A Testing and Evaluation Protocol to Assess Seal Coat Binder-Aggregate Compatibility

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Texas Department of Transportation

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16. Abstract: This research project was commissioned to develop a new testing protocol to assess the compatibility of asphalt binders and aggregates used in seal coats and surface treatments. The study encompassed physical, chemical and mechanical aspects of compatibility as well as the construction factors that affect the bond between the aggregate and binder in seal coats. Prior to the development of the testing protocol, a constructability review was undertaken to assess the conditions under which incompatibility between binders and aggregate occur. Four different tests were conducted on several aggregate-binder combinations; modified net adsorption test, TechMRT aggregate pull-out test, interfacial bonding (pull-out) test and the newly developed performance-based seal-coat aggregate-binder compatibility test. Based on the constructability review, aggregate-binder combinations were identified to represent the 'best' and 'worst' performing material combinations. These combinations were used to test the feasibility of the testing protocol to include both extremes of the performance spectrum. It was not the objective of this research project to test all possible aggregate-binder combinations using the newly developed testing protocol, but a sufficient number of tests were conducted to illustrate the feasibility of the newly developed testing protocol under the multitude of conditions experienced by TxDOT seal coats. In the end, several hundred tests were conducted using multiple aggregate-binder combinations. By analyzing the results of these four test methods, two testing protocols were recommended for possible implementation by TxDOT. The modified net adsorption test developed by the National Roads Authority of Ireland is recommended to evaluate the compatibility of uncoated aggregates and hot asphalt binders. The second testing protocol recommended for implementation is the performance-based seal-coat aggregate-binder compatibility test.

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Acknowledgement

Implementation Statement

The primary objective of this research project was to develop a new testing and evaluation protocol for aggregate-binder compatibility in seal coats that can be easily implemented. The researchers believe this objective has been met. A performance-based testing protocol was developed to evaluate various aggregate-binder combinations used in seal coats and surface treatments. The testing protocol has been verified using limited field performance data, and it has been demonstrated that the test method is capable of distinguishing between 'good' and 'poor' performing aggregate-binder combinations based on past experience in TxDOT. Furthermore, the effects of established construction and performance factors on aggregate-binder bonding have been demonstrated in the limited number of tests conducted in this research project. Further tests and field observations of seal coat test sections may be needed to fine-tune the test method and its evaluation protocol. The researchers feel good about the test method that has been developed, and strongly believe that TxDOT is getting a product (the testing protocol) that is effective, practical and economically suitable for seal coat and surface treatment applications. It is also ready for implementation.

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
AREA				
in²	square inches	645.2	square millimeters	mm²
ft²	square feet	0.093	square meters	m²
yd²	square yards	0.836	square meters	m²
ac	acres	0.405	hectares	ha
mi²	square miles	2.59	square kilometers	km²
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft³	cubic feet	0.028	cubic meters	m³
yd³	cubic yards	0.765	cubic meters	m³

NOTE: Volumes greater than 1000 l shall be shown in m³.

Symbol	When You Know	Multiply By	To Find	Symbol
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact)				
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m²	cd/m²
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in²	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm²	square millimeters	0.0016	square inches	in²
m²	square meters	10.764	square feet	ft²
m²	square meters	1.195	square yards	yd²
ha	hectares	2.47	acres	ac
km²	square kilometers	0.386	square miles	mi²
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m³	cubic meters	35.71	cubic feet	ft³
m³	cubic meters	1.307	cubic yards	yd³
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
TEMPERATURE (exact)				
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m²	candela/m²	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in²

(Revised September 1993)

* SI is the symbol for the International System of Units. Appropriate

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CHAPTER 1

INTRODUCTION

A seal coat, also known as a chip seal, surface treatment or surface dressing, is a thin surface treatment that consists of a single application of bituminous binder on an existing flexible pavement followed by a single layer of aggregate. The bituminous material is sprayed by a distributor truck and the aggregate (chips) are spread using a chip spreader. After the application of aggregate, rollers are used to embed chips into the binder. The typical embedment depth is approximately 30-50 percent of the nominal aggregate height. The thickness of the seal coat layer is governed by the nominal maximum size of the aggregate used.

A seal coat is generally applied to a pavement that is in relatively good condition to extend its service life by sealing the surface. Such a seal rejuvenates the existing pavement surface and also seals the cracks to prevent moisture migration into the pavement foundation. This type of treatment is classified as a preventive maintenance (PM) treatment. AASHTO defines preventive maintenance 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". Therefore PM involves early application of maintenance treatments when the pavement is in good structural condition. Seal coats are typically expected to provide useful service for about five to eight years. However, the service life of a seal coat varies depending on the traffic volume, climate, and numerous other variables.

1.1 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, and by making effective use of tax dollars.

The performance of seal coats depends to a great extent on the effectiveness of the aggregate-binder bond. Unlike hot-mix asphalt in which the aggregate particles are fully coated and protected by the binder, the seal coat system, which is only one aggregate thick, has to rely on the bonding established by a partial coating of the aggregate surface (approximately 30-50% of the aggregate surface area) by the bituminous binder. Adequate bond between the aggregate and binder will result in better seal coats, thus minimizing 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 use of material quantities. This can reduce binder cost and also the likelihood of bleeding/flushing problems.

The bond between aggregate and binder is a function of mechanical, chemical, electrostatic and adhesive bonding mechanisms. Both physical and chemical properties of

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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 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.

The adsorption of asphaltic components to the aggregate is promoted by the active sites on the aggregate surface. The covering of these active sites by various agents block their activity level. One such agent is the dust on aggregate surface. Surface dust can change the chemistry and physics of adhesion and result in weak bonding between the asphalt binder and the aggregate. Moisture state of aggregate can also affect the degree of bonding. Since seal coat aggregate is stored in stockpiles at the construction site, they are exposed to changing ambient conditions that have significant influence on the aggregate-binder bond.

In addition, field conditions during construction also play a critical role in the development of aggregate-binder bond. These conditions include the state of existing pavement, equipment used, and the changing weather conditions. This often leads to the notion among practitioners that seal coating is more of an "art" that leaves construction personnel and inspectors to make decisions at the field site which are critical to the performance of seal coat.

The viscosity of asphaltic materials is a function of its temperature. When the binder is less viscous, it will wet the aggregate better and penetrate into the pores of aggregates 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 binder application and 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 aggregate spreading and rolling. This is of particular significance when hot asphalts are 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.

1.2   Scope of the Research

Highway agencies such as TxDOT are in the process of implementing performance-based materials specifications. These specifications must be developed based on test methods that better simulate field conditions they are exposed to, both during construction and in 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 aggregate-binder combinations. TxDOT currently uses a number of tests to determine, separately, the suitability of aggregates and binders for seal coats. To determine the aggregate-binder compatibility, the agency currently has the standard test method Tex-216-F (Aggregate Retention Test) which determines the aggregate retention of seal coats. This procedure is not currently used by any district, possibly due to its inability to simulate field conditions in the lab environment. Therefore, a need exists for a performance-based testing

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protocol that simulates field conditions in the lab, in order to assess seal coat aggregate-binder compatibility. For this test method to be useful, it has to be able to identify the appropriateness of aggregate-binder combinations and also evaluate the suitability of various design parameter values such as percent embedment and aggregate spread delay for each combination.

This research report presents a detailed account of the tasks carried out in TxDOT research project 0-4362 titled *Develop a Testing and Evaluation Protocol to Assess Seal Coat Aggregate-Binder Compatibility*. The primary goal of this research project was to develop a new test method and an evaluation protocol to identify compatibility for seal coat aggregate-binder combinations.

This report is organized into seven chapters. This introductory first chapter is followed by a comprehensive literature review in Chapter 2. In Chapter 3, the constructability review and field studies including performance monitoring of field test sections is presented. The research work leading to the development of the new testing protocol (which has two parts) is presented in Chapters 4 and 5.

In Chapter 4, the details of research activities leading to the development of Part 1 of the testing protocol for the *Modified Net Adsorption Test* (NAT), is presented. The modified NAT is aimed at evaluating the compatibility between non-precoated aggregate and hot asphalt binders. It is also recommended to identify suitable hot asphalt precoating binders for aggregates that are earmarked for precoating. The modified NAT is not recommended for emulsified asphalts, cutback asphalts and precoated aggregates because the test conditions are not suitable for those materials. However, since a large majority of TxDOT seal coat projects use hot asphalts and precoated aggregates, the modified NAT can play a vital role in identifying effective aggregate-binder combinations.

In Chapter 5, the details of research activities leading to the development of Part 2 of the testing protocol are presented. This is the primary testing protocol developed for TxDOT seal coats that can be used for all types of seal coat binders and for precoated aggregate. This part of the test also allows its user to evaluate the performance of aggregate-binder combinations under various construction and field performance conditions. These conditions include pavement temperature at the time of sealing, time delay between binder application and aggregate spread, time delay between aggregate spread and rolling (for hot asphalt), aggregate dust content, Extent of precoating coverage, percent embedment of aggregate and climatic conditions. Therefore, this Part 2 of the test protocol is presented as a *performance-based* aggregate-binder compatibility test, and districts can use their own local construction and performance conditions as test parameter values.

Two other test methods were also used in this research to evaluate aggregate-binder bonding characteristics. They are the *Interfacial Bonding Test (IBT)* and the *Modified Australian Pull-Out Test*. These test methods and their results are discussed in Chapter 6 of this report under the heading *Additional Tests*. However, these test methods are not a part of the testing protocol recommended for implementation by TxDOT. The report is concluded in Chapter 7, which outlines the conclusions of the research findings and the researchers' recommendations for future work on this subject.

The following two products have been developed for possible implementation of this research, and they are included in Appendices A and B of this report.

1. Draft Test Procedures for Seal Coat Aggregate-Binder Compatibility

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2. Draft Updated Specifications for Seal Coats and Surface Treatments including New Acceptance Criteria

CHAPTER 2

LITERATURE REVIEW

A seal coat, also known as a chip seal, surface treatment or 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 aggregate (chip) spreader that spreads a single layer of aggregate to form a sealing surface. The process is illustrated in Figure 2-1. 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.

Figure 2-1 Typical Seal Coat Operation

2.1 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. Some maintenance treatments may fall into all three maintenance categories indicated above, distinguished only by the condition of the pavement they are applied to (Zaniewski and Mamlouk, 1999).

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2.1.1   Preventive Maintenance

A seal coat is generally applied to seal the surface of a pavement that is in relatively good structural condition, in order to extend its service life. Such a seal can reduce the oxidation of asphaltic material in the existing pavement surface and also seal small surface cracks to prevent moisture migration to the pavement foundation. This type of maintenance treatment is classified as a preventive maintenance (PM) treatment. Such treatments are also used on moderately distressed pavements, but the full benefits of the treatment may not be realized. Under emergency conditions, seal coats may only provide temporary improvement (emergency maintenance).

After the conceptual framework for a Pavement Management Systems was established, highway agencies tend to use PM in the early stages of the pavement in order to extend the service life. AASHTO (1999) defines preventive maintenance 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".   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.

Figure 2-2 illustrates the typical pavement life cycle. It shows that speed of deterioration increases rapidly in the latter part of the performance curve. Moreover, the cost of a possible rehabilitation at the late stages in the performance cycle is 3 to 4 times more expensive than a PM applied early in the life of pavement. Therefore, PM is a cost-effective tool to extend the service life of a pavement provided it is used at the appropriate time.
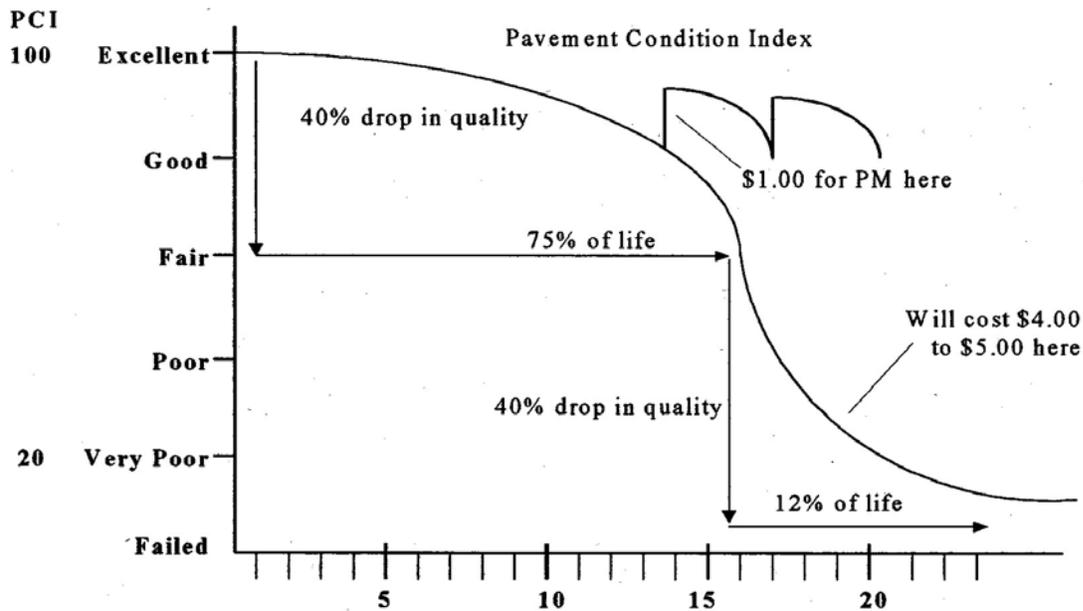
Figure 2-2 Typical Pavement Life Cycle (Hicks et al. 1999)

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2.1.2 Comparison of Various Preventive Maintenance Techniques

Table 2-1 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 2-1 Unit Costs and Expected Life of Maintenance Treatments (Hicks et al. 1999)

Treatment	Cost/m ($)[f]	Cost/yd^2 ($)	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:
[a] Application rate of 0.2 liters/m^2 (0.05 g/yd^2) of a 1:1 diluted CSS emulsion in water
[b] 7 kg/m^2 of ISSA Type II slurry
[c] 14 kg/m^2 of ISSA Type II microsurfacing
[d] 15 kg/m^2
[e] 30 to 44 mm/m^2
[f] Cost per meter per linear foot

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 EAC is calculated by the equation:

EAC = Unit Cost/Expected life of treatment in years

For instance, the EAC for a seal coat with an expected life of 5 years can be calculated as:

EAC for seal coat = $0.85/5 = $0.17/yd^2/year

EAC values for all the preventive maintenance treatments shown in Table 2.1 are presented in Table 2.2. It can be seen from Table 2-2 that fog seal has the lowest EAC followed by 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. It helps to minimize asphalt oxidation and 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 separately. Based on EAC evaluation, seal coats and fog seals are the most cost-effective treatments.

