeding was apparent in wheel paths
Brownwood II	5	some rock was scraped off by motor grader over winter season, no incompatibility problems
Brownwood III	5	N/A
Odessa	5	minor rock loss approx. 1 mile south of test section at low section of road where high waters caused by rainfall cover the road the rest of the road is in excellent condition, not considered as incompatibility
Abilene	3	rock loss is prevalent where hot mix patch exists beneath seal coat, fog seal was applied in December 02' to stop rock loss, fog seal was successful
Lubbock	2	very low embedment depth (10-15% loss, and as much as 100% loss)
El Paso	5	patches of rock loss at intersections
Lufkin	1	severe rock loss on inside wheel path of outside lane
Paris	5	N/A
Pharr	5	N/A
San Antonio	3	rock loss between the wheel paths, edge cracking and alligator cracking on the edges, minor bleeding on the wheel paths, low volume traffic

* 1 being the worst and 5 being the best.

3.3 Conclusions

The constructability review showed that seal coat performance may be strongly influenced by the construction process. Moreover, interviews with TxDOT seal coat personnel and engineers revealed that conditions during, and immediately after construction play a critical role in seal coat performance. Although the binder temperature in the distributor was around 300–350° F for hot applied asphalts, the measurement of binder temperature during the first few minutes after the binder spray showed that the binder cools down rapidly after it comes out of spray nozzles. Both the surface temperature of the existing pavement and the ambient temperature affect the binder temperature at the time of application of aggregate and rolling. In most test sections, the binder temperature was 5-14° C (10-25° F) above the surface temperature at the time of aggregate application. The ambient temperature at the time of construction varied from 79° F to 100° F. In the same manner, the surface temperature varied from 84° F upwards to 142° F. At 3 of 14 test sections, emulsified asphalts were used while the rest were hot asphalts. CRS-2P was the type of emulsion that used in all 3 test sections.

One general observation made in the follow up study was that loss of aggregate related to incompatibility always occurs outside or between wheel paths. This is an indication of how traffic loads help improve aggregate retention. Particularly in hot weather when the binder has a softer consistency, the traffic loads continuously compact the aggregate resulting in higher embedment depths and better aggregate retention. That may be the reason for raveling to occur outside and between wheel paths, and not on the

wheel paths. On the other hand, some test sections lost aggregate at intersection locations and driveways where turning movements and braking occur. Such aggregate losses may be due to a combination of high shear forces applied by braking and/or turning vehicles and lower shear strengths. These kinds of aggregate losses were not evaluated as losses due to incompatibility. In general, most of the test sections performed very well except Lufkin, Lubbock, San Antonio and Abilene test sections. In Lubbock and Abilene test sections, low traffic was noted. In Lufkin test section, the district engineer claimed that there was a thunderstorm soon after the construction.

CHAPTER IV

DEVELOPMENT OF NEW TESTING PROTOCOL

From the standpoint of aggregate-binder compatibility, the literature review conducted on the subject revealed that currently available test methods do not effectively simulate field conditions in the laboratory. Therefore, they cannot be effectively considered as performance-based tests. Climatic effects such as wetting/drying and freezing/thawing are not mimicked in the lab. The new testing protocol not only has to bring these factors into consideration, but also has to be simple enough to be performed in a TxDOT district lab, which has limited equipment availability. Besides, the method has to be relatively easy to perform and also inexpensive in line with the funds involved with seal coat work.

4.1 Specimen Preparation

The same steel plates that were used to collect field specimens were used to prepare laboratory seal coat specimens. In order to avoid spills, the plates were sealed on 4 sides using tape. The binder was heated to the desired temperature, which is 350° F for hot applied asphalts. The binder was poured to the sample plate placed on top of a weight scale, and the mass of binder corresponding to the desired binder rate was placed on the plate. The amount of binder needed is calculated by using the following procedure:

For a binder rate of 0.40 gal/yd²;

Plate dimensions: 15 cm x 15 cm

Binder thickness: $0.40 \text{ gal/yd}^2 \times 0.45271944 = 0.1811 \text{ cm}$

The volume of the binder = $15 \text{ cm} \times 15 \text{ cm} \times 0.1811 \text{ cm} = 40.75 \text{ cm}^3$

If we assume that the specific gravity of the asphalt is 1.02, then the mass is calculated by; $40.75 \text{ cm}^3 \times 1.02 \times 1 \text{ g/cm}^3 = 41.56 \text{ g}$. Therefore, in order to achieve rate of 0.40 gal/yd^2 on a $15 \text{ cm} \times 15 \text{ cm}$ plate, 41.56 grams of binder is needed.

The constructability review showed that in the construction process, the binder temperature at the time of spreading aggregate and rolling are critical. Consequently, when the desired temperature of aggregate application was reached, the aggregate was applied. At the desired temperature of rolling, the specimen was rolled with a roller weighing 236 kg (520 lb.) and covered with $\frac{1}{4}$ in thick rubber (Figure 4.1).

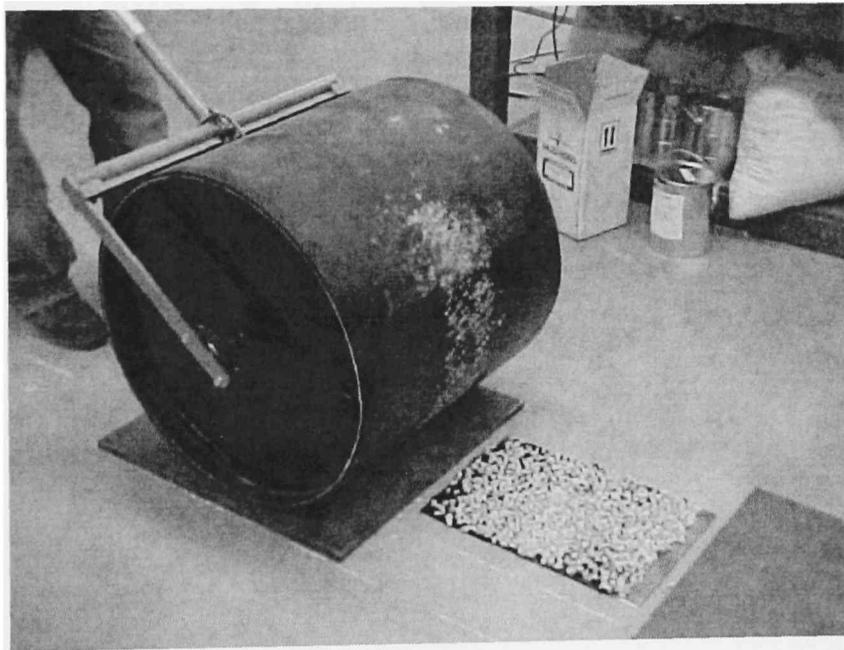


Figure 4.1: Specimens being compacted by the roller

4.2 Impact Testing

A certain amount of energy has to be applied to the specimens to initiate debonding failure between aggregate and binder. Instead of using a steel ball similar to the Viallet test, a Modified Proctor Hammer (MPH) was selected. It has a 45 cm (18 in.) vertical drop of a hammer weighing 4.5 kg (10 lb.). Preliminary tests showed that impact energy of MPH may cause failure between the specimen plate and the binder, instead of at the aggregate-binder interface, particularly when the specimen is tested at lower temperatures or when stiffer binders are tested. Therefore, the inverted specimen was placed on a rubber pad which was glued to a steel box (Impact Apparatus, Figure 4.2) and a 6.25 mm (0.25 in.) thick steel plate was placed on the top of the inverted specimen in order to evenly distribute the impact energy absorbed by the specimen and to reduce bending of the inverted specimen plate. The MPH is dropped 3 times on to the top plate. The percent aggregate loss is calculated from the ratio of number of aggregate particles lost and the total amount of aggregate particles in the original specimen.

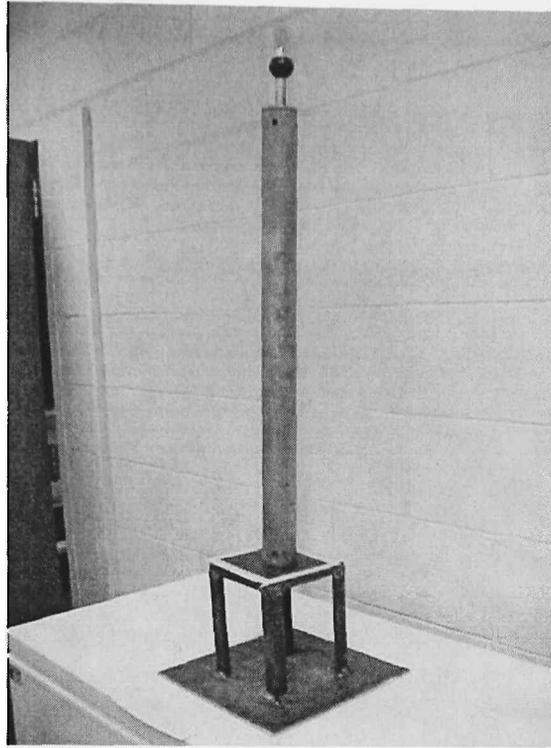


Figure 4.2: Impact Test Apparatus

The rubber pads, placed on 4 sides of the impact apparatus, damp the energy applied from the hammer and they hold the inverted specimen. This setup also limits the specimen surface area covered with the aggregate, since the aggregates placed on the edges of the specimen could cause the specimen to be disoriented. Therefore, for the specimens prepared in the lab, the aggregates were placed at least $\frac{1}{4}$ in far from each side of the steel plate.

4.3 Specimen Conditioning

After the preparation of the specimens, the specimens were subjected to various conditioning to select and simulate an appropriate field environment. Several freeze-thaw-soak and boil-soak protocols were tried on seal coat specimens. It was found that successive freeze-soak/thaw cycles and extended soaking were the most effective mechanisms of seal coat failure.

First, the effect of soaking was investigated. Table 4.1 shows the effect of different soaking durations. Identical specimens were tested after 0, 24, 48, 72, 96, and 120 hours of soaking at 25° C (70°F). In this test series, the specimens were tested at the end of each soaking time. However, if the soaking period was less than 120 hours, the specimens were continued to be soaked until a total of 120 hours of soaking was reached. and tested after each 24 hours intervals. For example, specimen #3 was first tested after 48 hours of soaking with the MPH. The number of aggregates lost due to de-bonding after 48 hours of soaking was counted and used to calculate the “percent initial loss.” After testing the specimen at 48 hours, the soaking continued until 120 hours of soaking was reached. But the specimens were tested at the end of 72, 96 and 120 hours. The number of aggregates lost was counted for each test interval and recorded. At the end of the 120 hours of soaking, all the aggregates lost was added and divided by the number of total aggregates. This ratio is named as “percent cumulative loss” and given at the very last column in the table. For specimen #6, the initial loss was equal to cumulative loss for being tested just once.

Table 4.1: Effect of Soaking Duration

Specimen #	# of Aggregates Lost						# of Total Aggregates	% Initial Loss	% Cumulative Loss
	0 Hours	24 Hours	48 Hours	72 Hours	96 Hours	120 Hours			
1	1	6	8	4	3	5	51	2.0	52.9
2		3	11	4	6	3	54	5.6	50.0
3			3	6	2	5	55	5.5	29.1
4				2	5	8	54	3.7	27.8
5					5	16	50	10.0	42.0
6						23	52	44.2	44.2

Specimens 1, 2, 3, and 4 had percent initial loss values between 2% and 5.6%.

Specimen 5 had a 10% initial loss and specimen 6 with 120 hours of soaking showed the highest percent initial loss of 44.2%. Based on cumulative loss at the end of 120 hours of soaking, specimens 1 and 2 both lost a little over 50% of their aggregates. Specimens 3 and 4 lost about the same percent of aggregate (27 to 29%), while specimen 5 lost 42% of its aggregates. These results show the compounding effect of the soaking duration and the number of times the specimens are subjected to impact loading. Moreover, when the cumulative losses of specimen 1 and 2 is compared to specimen 5 and 6, the effect of consecutive impact loading balances out the effect of soaking. It can be concluded that the time period of 120 hours gives reasonable amount of loss for evaluation purposes.

Since just one replicate was prepared per each condition, it is hard to draw conclusions from these results. On the other hand, this series of testing provides enough evidence to conclude that soaking has an effect on aggregate retention. Moreover, the

impact loading also affects the percent loss of the specimens by initiating the de-bonding at the aggregate-binder interface as projected.

4.4 Effect of Freeze-Soak/Thaw

Different types of conditioning such as wetting/drying, freezing/soaking, boiling, were tried to simulate field conditions in the lab. Different durations of conditioning such as 16 hours freezing 8 hours soaking, 2 hours of boiling have been also practiced in the developing process. Based on these trials, 16 hours of soaking followed by 8 hours of freezing was found to be the most effective one.

Freeze-soak/thaw cycles start with the soaking of seal coat specimen under water for 16 hours at room temperature 25° C (70° F). After 16 hours of soaking, the specimen is subjected to freezing at -25° C (-13° F) for 8 hours. This soaked freeze conditioning of 24 hours forms one “cycle”. This freeze-thaw process is continued with soaking the frozen specimen for 16 hours, which also includes thawing process. The durations of 16 hours and 8 hours were chosen by considering on their effectiveness as well as the practicality of conducting the test during a typical working day.

When freeze-soak/thaw cycles are followed with soaking, it has a significant effect on the amount of aggregate lost. Table 4.2 shows the results of specimens tested under different freeze-thaw and soaking conditions. Four identical specimens were prepared exactly the same using AC-5 w/2% Latex and a fifth specimen was prepared using a different binder (AC-15 5TR) but the same aggregate. Four different procedures were applied for different combinations of number of freeze-thaw cycles and the soaking

Table 4.2: Effect of Freeze-Thaw Cycles and Soaking Duration

	Conditioning				
	1 F-T Cycles + 48 hr Soak	2 F-T Cycles + 48 hr Soak	3 F-T Cycles + 48 hr Soak	3 F-T Cycles + 48 hr Soak	3 F-T Cycles + 24 hr Soak
Aggregate	Limestone	Limestone	Limestone	Limestone	Limestone
Binder	AC-5 w/2% Latex	AC-5 w/2% Latex	AC-5 w/2% Latex	AC-15 5 TR	AC-5 w/2% Latex
Binder Application Rate (gal/yd ²)	0.40	0.40	0.40	0.40	0.40
Binder Temp at Spreading Aggregate (F)	125.0	125.0	125.0	125.0	125.0
Binder Temp at Rolling (F)	105-110	105-110	105-110	105-110	105-110
# of Total Aggregates	82.0	79.0	55.0	61.0	48.0
# of Aggregates Lost	22.0	47.0	42.0	54.0	24.0
% Loss	26.8	59.5	76.4	88.5	50.0

duration at the end of freeze-thaw cycling. These procedures were 1 freeze-thaw (F-T) cycle + 48 hours of soaking, 2 F-T cycles + 48 hours soaking, 3 F-T cycles + 48 hours soaking, and 3 F-T cycles + 24 hours of soaking. The procedure with 3 F-T cycles + 48 hours soaking was also applied to the specimen prepared with the different binder. At the end of each conditioning procedure, the specimens were subjected to impact loading by MPH 3 times. The ratio of number of aggregates lost to the number of total aggregates was indicated as “percent loss” in the table. As mentioned previously, 1 F-T cycle is 16 hours soaking followed by 8 hours freezing. Results showed that 3 + 48 lost the most aggregate based on percent loss. The comparison between 2 + 48, 3 + 24 and 3 + 48 shows the effect of the soaking process after consecutive freeze-soak/thaw cycles. It can

be seen from the table that the binder application rate, binder temperature at the time of rolling and aggregate application were identical. This was because of isolating the factor of conditioning.

4.5 Sensitivity of the Protocol to Aggregate Type

Based on the preliminary tests discussed earlier, a conditioning of 3 F-T cycles + 48 hours of soaking followed by the impact loading with MPH was selected as the most effective protocol. At this phase of the study, specimens using the same binder (AC-5 + 2% Latex), but with four different seal coat aggregate types commonly used in Texas, were subjected to 3 freeze-thaw cycles followed by 48 hours of soaking and the impact loading with MPH. Results of these tests are shown in Table 4.3. Similar to the previous testing series, in order to isolate one factor, aggregate type, the other factors such as binder type, binder rate, binder temperature at the time of rolling and spreading the aggregate were kept identical. It can be seen from the table that the testing protocol is responsive to different types of aggregate. Based on the percent aggregate loss, lightweight aggregate performed the best (22.7% loss) compared to Gravel (Fordyce) aggregate which lost approximately 2.5 times more aggregate.

Table 4.3: Sensitivity of Testing Protocol to Aggregate Type

	Lightweight	Limestone	Gravel (Rhyolite)	Gravel (Fordyce)
Binder	AC-5 w/2% Latex	AC-5 w/2% Latex	AC-5 w/2% Latex	AC-5 w/2% Latex
Binder Application Rate (gal/yd ²)	0.40	0.40	0.40	0.40
Binder Temp at Spreading	125.0	125.0	125.0	125.0
Aggregate (F)				
Binder Temp at Rolling (F)	105-110	105-110	105-110	105-110
# of Total Aggregates	66.0	73.0	80.0	92.0
# of Aggregates Lost	15.0	25.0	36.0	72.0
% Loss	22.7	34.2	45.0	78.3

4.6 Verification of Testing Protocol

In the final phase of test protocol development, specimens brought from field test sections were tested with the new testing protocol developed. Preliminary studies showed that three consecutive freeze-soak/thaw cycles followed by 48 hours of soaking and the impact loading of MPH was the protocol most sensitive to extreme aggregate-binder compatibility variations. Since the surface of the specimens brought from the site visits were covered 100% with the aggregate, the aggregates placed $\frac{1}{4}$ in close to the sides were removed by hand. This was done because of the rubber pads on the 4 sides of the impact apparatus.

Three replicate specimens from each district test section were tested using the procedure illustrated previously, and the results are shown in Table 4.4. Results for the 15 test sections show that specimens from 9 test sections had 0% aggregate loss. The

other 6 sections showed some aggregate loss. Abilene test section lost 11.82% of its aggregate in average confirming the compatibility problem observed in the field monitoring follow-up study. San Antonio showed 6.56% loss, which also matched with the follow-up study observations. Lubbock test section lost 4.28% of its aggregate, which follows the loss pattern observed during the field monitoring follow-up study. The Fort Worth district showed a negligible amount of loss.

Table 4.4: Performance of Field Specimens under Proposed Test Protocol

District	Specimen # (% Loss)			Average
	1	2	3	
Ft. Worth	0.59	1.70	0.90	1.06
Odessa	0.00	0.00	0.00	0.00
El Paso	0.00	0.00	0.00	0.00
Brownwood I	2.60	1.30	4.65	2.85
Brownwood II	0.00	0.00	0.00	0.00
Brownwood III	9.30	4.17	5.56	6.34
Lufkin	0.00	0.00	0.00	0.00
San Angelo	0.00	0.00	0.00	0.00
Tyler	0.00	0.00	0.00	0.00
Paris	0.00	0.00	0.00	0.00
Lubbock	2.34	5.65	4.84	4.28
Childress	0.00	0.00	0.00	0.00
Abilene	13.51	11.67	10.29	11.82
Pharr	0.00	0.00	0.00	0.00
San Antonio	9.20	5.43	5.06	6.56

The incompatibility problem observed in the Lufkin test section could not be verified using this test method, but in that particular section the TxDOT district engineer claimed that the incompatibility was due to a heavy thunderstorm occurred within two days of construction. Since the specimens collected from the site visits immediately brought to the laboratory and aged for approximately one year, the specimens did not experience conditions similar to that occurred soon after construction. This may be the

likely reason why the testing protocol could not identify the incompatibility problem in Lufkin. Other test sections that showed excellent performance (Odessa, El Paso, Tyler, Ft Worth, Childress, San Angelo, Paris, Pharr, Brownwood I, II, and III) were verified by the test protocol except Brownwood I and III.

The results of the Brownwood III showed around 6.3% loss while Brownwood I showed 2.8% loss. Since the evaluation criteria have not been completed yet, 3% loss might be an acceptable percent loss for screening purposes. On the other hand, Brownwood I lost 6.3%, which created the most notable inconsistency between the test protocol and field observations. These mismatches could be explained by the effect of traffic because these sections have a considerably high amount of traffic compared to the other districts. Traffic has a significant effect on the performance of the seal coat as indicated earlier. Furthermore, significant bleeding was observed in the Brownwood I test section according to the field observations, which also indicates that the aggregate on the road is highly embedded into the binder.

It can be observed that laboratory prepared specimens showed higher aggregate losses compared to specimens brought from the field. But it should be noted that lab prepared samples started soaking 24 hours after specimen preparation, while the specimens brought from the field did not experience such extreme conditions soon after construction. Moreover, the field specimens waited almost a year in the lab environment without any conditioning whatsoever. This could be the reason for the strength of the field specimens.

4.7 Conclusions

The literature review showed that the current test methods are unable to effectively simulate field conditions in the lab environment. The climatic factors such as freezing/thawing and wetting/drying processes are not mimicked in the lab. Another finding of the literature review was that a certain amount of energy has to be applied to initiate the de-bonding at the aggregate-binder interface. Similarly to the use of steel ball in Viallet test, MPH is used for this purpose. In order to avoid de-bonding between the plate and binder, the impact energy applied by the MPH was damped using rubber pads and a ¼ in thick plate, placed on top of the inverted specimen. Basically, the impact loading consists of placing the inverted specimen on the impact apparatus, placing the ¼ in thick steel plate on top of inverted specimen and dropping the MPH 3 times on to this steel plate.

Developing of the new testing protocol continued with trying different conditioning options. Extended amount of soaking and consecutive freeze-soak/thaw cycling were found be the most effective ones. A series of tests incorporating these conditions showed that the 3 F-T + 48 hours of soaking is the best conditioning procedure. The specimens were subjected to impact loading after the conditioning of 3 F-T + 48 hours of soaking is completed. After developing the testing protocol, specimens prepared under identical conditions with different types of aggregates were tested with the new protocol. The protocol responded significantly to the specimens prepared with different types of aggregates. Since the actual field performances of these combinations were not known, it was not possible to draw a conclusion from these results.