Table 2-2 Cost-Effectiveness of Pavement Maintenance Treatments (Hicks et al. 1999).

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

2.1.3   Things a Seal Coat Can and Cannot Do

Seal coats can perform several functions in 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 can vary depending on the traffic volume, weather, and numerous other factors. The following are a list of surface deficiencies that can be corrected using seal coats:

1. Cracks: most cracks can be sealed as long as they are not very wide. A seal coat can effectively prevent water from seeping through narrow surface cracks thus minimizing moisture-induced damage in the pavement layers.
2. Raveling (Shelling): aggregate particles in old bituminous pavement surfaces often get dislodged due to oxidative hardening of asphalt. A seal coat can bind loose aggregate particles in place, rejuvenate the binder on the existing pavement surface, and prevent additional raveling.
3. Bleeding: the binder of an 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 properly applied seal coat can cover the bleeding spots with fresh aggregate, thus rejuvenating the surface.
4. Lack of skid resistance: a seal coat brings 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 or major surface 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 an existing pavement any stronger.
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.

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3. Smooth out rough pavement: if the old pavement is significantly rutted, shoved, or washboarded, a seal coat is not able to correct them effectively.
4. Bridge major cracks: cracks wider than about ¼ inches 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 the 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.

2.2   Seal Coat 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 including surface texture, porosity, surface chemistry, mineralogy, and surface charges of the aggregate influence its bonding with the binder.

Nevitt (1951) outlined aggregate properties that are desirable for successful seal coats. For seal coat aggregates, irregular shape is preferred but not essential. The aggregates that have cubical shape are the most desirable and the particles should be neither flaky nor splintered. The more spherical the aggregate particles are, the less surface area it presents for the binder to adhere to. The greater the aggregate surface in contact with the binder, the stronger will be the bond between the binder and the aggregate (Herbert, 1955).

Surface texture is the most important aggregate property that influences bonding and a surface that has a rough and gritty texture that is also resistant to wear, is most appropriate. The majority of aggregates do improve in both surface texture and shape characteristics from the crushing operation, and crushed aggregates are therefore desired for seal coat operations. However, selection of aggregate should be based on their specific physical and chemical characteristics, rather than based on whether they are crushed or not.

Most aggregates absorb asphalt to some degree. Asphalt absorption is directly related to the porosity and the pore size distribution of an aggregate. The total porosity is a good indicator for absorption. In general, asphalt absorption increases with coarse-pore porosities. Chemical composition of carbonate rocks influences aggregate porosity, and hence its asphalt absorption. (Lee, 1969)

Chemical and mineralogical characteristics of aggregate also have a strong influence on binder-aggregate bond. 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) in the presence of water, which would cause loss of aggregate (raveling) from a seal coat.

Aggregates by nature have highly heterogeneous surfaces. For example, an aggregate surface may contain a multiplicity of adsorption sites, some of which may be more active and polar than others and may also contain impurities such as moisture, dust or organic matter. The chemical interaction between liquid asphalt and the solid aggregate occurs predominantly at the more active sites. The overall chemical behavior of the adsorbent surface is directly related to

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the range of acidic and basic groups that exist on the surface (Labib and Zanzucchi, 1990). There are some positively charged sites, negatively charged sites and other sites where polarity changes depending on the conditions such as moisture content and temperature.

2.2.1 Aggregate Surface Chemistry

Surface chemistry of the aggregate has a significant influence on the bond between the aggregate and the binder. Every aggregate of a specific mineralogical type has a unique surface chemistry. Curtis et al. (1993) claimed that "…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….. When asphalt comes into contact with an aggregate surface, it can penetrate into the aggregate pores and the active sites on the particle surface attract the most polar and bondable asphaltic species on initial contact ". This asphalt that is in direct contact with the aggregate surface play a very important role in the performance of a seal coat, and it is this asphalt that must adhere, and remain adhered, under adverse conditions.

The aggregates provide a surface that is heterogeneous and has a variety of sites of different composition and levels of activity. These active sites frequently have electric charges that attract and orient the polar constituents of asphalt. The polar functionalities of the asphalt that is present at the point of contact of the aggregate adhere to the surface through electrostatic forces, hydrogen bonding or Van der Waals interactions. Any given asphalt molecule may not be too distant from inorganic surfaces either by contact with that surface or by contact with another asphalt molecule in contact with or influenced by the aggregate.

Gorman et al. (1998) studied the influence of aggregate surface chemistry in emulsion-aggregate interactions. In that study, the researchers pointed out the importance of contact angle measurement when surface and interfacial tension data cannot be obtained because it provides information on the *hydrophobicity* (affinity to repulse water) and *hydrophilicity* (affinity to attract water) of the aggregate surface. Gorman et al. (1998) used the term *contact angle* with the following 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."

Gorman et al. (1998) suggested the Washburn method to measure advancing water contact angle, as it provides a contact angle that is averaged over the whole contact surface. According to Gorman et al., an aggregate with a Washburn advancing water contact angle greater than 65° is an acceptable aggregate for slurry surfacing applications.

Zeta potential is another important property of the aggregate surface which can provide critical information regarding the surface charge present on the aggregate surface. The aggregate surface has an electrostatic interaction with the surfactant molecules. Gorman et al. (1998) found that the *Zeta potential* for all aggregate samples tested in their study was found to be negative under natural pH conditions. They speculated that it probably reflects the dominance of SiO_2 (45 to 72% composition in the aggregates tested).

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2.2.2 Precoated Aggregate

The existence of dust on the surface of seal coat aggregate is one of the most common causes of aggregate loss. Dust does not allow good adhesion between the aggregate and the binder. The most common solution adopted to overcome this problem is the use of precoated aggregate. Precoated aggregate is the aggregate that is covered with a very thin film of bituminous binder prior to the seal coat operation (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".

Khandhal and Motter (1997) conducted research on precoated aggregates for seal coats and surface treatments. They found that, in most cases, the rate of increase in aggregate knock-off loss for uncoated aggregate increased significantly after about 3% dust content (by weight). Secondly, they affirmed that "although no relationship was observed between the percent knock-off loss and percent water absorption or particle index of the aggregate, a correlation between the flakiness indices of the aggregates and the corresponding aggregate retention was observed" and they concluded that "when the percentage of the flat particles in the sample (or flakiness index) increases, the corresponding retention loss also increases".

In that study, five aggregates (AASHTO No. 8 size) with different mineral compositions containing 3% dust content were precoated to achieve five different degrees of precoating. All of the materials were subjected to Pennsylvania Aggregate Retention Test. Khandhal and Motter (1997) observed that increased precoating coverage decreased the initial aggregate retention loss, and that aggregate loss due to dust can be significantly reduced by precoating to achieve at least 90% coverage of the aggregate surface area with the precoating binder. Their results showed that with more than 90% precoating coverage, initial aggregate retention loss was reduced by as much as 80% when compared to non-precoated aggregates. 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, they noted that use of precoated aggregate with emulsified asphalt would slow the breaking duration of the emulsified asphalt so that the reopening time of the road to traffic will be delayed.

Janssen (1941) took a different approach to study the effect of precoating by precoating seal coat aggregate with furfural. He concluded that the treatment was effective in reducing the tendency of aggregate to strip, but overall, the performance of the treatment is not satisfactory.

2.3 Bituminous Binder

All bituminous binders used in seal coats are either asphalt cements or other products such as emulsified asphalt or cutback asphalt that are derived from it. Asphalt cement is known to be a complicated colloidal system of hydrocarbon compounds. The micellar theory of asphalt composition indicates that asphalt cements are composed of asphaltenes, resins, and oils (Curtis et al. 1989). Asphaltene is the heaviest and most polar fraction of asphalt cement. In this model, asphalt cement could be modeled as a colloidal suspension of asphaltenes in oils with resins acting as agents that prevent their coagulation. The chemical composition of asphaltene is considered as a combination of polyaromatic, alicyclic, and alkyl compounds that also contain heteroatoms such as oxygen, nitrogen and sulfur (Jeon and Curtis 1994). The heteroatoms are

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also found in five- or six-membered ring structures or as links between molecules such as ether. The heteroatoms are also present in functional groups such as sulfoxides, hydroxyls, carboxylic acids, ketones and nitrogen bases exhibiting different levels of basicity. All naturally occurring heteroatoms (i.e. nitrogen, sulfur, oxygen, and metals) contribute to increased polarity within these molecules. Polarity, which is defined as the separation of charge within a molecule, is an important characteristic of asphalt because it tends to assist the arrangement of the molecular constituents into orientations that enhance adhesion. Some of these asphalt molecules are relatively strong acids or bases and may be polyfunctional in nature. Oxidative weathering of asphalt can generate new sites that have greater polarity and are more numerous than those found in virgin (non-oxidated) asphalt (Petersen, 1975). Therefore, the propensity of the molecular structures in asphalt to self-associate will increase with aging. Also, the rate of association depends both upon the number of attractive sites and the magnitude of the attractive forces. The more numerous and stronger the attractions, the greater will be the propensity of the groups to associate. However, associative arrangements are slowed down by the high viscosity of the liquid asphalt.

2.3.1   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 in a storage tank (hot asphalt), by dissolving in a petroleum solvent (cutback asphalt), and by combining with water (asphalt emulsion). Today, due to environmental concerns, cutback asphalts are used less often.

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

The asphalt 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 accurate application. As conditions vary from one construction project to another, 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 including existing road surface characteristics, aggregate types, available equipment, and climatic conditions that may be encountered.

The primary rheological requirement for a bituminous binder is that its stability not be compromised under the highest service temperatures reached during its service life. For pavements in hot climates with no likelihood of frost, to sustain the stability of a pavement, as the temperature susceptibility of a particular asphalt binder 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). Polymer modification, using modifiers such as latex, SBS and SBR copolymers and tire rubber has become very common in the seal coat industry.

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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 of the blend. 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 to obtain desirable asphalt properties depends to a great extent on the virgin (or base) asphalt itself. Swelling is observed in polymer modified asphalts under certain conditions. The extent of swelling in asphalt-polymer blends is practically independent of temperature in 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 virgin asphalt has a significant influence on the properties of modified asphalt-polymer blends. An increase in the ring and ball softening point and the plasticity interval, and a decrease in the thermal sensitivity of asphalt-polymer blends, is not directly proportional to the polymer content and depends significantly on the virgin asphalt itself (Brule et al., 1988).

The use of rubberized asphalt to seal pavements was initially developed by the city of Phoenix in the 1960s, and is being used in seal coat applications to this day (Morrison and Hesp, 1995). This type of seal is often referred to as a hot rubber seal. Over 200 millions of tires are discarded every year in the United States. Therefore, the use of waste tire 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 rubber[1] and asphalt cement, the proportions by weight in the mixture shall be 25 ± 2% and 75 ± 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 may be added up to a maximum amount of 7.5% by volume of the hot asphalt-rubber mixture to obtain optimum viscosity for spray application and better "wetting" of cover aggregate. According to TxDOT specifications, in a mixture of Type II rubber[2] and asphalt cement, the proportions by weight of the asphalt cement (including extender oil, if needed) and rubber shall be 78 ± 2%, and 22 ± 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 of at least 30 minutes after all the rubber has been added, is needed prior to the mixing of rubber. Temperature of the material during the reaction period shall be between 375° F and 425° F.

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[1] Type I: Ground tire rubber.
[2] Type II: Mixture of ground tire rubber and high natural reclaimed scrap rubber.

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 is mixed into the asphalt for approximately 45 minutes at 347-428 °F at high temperatures to form a gel-like material. To this gel, some aromatic kerosene fraction is added to increase its workability. This "asphalt rubber" is mixed with aggregate to construct the pavement layer. 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".

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 3 to 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).

Nicholson (1998) identified several advantages to rubberized asphalt seals including resistance to reflective cracking, rutting, shoving, reduced tire-pavement noise, good color contrast for pavement markers and striping, and the environmentally friendly recycling of waste tire rubber. Another study by Olsen (1973) also point out the advantages of using rubberized asphalt. In that study, the McDonald method has been used and the proportion of rubber used was 25 to 30%. Olsen claimed that using rubber in larger proportions gave good results in controlling fatigue cracking. In addition, he indicated that, "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". However, he 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 the rheological properties of rubberized asphalt. They concluded that, "The viscosity of asphalt cement increases with the 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". The results of that study also showed that elastic stiffness 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 between the elastic stiffness 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 is increased".

An early study conducted by Itakura and Sugawara (1955) on the effects of rubber on binder properties concluded that heating and stirring time influence several physical characteristics of asphalt cement especially penetration, float test value, softening point, ductility and toughness. They observed that a 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. According to their conclusions, to improve the binder performance, a minimum of 3 to 5% of the natural rubber powder and 5 to 10% (based on the asphalt) of reclaimed rubber may be used in wearing courses.

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In addition to rubberized asphalt, asphalt is blended with smaller quantities of rubber (5% by weight of asphalt) and polymer to produce AC-15 5TR, and more recently AC-20 5TR, grades that are very popular in TxDOT seal coat applications. These binders have repeatedly shown good performance in a large number of seal coat projects constructed by TxDOT.

<u>2.3.2 Asphalt Emulsion</u>

Asphalt emulsions are widely used as a binder in seal coat operations. Asphalt emulsion is basically a mix of finely ground asphalt with water and an emulsifying agent. Emulsification of asphalt is the creation of small (1-5 micron) particles of asphalt that are coated with a chemical that provides a surface charge on them and allows the particles to stay apart in the water-asphalt suspension. The emulsified asphalt must however be converted to asphalt cement films in order to perform its intended functions such as coating, waterproofing and adhesion. The emulsified asphalt particles have an electrostatic charge on them and thus, two particles that approach each other are repelled due to the like charges they carry. However, particles cannot exist forever in a charged state. Once emulsified asphalt is applied, it goes through a process called "Breaking" that involves the separation of water from emulsion. 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 evaporation of water from the emulsified asphalt, will result in the development of desirable properties of the asphalt cement left on the pavement. The end result is a continuous cohesive asphalt film that holds the aggregate in place with a strong adhesive bond.

According to the Asphalt Emulsion Manual (Asphalt Institute 1999), 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 marginally wet aggregate may facilitate coating, excessive wetness 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 the water evaporation rate, emulsifier migration and water release characteristics. While breaking usually occurs more quickly at warmer temperatures, it 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 tire contact 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, quickens 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.

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

The charge density at the interface between the asphalt particle and the emulsified water medium, as measured by the *Zeta potential*, determines the stability of the emulsion. The thickness of double layer at the interface is determined by the molecular size of the emulsifier, and its ability to dissolve in the water phase (Hooleran, 1999).

The stability, and hence the success of an emulsified asphalt binder relies heavily on its ability to form films on aggregates. Film formation involves coalescence (without entrapment of water) of asphalt into a continuous film. Film formation is a function of kinetic factors such as temperature, viscosity, internal stability and thermodynamics of the system. 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 also enhance adhesion (Hooleran, 1999).

The mechanism of chemical interaction between emulsified asphalt and aggregate is not fully understood, but the following general description appears to represent agreement among most researchers.

The stable emulsified asphalt particles are attracted to the surface of the aggregate, where the emulsifier from the bulk solution interacts with the aggregate surface charges. This changes the equilibrium of the emulsified asphalt system, and it is destabilized and the charge on asphalt particles is neutralized. This leads to deposition of asphalt on to the aggregate surface. 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 rate 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 needed to push water away from the aggregate-binder interface. Cement is often used to enhance this phenomenon. The electrical charge on the aggregate surface may depend on the aggregate type and its mineral composition. Since both cationic and anionic emulsions are available, emulsifier choice relative to aggregate is a key issue (Hooleran, 1999).

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. 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. The researchers showed that by controlling the standard deviation of droplet size, it is possible to control viscosity and the rate of cohesion build

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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).

They correlated the breaking index of emulsions to the median diameter 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, and so 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 increases as the median diameter of the filler increases, for which the explanation is the same (it takes more filler with a smaller specific surface to break a given quantity of 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.

The properties of base asphalt used in an emulsion can have a significant influence on the properties of 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 of the binder, 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 the surfactant from bitumen, and the adsorption of bitumen on aggregate, could provide the kinetic control of the process. Moreover, surfactants with different reactivities 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 as a consequence of negative surface charge density. These results indicate that the surfactants used to manufacture cationic slow setting (CSS) emulsions have a stronger basic character than those 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 acidic 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 across the whole 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.

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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 a 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-type surfactants. 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 the natural acid groups in the bitumen. The adsorption of cationic surfactants on siliceous aggregates changes the surface charge from low negative potentials to positive potentials as high as 50 mV (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 emulsified asphalt 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 surfactant 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 emulsions that have less surfactant(s). However, it is generally common to use emulsions with high concentrations of surface-active agent, and it assumes that mineral particles are free, and that 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, and the surfactant which is 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 is 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) to 50 °C (122 °F). At low temperatures, siliceous materials result in setting times higher than for calcareous ones. As temperature increases, differences in setting times decrease for the different aggregates. The time of setting for

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different aggregates is about the same when the temperature reaches 50 °C (122 °F). The influence of the water vapor pressure that strengthens its influence from 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 temperature interval considered for any aggregate type. Siliceous aggregates result in the greatest differences. The residual adherence of bitumen on the aggregates is neither influenced by temperature nor by the amount of emulsifier. When SS emulsion is used with fine aggregates similar to that of a slurry seal, the variation of the rate of setting with temperature is more perceptible than in the case of mixtures with the RS emulsion. Using different concentrations of stabilizer agents within the pre-wetting water significantly increases the time of setting when non-pretreated aggregates are used. But, as the temperature increases, the differences decrease to zero at temperatures reaching 30 °C (86 °F) (Pinilla and Agnusdei, 1975).

2.4   Bond between Aggregate and Bituminous Binder

The performance of a seal coat depends to a great extent on the bond between the aggregate and bituminous binder. Unlike hot-mix asphalt where the aggregate particles are fully coated by bituminous binder, the seal coat system, which is only one aggregate thick, has to rely on the bonding established by a partial coating of the aggregate surface area (approximately 30-50%) by the binder. Providing adequate bond strength between the aggregate and binder will result in better seal coats by minimizing the occurrence of aggregate loss (raveling). In addition, better aggregate-binder bond will enable designers to use less binder thus reducing both the binder cost and the likelihood of bleeding/flushing problems.

2.4.1   Factors Affecting the Bond

The bond between asphalt and aggregate is a combination of mechanical, chemical, electrostatic and adhesive bonding mechanisms. It depends on both the chemical and physical properties of asphalt and aggregate (Brannan et al, 1991). Aggregate characteristics that influence bonding include porosity, surface texture, mineralogy and surface chemistry. Binder characteristics include chemical composition and its viscosity at the time of aggregate application. In the case of emulsified asphalts, the type of emulsion (cationic/anionic) and its breaking and curing characteristics affect bonding. In addition to the individual material characteristics indicated above, properties that correspond to the material pair (binder and aggregate) such as surface tension also play a key role in generating the interfacial bond.

Several mechanisms have been used to explain the adhesion between dissimilar materials such as asphalt and aggregate surfaces. Although these mechanism overlap to some extent, they have traditionally been assigned a chronological order as shown in Table 2-3 to reflect the degree to which adhesion occurs at the interface (Curtis et al. 1992).

The four types of interactions that are important in the asphalt-aggregate bonding are:

1. Dipole-ionic sites
2. Induced dipole-ionic sites
3. Dipole-dipole interactions
4. Dipole-induced dipole interaction

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Table 2-3 Aggregate-Binder Bonding Mechanisms (Curtis et al. 1992)

Chronology	Theory	Mechanism	Strength of Interaction
Wetting of aggregate	Adsorption	Van der Waals, dipole-dipole interactions	Weak, moderate to strong
Development of adhesion	Mechanical interlock	Wetting	Variable
	Diffusion	Inter-diffusion	Variable
	Chemical bonding	Covalent bond	Very strong
	Electrostatic attraction	Ion- dipole interactions	Strong
	Acid-Base	Hydrogen bonding, dipole-dipole/ionic bonding	Moderate to strong, Very strong
Failure	Weak boundary layer	Defects at the interface	Variable

Dipole interactions arise when molecules have atoms with different polarities. Induced dipoles occur when a polar molecule is influenced by an electric field, which may originate from an ionic site, another dipole, or a dipole field (Tanabe, 1970; Hair, 1967). It is unlikely that dispersion (London forces) play an important role in asphalt-aggregate interactions. Dispersion forces are probably more influential in asphalt intermolecular interactions that involve non polar molecules. Interaction of alkaline aggregates and asphalt with acidic components can be represented by the following simple acid-base reactions.

$$CaCO_3 + 2RCOOH \rightarrow (RCOO^-)_2Ca + CO_2 + H_2O$$

$$CaO + 2RCOOH \rightarrow (RCOO^-)_2 Ca + H_2O$$

$$Ca(OH)_2 + 2RCOOH \rightarrow (RCOO^-)_2Ca + 2H_2O$$
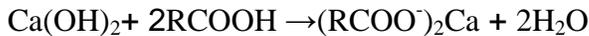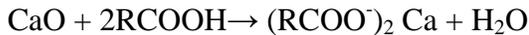
In a similar manner, amine compounds present in asphalt or added in the form of anti-stripping agents may react with acidic surfaces of siliceous aggregates to form surface complexes as shown by the chemical reaction equation below (Yoon and Tarrer 1988).

$$H_2SiO_3 + RNH_2 \rightarrow HSiO_2\text{-}NHR + H_2O$$
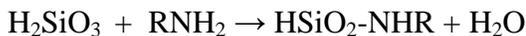
The adsorption of asphaltic components is promoted by active sites on the aggregate surface. The covering of these active sites by various agents would block their activity. One such agent is the dust on aggregate surfaces. Surface dust can change the chemistry of adhesion and result in weak bonding between the binder and the aggregate surface (Curtis et al., 1993). In addition, dust generated by specific aggregate types 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 varying ambient conditions that can have a bearing on the aggregate-binder bond.

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Curtis et al. (1993) indicated that short-range chemical interactions in asphalt are more feasible because of electrostatic interactions between a charged aggregate surface and the asphalt molecules with 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 have electrostatic interaction with other oppositely charged or partially charged asphalt molecules. If the interlinking of 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).

According to Graf (1986), the types and numbers of bonding interactions are important for overall adhesion and durability of the bond. The type of interfacial bond can characterize the durability of the bond, while the number of bonds formed corresponds to the degree of wetting of the aggregate surface. Graf also indicated that wetting action of the asphalt on aggregate surface serves as the vehicle for transporting active portions of the liquid asphalt to the active aggregate surface sites where adhesive bonding can occur. As the asphalt wets the aggregate surface, some of the asphalt's organic chemical functionalities bind with the constituents on the aggregate surface. Frequently, functional groups such as carboxylic acid and phenolic structures combine with the alkali metals that may be present on the solid aggregate surface to form water-sensitive salts such as sodium salts (Graf 1986). Consequently, these ionic asphalt-aggregate bonds weaken or dissolve (solubilize) over time with exposure to moisture (Yoon and Tarrer 1988). It is believed that the polyamine functional groups in commercial anti-strip agents to assist wettability are also moisture susceptible. Thus their ability to withstand solubilization and de-bonding over time can be compromised. To create a durable adhesive bond between asphalt and aggregate phases, the functional groups need to form hydrophobic bonds at the interface. Because the chemical compositions of both liquid asphalt and solid aggregate are heterogeneous, a method for matching of asphalt-aggregate bonding pairs is desirable.

Curtis et al. (1993) indicated that compounds with polar functional groups were more competitive for bonding sites and adsorbed much more strongly than nonpolar compound groups. Among these competitive species that were most competitive, those 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 active aggregate surface sites. 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. Curtis et al. (1993) showed that even the two most polar species containing sulfoxide and carboxylic acid functional groups were readily removed from the aggregate surfaces when in contact with 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 little difference between these two materials. They concluded, "The small changes in chemistry

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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".

Scott (1978) studied the de-bonding mechanisms between asphalt and aggregates in the presence of adhesion additives (antistripping agents), and found that these additives cause changes in the composition of the adsorbate. In the case of cationic additives, Scott observed that they are not exclusively adsorbed from hot asphalt to mineral, but adsorb together with other polar asphalt components. They explained that while adhesion additives will cause rapid initial adsorption and promote wetting of the surface, it may not be the ultimate equilibrium situation. After some time, the additive may no longer be the major component of the adsorbate. Both the nature of mineral surface and the asphalt appear to influence the effectiveness of an adhesion additive and the extent of its adsorption.

Scott (1978) also found that groups from asphaltene fraction molecules that contain oxygen predominate in the asphalt fractions adsorbed on the mineral surface. The exact composition of the adsorbate is based on the asphalt and the minerals along with the adsorption conditions. Cationic amine adhesion additives adsorb onto the mineral surface in combination with polar asphalt molecules. 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. According to Scott (1978), changes in pH 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.

Scott further stated that changes in the adsorption environment due to water at the interface 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).

Construction factors are as important as material factors to generate a good bond between the aggregate and the binder in a seal coat. These factors include climatic parameters such as temperature, humidity and wind, equipment processes such as rolling or sweeping, and also traffic control during and after the construction of seal coat. This often leads to the notion among practitioners that seal coating is more of an "art" that leaves the construction personnel and field inspectors to make decisions in the field that are often critical to the performance of the seal coat.

TxDOT specifications (TxDOT 1993) require that surface treatments such as seal coats shall not be applied when the air temperature is at or below 60 °F and falling, but may be applied when the air temperature is above 50 °F and rising. The air temperature shall be measured 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 viscosity of an asphaltic material 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 application of binder and aggregate spreading becomes a critical factor that determines aggregate-binder bonding effectiveness. As 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 aggregate spreading and rolling. This is of particular significance when hot asphalts are used. Rollers apply the mechanical energy needed to embed aggregates into the binder and they also facilitate the seating of each aggregate particle such that its center of gravity is at the lowest possible position, thus 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, thus causing aggregate loss.

Jackson et al. (1990) recommended that rolling 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 that a minimum of 3 pneumatic rollers traveling at a speed of 5 mph that can cover the full width of the asphalt shot be used. Shuler (1998) concluded that use of steel rollers should be avoided, particularly on high traffic volume facilities due to lack of compaction and potential for crushing of aggregate.

Another construction 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". 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".