To verify the new testing protocol, the specimens collected from field test sections were tested using the new test method. The comparison of the results of the lab specimens and the follow-up monitoring study provided satisfactory verification of the test method. The new testing protocol showed great promise in identifying incompatibility problems between seal coat aggregates and binders. Thirteen of the 15 test sections tested with the new testing protocol matched field observations. Among the 15 test sections tested, there are two test sections where field observations and lab test results did not match (Brownwood III and Lufkin test sections). Lufkin test section experienced incompatibility because of an early severe thunderstorm immediately after construction. Since the specimens brought from the field did not experience such conditions in the lab, because the specimens were retrieved immediately after the construction, it could be logical to observe 0% loss in the lab. Brownwood III district results highlight the fact that effect of traffic is not mimicked in the lab after the preparation of specimens. This can be addressed using appropriate evaluation criteria. For instance, the evaluation criteria can be based on the percent loss and the amount of traffic.

On the other hand, there was significant amount of difference between the performances of lab prepared specimens and the specimens brought from the construction visits. This can be explained by the fact that the specimens brought from the field did not experience severe conditioning soon after the construction while the specimens prepared in the lab were soaked within the next 24 hours after the preparation.

CHAPTER V

LABORATORY TESTING

After the testing protocol has been developed and the preliminary testing has been completed, an experimental plan was made to verify the new testing protocol. Several experimental factors, identified as causes for incompatibility during the constructability review, were incorporated in this plan. Specimens using different aggregate-binder combinations and were prepared and tested using the new test method. The experimental plan had three major parts, based on material used: hot applied asphalt, asphalt emulsion and precoated aggregate.

5.1 Specimen Preparation

Due to the potential corrosion of steel plates during the conditioning phase, it was decided to use aluminum plates in place of steel plates for specimen preparation. These plates had the dimensions (15 cm x 15 cm) as the steel plates, but the preparation of specimens was different from that adapted during the development of testing protocol. It was decided that 3 replicate specimens were prepared for each test condition in this series of experiments. Since it is not time-efficient to prepare a large number of specimens by hand, several changes were made to the specimen preparation process.

Instead of controlling the rate of application of binder by weight, it was controlled by the thickness of the applied. The 3 replicate plates were lined up to form a rectangle, whose dimensions were 15 cm x 45 cm, to ensure that they are prepared under the same

conditions. A blade, whose height from the sample plates was adjustable, was placed on two parallel rails on two sides of sample plates. The clearance between the sample plates and the blade controlled the thickness of the binder film, which represented the binder application rate. The aluminum plates, because of their smoother and more durable surface, helped to form a perfect binder film at the desired rate of application. Since the blade could slide on the rails, the process was easy, smooth and more consistently leaves a uniform binder coating. Figure 5.1 shows the specimen preparation set up. It can be seen that the blade is initially placed on one end of the sample plates. The binder heated in the oven to the required temperature (350° F for all hot asphalts and 170° F for emulsified asphalts) is poured in front of the blade and the blade is slid towards the other end of the sample plate in one smooth motion, creating a uniform binder film.

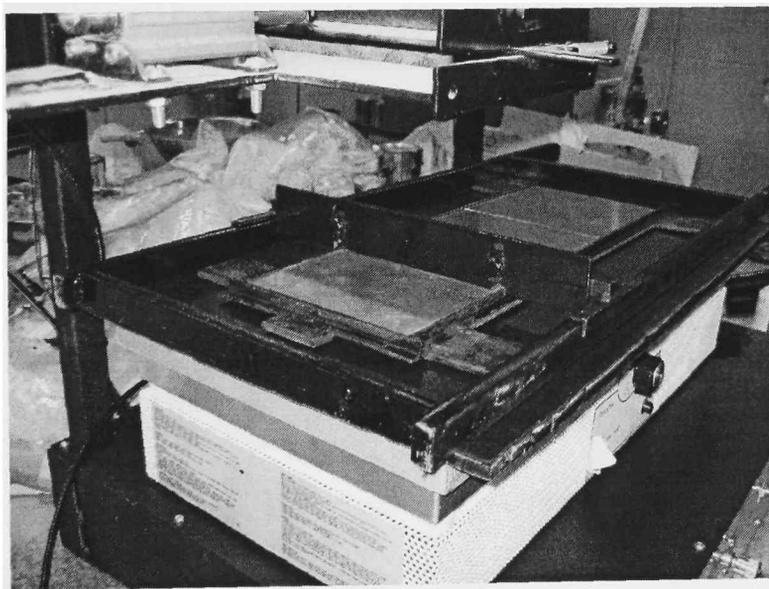


Figure 5.1: Specimen Preparation Set up

Two different surface temperatures, 70° F and 140° F, were used in the preparation of specimens to represent extreme field conditions. For the emulsified asphalt, the surface temperature was kept at 70° F. In the hot applied asphalt part, both surface temperatures (70° F and 140° F) were used, while in the precoating part the surface temperature was 140° F for hot asphalts and 70° F for emulsions. The surface temperature varied by placing the plates on a hot plate. The temperature of the specimen plates was controlled using the temperature regulator of the hot plate (Figure 5.2).

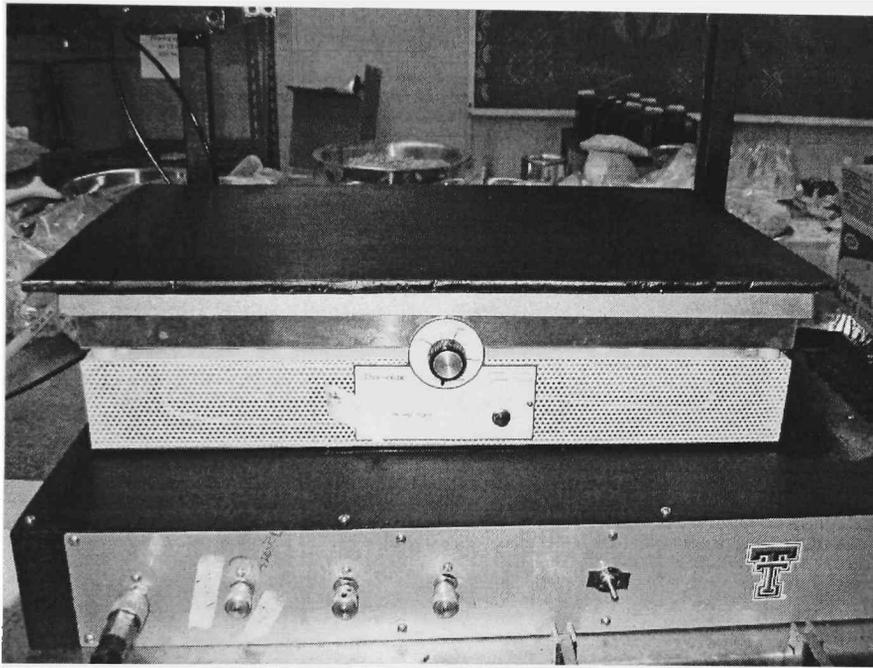


Figure 5.2: Hot Plate

After the binder is poured and spread at the required rate of application, the aggregate is placed to the binder film, when the binder temperature reached to the surface temperature. Before placing the aggregate, the binder temperature was measured using a non-contact infrared thermometer to make sure that the binder temperature was equal to

the surface temperature. TxDOT grade 4S aggregates were used throughout the experimental plan for all aggregate and binder types. The time elapsed after the spreading of the aggregate was one of the experimental factors. At the appropriate time, specimens were rolled 5 times. The roller was made up from a cylindrical steel drum filled with water and covered by a ¼ in thick rubber sheet to avoid the crushing of aggregate particles. It weighs 236 kg (520 lb.).

5.2 Hot Applied Asphalt

Based on performance history and results from other tests, two aggregate-binder combinations were selected to represent poor and excellent compatibility extremes. This way, the ability of the new testing protocol to identify the widest variety of material combinations could be verified. The two combinations were AC-5 w/2% Latex with Siliceous Gravel and AC-15 5TR with crushed Limestone. Specimens were prepared for four different experimental factors; embedment depth, dust content, surface temperature, time elapsed between application of aggregate and rolling. These four factors were selected based on the constructability review and their effect on aggregate-binder bond.

5.2.1 Embedment Depth

Seal coat is a surface treatment, where a single overlay of aggregate is partially covered with the asphaltic binder. The height of this partial asphaltic coating of the aggregates is often called as “embedment depth.” The embedment depth typically dictates the mechanical component of bond.

Figure 5.3 illustrates the factors that affect seal coat embedment depth. The aggregate and binder application rates, condition of the existing pavement, aggregate gradation and efficiency of compaction are the main factors that affect the embedment depth. Among these factors, the aggregate-binder application rates and the aggregate gradation, condition of the existing pavement are self-explanatory and the basic guidelines for these factors are available. On the other hand, the fourth factor “efficiency of compaction” is a factor that has to be investigated deeper than the others.

In Figure 5.3, the “efficiency of compaction” is divided into two parts; compaction under traffic and during construction. It is well known that traffic helps compaction of aggregates. In addition, the constructability review showed that loss of aggregate does not occur in the wheel paths. This can be explained by the compaction under traffic, which is affected by the ambient temperature and volume of traffic. Compaction during construction is a more complex factor because of the factors affecting it. Equipment related factors such as roller type and number of roller passes, are self explanatory but the factor “temperature of the binder at the time of rolling” has to be explained further. The stiffness of the binder is directly related to its temperature. In other words, if the binder is hot, it will be less stiff and can be easily compacted. Surface temperature, ambient temperature and the time elapsed between the spreading of aggregate and the rolling are the three factors that influence the heat loss of the binder after it sprayed from the distributor. Due to these factors, embedment depth becomes a complex phenomenon and it is difficult to control in the lab environment.

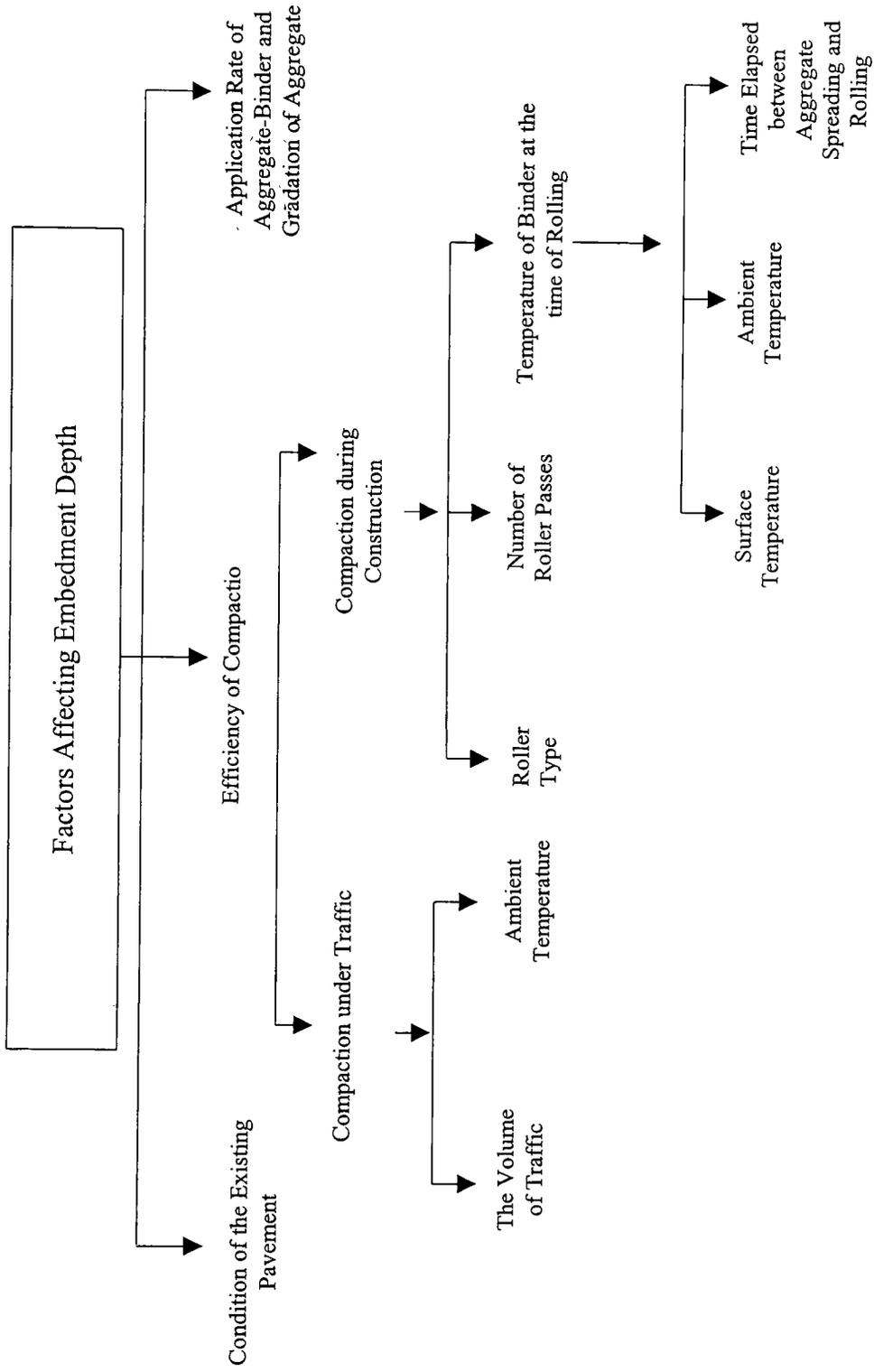


Figure 5.3: Factors Affecting Embedment Depth

Moreover, the same amount of binder and aggregate will not result the same embedment depth because of the other factors that affect embedment depth. As a result, the seal coat specimens were prepared to achieve 3 different embedment depths, 15%, 35%, and 50%. TxDOT design criteria were used to achieve the required embedment depth for crushed limestone, siliceous gravel and lightweight aggregate.

For limestone from Vulcan Materials Brownwood Pit:

$$A = 5.61E \left(1 - \frac{W}{62.4G}\right)$$

Where:

A = Asphalt Quantity (gal/yd²)

E = ed

e = design embedment (15%, 35% and 50%)

$$d = 1.33 \frac{Q}{W}$$

Q = Quantity of aggregate from board test, 22.04 lbs/yd²

W = Dry loose unit weight of aggregate, 90.42 lbs/ft³

$$d = 1.33 \times 22.045 / 90.42 = 0.32426$$

For 15% embedment E = 0.04864 in

For 35% embedment E = 0.1135 in

For 50% embedment E = 0.1621 in

G = Dry bulk specific gravity of the aggregate, 2.67

$$A = 5.61 \times 0.04864 \times \left(1 - \left(\frac{90.42}{62.4 \times 2.67}\right)\right) = 0.125 \text{ gal/yd}^2 \text{ (15\% embedment)}$$

$$A = 0.291 \text{ gal/yd}^2 \text{ (35\% embedment)}$$

$$A = 0.416 \text{ gal/yd}^2 \text{ (50\% embedment).}$$

For siliceous aggregate from Fordyce Company, Showers Pit:

$$Q = \text{Quantity of aggregate from board test, } 19.69 \text{ lbs/yd}^2$$

$$W = \text{Dry loose unit weight of aggregate, } 87.96 \text{ lbs/ft}^3$$

$$G = \text{Dry bulk specific gravity of the aggregate, } 2.61$$

$$A = 0.115 \text{ gal/yd}^2 \text{ (15\% embedment)}$$

$$A = 0.269 \text{ gal/yd}^2 \text{ (35\% embedment)}$$

$$A = 0.384 \text{ gal/yd}^2 \text{ (50\% embedment).}$$

Lightweight Aggregate from TXI Inc., Streetman Pit:

$$Q = \text{Quantity of aggregate (board test), } 8.34 \text{ lbs/yd}^2$$

$$W = \text{Dry loose unit weight of aggregate, } 46.15 \text{ lbs/ft}^3$$

$$G = \text{Dry bulk specific gravity of the aggregate, } 1.46$$

$$A = 0.110 \text{ gal/yd}^2 \text{ (15\% embedment)}$$

$$A = 0.233 \text{ gal/yd}^2 \text{ (35\% embedment)}$$

$$A = 0.333 \text{ gal/yd}^2 \text{ (50\% embedment).}$$

In order to use these design values with emulsified asphalt, the binder rates were adjusted based on emulsion having 70% asphalt 30% water. The application rates to achieve 35% embedment depth were calculated as follows:

$$\text{Limestone: } 0.291/0.70 = 0.416 \text{ gal/yd}^2$$

$$\text{Gravel: } 0.269/0.70 = 0.384 \text{ gal/yd}^2$$

$$\text{Lightweight: } 0.233/0.70 = 0.333 \text{ gal/yd}^2$$

5.2.2 Dust Content

Dust on cover aggregate is one of the most common factors that causes incompatibility. Although the aggregates are washed when they are first produced at the source, significant amount of dust may be generated during transportation and handling. Particularly, in the case of limestone and lightweight aggregate, the dust on the aggregate surface will affect the interaction with the binder resulting weak aggregate-binder bonding. It was decided to use aggregates with three different levels of dust; no dust, medium dust and high dust for preparation of the specimens.

Instead of exposing the dust artificially to the aggregate, it was decided to allow the aggregate to generate its own dust. This was done in the laboratory using the equipment from the Micro-Deval test. This test is typically used to evaluate the abrasion resistance of aggregates. In the original test procedure, 5000 grams of aggregate and 1500 grams of 1 in diameter steel balls are both inserted into a steel drum and closed with a lid. This cylindrical drum is placed on a rotation device and the drum is rotated for a specified duration. Preliminary tests showed that too much of dust is generated when steel balls are used, and also it does not accurately mimic field dust generation condition. Therefore, the Micro Deval setup is used without the steel balls but with the same amount of aggregate (5000 g). The duration of the drum rotation affects the amount of dust generated. Figure 5.4 shows the amount of dust generated with 2 min, 5 min and 15 min minutes of rotation. The trend line of these 3 points is linear. After these preliminary trials, the dust contents were evaluated 0, 2, and 5 minutes of drum rotation were selected to represent 0, medium and high dust contents. It was also observed that from the total of

from the total of dust generated, some dust stays in the steel drum without coating the aggregate surfaces. The amount of dust retained on the aggregate surfaces was determined and plotted in Figure 5.4 as “percent dust retained.”

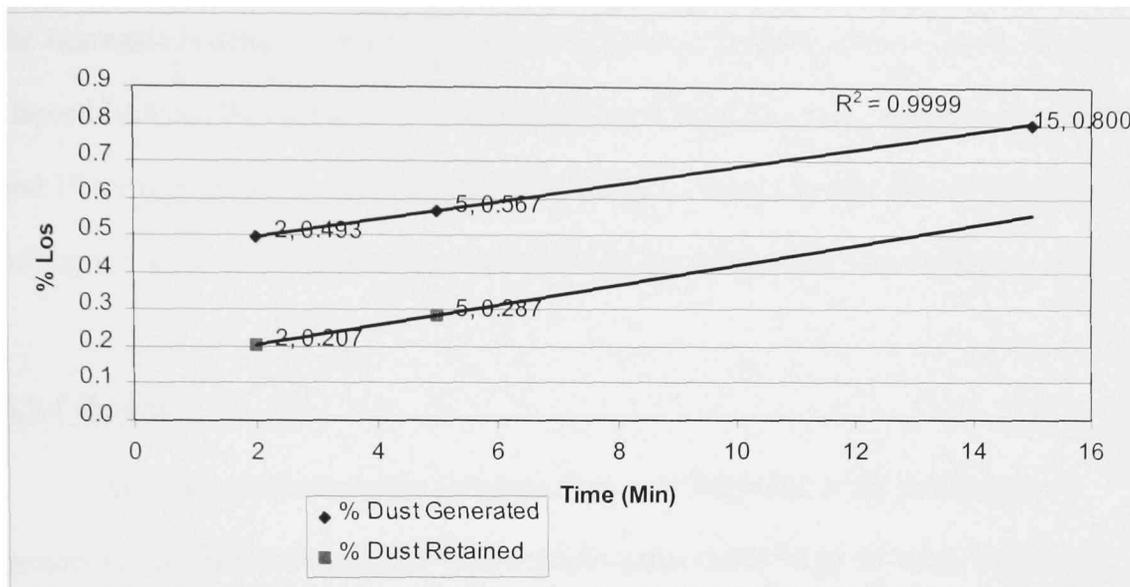


Figure 5.4: Amount of Dust Generated and Retained with Micro-Deval Test for Limestone, (Vulcan, Brownwood)

Before running the Micro-Deval test, all aggregate were washed and dried in the oven for 24 hours. The aggregate is stored in sealed bags until the preparation of specimens. As expected, the siliceous gravel did not generate any dust when subjected to the Micro-Deval rotation. Moreover, most gravels do not generate dust during handling an transportation. Therefore, dust content factor was only used as an experiment factor with aggregate tested in combination with AC-15 5TR binder.

Different types of aggregates generate different amount of dust. The Micro-Deval apparatus is capable of simulating this condition for a specific time of rotation, if it is included to the testing protocol.

5.2.3 Surface Temperature and Time Elapsed between Aggregate Spreading and Rolling

In this part of the experimental plan, the two extreme surface temperatures 70° F and 140° F were selected to prepare laboratory seal coat specimens. The application of the aggregate is delayed until the binder cools down to the surface temperature. The time elapsed between the spreading of the aggregate and the rolling was selected as 3 minutes and 10 minutes based on cooling rate of hot asphalts. Binder temperature at the time of rolling is measured using an infrared non-contact thermometer and recorded for each run.

5.2.4 Results

After the specimens were prepared, they were subjected to the conditioning protocol developed previously; 3 Freeze-Thaw cycles followed by 48 hours of soaking. Then impact loading using Modified Proctor Hammer (MPH) was applied three times. The number of aggregate particles in the specimen was counted before conditioning, after conditioning and after impact loading for each replicate. Percent loss for each replicate and the average loss for these three are calculated. Results for AC-15 5TR + Limestone combination are given in Table 5.1 and results for AC-5 w/2% Latex + Siliceous Gravel combination are given in Table 5.2. The first three numbers for each factorial are percent losses for replicate 1, 2, and 3, respectively. The number under each 3 series is the average. The dust content factor in Table 5.1 is indicated by 0 dust, 2 min Micro Deval (MD) and 5 min MD dust conditions. Surface temperatures of 70 °F and 140 °F and time elapsed between aggregate application and rolling of 3 minutes and 10 minutes were used.

Table 5.1: Percent Loss for AC-15 5TR + Limestone

	0 Dust			2 Min MD			5 min MD									
	70 F		140 F	70 F		140 F	70 F		140 F							
15% Embedment	3 min	61.11	85.07	62.50	66.67	84.75	83.33	57.53	53.52	61.64	98.65	92.42	92.00	85.92	70.31	58.90
	10 min	69.56			78.25			57.57			94.36			71.71		
35% Embedment	3 min	15.94	19.23	33.33	7.46	11.69	8.64	36.76	46.03	39.13	13.56	30.77	14.71	51.52	60.00	44.78
	10 min	22.84			40.64			19.68			52.10					
50% Embedment	3 min	11.39	10.14	14.12	24.29	39.06	46.30	2.90	8.54	4.76	20.63	11.76	21.54	5.63	7.04	9.72
	10 min	11.88			36.55			5.40			7.47			15.87		16.92
								6.59							12.49	

For instance, in Table 5.1, specimens prepared using AC-15 5TR and Limestone which is subjected to 2 min MD testing, at 140° F surface temperature, which is designed for 15% embedment and rolled after 3 minutes the application of the aggregate lost 57.57% in average. For this particular example, replicates 1, 2, and 3 lost 57.53%, 53.52%, 61.64% of aggregate, respectively. Three factors, surface temperature, dust content and roller time delay, are plotted against embedment depth and the charts are given in Figures 5.5, 5.6, and 5.7, respectively. In these figures, average percent loss is presented in z axis while the other two factors are shown in x and y axis. It can be easily seen that the new testing protocol effectively responds to the factors that affect the performance of the seal coat.

In Figure 5.5, the average percent aggregate loss due to change in dust content and percent embedment depth is plotted for 70° F surface temperature and 3 minutes roller delay. It can be seen that 15% embedment lost as much as 94% of its aggregate with 5 min MD dust content. The percent loss decreased to 70% when no dust existed on the aggregate. The difference between these two is approximately 24%. The specimens prepared for 35% embedment, lost as much as 52% with the 5 min MD dust content aggregate and when no dust was present, the percent aggregate loss is reduced to 23%. The difference of approximately 29% between these two conditions is comparable to the 24% aggregate retention difference for 15% embedment. This suggests that dust content affects the aggregate retention independent from the embedment depth and underlines the importance of it.

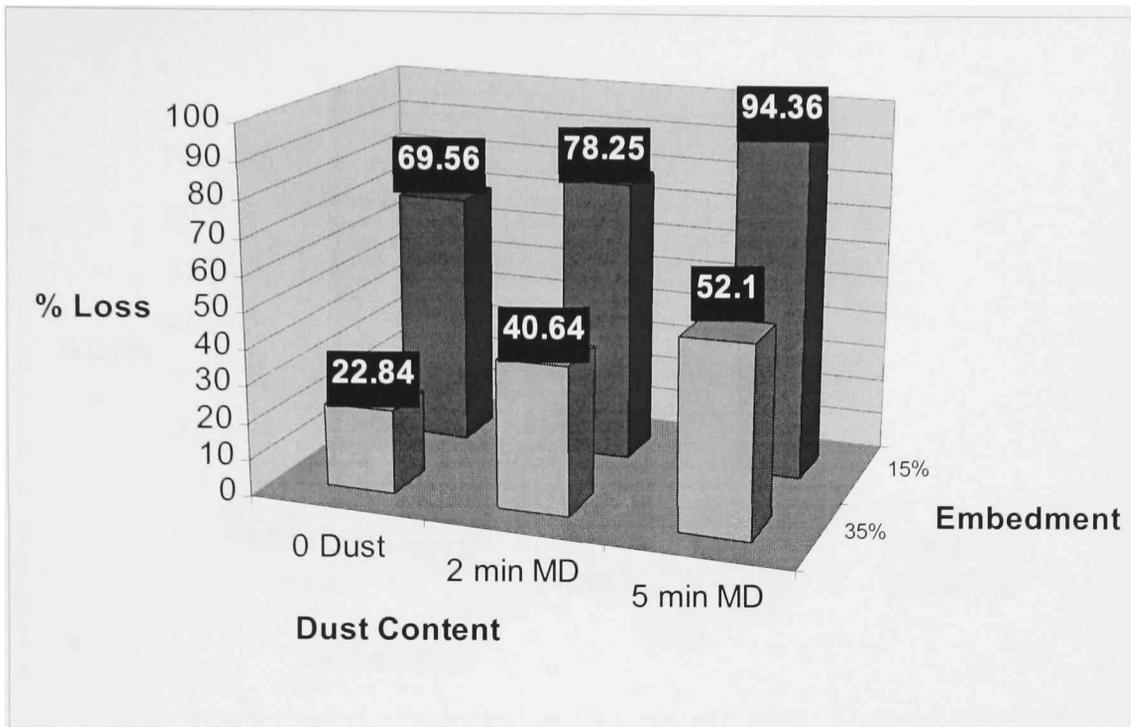


Figure 5.5: Percent Loss due to Dust Content and Percent Embedment Depth at 70° F and 3 minutes roller time delay for AC-15 5TR + Limestone

In Figure 5.6, the percent aggregate loss due to time elapsed between the aggregate application and rolling and embedment depth is plotted for 140° F surface temperature and 2 min MD dust conditions. The high difference between the 15% embedment and 35% embedment is very noticeable. This concludes that the aggregate retention improves exponentially with the increasing embedment depth. In addition, the roller time delay from 3 to 10 minutes does not appear to make a significant difference compared to the embedment depth. Also, the testing protocol showed almost similar trends for different roller time delays for the same embedment depths.

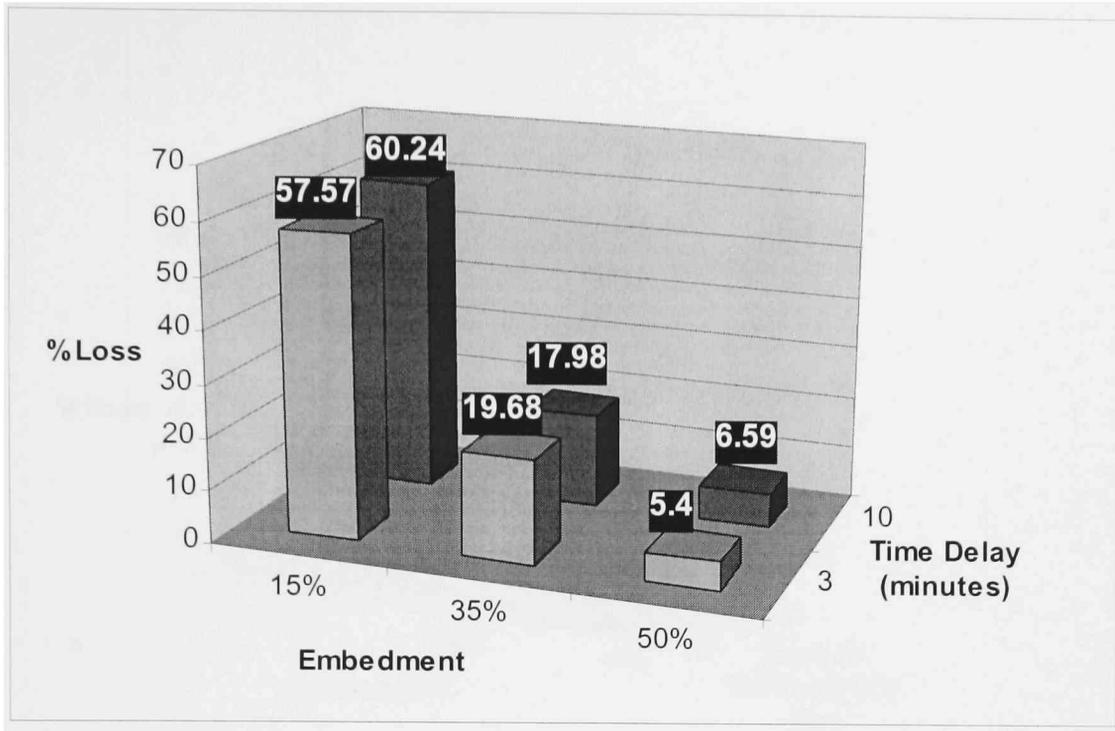


Figure 5.6: Percent Loss due to Time Elapsed between Aggregate Application and Rolling and Embedment Depth at 140° F and 2 min MD for AC-15 5TR + Limestone

Figure 5.7 is the plot of surface temperature versus embedment depth for 2 min MD and 3 min roller time delay. Embedment depths, 15% and 35%, show the same trend for 70° F and 140° F, but 35% and 50% do not. The aggregate loss was the lowest at 140° F and 50% embedment, at this temperature there was significant benefit gained by increasing percent embedment. However, at 70° F surface temperature, the increased percent embedment helped only up to 25%. The aggregate-binder bond showed high sensitivity to surface temperature even at high embedment depths. This leads to conclusion that bonding between the aggregate and the binder is dependent on a combination of percent embedment and surface temperature, and the field inspectors must ensure that hot asphalts shall not be applied when the surface temperature is low.

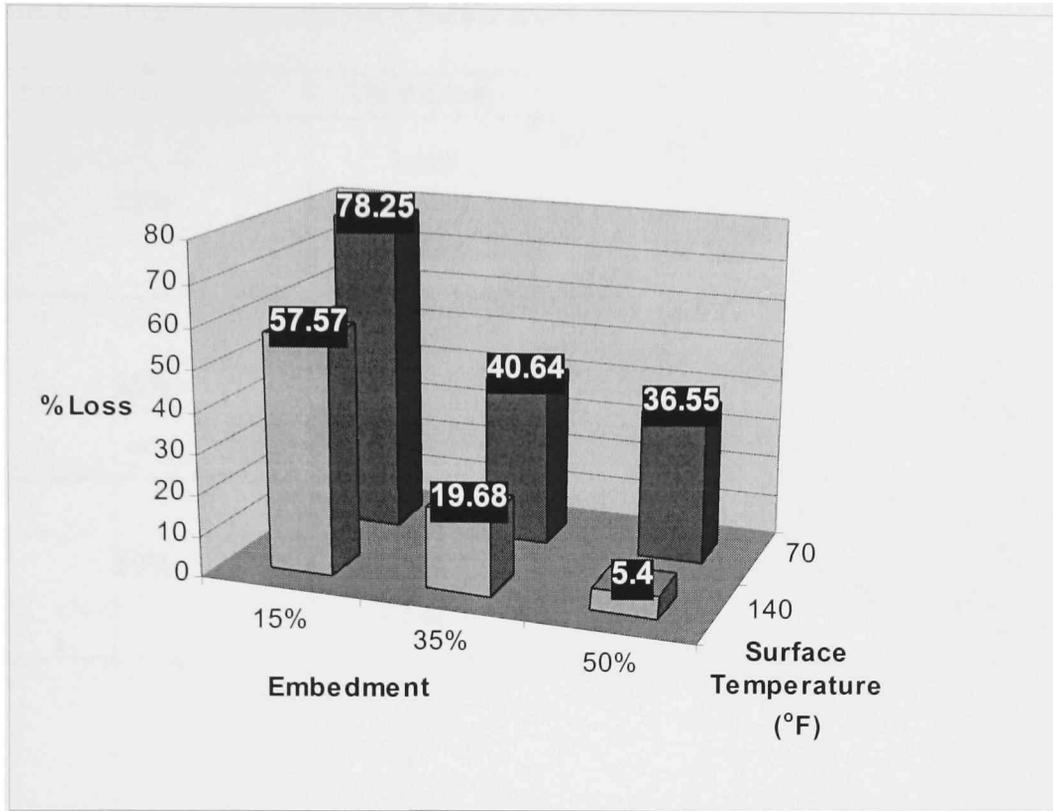


Figure 5.7: Percent Loss due to Surface Temperature and Embedment Depth for 2 min MD and 3 minutes roller time delay for AC-15 5TR + Limestone

The result of AC-5 with Siliceous Gravel is given in Table 5.2. The lowest percent loss for this combination is 33.4% and the highest is 88.2%. Comparison of these results with the previous combination suggests that the protocol was successful to identify the good and the poor material combinations.

The factors plotted in Figure 5.8 are similar to Figure 5.7, except that it is for the weaker bond combination of AC-5 w/2% Latex + Siliceous Gravel. As shown in the chart, none of the combinations performed well. There was a significant improvement when surface temperature increased from 70° F to 140° F for 35% and 50% embedment.

Table 5.2: Percent Loss for AC-5 w/2% Latex + Siliceous Gravel

Embedment Depth	Time Delay	70° F			140° F		
15%	3 min	91.3	90.3	83.1	73.6	82.9	93.5
		88.2			83.3		
	10 min				71.6	69.2	100.0
					80.3		
35%	3 min	77.0	89.5	93.8	32.9	57.1	84.9
		86.8			58.3		
	10 min				56.1	71.0	77.6
					68.2		
50%	3 min	50.8	65.2	53.2	20.5	42.4	42.3
		56.4			35.1		
	10 min				28.2	35.5	36.6
					33.4		

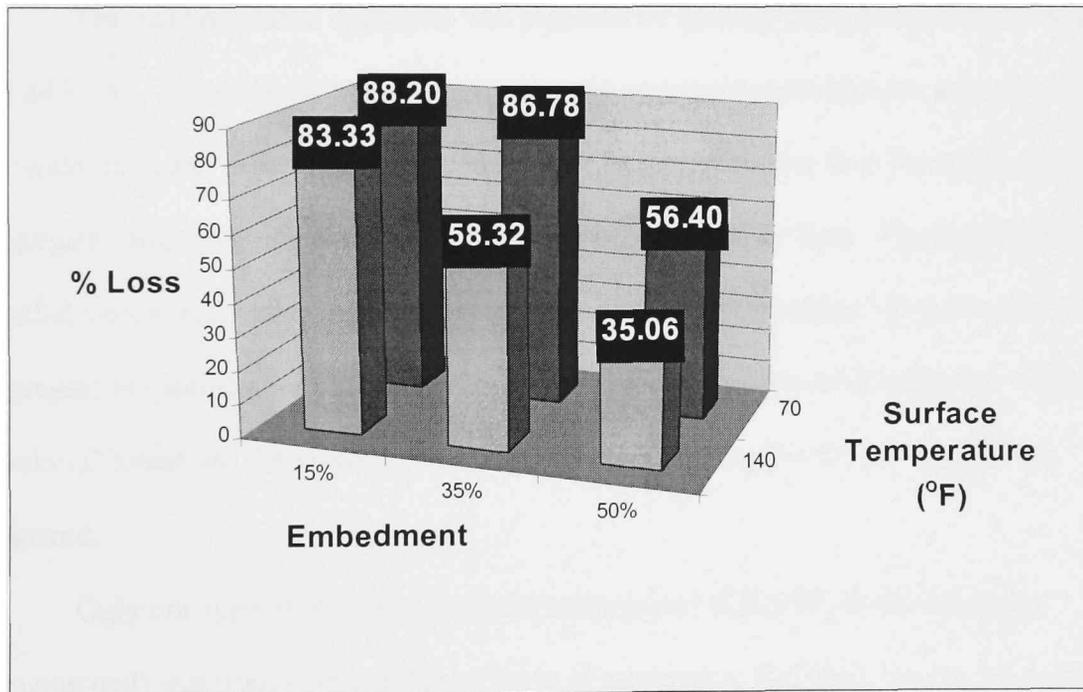


Figure 5.8: Percent Loss due to Surface Temperature and Embedment Depth with 3 minutes roller time delay for AC-5 w/2% Latex + Siliceous Gravel

5.3 Asphalt Emulsions

In this part of the study, laboratory specimens were prepared using emulsified asphalt (CRS-2P) with 3 types of aggregate were tested with the testing protocol developed. It is known that the moisture state and breaking/curing processes affect the performance of the bonding between the aggregate and the emulsified asphalt. Therefore, specimens using aggregates with saturated surface dry (SSD) condition and air dry condition were prepared. Moreover, aggregates were applied both before and after the emulsion broke. Curing time of the emulsion have been also considered as an experimental factor, but since the specimen conditioning starts with soaking, the early curing times caused the emulsion to dissolve in water, which results a total failure in few hours.

The SSD condition aggregate was prepared by soaking the aggregate under water for 24 hours. Immediately before application, the aggregate particles were taken out of the water bath and dried with a towel to remove the free moisture from the surface of the aggregate. Breaking process of the emulsion was controlled by time. The aggregate was applied immediately after the binder to simulate the “before breaking” situation, the aggregate is applied after 75 minutes from the binder application to simulate the “after breaking” situation. The testing procedure started 30 hours after the specimens were prepared.

Only one type of emulsion from the same source (CRS-2P, Koch materials, Brownwood) was used with 3 different types of aggregates, Siliceous Gravel, Limestone

and Lightweight aggregate. Gravel was not tested in SSD condition, since it has a low adsorption capacity.

The surface temperature was 70° F for all aggregate applications and the design percent embedment was 35%. The asphalt to water ratio in the emulsion was 70% asphalt to 30% water. The actual asphalt residue embedment was designed based on this emulsion composition. The result of this series of tests is given in Table 5.3.

Table 5.3: Percent Loss for CRS-2P + Siliceous Gravel, Limestone, and Lightweight

Moisture Content	Timing of Aggregate Application	Siliceous Gravel			Limestone			Lightweight		
Air Dry	Before Breaking	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.56
		0.00			0.00			0.85		
	After Breaking	5.66	5.45	3.13	29.79	14.58	13.56	0.00	0.00	0.00
		4.75			19.31			0.00		
SSD	Before Breaking				0.00	0.00	0.00	0.00	0.00	3.64
					0.00			1.21		
	After Breaking				0.00	2.00	0.00	0.00	0.00	2.90
					0.67			0.97		

In general, the performance of emulsified asphalt specimens were far better than specimens prepared with the hot applied asphalt. The testing protocol was found to be successful in distinguishing the specimens whose aggregate was applied before breaking and after breaking, except for lightweight aggregate. The lightweight aggregate performed well in both cases not showing a significant amount of loss. This type of aggregate is porous and the emulsion, which is highly liquid at the time of application, can penetrate easily into these pores. This often leads to more effective bonding, as it is evident from the test results. Limestone showed the most aggregate loss in the “after

breaking” situation under “air-dry” moisture condition. Siliceous gravel also lost 4.75% aggregate when it was applied after the emulsified asphalt broke, while it did not lose any aggregate when it was applied before breaking.

Aggregate under the SSD condition performed better than the air dry aggregate. The field observations suggest that SSD condition aggregate improves the bonding between the aggregate and the binder, when emulsified asphalt is used. Under SSD condition, limestone and lightweight aggregate did not lose significant amounts of aggregate in both before breaking and after breaking aggregate application.

5.4 Precoated Aggregate

In the third phase of laboratory testing program, specimens prepared with precoated aggregate were tested with the new testing protocol. It is well known that that the dust on aggregate is one of the common causes of incompatibility. The aggregates that generate significant dust during transportation and handling have to be precoated to eliminate incompatibility problems.

The constructability review showed that districts follow two procedures in precoating seal coat aggregate. Lightweight aggregate is typically precoated with CSS-1h and the limestone aggregate is typically precoated with PG 64-22. Therefore, same combinations were used to prepare precoated aggregate in the lab. In addition, specimens were prepared using aggregate with two different precoating levels, up to 50% and more than 90%. AC-15 5TR, AC-15P, and CRS-2P were used as seal coat binder. The precoating levels were based on visual observation. The binder to aggregate ratio of for

the two precoating levels was determined by trial and error. For limestone, the binder to aggregate ratios were 0.2% and 0.5% by weight for up to 50% and more than 90% precoating levels, respectively. For lightweight aggregate, these ratios were 1.7% and 4.5% for up to 50% and more than 90% precoating levels, respectively. The aggregates were washed and oven dried prior to precoating so that there was no dust at all. Since AC-15P is not typically used with precoated lightweight aggregate in field practice, that combination was excluded.

In this phase, when specimens were prepared in the laboratory, surface temperature for hot applied asphalts was 140° F and for emulsified asphalt was 70° F. All aggregates were applied after the binder cooled down to surface temperature. The design embedment depth for all combinations was 35%. The time elapsed between the aggregate application and the rolling was 3 minutes. The temperature of the binder at the time of rolling was also measured and recorded and given in Appendix B.

The results of the precoated aggregate testing are indicated in Table 5.4. The overall performance of the precoated aggregates was far better than the uncoated aggregates. The new testing protocol was successful to identify the precoating level for AC-15P. For the other binders, there was no significant difference between the precoating levels since all of them did not lose more than 0.01% of their aggregate. This shows how precoating improves the performance of the seal coat if it is used with the appropriate precoating and seal coat binder. On the other hand, since the aggregates were washed and oven dried before precoating, that would also have improved the performances of these specimens.

Table 5.4: Percent Loss for Precoated Aggregate Specimens

	Uncoated Lightweight			Uncoated Limestone			Emulsion Precoated (CSS-1h)						AC Precoated (PG 64-22)					
	8.62	16.92	8.20	13.56	30.77	14.71	Up to 50% Coating	More than 90% Coating	Up to 50% Coating	More than 90% Coating	Up to 50% Coating	More than 90% Coating	Up to 50% Coating	More than 90% Coating	Up to 50% Coating	More than 90% Coating		
AC-15 5TR	11.25			19.68			0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
AC-15P				54.00	62.50	54.72							0.00	14.55	14.04	2.13	5.88	1.79
				57.07									9.53					
CRS-2P	0.00	0.00	2.56	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
	0.85			0.00			0.00						0.00					

5.5 Suggested Evaluation Criteria for the Testing Protocol

Table 5.5 gives a simple statistical analysis of results for the AC-15 5TR + Limestone combination. The 5 to 10% range is the most occurred data range. The 2 min MD with 50% embedment at 140° F lost 5.40% and 6.59% aggregate at 3 min and 10 min roller time delays, respectively. A loss of 7.50% occurred for 5 min MD at 140° F 3 min time delay. A loss of 9.26% occurred for 0 dust, 35% embedment at 140° F 3 min time delay. These 4 instances are considered to be acceptable. Therefore 0 to 10% loss range was considered as “very good.” Two data points fell in the 10 to 15% range. 11.88% loss for 50% embedment at 70° F for 0 dust 3 min delay and 12.49% for 5 min MD, 50% embedment at 140° F with 10 min delay. These may be susceptible to some incompatibility one having an extremely cold surface temperature and the other having an extreme dust content and delayed rolling. The losses of 18.0% and 19.7% resulted from 2 min MD, 35% embedment at 140° F with 3 min and 10 min rolling delay, respectively. These combinations are above average combinations. Therefore, 10 to 20% range is considered as “maybe incompatible,” if field conditions are adverse. Loss of 22.8% was the result of 35% embedment, 0 dust at 70° F, 36.5% was the result of 50% embedment, 2 min MD at 70° F and 40.6% was the result of 35% embedment, 2 min MD at 70° F. The last two combinations are highly susceptible to incompatibility, but the first one may still perform adequately, because of 0 dust content and 50% embedment. Therefore, 20 to 30% is rated as “marginally compatible.” Any percent loss beyond 30% may be classified as “highly incompatible,” when tests are conducted under laboratory conditions that simulate local conditions. The evaluation criterion established is given at Table 5.6.