Even when a seal coat is constructed under conditions that are desirable for such work, its performance with regard to aggregate-binder bond is often affected by the weather following the seal coat operation. It is commonly known that a seal coat, once in place, needs a "conditioning period" to develop bonding between the aggregate and the binder before cold weather and/or heavy rains set in. Therefore, it is not advisable to construct seal coats towards the end of the summer construction season because of the lack of time available for the development of bond between the aggregate and binder.

2.4.2   Moisture Induced Stripping at Aggregate-Binder Interface

The bonding between aggregate and binder has to be strong and durable enough to satisfactorily carry out its intended functions during service life. Any weakness or loss of bonding will produce problems throughout the life of pavement. Water or moisture is known to be a major cause of adhesion loss between aggregate and bituminous binder. Tarrer (1986) defined stripping in asphalt pavements as "…the loss of adhesive bond between the aggregate surface and asphalt binder due to adverse action of water or moisture, resulting in the displacement of asphalt cement films from aggregate surfaces by water. Fromm (1974) defined

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stripping as the process caused by water penetrating between asphalt film and the aggregate surface. Curtis et al. (1993) identified the following modes of failure associated with stripping:

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

In seal coats, there are several factors that influence debonding failure between aggregate and the bituminous binder. The moisture sensitivity of the interfacial bond between asphalt and aggregate is one of the primary causes of debonding at the interface. Therefore, to better understand and control seal coat stripping problems, it is necessary to investigate the fundamental chemistry of the aggregate/asphalt interfacial bond in the presence of water. The identification of interactions at the asphalt-aggregate interface that are water sensitive, together with an understanding of asphalt adsorption onto aggregates may permit matching of satisfactory asphalt-aggregate pairs and/or modification of asphalts and aggregates that can yield stronger, more permanent interfacial bonds (Fromm,1974; Petersen et al., 1974).

Stripping of the asphalt film from an aggregate surface is influenced by mechanical, thermodynamic or interfacial energy, and/or chemical changes that lead to a loss of adhesion. Each of these mechanisms, either individually or through synergy, may occur in any given instance of stripping. While some physical properties of the aggregate may affect stripping, no correlation was observed between the aggregate's pore volume and surface area and the stripping propensity. Chemical interactions of the aggregate surface in the presence of water were the more important factors for stripping. The pH of water at the asphalt-aggregate interface can change significantly with time as water reacts with components on the aggregate surface. Apparently, this change in pH generate significant shifts in the strength of the bonding forces between the aggregate and the asphalt, and in turn, increases stripping propensity (Hughes et al. 1960; Scott 1978).

Certain metal ions on the aggregate surface such as sodium and potassium cause a rise in the pH level of the interfacial water that can disrupt the asphalt-aggregate bond. The silicate structure of silica-rich aggregates can become swollen by absorption of water, and thus the asphalt film may be more easily stripped away due to rupture of the weaker gel-like structures that are formed (Terrel and Al-Swailmi 1994). Therefore, stripping damage can be expected in the case of aggregates that increases the pH of contacted water.

The surface charge of the aggregate is as important to adhesion as specific chemical interactions. The functional group types of asphalt adsorbed on an aggregate surface consist mainly of the acid groups of the asphalt. An acid molecule is characterized by the carboxylic acid (R-COOH) group. In the presence of water, the acid molecules are hydrolyzed into two ions; the carboxyl anion(R-COO$^-$) and the cationic hydrogen ion (H$^+$). This change results in a negative polarity of the asphalt at the interface. The increase of pH in the presence of water at the aggregate surface increases the extent of dissociation of the acid molecules. Since most mineral surfaces are also negatively charged at varying intensities, repulsion develops between the two negatively charged surfaces (i.e. asphalt and aggregate). The intensity of the repulsive forces developed depends on the intensity of surface charges at the point of contact. This ionic

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repulsion between two negatively charged surfaces at the asphalt/aggregate interface can lead to the separation or stripping of the asphalt and the aggregate (Thelen, 1958; Scott, 1978)

Tarrer (1986) presents several mechanisms to explain stripping and they can be classified into three types; mechanical, thermodynamic, chemical. The mechanical concept models the mechanical adhesion which is governed by the surface texture and shape of aggregate particles. According to Tarrer, asphalt or the oils from it, normally penetrates 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, and therefore less readily stripped by water. In addition to aggregate porosity and surface irregularity, the mechanical bond also depends on factors such as the size of the aggregate's individual crystal faces and the absorption capability of the aggregate. 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. According to Tarrer (1986), chemical and electrochemical properties of aggregate surface in the presence of water were the most important factors for stripping.

The thermodynamic or surface energy concept involves the wetting behavior of asphalt at the asphalt-aggregate-water-air interface. Adhesion results from the interfacial energy relationships at the aggregate-asphalt-water-air interface. According to Tarrer, when asphalt spreads over and wets the aggregate surface, a change in energy called "adhesion tension" takes place, which depends on closeness of contact, mutual affinity of the two materials and the time of contact. An aggregate tends to get 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, and in such situations, water tends to strip an asphalt coating at the interface.

The chemical concept suggests that when aggregate is coated by asphalt, selective adsorption occurs at the interface, followed by a chemical reaction. According to Tarrer, "…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".

Fromm (1974) suggested that emulsion formation is a mechanism by which asphalt stripping occurs, and if it can be retarded or prevented, pavements can be made to last longer. Water may enter and penetrate asphalt films by spontaneous emulsion formation. The bond formed by an asphalt film on the aggregate surface may be breached at an air-water-asphalt interface by interfacial tension. According to Fromm, asphalts vary in their rate of emulsification depending on the nature of the asphalt, and the rate and extent of emulsification may be increased or decreased with use of different additives.

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. He concluded that because of low pore volume and surface area, granite has a relatively high moisture sensitivity. Low pore volume or surface area is usually an indication of a smooth, crystalline aggregate surface with low surface roughness. Therefore, the pore volume and the surface area of an aggregate are two primary physical properties on which its moisture susceptibility is dependent.

Tarrer (1986) also made another important observation with regard to physical surface properties of aggregates. He compared the interfacial bonding of asphalt with dolomite and

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crushed quartz gravel, and observed that "Although dolomite had a higher surface area, it also had more stripping propensity because of its smaller pore size compared to crushed 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.

According to Tarrer (1986), the pH of the water in contact with the aggregate-asphalt system also influences stripping. He claimed that "During the aggregate coating with asphalt, the aggregate is likely to adsorb some components of asphalt, probably its more polar species, thus forming hydrogen bonds or salt links. These links between the adsorbed asphalt component and the aggregate surface are likely to…..increase pH of contacting water….stripping damage may be expected for an aggregate, which changes the pH of contacting water to a relatively high value".

A study made by Curtis et al. (1992), focused on the influence of aggregate chemistry on adsorption and desorption of asphalt, suggested two mechanisms to explain stripping. The researchers suggested "….water is drawn from the air voids through the asphalt surrounding the aggregate and onto the asphalt-aggregate interface, decreasing the Gibbs' free energy and diminishing the strength of the bond between the asphalt and the aggregate." In this study, Curtis et al. (1992) found that the accurate measurement 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 for stripping. On the other hand, Curtis et al. (1992) concluded that the role of a higher Gibbs' free energy of the asphalt-aggregate interface is to increase tensile strength at the interface and increase resistance to de-bonding.

Nguyen et al. (1996) also studied the mechanism of stripping, and observed that bonding between asphalt, an organic material, and a siliceous aggregate is governed mostly by weak secondary forces, which are generally less than 25kJ/mole, whereas the magnitude of the bonds water forms with silica oxide surfaces are substantially higher, typically in the range of 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 SiO_2, 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 the interface is exposed to water or high relative humidity. 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. On the other hand, the asphalt, with its low basicity and high acidity, interacts weakly with acidic SiO_2 but adheres strongly with basic aggregates, such as limestone. The strong asphalt-basic aggregate bonds can resist the water-SiO_2 interactions, thus preventing stripping of asphalt from the aggregate. Modifying siliceous aggregate surfaces with a monolayer of a base, such as amine, will render the surface basic. This surface would form strong bonds with the acidic asphalts (Nguyen et al., 1996).

2.4.3   Antistripping Additives

As discussed in the preceding section, stripping results in weakness and loss of adhesive bond between the aggregate and binder in the presence of water. In order to eliminate this problem, a great number of antistripping additives, which are actually surfactants, are available in the market. Many of these chemical antistripping additives, which are mostly proprietary

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chemical compounds, have shown to reduce stripping damage (Tarrer, 1986). These antistripping additives are added to asphalt cement to improve 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). Lime is also a very common antistripping additive.

TxDOT specifications require that 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 (TxDOT 2004).

Tarrer (1986) tested eight different antistripping additives, consisting of primary and secondary amines and other organic nitrogen compounds, and the following conclusions were made:

- All the adhesion additives studied were capable of reacting with asphalt components at typical asphalt handling temperatures. This reaction causes the additive concentration in asphalt to decrease while the asphalt is stored at a high temperature.
- The contact angle of the asphalts were markedly reduced when antistripping additive were used, resulting in better wetting of the aggregate surface.
- In general, all the commercial antistripping additives tested were found to improve stripping resistance at least to some extent, depending on their concentration.
- With each additive, a specified resistance to stripping could be attained by sufficiently increasing its dosage.
- The optimum dosage of an additive varied depending on the asphalt and the aggregate.
- Direct application of additive to the aggregate surface was observed to require a lower amount of additive (about 5%) than that required when the additive was added 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 hot asphalt cement.
- The performance of an asphalt-aggregate mixture was improved remarkably by storing (curing) the mixture at approximately 300 °F for several hours. The improved adhesion of the asphalt to the aggregate surface during curing is most likely not due to diffusion effects but rather some type of coupling reactions between the asphalt and the aggregate surface.

Halstead (1985) and Tarrer (1986) observed that addition of most antistripping additives softened the asphalt. This effect varied for different asphalt-additive combinations. Age hardening of asphalt can be reduced by the addition of amine type antistripping additives. Research done by Halstead (1985) indicates a reduction in temperature susceptibility and an improvement in aging characteristics when antistripping additives are used. It appears that the additives affect properties of asphalts primarily by disruption or altering the molecular interactions of polar species in the asphalts.

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2.5   Test Methods used to Characterize Aggregate-Bitumen Bond

A number of test methods have been developed to characterize the aggregate-bitumen bond in hot mix asphalt (HMA) and surface treatment applications. Several of these test methods are used by practitioners, while others have been limited to research investigations. In HMA applications, the significance of aggregate-bitumen bond is related to moisture-induced stripping of the bond. In surface treatment applications, its significance is related to aggregate retention.

2.5.1   Moisture Sensitivity Tests

2.5.1.1 Boil Test

Boil test is one of the most commonly used tests to evaluate the stripping potential of aggregates. It is widely used due to its simplicity and reasonably accurate results. There are many variations of the boil test, but its basic test procedure involves the following steps:

- Soak 100 grams of aggregate sample in distilled water for 24 hours and towel dry
- Preheat aggregate sample in a stainless steel bowl and keep in an oven at 300° F for 1 hour
- Add 5.5 grams of asphalt cement heated at 275° F for 10 minutes to the preheated aggregate
- Mix asphalt cement and aggregate with a hot spatula for 2 minutes and keep in an oven maintained at a specified temperature (reheating temperature) for a specified period of time (reheating time)
- Cool the mix sample to room temperature and put the mixture in 250 ml of boiling water in a 400 ml beaker on a hot plate
- Maintain the water at a slow boil for 10 minutes while stirring with a glass rod at 4 minutes and 8 minutes for 10 seconds each time
- Allow mixture to cool down to room temperature while still in the water
- Drain water from the beaker and place the mixture on a paper towel and allow to dry
- Evaluate the amount of stripping by visual observation and record it as a percentage of asphalt coating retained on the aggregate after boiling

Tarrer (1986) stated that the boiling water test procedure has reasonable correlation with field performance. He found that the 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 increased considerably.

Two other tests that are often used to determine the stripping potential of aggregate-binder systems are the freeze-thaw pedestal test and the indirect tensile test. Both these tests involve the conditioning of asphalt concrete test specimens, either by soaking or by using a soak-free-thaw procedure before testing the cylindrical specimens for indirect tensile strength.

Parker and Gharaybeh (1988) compared the three stripping tests, boil test, freeze-thaw pedestal test and the indirect tensile test (ITT) on five different aggregates whose field stripping potentials were also known. The researchers concluded the following:

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- The tests correctly matched laboratory and field performance of three of the five aggregates.
- In some cases, the tests may not be valid indicators of stripping, or the subjective reported field performance may not be valid for specific mixes.
- 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.
- Two aggregates that provided mixes with high indirect tensile strengths under control conditions had different stripping potentials.

Paul (1995) compared the three tests, Louisiana ten minute boil test, freeze-thaw pedestal test and indirect tensile test (ITT) based on their ability to measure stripping potential, and made the following conclusions:

- The boil test was discriminating with respect to aggregate source, antistripping additive and asphalt cement source. Even though this is a subjective test, no significant differences were found between raters for the same asphalt-aggregate combinations.
- The indirect tensile test (ITT) was able to distinguish the performance between different aggregate-asphalt 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 mixes with low efficiency antistripping additives and those with no additives.
- As a diagnostic test, the freeze-thaw pedestal test accurately identified potential moisture problems for 8 of 12 mixtures or aggregate components, when compared to actual 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 a similar record of identifying potential moisture problems.
- The freeze-thaw pedestal test was capable of discriminating between all antistripping additives including hydrated lime, but did not demonstrate differences in performance between lime slurry and dry lime as the other methods did. Also, most mixes tested with the freeze-thaw pedestal test did not indicate poor performance, which may indicate that this test is not useful for establishing job mix performance.