It should be noted that this evaluation criterion is based on the performance of lab prepared specimens. This criterion should not be used for the specimens brought from the site.

Table 5.5: The occurrence of data for the corresponding ranges based on percent loss for AC-15 5TR + Limestone

Data Range (%)	# of Occurrences	% Losses Recorded within the Corresponding Range
0 to 5	0	N/A
5 to 10	4	5.40, 6.59, 7.50, 9.30
10 to 15	2	11.9, 12.49
15 to 20	2	18.00, 19.70
20 to 30	1	22.8
30 to 40	1	36.5
40to 50	1	40.6
50 to 65	3	52.1, 57.60, 60.20
65 to 90	3	69.6, 71.70, 78.20
90 to 100	1	94.4

Table 5.6: Suggested Evaluation Criteria

Data Range (% Loss)	Compatibility Evaluation
0 to 10	Very Good
10 to 20	Maybe Incompatible
20 to 30	Marginally Compatible
30 to 100	Incompatible

5.6 Conclusions

After the development of the new testing protocol has been completed, the effects of factors known to cause incompatibility were tested with the new test method. In addition, different aggregate binder combinations were also tested. The laboratory

testing program was divided into three phases: hot applied asphalt, asphalt emulsion and precoated aggregate.

In the first phase, two different material combinations were selected based on field experience. The combination of AC-15 5TR + Limestone was selected as the “good” combination while AC-5 w/2% Latex + Siliceous Gravel was selected as the “poor” combination. For the good material combination, specimens were prepared to investigate the effect of dust content, surface temperature, embedment, and roller time delay. For the poor material combination, dust content was not considered because dust is not a typical problem with gravels.

The new testing protocol identified typical different levels of experimental factors that cause incompatibility. According to the test results, AC-15 5TR + Limestone performed better than AC-5 w/2% Latex + Gravel as expected. These results indicate that the new testing protocol can successfully discriminate the good and poor material combinations. Subsequently, the protocol responds to the factors of incompatibility including dust content, surface temperature, percent embedment and roller time delay.

In the second phase of the laboratory testing, specimens prepared with CRS-2P emulsion and 3 different types of aggregates (limestone, lightweight and siliceous gravel). In addition to testing the suitability of material combinations, effects of aggregate moisture state and breaking process of emulsion were also investigated. The specimens prepared under the SSD moisture condition aggregates performed better than the ones with air-dry condition. This is agreeable to the field experience with emulsions. On the other hand, the specimens in which aggregate was placed after the emulsion broke,

performed worse compared to those in which aggregate was placed before the emulsion broke for limestone and siliceous gravel. These results confirm the ability of the new testing protocol recognizing the factors that cause incompatibility.

In the third phase of the laboratory testing, specimens prepared with precoated aggregate were tested. Lightweight aggregate precoated with CSS-1h and limestone aggregate precoated with PG 64-22, were used for preparation of specimens, while AC-15 5TR, AC-15P, and CRS-2P were used as seal coat binder. Precoating level was also investigated as a factor that affects the performance of the seal coat. The new method responded to the precoating level for AC-15P and the overall performances of precoated aggregates were found to be far better than the uncoated aggregates of same type.

CHAPTER VI

INTERFACIAL BONDING (PULL-OUT) TEST

The performance of a seal coat mainly depends on the effectiveness of the aggregate binder bond. In this study, the aggregate cut into a cubical shape is embedded into the binder. After a certain time was elapsed, the aggregate particle was pulled out from the binder at a specific loading rate and the load-displacement distribution was recorded. This testing procedure was named Interfacial Bonding Test (IBT).

6.1 Test Procedure and Experimental Plan

Interfacial bonding test is a tensile test of a single aggregate, which is cut into a rectangular prism and embedded into the asphaltic binder. The aggregate particles were cut using a precision electric saw. Before embedding the aggregate into the binder, the dimensions of each particle were measured using a digital caliper. Later, these dimensions were used to calculate the stress and percent embedment. A motorized loading frame was used for testing. A digital load cell attached to the loading frame, and connected to a PC was used to measure the load and to record the load-displacement data. The loading rate used for the tensile test was 2 in/min for all IBT experiments. After the aggregate particle is pulled out the thickness of the binder film is measured and the percent embedment was calculated. Similar to the previous experiments described in Chapter V, experiments were conducted using both hot applied asphalt and asphalt emulsion.

6.2 Test Results

6.2.1 Hot Applied Asphalt

Specimens were prepared with AC-15 5TR + Limestone and AC-5 w/2% Latex + Rhyolite Gravel combinations. Factors such as the binder temperature at the time of aggregate application and aggregate dust content were the factors investigated in this phase. Tests were conducted with the aggregate particles placed in two positions, one fully embedded to the binder and the other one is just the bottom of the particle is touching to the binder (0 embedment). The difference between these two will provide the information about how embedment depth improves performance of the bond.

Table 6.1 shows the results for AC-5 w/2% Latex + Rhyolite Gravel. The first two columns indicate the dust content of the aggregate and the binder temperature at the time of aggregate application respectively. The third column shows the peak pull-out stress when just the bottom of the aggregate is in contact with the binder (0 embedment). The fourth column shows the peak pull-out stress measured when the aggregate is in fully embedded position. At the following column, the percent embedment of each aggregate particle is given. Sixth column is the pressure calculated per one percent embedment by dividing the peak stress value of the fully embedded by percent embedment. The last column shows the peak pull-out stress calculated from the difference between the two embedment positions (fully embedded and 0 embedment) divided by the percent embedment. In Table 6.2, the results for AC-15 5TR + Limestone material combination is presented in a similar manner.

Table 6.1: Peak Stress Pull-out for AC-5 w/2% Latex + Rhyolite Gravel

Aggregate Dust Content	Surface Temp (°F)	0 Embedment (psi)	Fully Embedded (psi)	% Embedment	1% Embedment (psi)	1% Embedment Comparison (psi)
Clean Aggregate	100	18.56	100.60	20	5.030	4.102
	140	17.19	121.32	23	5.275	4.527
Dusty Aggregate	100	21.00	96.07	20	4.804	3.754
	140	33.81	96.48	15	6.432	4.178

It can be seen that the results of the IBT are consistent within the factors tested. The clean aggregates performed better than the dusty aggregate, while the aggregates placed at 140° F performed better than the ones that were placed at 100° F. Based on 1% embedment comparison between the fully embedded and the 0 embedment position, the peak stress values follow this trend. As expected, based on 1% embedment comparison, the highest pull-out stress comes from the clean aggregate applied at 140° F and the lowest stress comes from the dusty aggregate applied at 100° F.

Figure 6.1 is the plot of peak stress for 1% embedment due to dust content and binder temperature. The consistency of the results can easily be seen within the factors plotted.

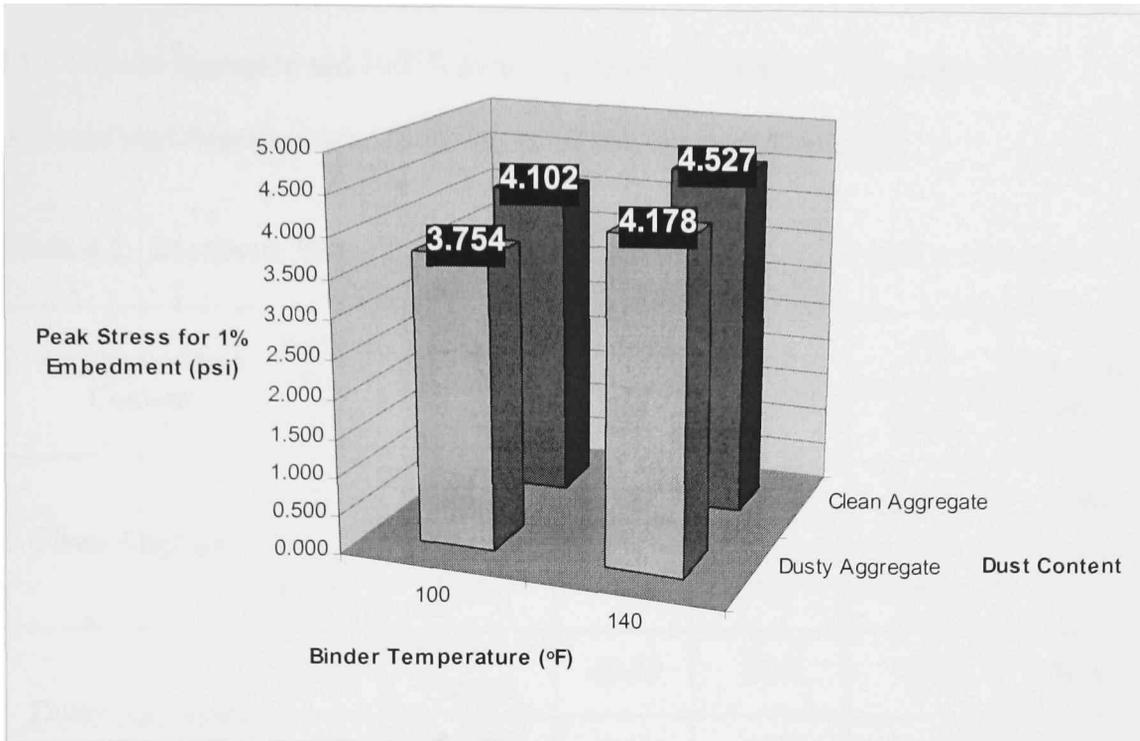


Figure 6.1: Peak Pull-out Stress versus Dust Content and Temperature of Binder at the time of Aggregate Application for AC-5 w/2% Latex + Rhyolite Gravel

Based on 1% embedment, the clean aggregates of AC-15 5TR + Limestone showed higher stress values than AC-5 w/2% Latex + Rhyolite Gravel. On the other hand, dusty aggregate showed lower stress values than AC-5 w/2% Latex + Gravel. This can be explained by the fact that the gravels do not have dust naturally while the limestone does. Another possible explanation is, during the preparation of dusty aggregate, there was not a perfect control over the dust content. This means the dust content of the limestone that is used in the tests might be much higher than the gravels since it was not controlled precisely. The binder was not able to wet the aggregate when it is in dusty and 0 embedment condition. Therefore, the strength of the bond was not measured at those conditions.

Again for this combination, the highest and the lowest stress values comes from 140° F clean aggregate and 100° F dusty aggregate, respectively. The stress values decrease significantly when the aggregate surface is covered with dust.

Table 6.2: Interfacial Bonding Test Peak Stress Results for AC-15 5TR + Limestone

Aggregate Dust Content	Surface Temp (°F)	0 Embedment (psi)	Fully Embedded (psi)	% Embedment	1% Embedment (psi)	1% Embedment Comparison (psi)
Clean Aggregate	100	24.77	84.27	15.0	5.618	3.967
	140	14.47	90.38	10.6	8.526	7.161
Dusty Aggregate	100	No Bond Formed	42.64	21.8	1.956	N/A
	140	No Bond Formed	75.07	22.2	3.382	N/A

6.2.2. Asphalt Emulsions

Specimens prepared using CRS-2P emulsified asphalt with lightweight aggregate and siliceous gravel were tested with IBT. Factors such as breaking time, curing time, and aggregate moisture content were also incorporated in the test program. Similar to the previous series of experiments, two moisture states for the aggregates, air dry and SSD were used. Curing times of 3 hours and 10 hours after the application aggregate were used. Effect of breaking process was controlled by applying aggregate immediately after the application of the emulsion (before breaking) and 75 minutes after application of the

emulsion (after breaking). Different from tests with hot asphalt, only the fully embedded condition was tested.

Peak pull-out stress results measured are given in Table 6.3. Since all aggregates were fully embedded, only the stress value per 1% of embedment is calculated. The stress values are much lower than the stress values for hot asphalt. This is probably due to emulsion was still soft and not completely cured yet at the time of testing. Stress values of SSD condition aggregates are lower than the ones prepared with air dry condition aggregates. These results contradict with the earlier new testing protocol results presented in Chapter 5. The curing times of the IBT specimens were 3 and 10 hours while the other specimens cured 30 hours. This is an indication of the effect of water to the aggregate emulsion bonding. The bond gains its full strength when there is no water at the system. This statement also suggests that the emulsions were more vulnerable to the various weather conditions soon after construction.

The peak stress values for after breaking applied aggregate were lower than before breaking applied aggregates for the SSD condition aggregate. However, the air dry aggregates did not follow this trend.

6.3 Conclusions

In this part of the study, the bonding between the aggregate and the binder were investigated. It is mentioned that this bonding is the main factor that affects the performance of the seal coats. However, there are various factors that also influence bonding. It can be concluded that there is a correlation between the strength of the

aggregate binder bond and the factors of incompatibility. The factors for hot asphalt tested were dust content and the binder temperature at the time of aggregate application. For emulsified asphalt, the factors such as curing time, breaking time, and moisture content were investigated.

Table 6.3: Interfacial Bonding Test Peak Stress Results for CRS-2P with Limestone and Siliceous Gravel

Moisture Content	Curing Time (Hours)	Before Break						After Break					
		Siliceous Aggregate (psi)	% Embedment	Stress per 1% Embedment (psi)	Limestone (psi)	% Embedment	Stress per 1% Embedment (psi)	Siliceous Aggregate (psi)	% Embedment	Stress per 1% Embedment (psi)	Limestone (psi)	% Embedment	Stress per 1% Embedment (psi)
Air-Dry	3	24.07	48	0.501	21.63	52	0.416	13.08	50	0.262	20.64	39	0.536
	10	35.78	48	0.745	21.52	43	0.500	30.3d	40	0.759	28.38	41	0.697
SSD	3	22.23	40	0.556	5.30	41	0.129	24.22	35	0.692	2.83	89	0.0318
	10	16.59	48	0.345	23.09	47	0.491	38.89	40	0.972	19.92	45	0.443

The effect binder temperature at the time of aggregate application was identified with the IBT precisely. Both material combinations responded to this factor but AC-15 5TR + Limestone improved much more than AC-5 w/2% Latex + Rhyolite Gravel. This might be because of the surface texture of the aggregate. The binder AC-5 w/2% Latex is

less viscous than the AC-15 5TR at 100° F and 140° F. One could think AC-5 will more effectively penetrate into the pores of the aggregate due to its high fluidity, but the results show the opposite trend. Surface texture of the aggregate might be the reason for these results.

The IBT also responded to the dust content factor. Due to the lack of control over the dust content, the results do not show any clear trend for 0 embedment. On the contrary, the fully embedded position results suggest that bond strength decrease significantly when dust is present, particularly for the limestone.

The comparison of the two material combinations shows that AC-15 5TR + Limestone is the better combination, but this is when no dust is present at the aggregate surface. The performance of this material combination decreased significantly when dust is present on the aggregate surface.

The peak bond stresses were much lower for emulsified asphalt than for hot applied asphalt. This could be due to the low curing times used for emulsion. The emulsion was still soft when tested even for the longest curing time, 10 hours. The IBT was successful in identifying the before/after break effect for SSD condition aggregates, while it was unsuccessful for air-dry condition aggregates. Moreover, the IBT distinguished the difference between two curing times.

IBT can be used to evaluate the aggregate-binder bond for different material combinations and factors. The test is sensitive for many factors that affect the performance of the bonding.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

A new performance based testing protocol for seal coats (chip seals) and surface treatments is developed in order to identify the problems associated with aggregate-binder incompatibility. This protocol is easy to perform and the equipment needed can be assembled as at a reasonable cost. It can be easily performed in a TxDOT district lab, which has limited equipment availability.

The process of developing the new protocol started with conducting a literature review on the subject. This study revealed that available test methods are unable to predict performance of the bonding primarily because they do not effectively simulate the field conditions in the laboratory. The effects of wetting/drying and freezing/thawing have to be effectively reproduced for the procedure to be successful. The protocol development continued with constructability review. This review was done in two parts. The first part included site visits during construction while the second part was a follow-up monitoring study. The goal of the first part was to come up with scenarios of incompatibility. Moreover, information about material combinations and seal coat construction practices were collected. Field seal coat specimens were collected on steel plates and were brought back from these visits along with the material samples. A year after the construction site visits, the follow-up evaluation study is made to observe the performances of these test sections. The comparison of the follow-up study and the

testing of the specimens brought from the field with the new protocol verified the new testing protocol.

The laboratory portion of the study started with developing the new protocol. Various specimen conditioning methods and durations were tried but 3 cycles of 16 hours soaking at room temperature and 8 hours of freezing at -25°C (-13°F) followed by 48 hours of soaking at room temperature was found to be the most effective conditioning protocol. After this conditioning process the specimens were subjected to impact loading with MPH. The specimens brought from the field visits were also tested with this protocol. The test results of the specimens brought from 13 test sections matched with the follow-up study results, while 2 sections, Lufkin and Brownwood III, did not. The Lufkin test section did experience an early thunderstorm after the construction and the Brownwood test section have high volume traffic. These might be the possible explanations of these mismatches, because the specimens brought to the lab did not experience such storm and traffic loads. It is known that the next days after the construction is critical on the performance of the seal coat. On the other hand, the compaction under the traffic helps the seal coat by providing high embedment depth. Due to this, most of the time the aggregate loss occurs outside and between the wheel paths not on the wheel paths. The specimens brought from the field aged for a year any kind of conditioning or compaction in the lab and this might be the reason for mismatches.

After the testing protocol was established, the laboratory testing part of the study was conducted. The experimental plan was divided into three parts; hot applied asphalt,

asphalt emulsion and precoated aggregate. Factors including dust content, embedment depth, the time elapsed between the application of aggregate and rolling and surface temperature were included in the experiments. In addition, a “good” and a “poor” combination of aggregate and the binder were also used in the preparation of the specimens. AC-15 5TR + Limestone (good) and AC-5 w/2% Latex + Siliceous Gravel (poor) were those material combinations. The dust content factor was tested with only limestone since dust is not a problem with siliceous gravel. Micro Deval (MD) test is used to evaluate the dust generation potential of aggregate. Washed and oven dried aggregate was used for 0 dust while 2-min MD and 5-min MD were used for medium and high dust contents. The MD test was used to simulate the dust generated in transportation and handling of aggregates and it is a part of the testing protocol. Results from new test method showed that it can identify between “good” and “poor” performing seal coats.

In the emulsion phase of the experimental plan, the effect of aggregate moisture content and the breaking process were tested using specimens prepared with CRS-2P emulsified and three aggregates (Limestone, Lightweight Aggregate, and Siliceous Gravel). The overall performances of the emulsion specimens were far better than the hot asphalt ones. The lightweight aggregate did not lose any aggregate in the after breaking scenario while gravel and limestone lost significant amounts of aggregate. It can be concluded that the test can identify the effect of breaking process of emulsions on bonding. The aggregates with two different moisture contents, air-dry and SSD, were used in preparing specimens. Based on field experience, the SSD condition aggregates work well with emulsions. The test results also led to the same conclusion.

The pre-coated aggregate part included testing the effects of pre-coating binder type and pre-coating level for different aggregate-binder combinations. Limestone pre-coated with PG 64-22 and Lightweight pre-coated with CSS-1h were the pre-coated aggregates. The seal coat binders used were: CRS-2P, AC-15 5TR, and AC-15P. The pre-coating level was based on percent coating from visual evaluation and was determined as 50% and 90% pre-coating levels.

The performances of pre-coated aggregates indicate that the problem associated with dust content can be solved with pre-coating. The uncoated aggregates can result in significant aggregate loss, which may be rectified by pre-coating. Within the specimens prepared with AC-15P, the test identified the effect of different pre-coating levels. Both emulsion and AC pre-coated aggregates performed excellently.

The results of different material combinations under various experimental conditions suggest that the limestone should be pre-coated with hot asphalt. The use of AC-5 with siliceous gravel is not recommended due to high probability of failure. On the contrary, use of CRS-2P with Lightweight is highly recommended. When siliceous gravel is used with CRS-2P, the breaking process of the emulsion should be controlled precisely to avoid possible failure. When AC-15P is used with limestone, the aggregate should be pre-coated, preferably to at least 90% pre-coating level.

The variability of results between replicates was significant at some instances. Care must be taken to use the same condition when all replicates are tested. This may be particularly important during the impact test procedure. There are a lot of factors come into play with the impact apparatus. During the drop of MPH, the load bounced back

from the steel plate and hits the plate once more. In addition, the specimen plate and the ¼ in top plate were not fixed. Sometimes these plates displaced during testing. In order to eliminate this problem, the walls of the box had to be made higher and the specimen plate and the top plate will be placed as previously. The higher walls will not allow the plates to displace, thus causing all the energy to be absorbed by the test specimen.

The last phase of the study was the investigation of the bonding between the aggregate and the binder using the Interfacial Bonding (pull-out) Test. This was conducted for hot applied asphalt and emulsified asphalt. The aggregates cut into cubical shapes embedded into the binder were pulled at a constant loading rate, while the load values were captured to a PC. These load values were converted to stresses using the known area of the aggregate particles. The embedment depth was also measured.

For hot asphalt, specimens prepared using AC-5 w/2% Latex + Rhyolite Gravel and AC-15 5TR + Limestone were prepared. Factors such as dust content and binder temperature at the time of aggregate application were tested. The specimens were prepared with 0 embedment and fully embedded positions.