In the end, Paul (1995) stated that each of the test methods he evaluated can be used to predict potential moisture susceptibility problems. However, he cautioned about the shortcomings associated with each test method; boil test only examines the effect of coarse aggregate on stripping and the pedestal test can take up to 20 days to complete. Paul also indicated that while the ITT examines the entire mixture, it is not discriminating with respect to antistripping additive type or possibly its dosage. He recommended that the boil test, because it is a quick and easy test to perform, should be used to establish the dosage of the antistripping additive to be used.

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2.5.1.2   Strategic Highway Research Program (SHRP) Net Adsorption Test

The net adsorption test (NAT) is a product of the Strategic Highway Research Program, and it was developed to evaluate moisture susceptibility of different aggregate-binder combinations for hot mix asphalt concrete (SHRP 1993). It is one of the first efforts to evaluate aggregate-binder moisture susceptibility by using principles of chemistry and surface science. This test procedure has three parts. First, an asphalt-toluene solution is allowed to flow through an aggregate sample in a constant temperature recirculating column and adsorb onto the aggregate. As the asphalt-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 and the final (after 7 hours of adsorption) concentration of asphalt in the solution are determined by measuring the light absorbance at 410 nm using a UV-visible spectrophotometer. Second, a small specified amount of water is introduced to the asphalt-toluene solution to investigate the desorption (stripping) potential of adsorbed asphalt. 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 to the aggregate, and the stripping potential of that asphalt-aggregate combination. The difference between the amounts of asphalt adsorbed before and after desorption serves as an indicator of the water sensitivity of the asphalt-aggregate combination (Curtis et al., 1992).

The concentration of the asphalt in solution is 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 of asphalt in toluene (g/L).

The equations for calculating the amount of asphalt 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 are concentrations of asphalt solution before and after adsorption, respectively;
A_0 and A are visible light absorbance 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 involves introducing 1150 μl of water into approximately 126 ml of asphalt-toluene solution and allowing the solution plus water to re-circulate for 2 hours through

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the aggregate column. After this step is completed, the absorbance of the asphalt solution is measured at 410 nm, and the asphalt concentration is determined using the following equations:

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

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 if 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.

$$\text{Net adsorption} = A_{sp} - D_w.$$

This test procedure uses an initial concentration of 0.6 grams of asphalt per liter of 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. Sometimes this step can be shortened if satisfactory results are obtained sooner. Reaching the equilibrium is essential in the desorption step to achieve satisfactory results. The use of three replicate specimens along with one control specimen is recommended for the test. Based on the net adsorption results of aggregate-binder combinations, SHRP suggests the following criteria to evaluate aggregate-binder adhesion performance (Table 2-4).

Table 2-4 SHRP Criteria for Aggregate-Binder Adhesion Performance (Curtis et al. 1993)

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

2.5.1.3 Scaled-up Net Adsorption Test

After the SHRP net adsorption test was developed by Curtis et al. (1993), the researchers continued to work on refining the test to make it more economical and efficient. The result of these research efforts is the "scaled-up" net adsorption test which had the following objectives.

- Maximize the quantity of aggregate used in the test while minimizing the amount of solvent
- Use aggregate gradations as close as practical to those used in pavements
- Keep the relationship between materials quantities the same as the initial study

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- Adjust the testing protocol so that the test can be completed in a typical working day
- Facilitate the 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 grams. In order to reduce the amount of solvent used, the volume of solvent was halved and the amount of asphalt dissolved was doubled to keep the ratio of asphalt to aggregate between the two procedures the same. 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 test preparation time and 30 minutes allowed for cleanup, this procedure could be completed during normal laboratory operating hours in one day.

The scaled-up test method showed higher levels of initial adsorption when compared to the previous method. 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 re-circulated more times. The net adsorption values were also correspondingly higher for the scaled-up test than for the original SHRP 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, and to screen material combinations for suitability in pavement construction.

2.5.1.4 National Roads Authority of Ireland Net Adsorption Test (NRA NAT)

The National Roads Authority (NRA) of Ireland developed a modified version of the NAT for chip seals (seal coats). In this procedure, the circulating column was replaced by a mechanical shaker. The test consists of the following 4 steps and the total test duration is 24 hours.

- Three test flasks are prepared for each aggregate-binder combination and a fourth flask is prepared as a control sample. The control sample is carried through the entire test procedure to ensure that there is no material on the surface of the aggregate that will interfere with the test
- A bitumen-toluene solution of known concentration (approximately 1 g of bitumen to 1 liter of toluene), is prepared
- Fifty grams of aggregate, graded as shown in Table 2-5, is added to each flask including the control flask. Walsh et al. (1995) indicated that the precision of this test method was excellent when the specified gradation 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.
- 2 ml of water is added to each flask and shaking is continued for a further period of 16 to 17 hours.

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Table 2-5: 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:

Initial concentration of bitumen-toluene solution $= A_1$
Concentration after 6 hours in contact with the aggregate $= A_2$
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 (V) $=$ 136 ml
Percent Net Adsorption $= \frac{A_n}{A_i} \times 100$

One major drawback of this test method for surface treatment applications is that, for emulsified asphalt, which is a popular binder type for seal coats, only its base asphalt can be used. Walsh et al. (1995) tested the residue of the bitumen emulsions using NAT by extracting it from controlled evaporation in a stream of air under constant pressure and constant temperature for 18 hours. They commented that

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"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 bitumen. However, there is no reason why the test should not be used to rank various bitumen emulsions with aggregate".

The differences of the initial asphalt adsorption were quite small for a given aggregate, although the asphalts differed substantially in their chemical composition and characteristics. Asphalt cements exhibited high and low levels of adsorption on the same aggregates, but the magnitude of the differences among different aggregates for the same asphalt was quite large. The amounts obtained for initial adsorption were high for limestone and low for granites. Statistical analysis of the test results showed that initial and final adsorption was a function of aggregate properties such as potassium oxide content, while little influence was observed from other variables such as binder composition. The initial and net adsorption values were higher for the scaled-up net adsorption test than the original SHRP method due to increased solution flow rate in the scaled-up test. The difference between the initial and net adsorption values for the original SHRP test versus the scaled-up test was highly variable due to the variations in the test method and material parameters. Due to the satisfactory performance of the scaled-up test, the authors recommended minus No. 4 fraction of unwashed aggregate for the net adsorption test to represent most accurately the aggregates used in the road pavements (Walsh 1995).

2.5.2   Aggregate Retention Tests

All the tests described above do a reasonable job of evaluating the stripping potential of asphalt-aggregate combinations. However, none of these methods can effectively evaluate the compatibility of aggregate and binder used in a seal coat because such compatibility depends on a number of field conditions. The following tests have been developed to evaluate seal coat aggregate-binder compatibility.

2.5.2.1 Australian Aggregate Pull-Out Test (RTA T-238)

The Road and Traffic Authority (RTA) of New South Wales, Australia (RTA 2001) have been using a test protocol to evaluate the aggregate-binder compatibility in their seal coats. This test method (RTA T-238) allows the user to assess the extent of adhesion between aggregate and a bituminous binder under wet and dry test conditions. The test is conducted at the room temperature using cutback asphalt binders.
This test protocol has provision for both precoated and uncoated aggregate as well as binders with or without modifier. Binders are usually cutback to give the desired viscosity at test temperature. The target viscosity of this cutback binder for the test should be around 15,000 stokes. About ten samples are used to evaluate each binder-aggregate combination. Each sample is made up of a 3 mm thick asphalt film with 20 aggregate particles embedded into it. In addition to the "as received" condition, the testing protocol also allows for the evaluation of four aggregate surface conditions; clean and dry, dusty, saturated-surface-dry, saturated-surface-wet.
Preparation of the test specimen involves heating the binder until it becomes fluid and pouring a sufficient amount in a metal tray to produce a 3 mm thick layer. Now, the aggregate

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particles are manually pushed into the binder to replicate the seal coat operation. Each seal-coat sample prepared above is tested after curing under soaked or un-soaked condition. The test involves pulling the aggregate out of the binder with the aid of a pair of long nosed pliers while the aggregate is still under water at room temperature. Once pulled from the asphalt, aggregates are checked for the percentage of coated area, and they are categorized into three groups, i.e. completely stripped, partially stripped and not stripped. The final results are expressed in terms of percent aggregate surface area stripped, by taking a weighted average of these three categories.

2.5.2.2 Pennsylvania Aggregate Retention Test

In a study conducted by Khandhal and Motter (1997) to investigate the effect of precoating on seal coat aggregate retention, the Pennsylvania Aggregate Retention Test was presented. The equipment used in this test consists of 8-inch sieves, 8-inch pans, a sieve shaker, rubber pads, a compression machine and a balance. The test procedure for an emulsified asphalt binder is outlined below.

1. Apply the bituminous binder at the appropriate application temperature on the back of an 8-inch separator pan to obtain an application rate of 0.25 gal/yd^2.
2. Apply the cover aggregate at a rate of 17.4 lb/yd^2 using a Mary Ann® laboratory sieve shaker.
3. Within 15 minutes of the application of aggregate, the pan is covered with a 7-1/2 in diameter and ¾ inch 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 corresponds to that of pneumatic-tired rollers commonly used in seal coat construction.
4. After compaction, the bearing pad is removed and the pan containing emulsion and aggregate is cured under ambient temperature for 24 hours. The weight of pan and the seal is measured after curing.
5. 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:

$$\text{\% Initial loss} = \frac{B}{A} x100$$

   Where: A = Weight of total aggregate (300 grams)
             B = Initial loss in grams.
6. After the initial loss is determined, the pan containing emulsion and aggregate is placed upside down at the top of five ½ 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:

$$\text{\% Knock-off loss} = \frac{C}{A - B} x100$$

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C is the knock-off loss in grams.

7. The total loss (initial loss + knock-off loss) is calculated as follows:

$$\% \text{ Total loss} = \frac{B + C}{A} x100 \,.$$

2.5.2.3 Vialet Test

The Vialet drop test was developed by the LCPC in Paris in the early 1960's. This test method was used to determine aggregate design quantities and the strength of asphalt-binder bond. This test uses a 0.62 cm (0.25 in.) thick steel plate 17.5 cm (7 in.) square as a sample preparation medium. A 0.2 cm (0.25 in) rim was used around the plate to prevent binder runoff. An impact force was applied to the inverted plate containing the chip seal sample by dropping a 5 cm (2 in.) diameter steel ball from a height of 45 cm (18 in.).

Both the plate and the binder (emulsion) are preheated to 60°C (140° F). A quantity of binder corresponding to the appropriate field application rate is applied to the steel plate. The plate is rotated until the binder is evenly distributed over the surface. Aggregate is also applied in a manner suitable to mimic field conditions. Rolling was simulated with a rubber tire with an inflation pressure of 14.8 kg/cm^{2} (40 psi) and weighing 396 to 440 lbs (180 to 200 kg). 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 are tested at 10 minutes, 30 minutes, 2 hours, 5 hours, and 24 hours after the aggregate is applied. All samples are stored at 25° C (77° F) and a relative humidity of less than 30%. The initial weight of each plate containing the seal coat sample is obtained, and 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;

$$\% \text{ Retained} = 100 - \frac{(Original\ Weight - Weight\ after\ Invert)}{(Original\ Weight)} x100 \,.$$

While the plate is re-inverted in the apparatus, a steel ball is dropped at the center of the plate three times within a 10-second period. A final weight is taken, and the percent material retained after impact is calculated. The test data can be used to evaluate the rate of set of the binder by monitoring the increase in material retained with time. The aggregate lost during the initial invert testing at the 5-hour 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 Vialet test.

1. Problems with excess aggregate can be identified with the 5 hour or 24 hour initial invert Vialet test.
2. Problems with aggregate pick up on rollers during construction are related to the 10-min impact Vialet test results showing less than 30% material retained.

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3. Problems of surface damage due to early brooming are indicated by the 30-min impact Vialet 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-hour impact Vialet test results greater than 80% material retained.

The test has also been used to measure the adhesion of binder and aggregate. Coyne (1988) made several modifications to the Vialet Test. Gallon-can lids were used instead of 200-mm square steel plates. The can lids, with the aid of ¼-inch 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 Vialet 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" Vialet test was found to be quite effective.

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CHAPTER 3

CONSTRUCTABILITY REVIEW

The success of a new testing protocol depends on its ability to accurately predict the field performance. Therefore, a field constructability review was undertaken as an integral part of this research to evaluate the field performance of seal coat test projects for which researchers collected data during construction. The research team visited 15 of the 25 TxDOT districts and test sections were selected from 14 of those districts. Figure 3-1 shows the TxDOT districts that were visited for this constructability review.
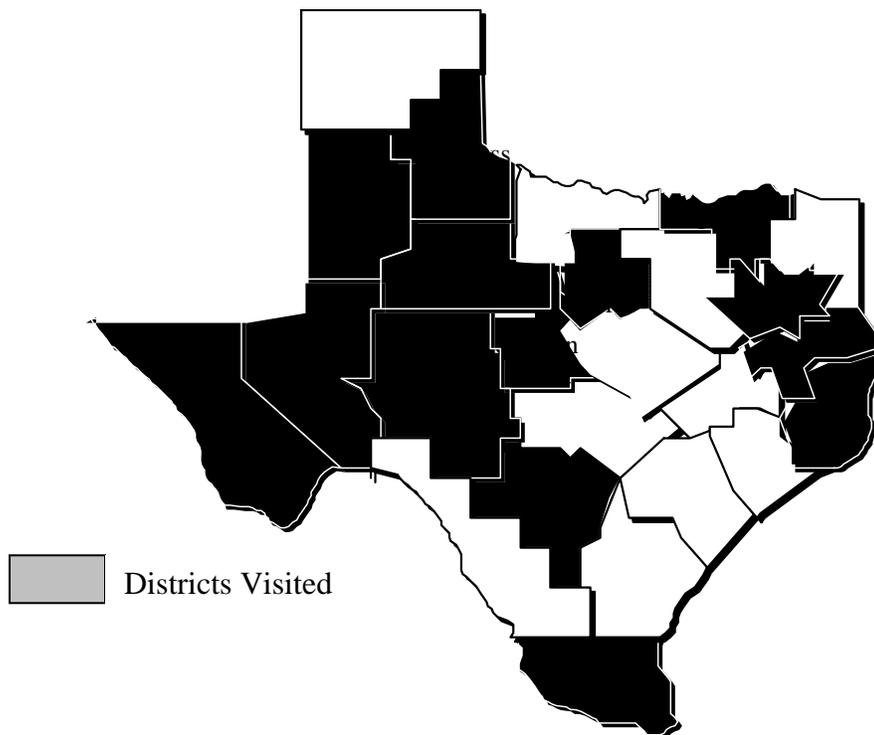
Figure 3-1 Districts Visited for the Constructability Review

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

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was to come up with different scenarios of incompatibility, and a database of field information was created.

The site visits were coordinated with each district, and 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.

For each district visited, a data collection sheet (Figure 3-2) was filled out. It included information such as location of test section, climatic data during and immediately after construction, existing surface condition, and construction process information. 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 inside the distributor. The binder temperature was also recorded in three instances using a infra-red non-contact thermometer; just after the shooting, when the aggregate was spread, and when the new seal coat was rolled. The time difference between binder application and aggregate spreading, as well as between aggregate spreading and rolling, were also recorded. 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 and still images of the construction process were also collected.

3.1.2   Sample and Material Collection

For every district visited, the researchers 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 roadway features, but typically, 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 (Figure 3-4). These samples were transported back to the Texas Tech University Asphalt Laboratory and stored in a library of seal coat samples.

District:	Country:	Highway:	Location:	Date:

Surface Before Construction: ☐ New Patch ☐ Old Patch ☐ Flushing ☐ Bleeding ☐ Dusty ☐ Level-Up ☐ Other

Time of Construction:

Ambient Temp.: Average: High: Low: Humidity:

Weather Conditions During Construction: ☐ Sunny ☐ Cloudy ☐ Partly Cloudy ☐ Windy **Comments:**

Existing Surface: ☐ HMAC Gr. ☐ Seal Coat ☐ Slurry Seal ☐ Other **Color:** ☐ Oxidized ☐ Slightly Oxidized ☐ Not Oxidized

Condition of the existing surface: ☐ Bleeding ☐ Flushed ☐ Smooth ☐ Slightly porous ☐ Slightly pocked, porous ☐ Badly pocked

Aggregate Source: Grade: **Condition:** ☐ Dusty ☐ Air Dry ☐ Moist ☐ 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: ☐ Uniform ☐ Variable **WP 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.) ☐ Before spread ☐ After spread **Time to break (min):**

Time difference between the spreading of aggregate and rolling:

Roller Type: ☐ Lightweight ☐ 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 used in Construction Site Visits

During the laboratory portion of this research, 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. Results from these laboratory tests and related discussion are presented elsewhere in this report.

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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 each test project site. These materials were used to make additional laboratory specimens for developing the laboratory test protocol and related 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 3-3 Collection of Field Specimens at Test Projects (Insert Shows Specimen Plates Placed on Existing Pavement)

Figure 3-4 Typical Seal Coat Specimen Collected from the Field

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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 when the seal coat project is subjected to different seasonal and traffic effects. Therefore, approximately one year after the initial site visits, the research team made a follow-up visit to evaluate the performance and check the integrity of each test section. Particular emphasis was placed 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. Data collected on the performance of each seal coat test section were used as field verification of the test protocol developed during this study (see Chapter 4). The specimens that were brought to Texas Tech from previous year's site visits were tested with the new testing protocol. The comparison of results from the new testing protocol and the actual field performance of each test section was used to demonstrate the capability of the new test protocol to recognize any problems of material incompatibility.

More than half of the test sections visited, including those in 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 causes other than aggregate-binder compatibility. The test sections in San Angelo District performed well in general, although district personnel 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 had bladed off some sleet and snow after a winter storm. The projects in Odessa District 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. For that reason this section is included in the excellent category. Pharr district has not experienced any problem at all, and 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

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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). District personnel indicated that a possible cause for this severe rock 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 were 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 Illustration of Rock Loss due to Aggregate-Binder Incompatibility

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 emulsion binder used in all three of these test sections.

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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 covered the road; rest of the road is in excellent condition, not considered as having incompatibility problems
Abilene	3	rock loss observed where seal coat was applied over recent hot mix patch, fog seal was applied in December 2002 to stop rock loss, fog seal was successful
Lubbock	2	very low embedment depth (10-15% loss in several places, and as much as 100% loss in a few locations)
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.

One general observation made in the follow up study was that loss of aggregate related to incompatibility always occurred outside or between wheel paths. This is an indication of how traffic can help improve aggregate retention. Particularly in hot weather when the binder has a softer consistency, traffic will continuously compact the aggregate resulting in higher embedment depths and better aggregate retention. This could well 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

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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 when the newly applied seal was opened for traffic prematurely. Such loss of aggregate was 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 personnel who were interviewed attributed the aggregate loss to a thunderstorm that occurred soon after construction was completed.

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CHAPTER 4

THE MODIFIED NET ADSORPTION TEST

As discussed in chapter 2, moisture is a major threat to the satisfactory functioning of asphalt pavements. However, there are very few laboratory test methods available that can accurately predict the effects of moisture on asphalt-aggregate combinations. In the late 1980's, The Strategic Highway Research Program (SHRP) developed a test method to evaluate the moisture susceptibility of hot mix asphalt mixes; *SHRP Standard Test Method 1013: Measurement of Initial Asphalt Adsorption and Desorption in the presence of Moisture* (Curtis et al. 1990). Results from this test method produced evidence to suggest that adhesion failure is primarily caused by undesirable aggregate characteristics and to a lesser extent by the asphalt binder characteristics. However, Woodside et al (1993) identified several limitations with regard to the specifications, laboratory equipment and reporting of the SHRP test results. They stated that although the SHRP method is effective in predicting the moisture sensitivity of the bond between a particular asphalt binder and an aggregate, it does not provide adequate information regarding the amount of bitumen initially adsorbed and subsequently desorbed after the water is added to the system. Based on research conducted at the University of Ulster in Ireland, Woodside et al (1993) presented a modified version of the SHRP test. Walsh et al. (1995) modified the test method of Woodside et al. even further to extend its applicability to seal coats (i.e. chip seals). This modified net adsorption test (NAT) developed by Walsh et al. was used in this research to evaluate the effectiveness of aggregate-bitumen bond for this research project on seal coats.

4.1 Experimental Design

One of the limitations of using the modified NAT is that it cannot be used for emulsified asphalt seal coats and the seal coats that use precoated aggregate. The researchers explored the viability of a number of tests for emulsified asphalt-aggregate bond, and the only test method that had some promise was the zeta-potential test. However, this test uses finely ground aggregate particles, and its practical validity in day-to-day TxDOT seal coat work is questionable. For these two types of seal coat materials (i.e. emulsified asphalt seal coats and seal coats with precoated aggregates), the researchers recommend the *Performance-Based Seal Coat Aggregate-Binder Compatibility Test* outlined in Chapter 5. This performance-based test method is also recommended for the materials tested using the modified NAT because of its ability to simulate a number of field conditions that influence seal coat performance.

The experimental design for the modified NAT includes two factors; asphalt binder, aggregate. The results were used to calculate three parameters that represent adhesion of asphalt on the aggregate and its stripping potential; initial adsorption, final adsorption and percent net adsorption. Asphalt cement and aggregate, each obtained from five different sources, were selected for the first round of tests. Five additional asphalt cement-aggregate combinations were sampled from field projects to compare results from modified NAT to field performance in actual projects.

The five sources of aggregates and binders used in the first round of laboratory tests are presented in Tables 4-1 and 4-2 respectively. These material sources were selected based on

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their heavy usage in TxDOT seal coat projects. In Table 4-2, asphalt binder samples taken directly from plants are referred to as plant samples and those taken from field projects are referred to as field samples.

4.2 Testing Protocol for Modified Net Adsorption Test (NAT)

This test method, as described previously in this chapter, involves an *adsorption* and d*esorption* phase of asphalt cement on aggregate. The asphalt is in contact with the aggregate in the form of an asphalt-toluene solution. The moisture sensitivity of the asphalt-aggregate bond is evaluated using the parameter *percent net adsorption* as suggested in the SHRP testing protocol developed by Curtis et al. (1990), and also by Woodside et al. (1993) at the University of Ulster.

Table 4-1 Sources of Aggregate used for the Modified NAT Laboratory Program

Aggregate Type	Source
Limestone	Vulcan Materials, Brownwood Pit
Lightweight	TXI, Streetman Plant
Rhyolite	Trans-Pecos Materials Hoban Pit
Siliceous Gravel	Delta Materials, Marble Falls Pit
Sandstone	Martin Marretta Materials, Sawyer Pit

Table 4-2 Sources of Asphalt Binder used for the Modified NAT Laboratory Program

Asphalt Binder	Sample Type	Sources
AC-15 5TR	Plant	Trumbull Asphalt, Houston Plant
	Plant	Alon USA, Big Spring Plant
	Field	Trumbull (Wright) Asphalt taken on 06/18/02 from Lufkin District, San Jacinto Co., US 59 NB shoulder
AC-5 + 2% latex	Plant	Alon USA, Big Spring Plant
	Field	Eagle Asphalt (Corpus Christi) taken on 8/23/02 from Odessa District, Midland Co., FM 1379
AC-10 + 2% latex	Plant	Alon USA, Big Spring Plant
	Field	Eagle Asphalt (Corpus Christi) taken on 06/26/02 from Lubbock District, Floyd Co., FM 602 South of U.S 62
AC-15P	Plant	Koch Materials, Saginaw Plant
	Field	Eagle Asphalt (Corpus Christi) taken on 08/14/02 from San Antonio District, Kendall Co., RR 473 W
PG 64-22	Plant	Lion Oil Co., El Dorado Plant in AR

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4.2.1. Apparatus and Reagents

The following apparatus and reagents were used in the modified NAT.

- Shaker Table capable of holding eight 500ml Erlenmeyer flasks (Figure 4-1)
- Spectrophotometer capable of providing a continuous 410 nm wavelength with an accuracy of ±2nm, holding standard 10 mm path length cuvettes (Figure 4-2)
- Spectrophotometer cuvettes capable of 4.5 ml and 10 mm path length
- 500ml Erlenmeyer Flasks
- Volumetric Flasks with 25ml and 1000ml capacities
- Filter paper: Whatman No. 42, 125mm diameter
- 250ml graduated glass cylinder
- 10ml pipettes
- Analytical Balance with precision up to 3 decimals
- Aggregate Drying Oven capable of maintaining 135 °C
- Toluene: UV grade / Spectroanalyzed grade
- Distilled water

Figure 4-1 Mechanical Shaker

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Figure 4-2 Spectrophotometer

4.2.2 Preparation of Aggregate Samples

Each aggregate-asphalt combination needs four 50g aggregate samples with the standard gradation shown in Table 4-3. They are dried in an oven (uncovered) at 135°C for approximately15 hours. The samples are removed from the oven at least 15 minutes prior to being placed in a 500ml Erlenmeyer flask to prepare the stock solution. The following table gives the quantities of aggregate for each size to make up the 50g test sample.

Table 4-3 Standard Grading of Aggregate for the Modified NAT (Walsh et al. 1995)

Sieve Size	Percent Retained	Wt. Retained (g)
2.36 mm	8.0	4.3
1.18 mm	25.0	13.5
600 µ m	17.0	9.1
300 µ m	23.0	12.4
150 µ m	14.0	7.5
75 µ m	6.0	3.2
	Total	50.0

4.2.3 Preparation of Stock Solution

In a 1000ml volumetric flask, 0.5g of each asphalt sample (±0.001g) is weighed and dissolved in 500ml of toluene to produce a stock solution with a concentration of 1g/liter.

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4.2.4   Test Procedure

The test takes nearly 24 hours to complete and the test procedure can be divided into the following four main steps.

4.2.4.1 Preparation of Test Samples and Control Sample

Each aggregate sample is placed in a labeled 500ml Erlenmeyer flask. Of the four Erlenmeyer flasks, three are test samples and the fourth is used as the control sample. 140ml of a freshly-prepared asphalt-toluene stock solution is added to each of the three test flasks while 140ml of pure toluene is added to the control flask to check if the aggregate would interfere with the adsorption measurements. All four flasks are placed on a mechanical shaker operating at 300 rpm.

4.2.4.2 Measurement of Initial Absorbance of the Stock Solution

Approximately 4ml of the prepared stock solution is diluted with pure toluene in a dry, clean 25ml volumetric flask and the initial absorbance of the asphalt-toluene solution is measured at a wavelength of 410 nm using the spectrophotometer.

4.2.4.3 Measurement of Initial Adhesion

After the Erlenmeyer flasks are shaken for 6 hours, 4ml of solution from each flask is filtered and diluted with toluene in a clean dry 25ml volumetric flask and the diluted solution is then transferred into the spectrophotometer cuvette and light absorbance at 410 nm is determined. After the addition of 2ml of distilled water to each of the 250ml Erlenmeyer flasks, flasks are put on the shaker table for shaking overnight.

4.2.4.4 Measurement of Final Adhesion

After 15 to 17 hours of shaking, a final absorbance is determined using the procedure explained in 4.2.4.2 above.

4.2.5   Calculations

Two calculation procedures adopted in this research to evaluate the adsorption and stripping potential for asphalt-aggregate combinations were the SHRP NAT procedure (Curtis et al. 1990) and the modified NAT procedure (Walsh et al. 1993). The following notations and symbols were used in the calculation procedures used.