For emulsified asphalt, factors such as moisture content, curing and breaking processes of emulsions were investigated. CRS-2P emulsified asphalt with Limestone and Siliceous Gravel were used as material combinations. All the aggregates were in fully embedded positions for emulsified asphalt.

The AC-15 5TR + Limestone combination performed better than the AC-5 + Gravel combination when no dust was present. On the contrary, when dust was present on limestone aggregate surface, the peak stress values decreased noticeably. When the

binder temperature was high at the time of applying aggregate, the stress values increase based on per 1% embedment peak stress. For the 0 embedment position with AC-15 + Limestone, the binder did not wet the aggregate when it was dusty, resulting in zero bond.

The IBT did not respond to the breaking factor in the emulsion part, primarily because even after 10 hours, the emulsion was still curing. However, different curing times resulted in different peak stress values, indicating that the more the emulsion cures, the higher the stress values are. Aggregate in the SSD condition had lower stress values compared to the air-dry condition. The peak stress values were much lower than for hot applied asphalt due to the short curing times.

The IBT is sensitive to the aggregate-binder condition and factors affecting the aggregate-binder bond. It can be used to measure the bond strength for different material combinations under different conditions. Further research should be done on modeling the stress-displacement patterns for different aggregate and binders.

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APPENDIX A
DATA COLLECTED FROM CONSTRUCTABILITY REVIEW

Table A-1. Data Collected from the Constructability Review

Date	District	County	Highway	Location
8/6/2002	TOWNWOOD	Coleman	US84SB	Taylor-Coleman County Line
8/5/2002	Brownwood III	Stephens	FM 1148	Possom Kingdom Lake
7/25/2002	Brownwood II	Coleman	FM 503	Underwood Dr. (between Voss & Valera)
7/1/2002	Abilene	Haskell	FM1720	7.8 mi. E of U.S. 277
7/11/2002	Fort Worth	Tarrant	SH 121/Shoulder	North Near DFW
6/26/2002	Lubbock	Floyd	FM 602	S. of U.S. 62, 1.6 mi. N. of TRM 194
6/18/2002	Lufkin	San Jacinto	U.S. 59 NB/ Shoulder	S. City Limit of Shephard, TRM 456 + 1.8 mi.
6/19/2002	Lufkin	San Jacinto	Loop 424	across from EMS station
6/18/2002	Lufkin	San Jacinto	U.S. 59 NB/ Main Lane	S. City Limit of Shephard, TRM 456 + 1.8 mi.
8/23/2002	Odessa	Midland	FM 3095 SB	FM 1379 intersection
8/14/2002	San Antonio	Kendall	Ranch Road 473	Ref # 480
8/22/2002	Tyler	Rusk	U.S. 84 WB	1140 ft. W. of TRM 832
9/4/2002	El Paso	El Paso	FM 76	1000 ft. S. of Horizon Rd.
9/13/2002	Childress	Wheeler	Fm 1443 SB	approx. 2 mi. N. of IH 40
8/30/2002	Paris	Fannin	FM 1550	N. of Laddonia, 2 mi. E. of SH 50/34
6/6/2002	San Angelo	Coke	U.S. 277	25 mi. N. of S.A., N. & S. of Tennyson
6/7/2002	San Angelo	Coke	U.S. 277	U.S. 277 SB, shoulder, TRM 368
Date	District	Time of Construction	Ambient Temp. (°F)	Weather Conditions
8/6/2002	Brownwood II	3:00 PM	95	Partly Cloudy
8/5/2002	Brownwood III		102	Cloudy
7/25/2002	Brownwood II	2:30 PM		Partly Cloudy
7/1/2002	Abilene	5-6 PM	70	Cloudy, Windy
7/11/2002	Fort Worth	2:00 PM	83	Sunny
6/26/2002	Lubbock	9:00 AM	79	Overcast
6/18/2002	Lufkin	4-5 PM	79	Partly Cloudy
6/19/2002	Lufkin	9-10 AM	79	Partly Cloudy
6/18/2002	Lufkin	4-5 PM	77	Partly Cloudy
8/23/2002	Odessa	11:00 AM	90	Sunny
8/14/2002	San Antonio	9:30 AM		Partly Cloudy
8/22/2002	Tyler	11:30 AM	84	Partly Cloudy
9/4/2002	El Paso	11:45 AM		Sunny
9/13/2002	Childress	12:30 PM		Cloudy
8/30/2002	Paris	12:20 PM	85	Partly Cloudy
6/6/2002	San Angelo	1:30-4 PM	95	
6/7/2002	San Angelo	9:30 AM	80	

Table A-1. Data Collected from the Constructability Review (continued)

District	Surface Before Construction	Surface Temp. (°F)	Condition of Existing Surface	Comments
Brownwood II	Flushing	137	Flushed asphalt	Slightly pocked, porous, oxidized surface.
Brownwood III			Flushed asphalt on WP	Badly pocked, porous, oxidized surface (bet. WP)
Brownwood II		130	Slightly porous	
Abilene	Old Patch	105	Flushed asphalt	Not Oxidized
Fort Worth	New Patch	142	New Patch	
Lubbock			Smooth	Slightly Oxidized
Lufkin	Seal Coat	125		Slightly pocked, porous, oxidized surface.
Lufkin	HMAC	94	Flushed asphalt	Slightly porous, slightly oxidized
Lufkin	HMA	115	Smooth	Slightly porous, slightly oxidized
Odessa	Slight Flushing			Slight porous, slight oxidized surface
San Antonio	Flushing	100	Flushed Asphalt	Slight pocked, porous, oxidized surface
Tyler	HMA	110	Smooth Non porous	Slightly ocked, porous, oxidized surface
El Paso		116	HMAC Gr. Oxidized	
Childress	Seal Coat	95	Seal Coat	Slightly Pockedm prous
Paris	Old Patch, Old Seal		Smooth	Slightly pocked, porous
San Angelo				Slightly porous, slightly oxidized
San Angelo		92		
District	Aggregate Source	Aggregate Grade	Aggregate Condition	Aggregate Ratio (cu.yd.:sq.yd)
Brownwood II	TXI	L Gr 3	Uncoated	1:90
Brownwood III	Zack Burkett	PB Gr 4	Precoated	1:120
Brownwood II	Vulcan (Brownwood)	Gr 4M	Uncoated	1:90
Abilene		PB Gr 3	Precoated	1:105-112
Fort Worth	TXI	L Gr 5	Uncoated	1:120
Lubbock	Capital (Midland)	PB Gr 7	Precoated	1:110
Lufkin	TXI	P Gr 3	Precoated	1:100
Lufkin			Precoated	
Lufkin	TXI	PL Gr 3	Precoated	1:100
Odessa	Clemmons	P Gr 4	Precoated	1:125
San Antonio	Capital	PA Gr 4M	Precoated	1:110
Tyler	TXI	PL Gr 4	Precoated	1:115
El Paso	Jobe	P Gr 3M	Precoated	1:90
Childress	Bundy Pit	PB Gr 4M	Precoated	1:125
Paris	Marin Marrietta Sawyer,OK	P Gr 3	Precoated	1:90
San Angelo	CSA Materials (Turner Pit)	P Gr 3	Precoated	1:100
San Angelo	CSA Materials (Turner Pit)	P Gr 3	Precoated	1:100

Table A-1. Data Collected from the Constructability Review (continued)

District	Binder Source	Binder Type	Binder Rate (gal/yd ³) 0.52 to 0.54	Binder Rate Distribution Across Lane
Brownwood II	Koch Brownwood	CRS-2P		Variable
Brownwood III	Eagle Asphalt (Corpus)	AC15-5TR	.40	Variable
Brownwood II	Koch Brownwood	CRS-2P	.67	Uniform
Abilene		AC15-5TR	0.46	Uniform
Fort Worth	Koch Saginaw	CRS-2P	0.46	Uniform
Lubbock	Eagle Asphalt (Corpus)	AC 10 w/ 2% latex	0.45	Uniform
Lufkin	Wright Asphalt Trumbull	AC15-5TR	0.49	Uniform
Lufkin	Wright Asphalt (Trumbull)	AC15-5TR	0.48	Uniform
Lufkin	Wright Asphalt (Trumbull)	AC15-5TR	0.49	Uniform
Odessa	Eagle - Corpus	AC-5 with 2% latex	0.35	Uniform
San Antonio	Eagle Asphalt	AC-15P	0.3	Uniform
Tyler	Wright (Channelview, Tx)	AC15-5TR	0.39	Uniform
El Paso	Chevron (El Paso)	PG 58-28 w/19% TR	0.37	Uniform
Childress	Alon	AC-5 with 2% latex	0.5	Uniform
Paris		AC15-5TR	0.44	Uniform
San Angelo	Eagle Asphalt (Corpus)	AC15-5TR	0.41	Uniform
San Angelo	Eagle Asphalt (Corpus)	AC15-5TR	0.44	Uniform
District	Temp. of Binder in Distributor (°F)	Temp. of Binder just after Shot (°F)	Temp. of Binder when Spreading Aggregate (°F)	Temp. of Binder at Rolling (°F)
Brownwood II	170-180	132	112	112
Brownwood III	350	295	135	
Brownwood II	170	137	104	104
Abilene	340		108	106
Fort Worth	170			
Lubbock	260		92	103
Lufkin	350	136	130	115
Lufkin	350		112	108
Lufkin	350	136	130	128
Odessa	345	290	125	
San Antonio	350	140	108	108
Tyler	355	195		
El Paso	385		140	137
Childress	345		130	101
Paris	350			
San Angelo	350			
San Angelo	350	266		

Table A-1. Data Collected from the Constructability Review (continued)

District	Delta Time between application of binder and spreading of aggregate (min)	Delta Time between spreading of aggregate and compaction (min)	Roller Type
Brownwood II	7.5	3	Med. (3)
Brownwood III	4	5	Med. (4)
Brownwood II	7	5	
Abilene	4	5	Med.
Fort Worth	5.5	1	Med.
Lubbock	3	1	Med.
Lufkin	3	7	Med.
Lufkin	3.5	6	PT 240 R Ingersoll Rand
Lufkin	4	4	Med.
Odessa	2	2	Light
San Antonio	2	0.25	Cat PS-180
Tyler	3	0.0800	Dynapac
El Paso	5	0.5	Heavy
Childress	4	4.5	Medium
Paris	8	7	Cat 1508
San Angelo			
San Angelo	3	1	
District	Plot Cars Used?	How Long?	Actual Embedment Depth
Brownwood II			
Brownwood III			
Brownwood II	Yes		
Fort Worth			
Lubbock	No		
Lufkin	No		
Lufkin	Yes	5 min Intervals	
Lufkin	No		
Odessa	Yes		
San Antonio	Yes		
Tyler	Yes	5 min intervals	
El Paso	Yes	5 min intervals	10% or less
Childress	Yes	4-6 min intervals	
Paris	Yes		
San Angelo			
San Angelo			

Table A-1. Data Collected from the Constructability Review (continued)

District	Roller Passes	Additional Remarks	Field section have shaded areas?
Brownwood II	3	Samples not brushed at site, will be brushed in Lab. Retrieved 1 hour after rolled.	No
Brownwood III			
Brownwood II			
Abilene	3	very low traffic (maybe 10 cars in 4 hrs)	No
Fort Worth	2		No
Lubbock	3		No
Lufkin	6	TxDOT Inspector noted that there were too many fines in the aggregate.	No
Lufkin			Yes
Lufkin	5	ADT ~ 42,600 ypd	No
Odessa	3		No
San Antonio	3	Samples not brushed at site, will be brushed in Lab. Retrieved 1 hour after rolled.	No
Tyler	3	Plates were not brushed.	Few
El Paso	5		No
Childress	1hr/2000 sq.yds		No
Paris	3		Yes
San Angelo			
San Angelo			

APPENDIX B
STRESS-DISPLACEMENT CURVES OF
INTERFACIAL BONDING TEST

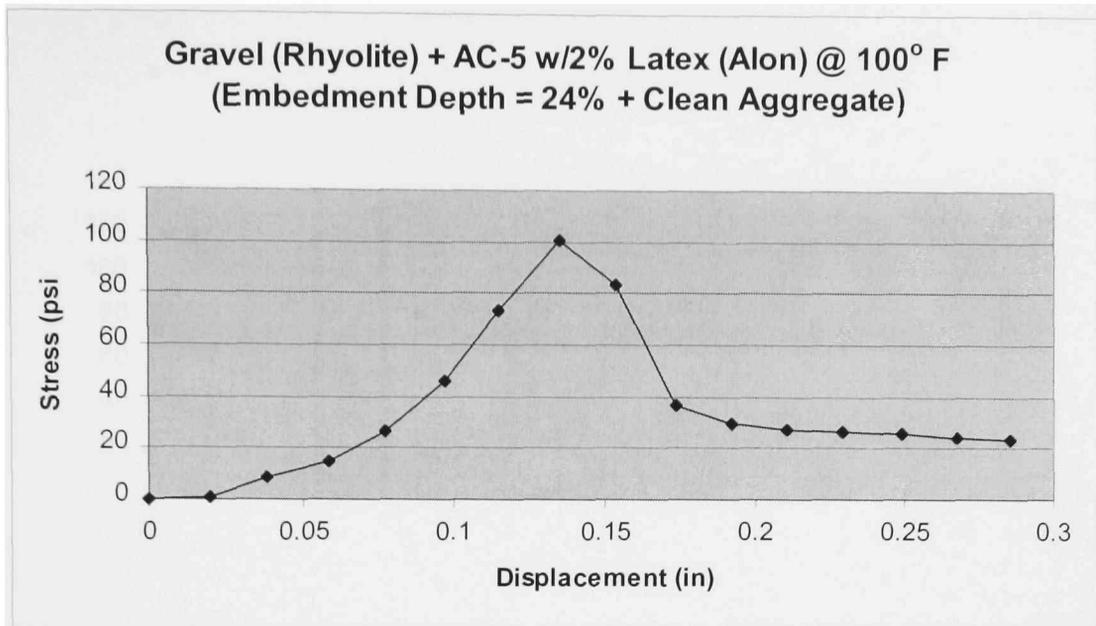


Figure B-1: Stress-Displacement Curve of Rhyolite + AC-5 @ 100° F (Fully Embedded)

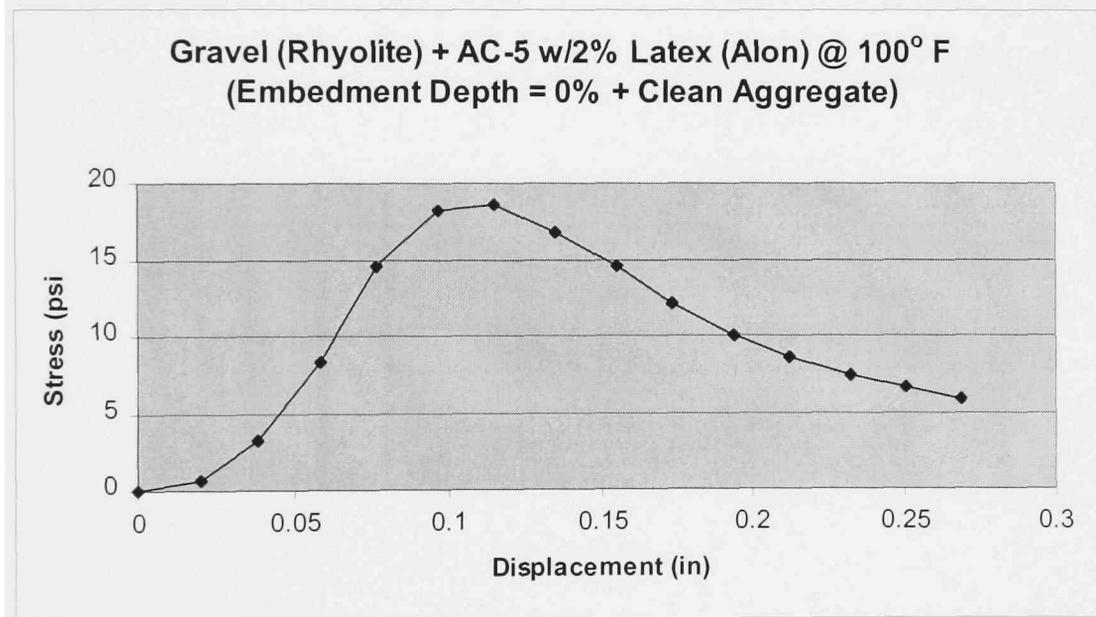


Figure B-2: Stress Displacement Curve of Rhyolite + AC-5 @ 100° F (0 Embedment)

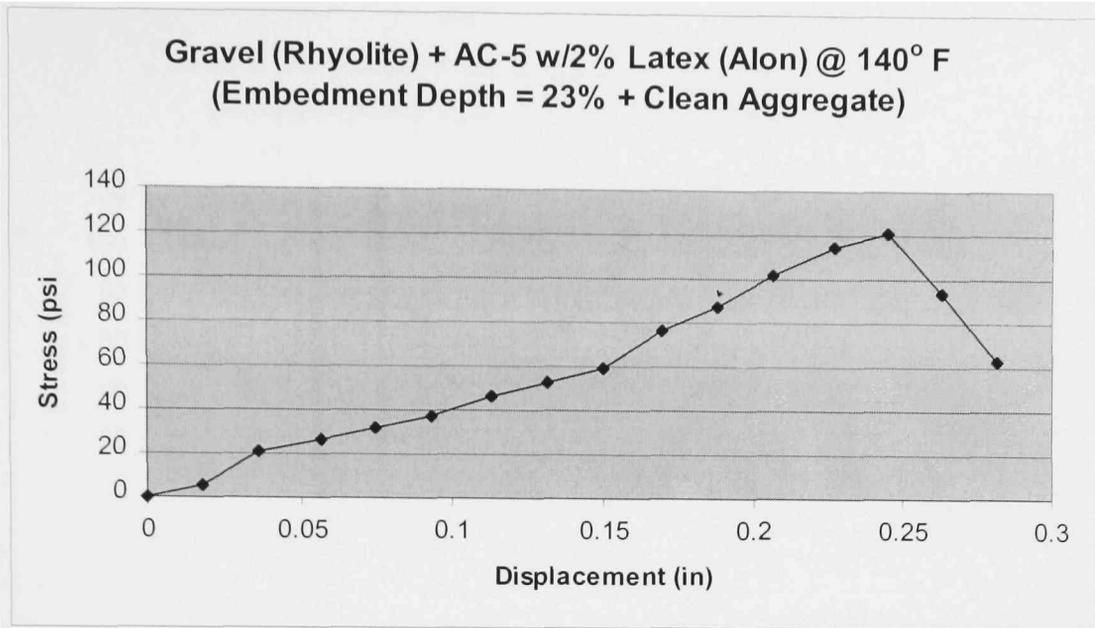


Figure B-3: Stress Displacement Curve of Rhyolite + AC-5 @ 140° F (Fully Embedded)

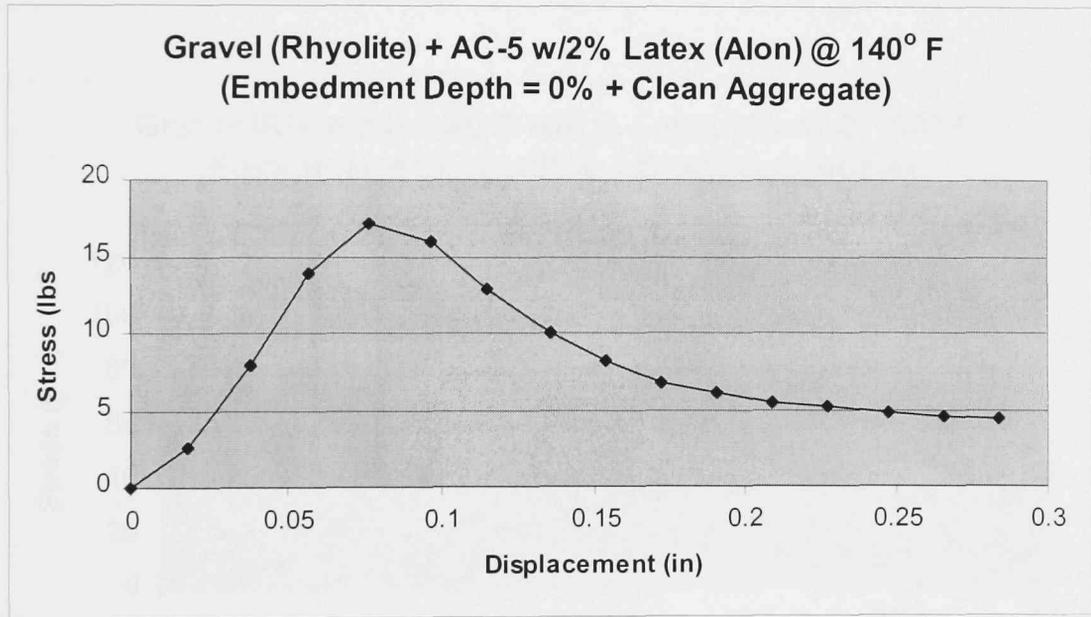


Figure B-4: Stress Displacement Curve of Rhyolite + AC-5 @ 140° F (0 Embedment)

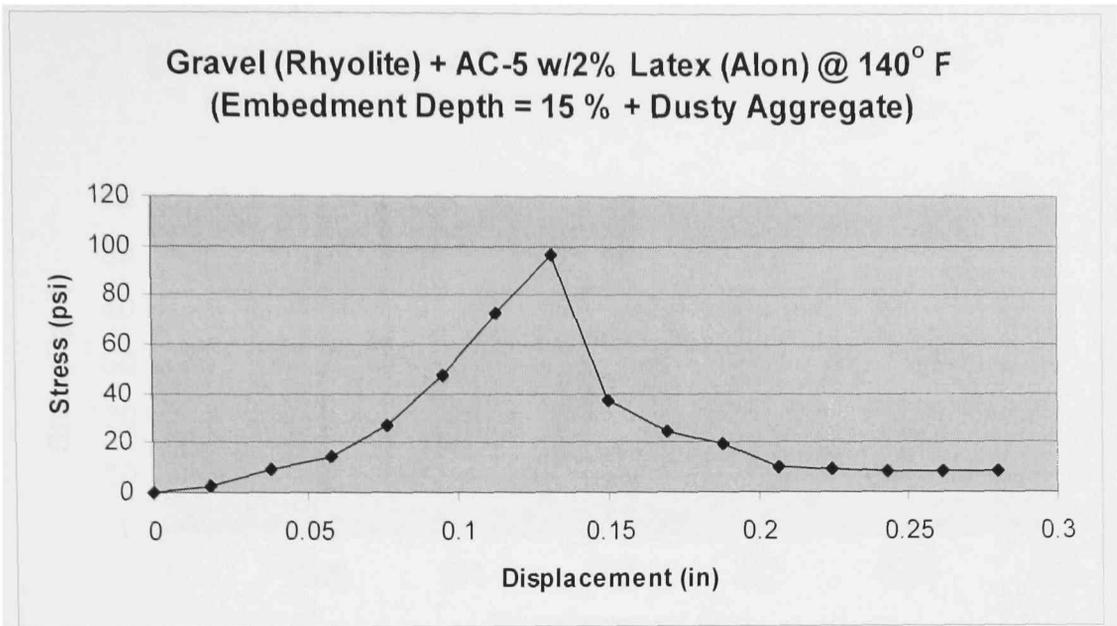


Figure B-5: Stress-Displacement Curve of Rhyolite + AC-5 @ 140° F (Fully Embedded, Dusty)