A_i = Initial adsorption, i.e. amount of bitumen initially adsorbed to the aggregate surface, mg/g
A_n = Net Adsorption, i.e. amount of bitumen remaining on aggregate after water is added, mg/g
A_{max} = Theoretical maximum absorbance reading, representing the condition when all the bitumen in the solution has been absorbed by the aggregate
$\% A_i$ = Percent initial absorption

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V = volume of solution in the flask, 140 ml
W = weight of aggregate, in grams
C = Initial concentration of bitumen in solution, (typically 1g/liter)
A_1 = Initial absorbance reading
A_2 = Absorbance reading after 6 hours of shaking (without moisture)
A_3 = Absorbance reading after 16-17 hours of shaking with moisture in the system
A_n = Net adsorption, mg/g
$\%A_n$ = Percent net absorption, i.e. % bitumen remaining on the aggregate after the test
V_r = Volume of solution in the flask when measurements to calculate A_3 is obtained (i.e. 136 ml)

4.2.5.1 SHRP Calculation Procedure

$A_i = VC*(A_1-A_2)/ (WA_1)$
$A_n= V_rC (A_1-A_3)/ (WA_1)$
$\%A_n = (A_n / A_i) * 100$

4.2.5.2 University of Ulster (UU) Calculation Procedure

$A_i = VC*(A_1-A_2)/ (WA_1)$
$A_n= V_rC (A_1-A_3)/ (WA_1)$
$A_{max} = VC * (A_1-0) / (WA_1)$
$\%A_i = (A_i / A_{max}) * 100$
$\%A_n = (A_n / A_{max}) * 100$

Where $\%A_n$ is the percent net adsorption, which is different from $\%A_n$ calculated under the SHRP procedure.

4.3   Results and Discussion

The criteria developed by SHRP to evaluate the performance of aggregate-binder adhesion based on the percent Net Adsorption Test (NAT) results are summarized in Table 4-4. The results from the modified NAT are presented for each of the five aggregate types tested, are discussed in section 4.3.1 below.

Table 4-4 SHRP Evaluation Criteria for Aggregate-Binder Adhesion (Walsh et al. 1995)

SHRP Percent Net Adsorption ($\%A_n$)	Expected Performance of Aggregate-Binder Bond
>70	Good
55-70	Marginal
<55	Poor

4.3.1   Evaluation of the Performance of Aggregate-Binder Bond

The results from the modified NAT are summarized in the following section for the two evaluation parameters (initial adsorption and percent net adsorption) using both the SHRP

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method and the revised University of Ulster method. The percent initial adsorption using the University of Ulster calculation method showcase the affinity of an aggregate to a given asphalt cement. On the other hand, the percent net adsorption value shows the resistance of the aggregate-binder bond to stripping in the presence of moisture.

4.3.1.1 Crushed Limestone

Limestone is a sedimentary rock that is used by many TxDOT districts in both seal coat and hot mix asphalt (HMA) applications. The limestone aggregate used in the net adsorption test program was sampled from the Vulcan Materials Brownwood pit and it was tested with five different plant asphalts as indicated in Table 4-3 (PG 64-22 from Lion Oil, AC-15 5TR from Alon USA in Big Spring, AC-15 5TR from Trumbull Asphalt, AC-15P from Koch Materials Saginaw plant, and AC-10 + 2% latex from Alon USA in Big Spring.). Three field binders collected from construction project sites were also tested with the limestone aggregate; AC-5 + 2% latex from Eagle Asphalt in Corpus Christi, AC-15 5TR from Trumbull Asphalt, AC 15P from Eagle Asphalt in Corpus Christi. Table 4-5 shows the results for the modified NAT for limestone aggregate with plant and field asphalts indicated above. Figure.4-3 illustrates the percent initial adsorption and percent final adsorption based on the revised University of Ulster (UU) Method and percent net adsorption based on the SHRP Method, for limestone aggregate-binder bonding.

Table 4-5 Adhesion and Stripping Potential of Limestone with Seal Coat Binders

Asphalt Binder Grade(Source)[*]	SHRP			University of Ulster (UU) Method		
	A_i	A_n	%A_n	A_{max} VC $(A_1-0)/(WA_1)$	%A_i $(A_i/A_{max})*100$	%A_n $(A_n/A_{max})*100$
PG 64-22 (P-Lion Oil)	1.55	0.69	69.84	2.80	55.18	24.59
AC-5+2% Latex (F-Eagle)	1.62	1.10	68.03	2.80	57.72	39.21
AC-10+2% Latex (P-Alon USA)	1.39	0.95	67.99	2.80	49.82	33.80
AC-15P (P-Koch)	1.79	1.14	63.99	2.80	63.76	40.73
AC-15P (F-Eagle)	1.39	0.94	67.84	2.80	49.60	33.63
AC-15 5TR (P-Alon USA)	2.21	1.81	82.21	2.80	78.76	64.75
AC-15 5TR (P-Trumbull)	1.70	1.37	81.20	2.80	60.71	48.93
AC-15 5TR (F-Trumbull)	1.54	1.21	78.63	2.80	55.16	43.38

[*]F – Field Asphalt, P- Plant Asphalt

Figure 4-3 shows several interesting observations regarding the bond between crushed limestone seal coat aggregate from the Vulcan Materials Brownwood pit and several key seal coat binder sources. Based on the SHRP net adsorption test criteria presented in Table 4-4, it can be seen that all three AC-15 5TR sources showed 'Good' resistance to stripping. All other binder sources with the exception of PG 64-22 fell into the category of 'moderate' resistance to stripping. The PG 64-22 from Lion Oil Company in El Dorado showed 'poor' resistance to stripping. This binder grade is commonly used to precoat limestone aggregate used in TxDOT seal coats. The parameter '% Net Adsorption', shows the potential for stripping of initially adsorbed binder due to the presence of moisture. It does not provide an indication of how much binder was adsorbed to begin with, and therefore, it is important to use the % NAT parameter in combination with the '% Initial Adsorption' as calculated by the University of Ulster (UU) procedure. There is no standard set of guidelines to assess the extent or the effectiveness of initial adsorption. However, some relative comparisons can be made from Figure 4-3. The AC-15 5TR binder sampled from the Alon USA plant in Big Spring showed the best coating (almost 80% of available asphalt) of the limestone aggregate. Its adsorption was significantly higher compared to the other binders. Two other binders provided over 60 percent usage of available binder for coating. They were the AC-15 5TR binder from Trumbull Asphalt and AC-15P from Koch Materials in Saginaw. The lowest initial adsorption levels were for AC-10 from Alon USA and AC-15P sampled from one of Odessa district projects.
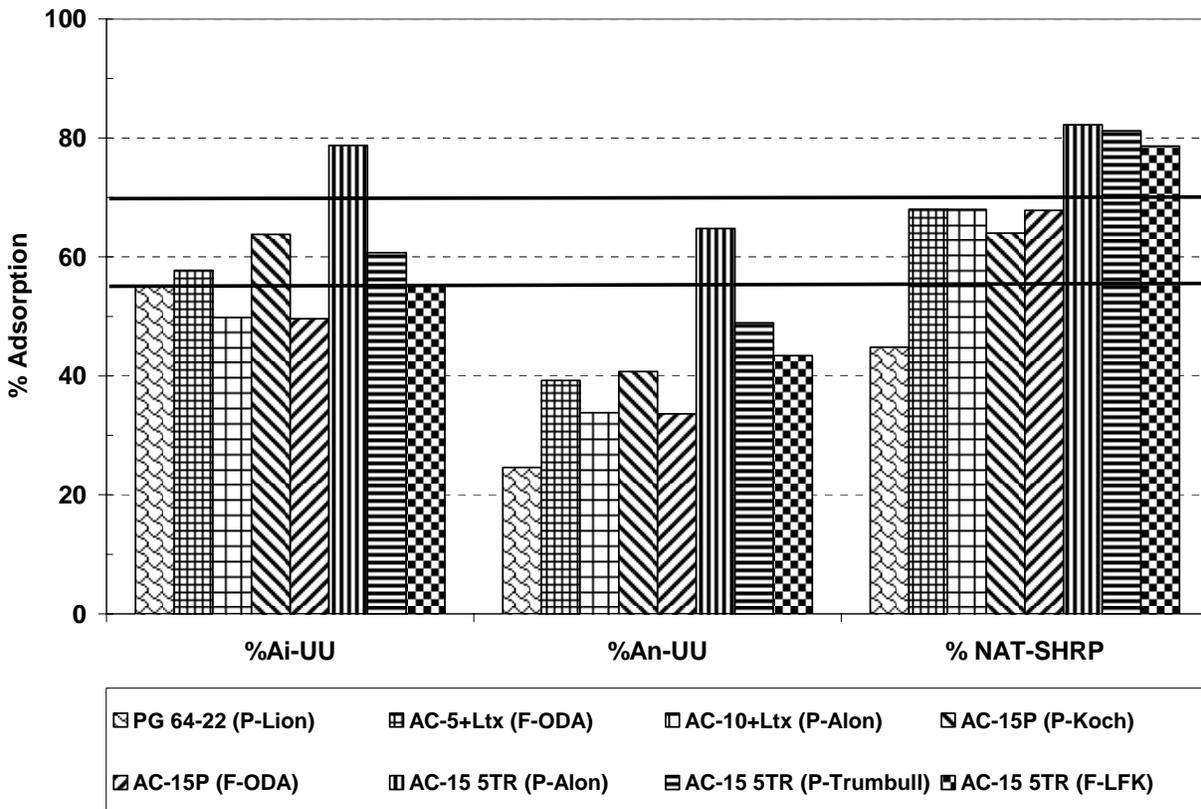
Figure 4-3 Adhesion and Stripping Potential between Limestone and Seal Coat Binders

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Figure 4-3 also seems to indicate that for the limestone aggregate, the asphalt cements sampled directly from the plants performed better than those sampled from the field projects for the same binder grade. Although the initial adhesion of PG 64-22 on limestone was good, the percent net adsorption showed a 'poor' rating, and of the material combinations tested, it was the only material combination to have that rating. This behavior can be predicted on the basis that acidic material (e.g. carboxylic acids) strongly adsorbs onto some aggregate surfaces like limestone and can react with water to form carboxylate salts and free carboxylic acid. Plancher et al. (1977) found that at the asphalt-aggregate interface, there were higher concentrations of free carboxylic acid molecules than carboxylate salts for limestone when compared with other aggregates.

4.3.1.2 Expanded Shale Lightweight Aggregate

Several TxDOT districts use the expanded shale artificial lightweight aggregate for seal coat work. The Texas Industries (TXI) plant, which is the only source for this aggregate in the state, is located in Streetman, Texas south of Dallas along IH-45. This aggregate has excellent polish resistance, low specific gravity and high porosity. It is used both in uncoated and precoated forms. Uncoated lightweight aggregate is commonly used with emulsified asphalts, particularly CRS-2P. The Net adsorption testing protocol does not allow the testing of precoated aggregates. Therefore, only the uncoated lightweight aggregate was tested with four different plant binders and one field sampled binder, as shown in Table 4-6. The results are illustrated in Figure 4-4.

Table 4-6 Adhesion and Stripping Potential of Lightweight Aggregate with Seal Coat Binders

Asphalt Binder Grade(Source)[*]	SHRP			University of Ulster (UU) Method		
	A_i	A_n	%A_n	A_{max} VC $(A_1-0)/(WA_1)$	%A_i $(A_i/A_{max})*100$	%A_n $(A_n/A_{max})*100$
PG 64-22 (P-Lion Oil)	1.51	0.89	59.10	2.80	53.79	31.79
AC-15 5TR (P-Alon USA)	1.46	0.95	65.55	2.80	52.15	34.00
AC-5+2%Latex (F-Eagle)	1.71	1.38	80.87	2.80	61.00	49.33
AC-10+2%Latex (P-Alon USA)	1.22	0.84	68.68	2.80	43.64	29.85
AC-15P (P-Koch)	1.98	1.29	65.58	2.80	70.70	46.21

[*]F – Field Asphalt, P- Plant Asphalt

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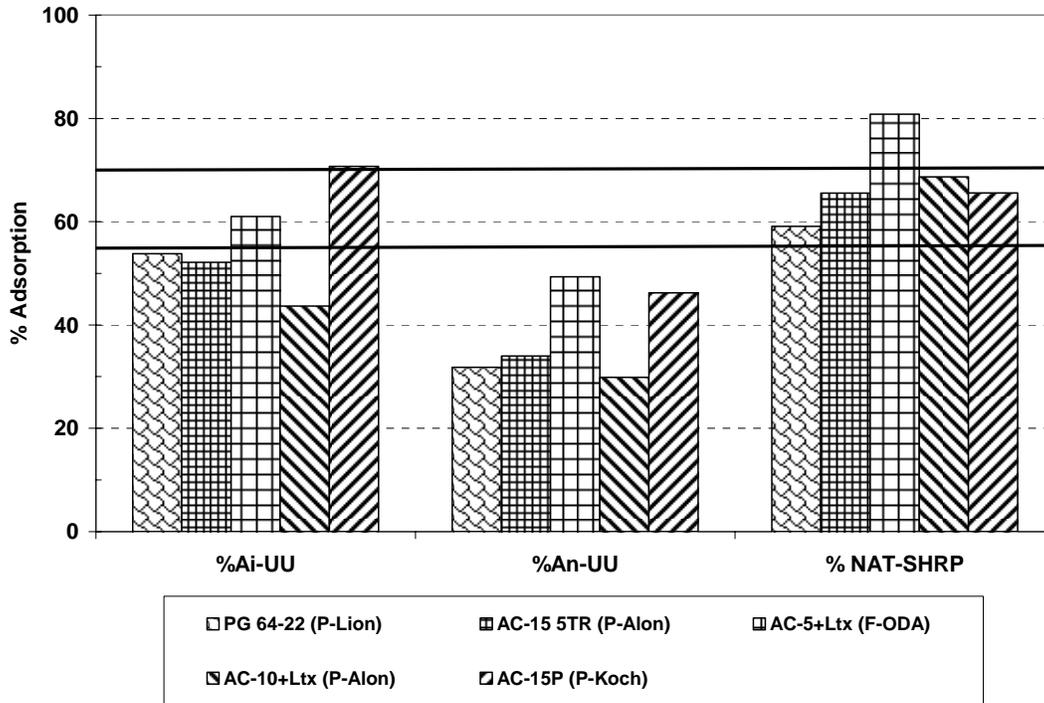
Figure 4-4 Adhesion and Stripping Potential between Lightweight Aggregate and Seal Coat Binders

According to Figure 4-4, the initial adhesion of binders on lightweight aggregate was generally lower than for limestone aggregate from Vulcan Materials Brownwood pit. The highest initial adsorption (70%) was achieved with AC-15P from Koch Materials Saginaw plant. The only other binder that provided an initial adsorption of more than 60 percent was AC-5 (with latex), sampled from an Odessa district project. The lowest initial adsorption, at a little over 40%, was with AC-10 (with latex). As for the effectiveness of adhesion against stripping, the AC-5 (with latex) provided good stripping resistance as per SHRP guidelines. All other binders tested showed "marginal' stripping resistance. It was interesting to note that the AC-15P binder that was adsorbed the most initially did not perform as well under stripping, and showed a % net adsorption of 66%.