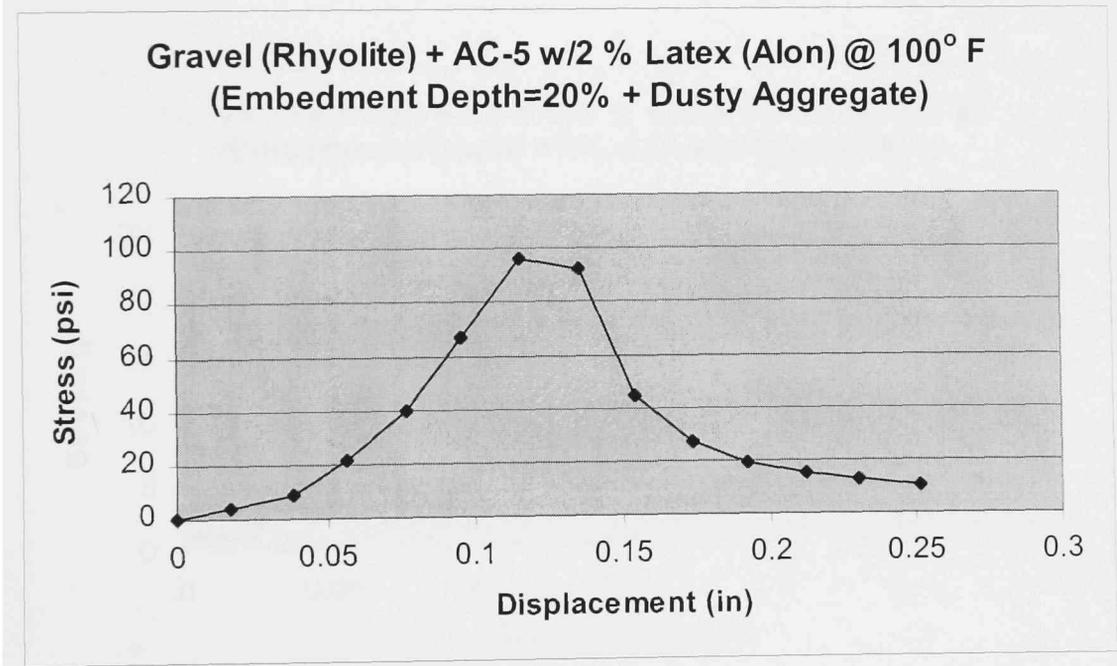


Figure B-6: Stress-Displacement Curve of Rhyolite + AC-5 @ 100° F (Fully Embedded, Dusty)

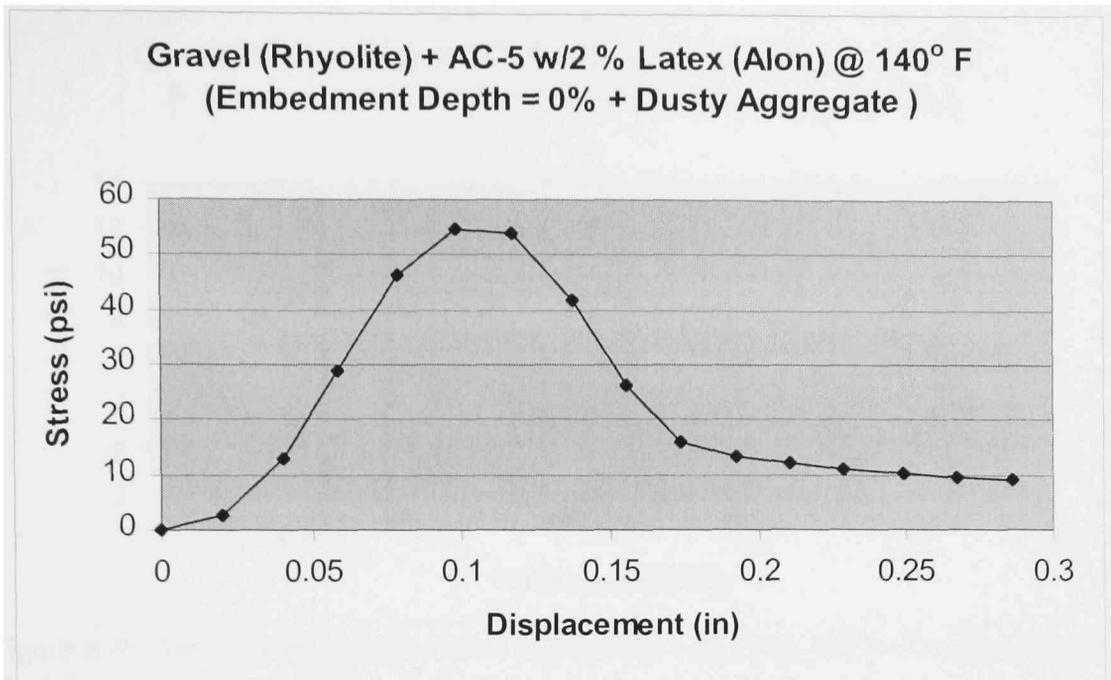


Figure B-7: Stress-Displacement Curve of Rhyolite + AC-5 @ 140° F (0 Embedment, Dusty)

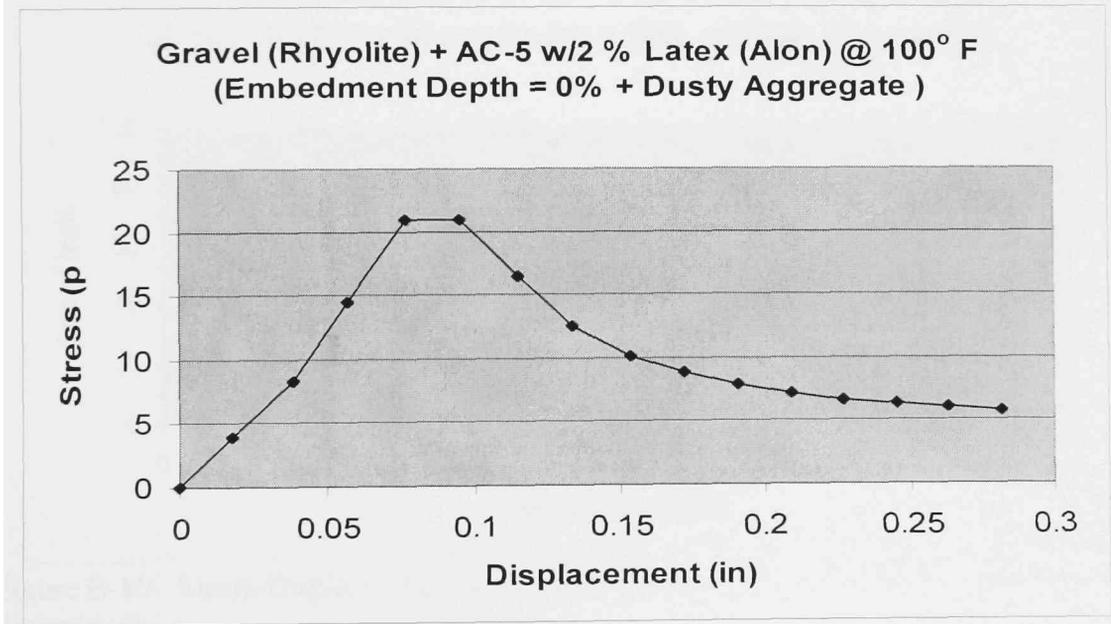


Figure B-8: Stress-Displacement Curve of Rhyolite + AC-5 @ 100° F (0 Embedment, Dusty)

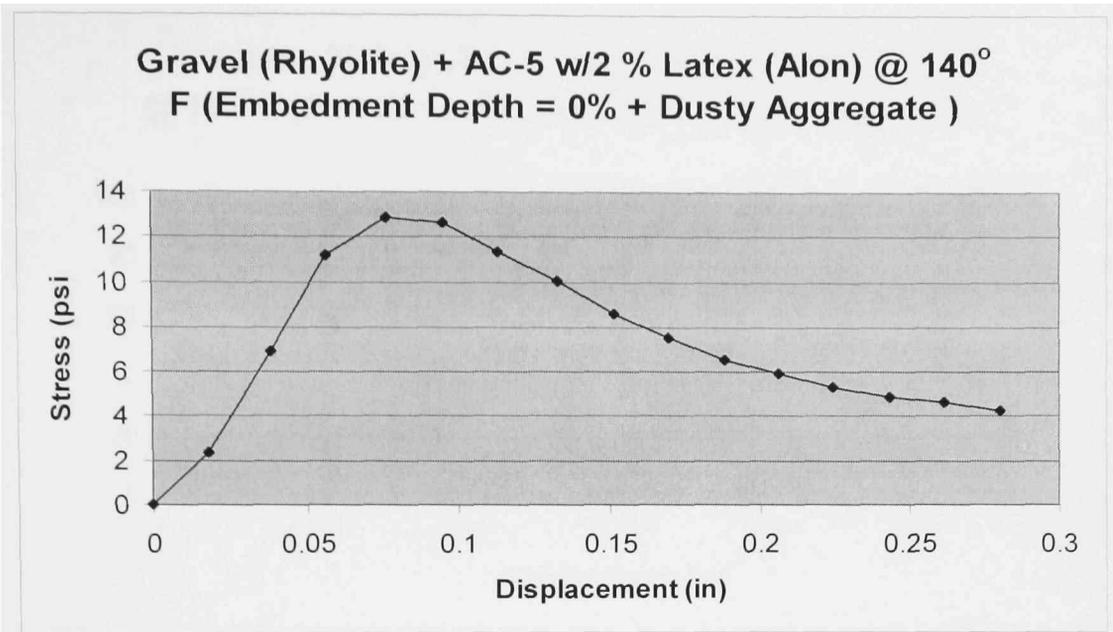


Figure B-9: Stress-Displacement Curve of Rhyolite + AC-5 @ 140° F (0 Embedment, Dusty)

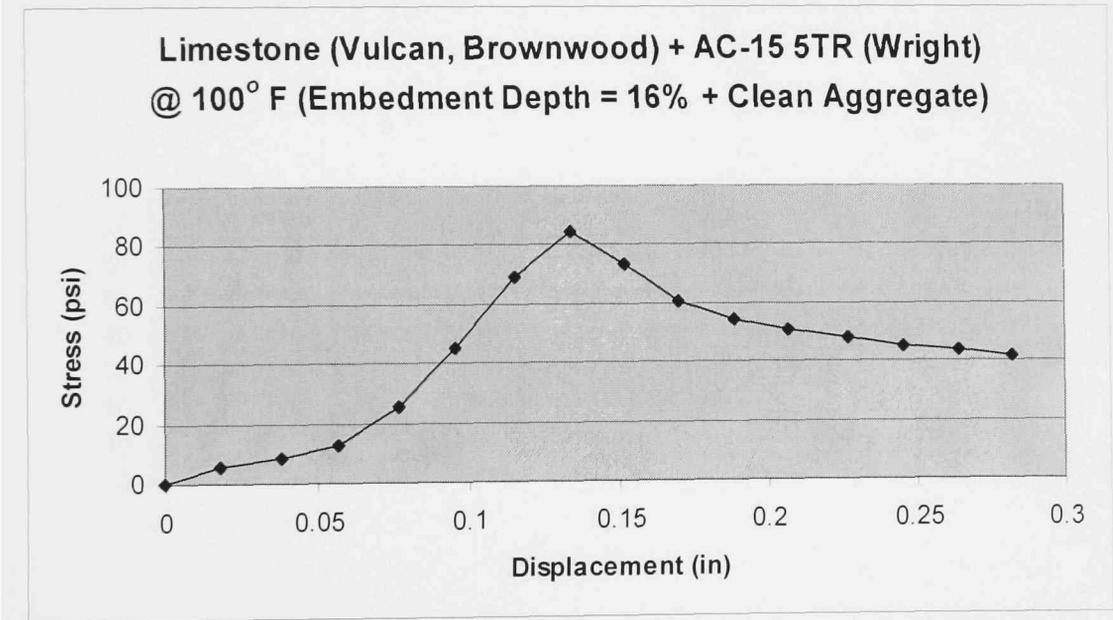


Figure B-10: Stress-Displacement Curve of Limestone + AC-15 @ 100° F (Fully Embedded)

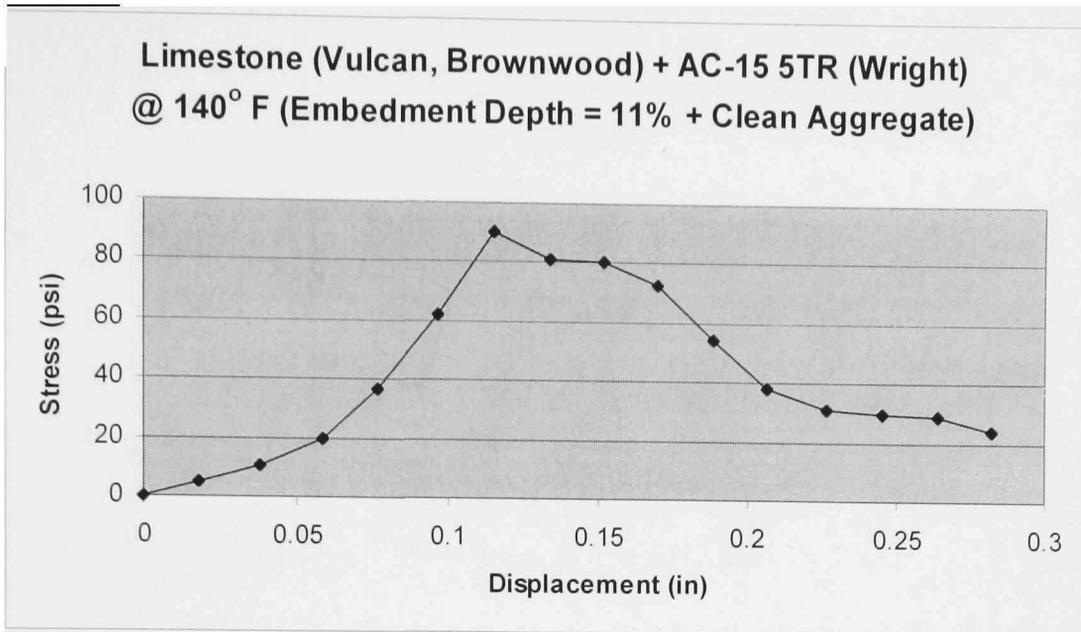


Figure B-11: Stress-Displacement Curve of Limestone + AC-15 @ 140° F (Fully Embedded)

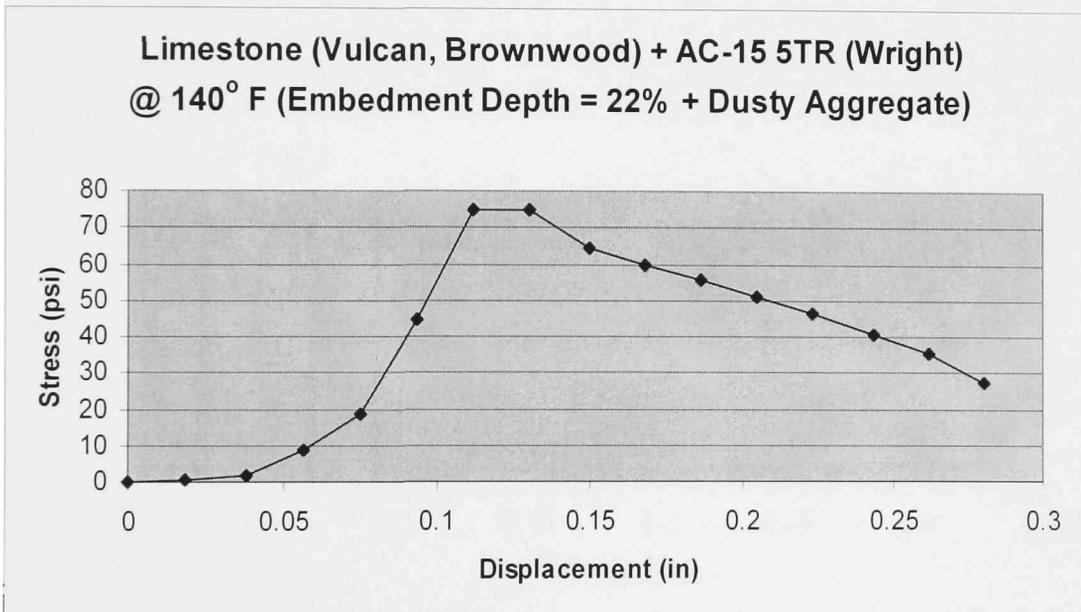


Figure B-12: Stress-Displacement Curve of Limestone + AC-15 @ 140° F (Fully Embedded, Dusty)

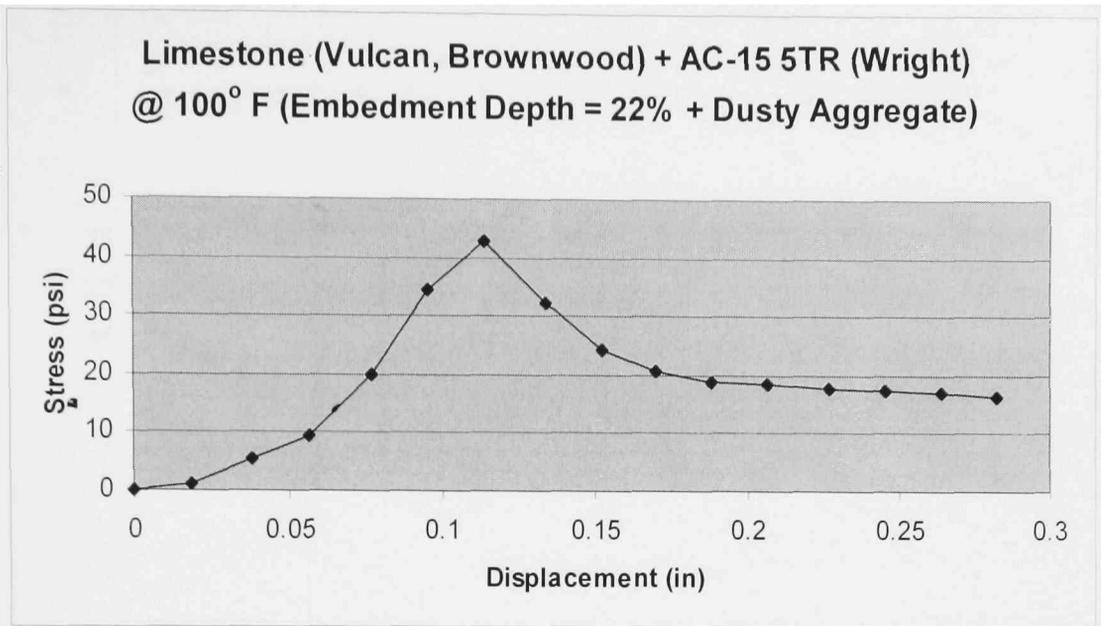


Figure B-13: Stress-Displacement Curve of Limestone + AC-15 @ 100° F (Fully Embedded, Dusty)

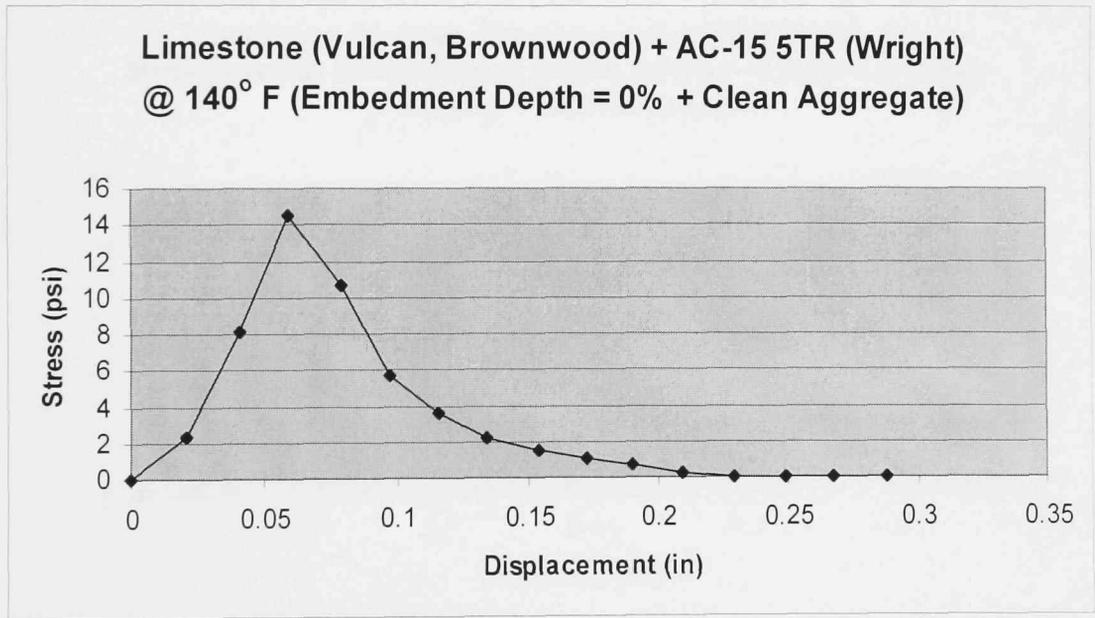


Figure B-14: Stress-Displacement Curve of Limestone + AC-15 @ 140° F (0 Embedment)

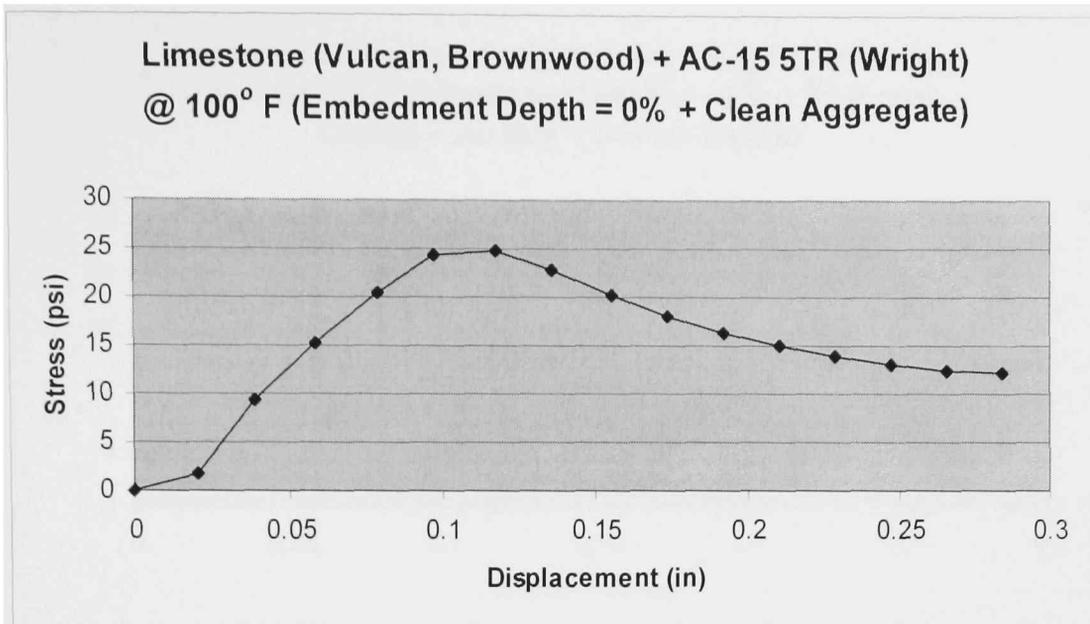


Figure B-15: Stress-Displacement Curve of Limestone + AC-15 @ 100° F (0 Embedment)

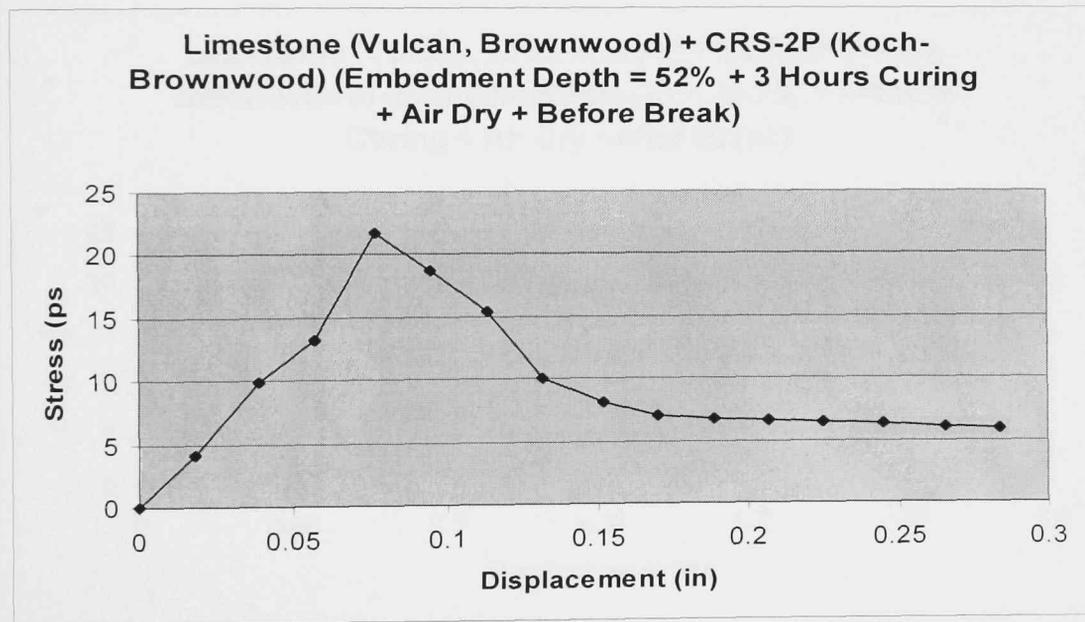


Figure B-16: Stress-Displacement Curve of Limestone + CRS-2P (3 Hours Curing + Air Dry + Before Break)

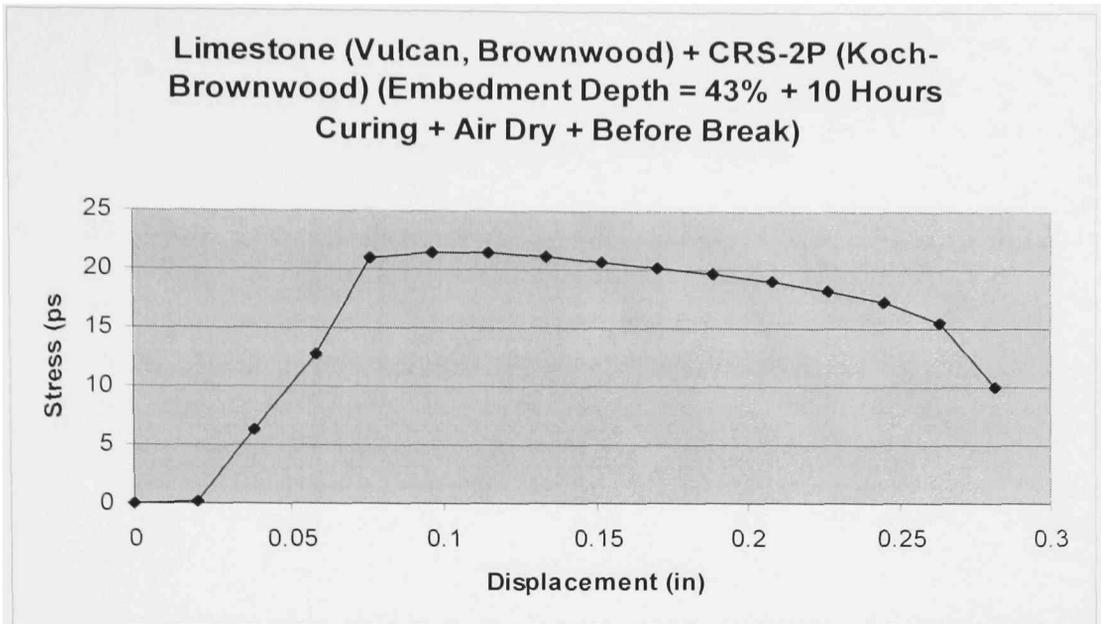


Figure B-17: Stress-Displacement Curve of Limestone + CRS-2P (10 Hours Curing + Air Dry + Before Break)

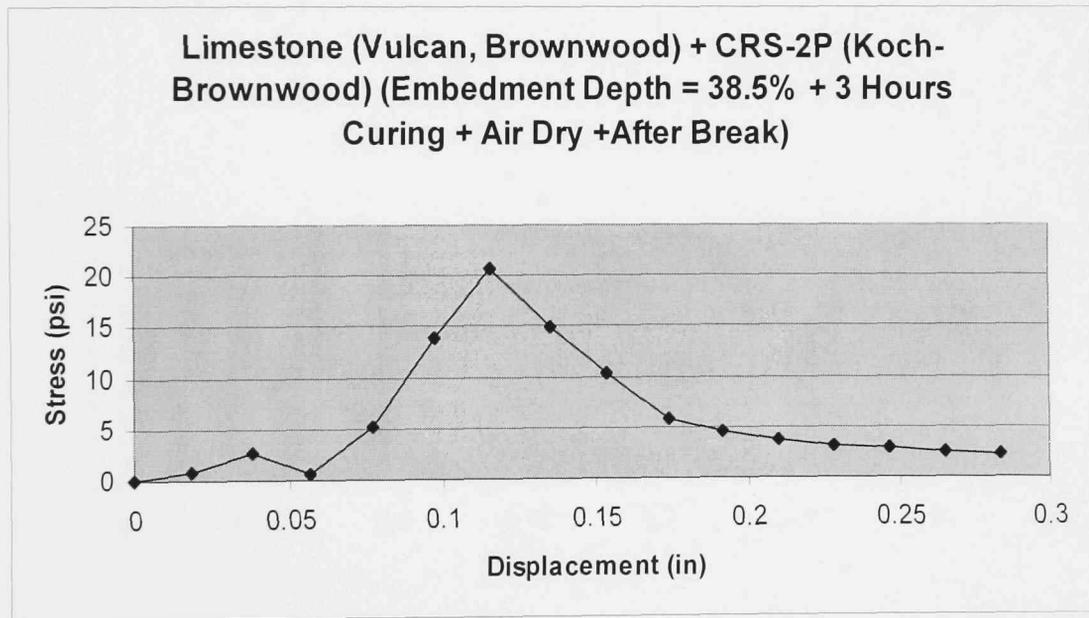


Figure B-18: Stress-Displacement Curve of Limestone + CRS-2P (3 Hours Curing + Air Dry + After Break)

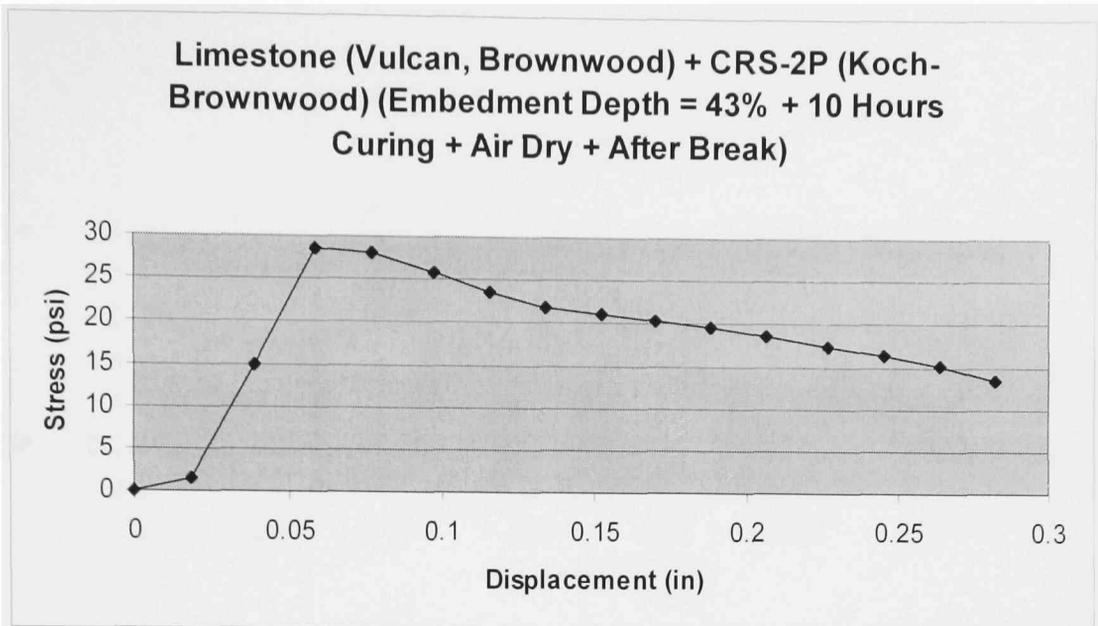


Figure B-19: Stress-Displacement Curve of Limestone + CRS-2P (10 Hours Curing + Air Dry + After Break)

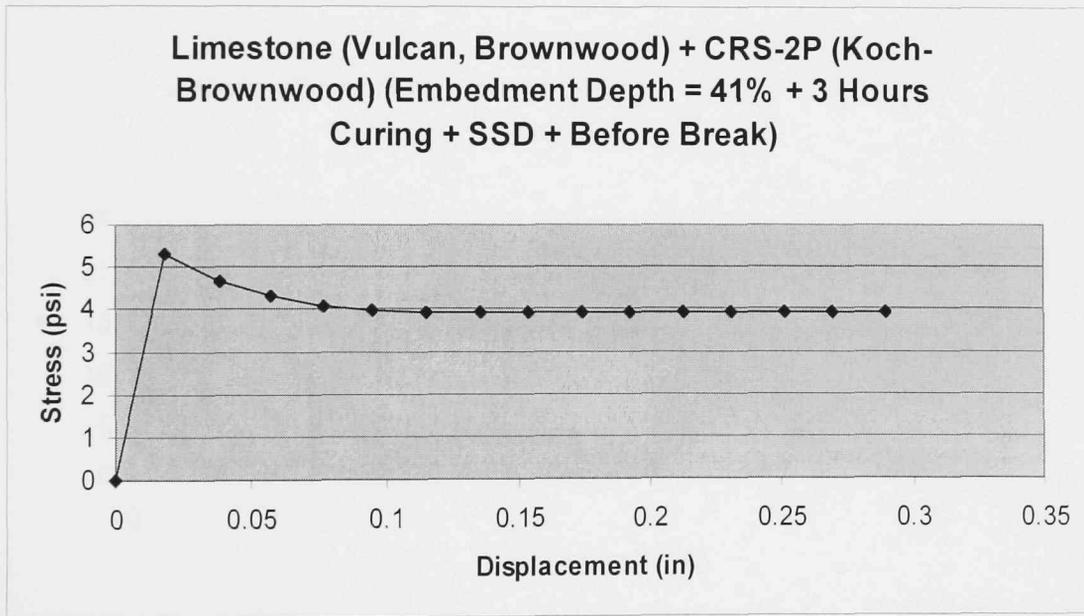


Figure B-20: Stress-Displacement Curve of Limestone + CRS-2P (3 Hours Curing +SSD + Before Break)

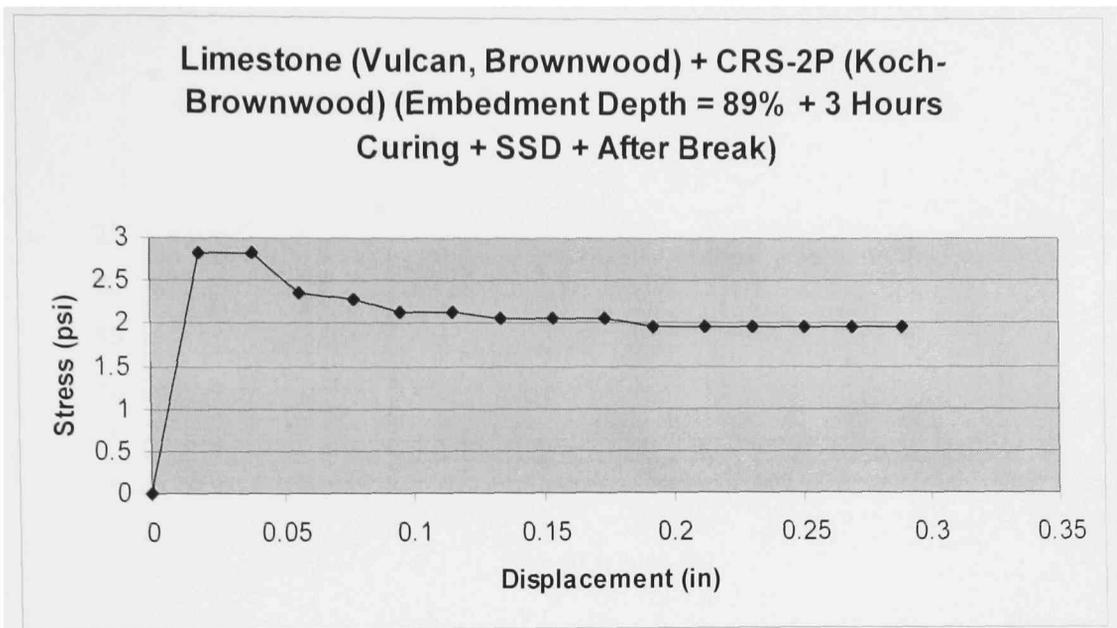


Figure B-21: Stress-Displacement Curve of Limestone + CRS-2P (3 Hours Curing + SSD + After Break)

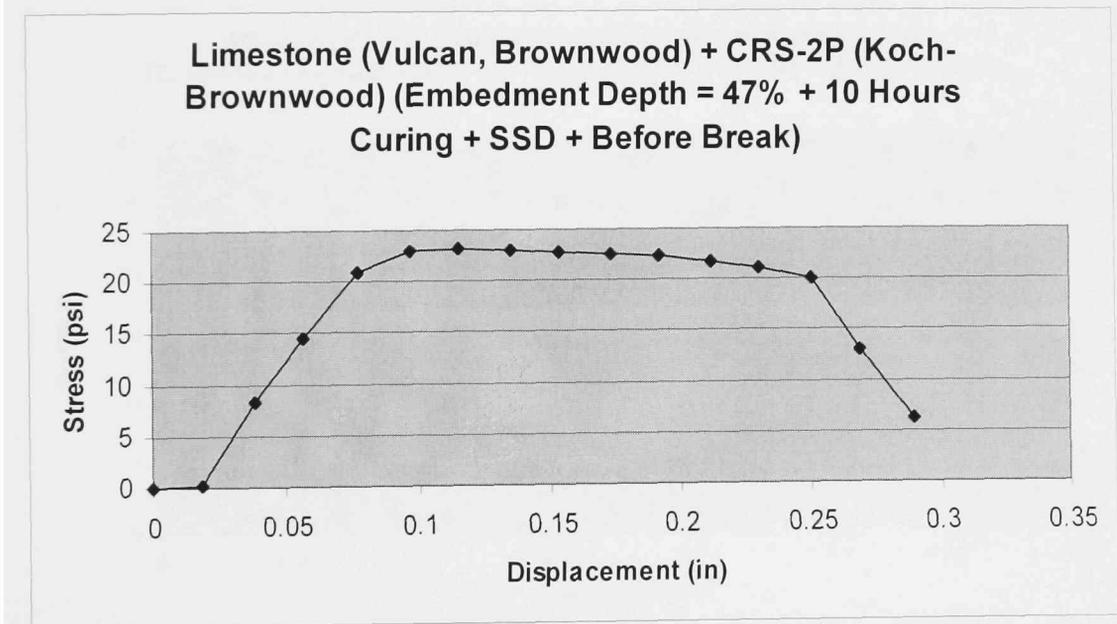


Figure B-22: Stress-Displacement Curve of Limestone + CRS-2P (10 Hours Curing + SSD + Before Break)

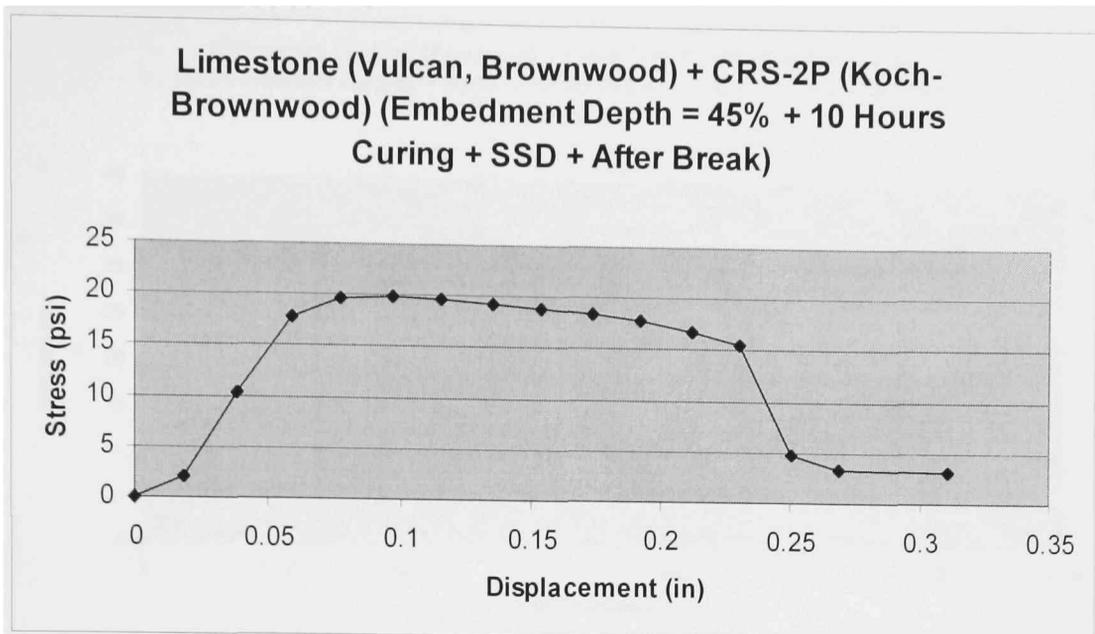


Figure B-23: Stress-Displacement Curve of Limestone + CRS-2P (10 Hours Curing + SSD + After Break)

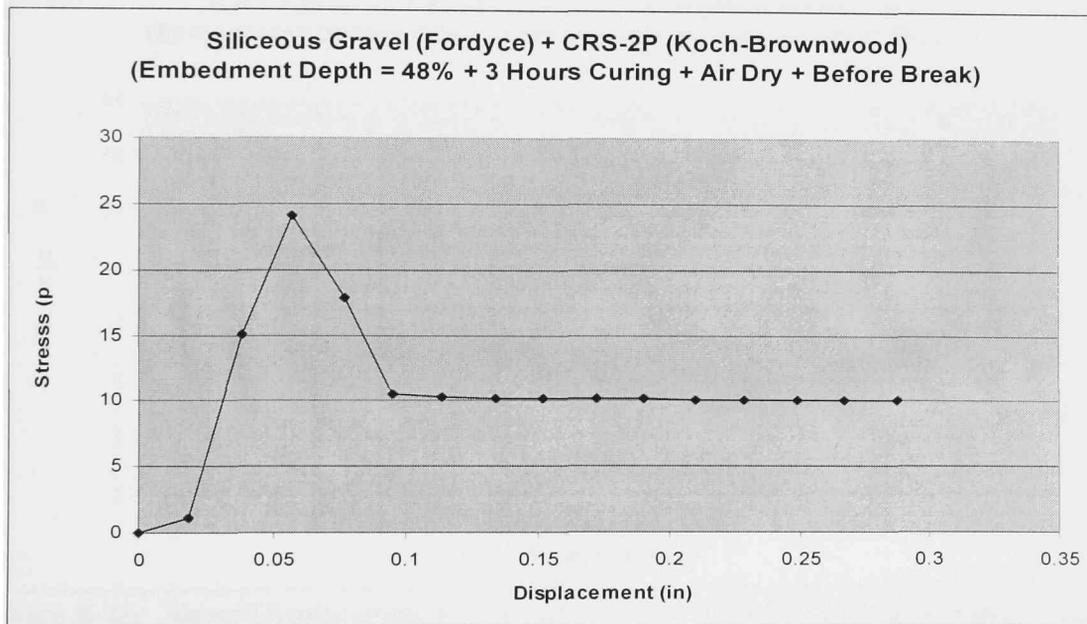


Figure B-24: Stress-Displacement Curve of Fordyce + CRS-2P (3 Hours Curing + SSD + Before Break)

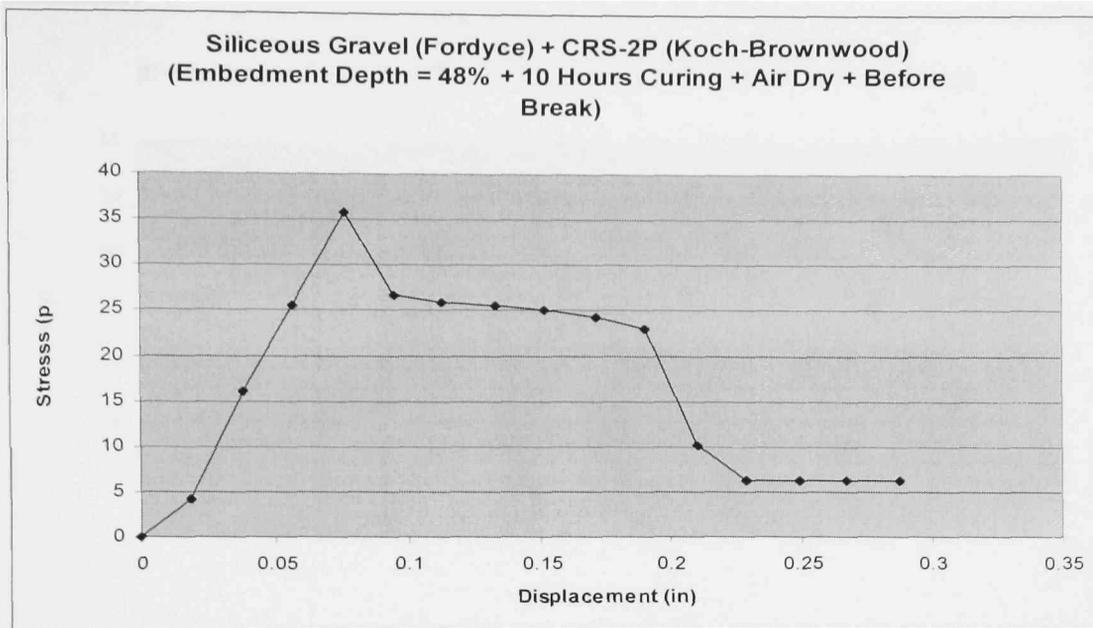


Figure B-25: Stress-Displacement Curve of Fordyce + CRS-2P (10 Hours Curing + Air Dry + Before Break)

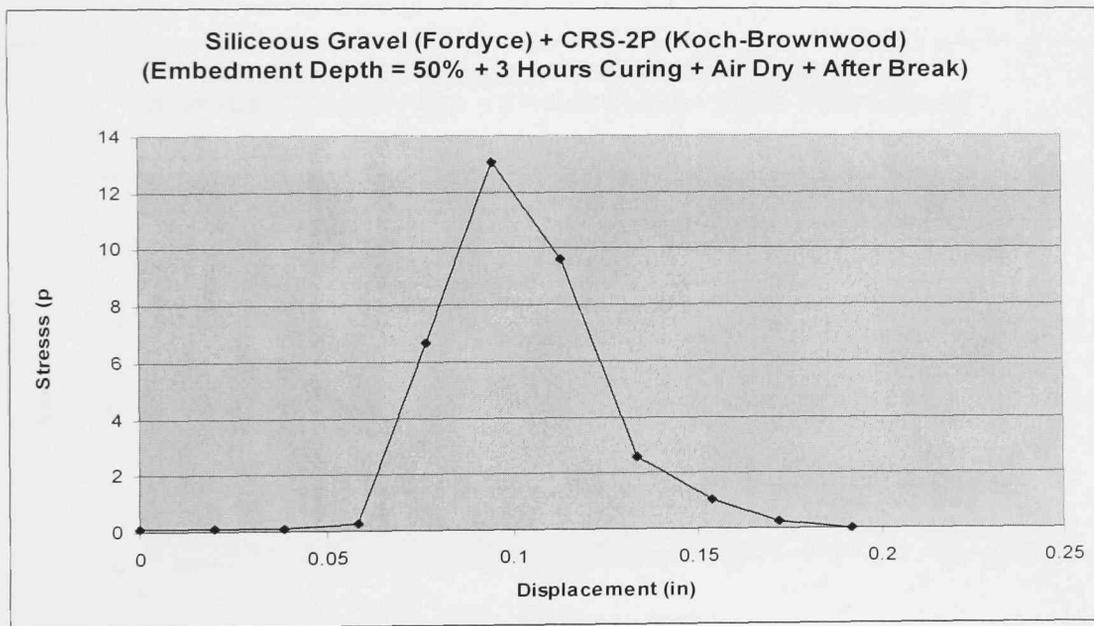


Figure B-26: Stress-Displacement Curve of Fordyce + CRS-2P (3 Hours Curing + Air Dry + After Break)

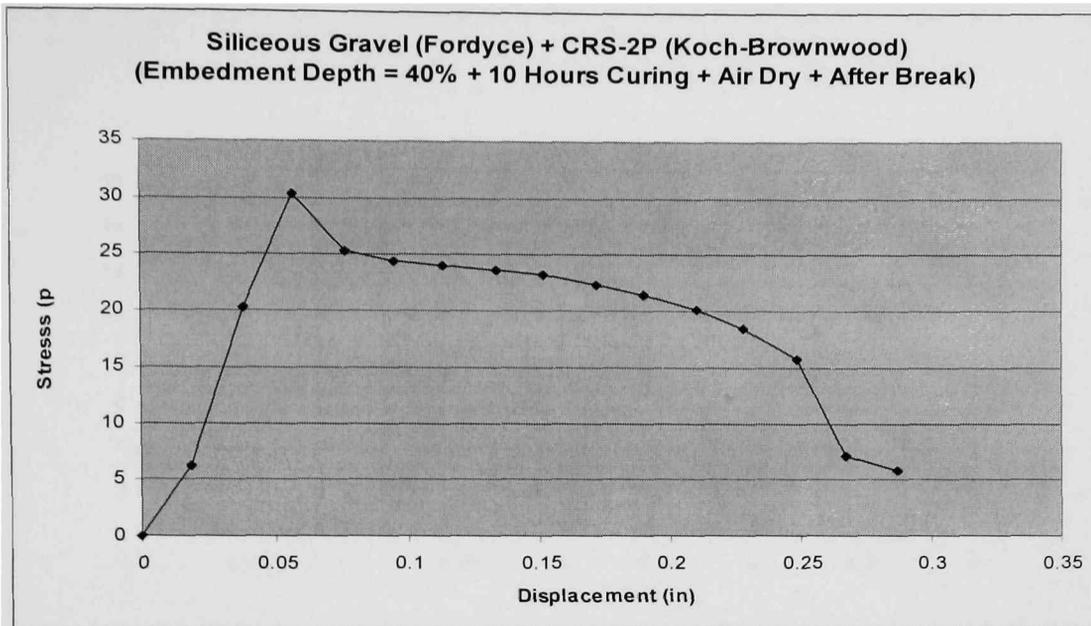


Figure B-27: Stress-Displacement Curve of Fordyce + CRS-2P (10 Hours Curing + Air Dry + After Break)

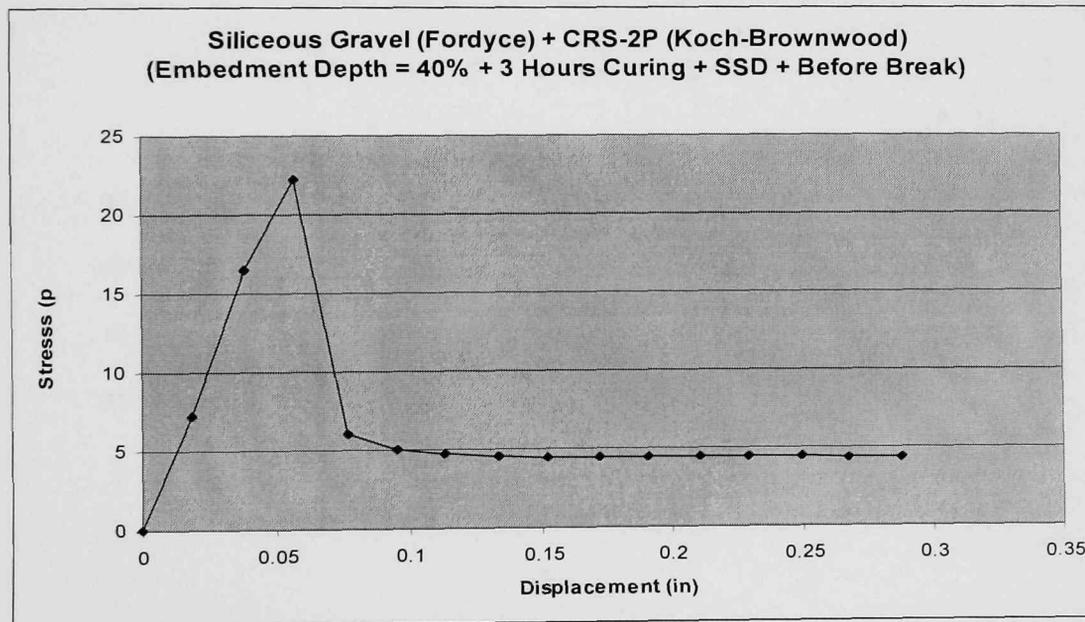


Figure B-28: Stress-Displacement Curve of Fordyce + CRS-2P (3 Hours Curing + SSD + Before Break)

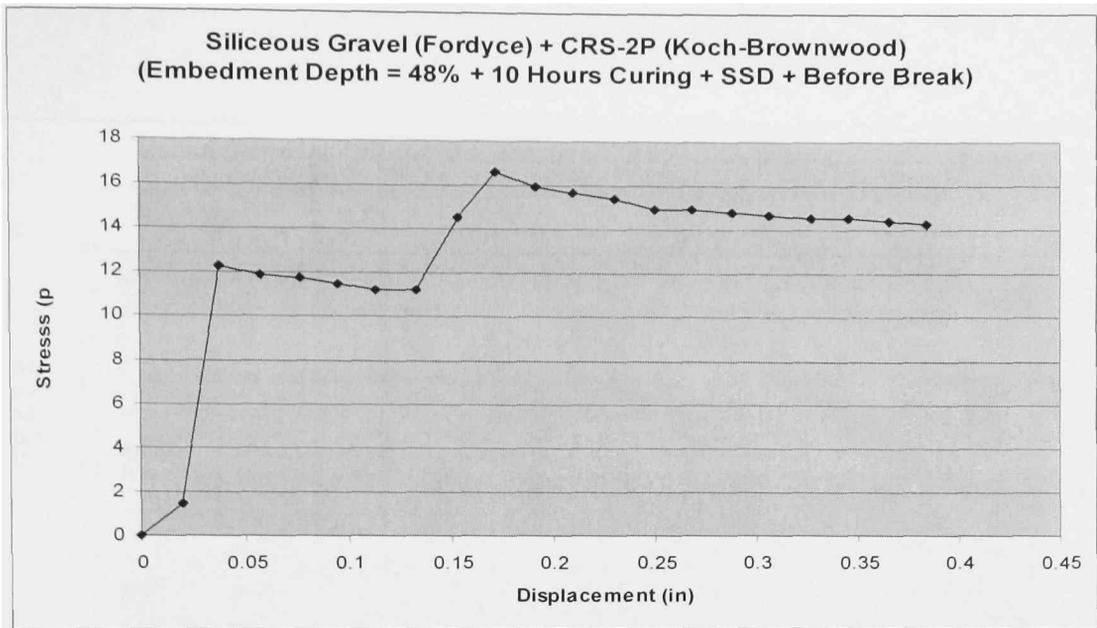


Figure B-29: Stress-Displacement Curve of Fordyce + CRS-2P (10 Hours Curing + SSD + Before Break)

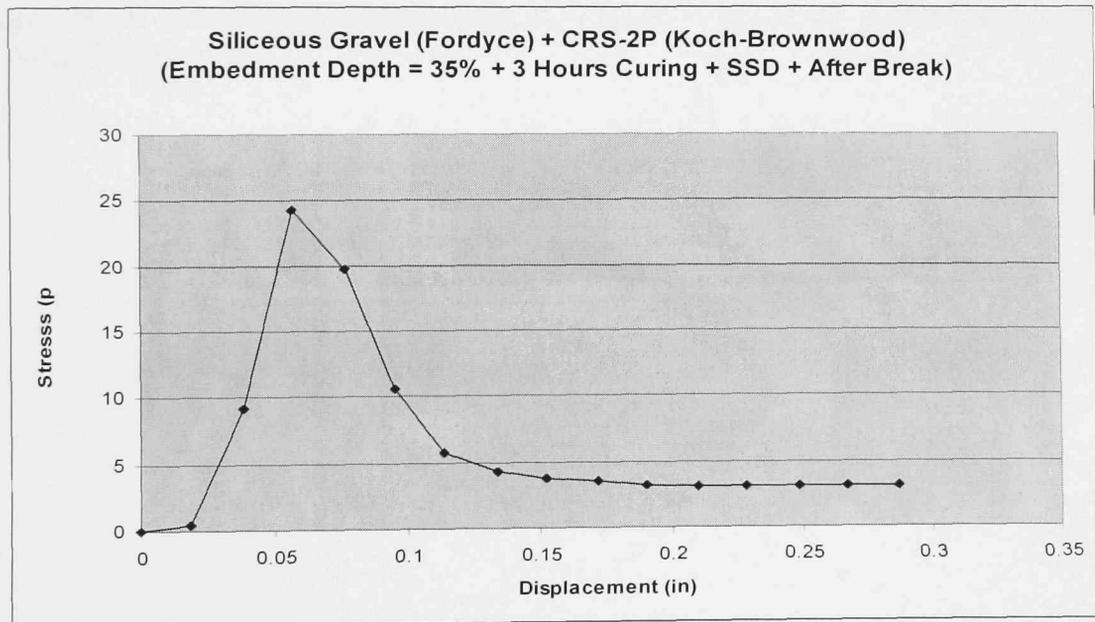


Figure B-30: Stress-Displacement Curve of Fordyce + CRS-2P (3 Hours Curing, SSD + After Break)

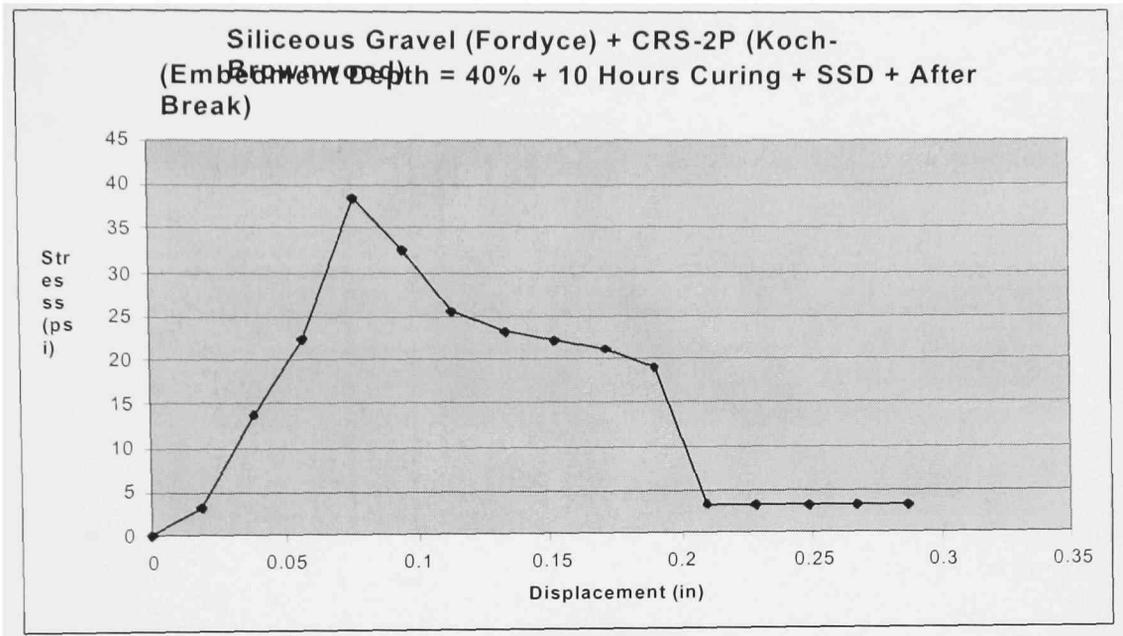


Figure B-31: Stress-Displacement Curve of Fordyce + CRS-2P (10 Hours Curing + SSD + After Break)

APPENDIX C
DIMENSIONS AND SPECIFIC GRAVITIES
OF THE AGGREGATES USED IN
INTERFACIAL BONDING TEST

Table C-1: Dimensions of the Aggregates used in IBT

Aggregate Type	Binder Type	Condition of the Aggregate	Area (mm ²)	Height Measurements (mm)
Gravel (Rhy)	AC-5 w/2% L	100°F,Clean,FE ^a	153.14	12.91, 12.80, 12.67
Gravel (Rhy)	AC-5 w/2% L	100°F,Clean,0E ^b	155.68	12.30, 12.31, 12.38
Gravel (Rhy)	AC-5 w/2% L	140°F,Clean,FE	156.24	13.21, 13.20, 13.03
Gravel (Rhy)	AC-5 w/2% L	140°F,Clean,0E	151.64	12.60, 12.64, 12.46
Gravel (Rhy)	AC-5 w/2% L	140°F,Dusty,FE	171.98	12.32, 12.32, 12.35
Gravel (Rhy)	AC-5 w/2% L	100°F,Dusty,FE	127.33	12.69,12.81
Gravel (Rhy)	AC-5 w/2% L	140°F,Dusty,0E	134.12	8.93, 8.88, 8.81
Gravel (Rhy)	AC-5 w/2% L	100°F,Dusty,0E	99.17	
Gravel (Rhy)	AC-5 w/2% L	140°F,Dusty,0E	166.19	
Limestone	AC-15 5TR	100°F,Clean,FE	161.08	12.28, 12.24, 12.21
Limestone	AC-15 5TR	140°F,Clean,FE	140.18	12.09, 12.03, 11.87
Limestone	AC-15 5TR	140°F,Dusty,FE	143.69	11.78, 11.68, 11.43
Limestone	AC-15 5TR	100°F,Dusty,FE	156.12	12.55, 12.62, 12.73
Limestone	AC-15 5TR	140°F,Clean,0E	140.88	
Limestone	AC-15 5TR	100°F,Clean,0E	122.41	
Limestone	CRS-2P	3 Hours, AD ^c ,BB ^d	116.87	9.68, 9.69, 9.89
Limestone	CRS-2P	10 Hours, AD, BB	140.27	12.01, 11.92, 11.73
Limestone	CRS-2P	3 Hours, AD, AB ^e	123.13	13.27, 13.20, 13.09
Limestone	CRS-2P	10 Hours, AD, AB	93.18	12.48, 12.48, 12.49
Limestone	CRS-2P	3 Hours, SSD ^f , BB	155.74	13.74, 13.68
Limestone	CRS-2P	3 Hours, SSD, AB	163.84	9.85, 9.92, 10.30
Limestone	CRS-2P	10 Hours, SSD, BB	155.86	13.65, 13.47, 13.47
Limestone	CRS-2P	10 Hours, SSD, AB	145.05	11.68, 11.79, 11.9
Gravel (Fdy)	CRS-2P	3 Hours, AD, BB	101.86	11.95, 12.00, 12.05
Gravel (Fdy)	CRS-2P	10 Hours, AD, BB	101.32	12.07, 12.09, 12.15
Gravel (Fdy)	CRS-2P	3 Hours, AD, AB	100.60	11.90, 12.01, 12.02
Gravel (Fdy)	CRS-2P	10 Hours, AD, AB	98.60	12.28, 12.20, 12.25
Gravel (Fdy)	CRS-2P	3 Hours, SSD, BB	98.64	12.08, 12.08, 12.05
Gravel (Fdy)	CRS-2P	10 Hours, SSD, BB	95.64	12.14, 12.17, 12.26
Gravel (Fdy)	CRS-2P	3 Hours, SSD, AB	99.62	12.03, 12.10, 12.14
Gravel (Fdy)	CRS-2P	10 Hours, SSD, AB	100.50	12.11, 12.10, 12.05

^a Fully Embedded

^b 0 Embedment

^c Air Dry

^d Before Break

^e After Break

^f Saturated Surface Dry

Table C-2: Specific Gravities of the Aggregates

Aggregate Type & Source	Condition of the Aggregate	Weight (g)	Specific Gravity
Limestone	SSD	2028.4	2.668
	Submerged	1273.8	
	Oven Dry	2013.1	
Gravel (Rhyolite)	SSD	2932.2	2.412
	Submerged	1757.5	
	Oven Dry	2833.6	
Gravel (Fordyce)	SSD	3126.3	2.612
	Submerged	1937.9	
	Oven Dry	3104.2	
Lightweight	SSD	1931.4	1.462
	Submerged	781.8	
	Oven Dry	1681.2	
Sandstone	SSD	3181.9	2.491
	Submerged	1930.0	
	Oven Dry	3118.2	

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