4.3.1.3 Rhyolite

Rhyolite is a siliceous gravel that is sold by Trans-Pecos Materials near Pecos, Texas, and it is used as a seal coat aggregate by several west Texas districts that are in close proximity to the aggregate pit. This aggregate has the physical characteristics of high surface area and very low porosity. The results of modified NAT for rhyolite aggregate bonding with different seal coat binders are presented in Table 4-7 and Figure 4-5. As with previous tables and charts in this section, the percent initial and percent final adsorption calculated using the 'UU' procedure and the percent net adsorption calculated using the SHRP procedure are presented.

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Table 4-7 Adhesion and Stripping Potential of Rhyolite with Seal Coat Binders

Asphalt Binder Grade(Source)[*]	SHRP			University of Ulster Method		
	A_i	A_n	$\%A_n$	A_{max} VC $(A_1\text{-}0)/(WA_1)$	$\%A_i$ $(A_i/A_{max})*100$	$\%A_n$ $(A_n/A_{max})*100$
PG 64-22 (P-Lion Oil)	1.52	1.03	67.71	2.8	54.16	36.81
AC 15-5TR (P-Alon USA)	1.79	1.34	75.14	2.8	63.78	47.95
AC 5+2%L (P-Alon USA)	1.73	1.00	57.67	2.8	61.92	35.71
AC 5+2%L (F- Eagle)	1.86	1.56	84.17	2.8	66.32	55.82
AC 15P (P- Koch)	1.50	1.13	75.39	2.8	53.74	40.48
AC 10+2%L (P-Alon USA)	1.22	1.03	84.15	2.8	43.69	36.77
AC 10+2%L (F-Eagle)	1.48	1.27	85.97	2.8	52.88	45.53

[*]F – Field Asphalt, P- Plant Asphalt

As shown in Figure 4-5, the percent initial adsorption of the tested asphalt cements on rhyolite aggregate were all below 70 percent. Three binder sources provided initial adsorptions in excess of 60 percent (AC-15 5TR from Alon USA plant, AC-5 with latex from the Alon USA plant and AC-5 with latex obtained from a field project in Odessa district). The AC-10 (with latex) binder showed the lowest initial adsorption with the rhyolite aggregate at 44 percent. As for the stripping potential, percent net adsorption of rhyolite with five asphalt cements (AC-15 5TR from Alon USA plant, AC-5 with latex from the Odessa district field sample, AC-15P from Koch Materials Saginaw plant, AC-10 with latex from Alon USA plant and the AC-10 with latex field sample from Lubbock district) showed 'good' stripping resistance with rhyolite aggregate. The PG 64-22 precoating binder and the AC-5 with latex binder from the Alon USA plant showed 'marginal' stripping resistance. In summary, it can be said that the rhyolite aggregate has moderate affinity for asphalt, and once bonded, all binders showed at least 'marginal' stripping resistance.

3.4.1.4 Sandstone

Sandstone rocks are sedimentary and can be both acidic and basic in nature. They have the physical characteristics of rough surface texture and moderate porosity. The sandstone for this research was sampled from the Martin Marietta Materials pit in Sawyer, Oklahoma. This is a popular source for seal coat aggregate in the north and northeast Texas districts. It was tested with four different plant sampled asphalts and one field sampled asphalt. The results are shown in Table 4-8 and Figure 4-6.

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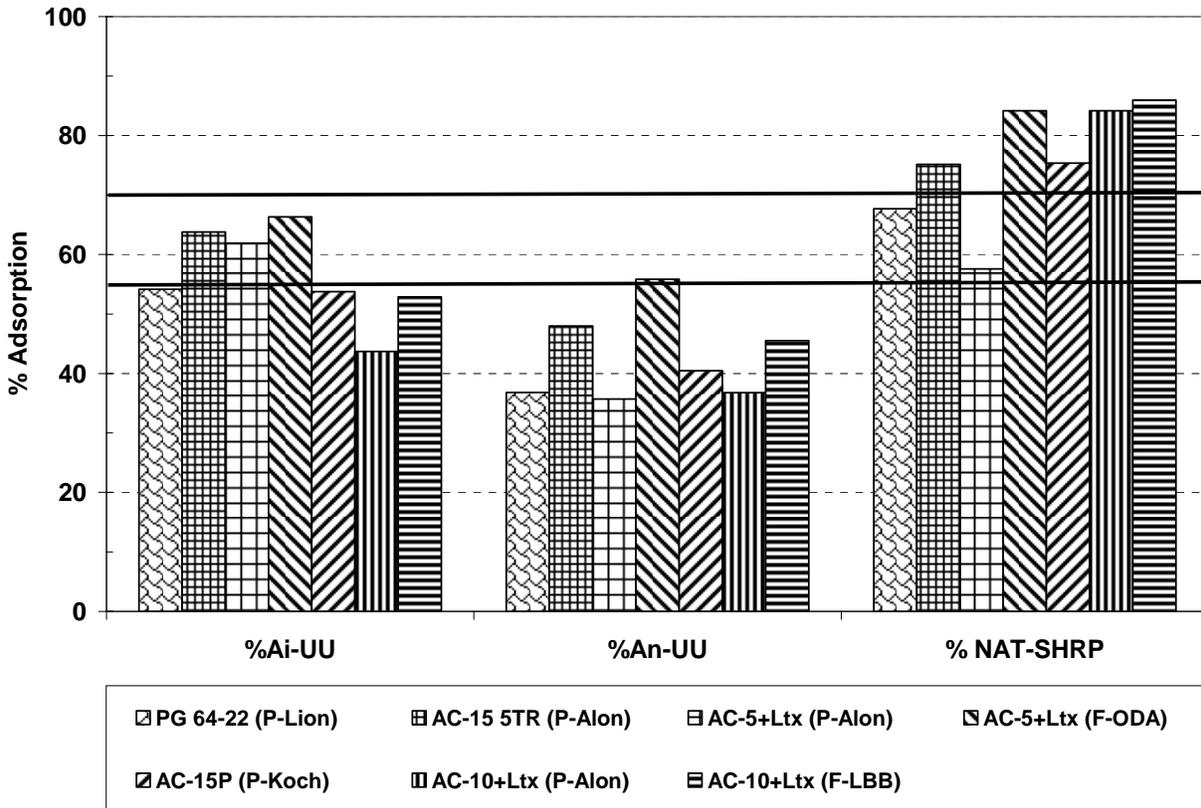
Figure 4-5 Adhesion and Stripping Potential between Rhyolite and Seal Coat Binders

Table 4-8 Adhesion and Stripping Potential of Sandstone with Seal Coat Binders

Asphalt Binder Grade(Source)[*]	SHRP			University of Ulster Method		
	A_i	A_n	$\%A_n$	A_{max} VC $(A_1\text{-}0)/(WA1)$	$\%A_i$ $(A_i/A_{max})*100$	$\%A_n$ $(A_n/A_{max})*100$
PG 64-22 (P-Lion Oil)	1.46	1.13	77.35	2.8	52.25	40.4
AC 15-5TR (P-Alon USA)	1.59	1.17	73.8	2.8	56.66	41.82
AC 5+2%L (F- Eagle)	1.47	1.18	80.68	2.8	52.37	42.25
AC 10+2%L (P-Alon USA)	1.15	0.83	72.06	2.8	40.89	29.46
AC 15P (P- Koch)	1.37	0.98	71.47	2.8	48.82	34.88

[*] F – Field Asphalt, P- Plant Asphalt

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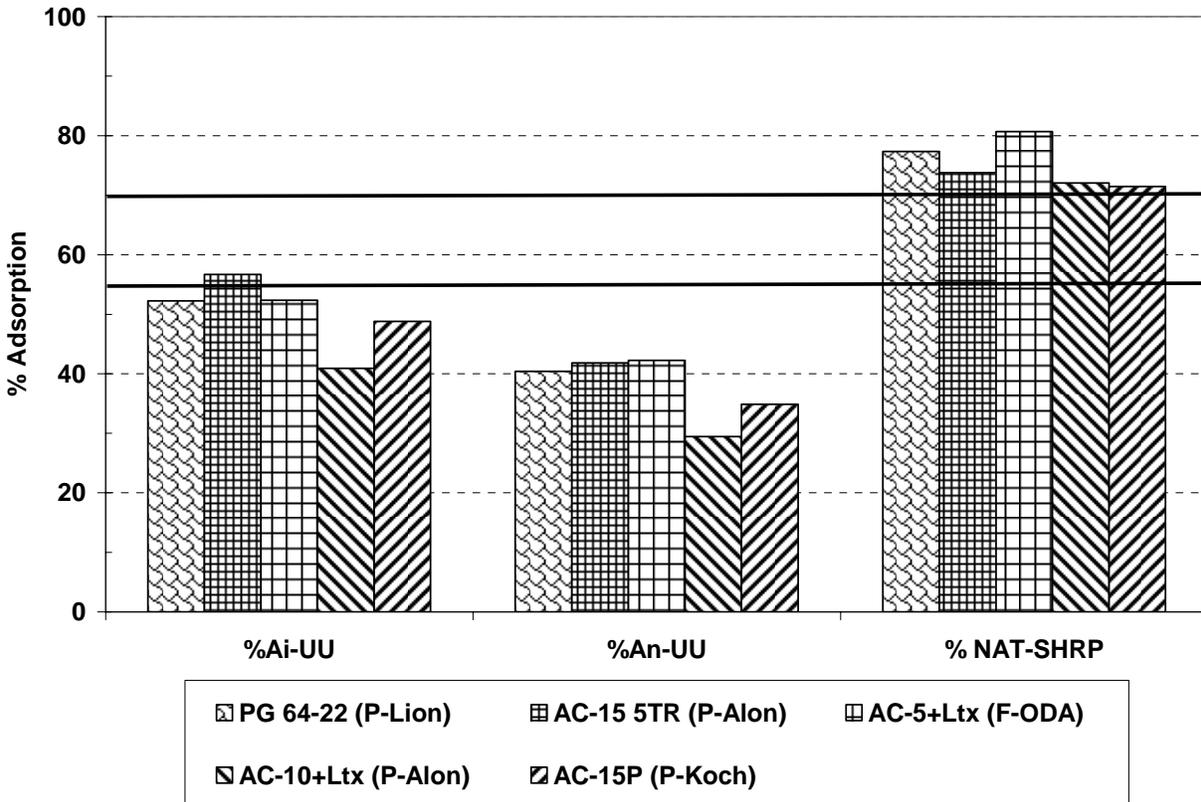
Figure 4-6 Adhesion and Stripping Potential between Sandstone and Seal Coat Binders

Figure 4-6 shows that initial adhesion of asphalt cements with sandstone aggregate was in general not as good as for limestone, lightweight aggregate and rhyolite. Three binders provided over 50 percent initial adsorption (PG 64-22, AC-15 5TR from Alon OSA plant and AC-5 with latex from the field project in Odessa district). The asphalt cement AC-10 (with latex) from the Alon USA plant showed the lowest initial adsorption with sandstone aggregate. With regard to the percent net adsorption from the SHRP testing protocol, sandstone displayed good resistance to stripping with all the binders. The sandstone aggregate is considered to have neutral (both acidic and basic) characteristics, and therefore its performance with respect to initial and final adhesion with almost all asphalts it was tested with, produced similar results.

4.3.1.5 Siliceous River Gravel

Siliceous gravel aggregate is known for its hardness, smooth surface texture and very low porosity. For this testing program, it was tested with four different plant sampled seal coat asphalt cements along with one field sample collected from a seal coat construction project. The results are summarized and illustrated in Table 4-9 and Figure 4-7 respectively.

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Table 4-9 Adhesion and Stripping Potential of Siliceous Gravel with Seal Coat Binders

Asphalt Binder Grade(Source)*	SHRP			University of Ulster Method		
	A_i	A_n	$\%A_n$	A_{max} VC $(A_1-0)/(WA1)$	$\%A_i$ $(A_i/A_{max})*100$	$\%A_n$ $(A_n/A_{max})*100$
PG 64-22 (P-LionOil)	1.25	0.96	76.76	2.8	44.70	34.28
AC 15-5TR (P-AlonUSA)	1.15	0.99	85.68	2.8	41.07	35.22
AC 5+2%L (F- Eagle)	1.47	1.13	77.03	2.8	52.59	40.51
AC 10+2%L (P-AlonUSA)	1.12	0.72	63.93	2.8	40.1	25.62
AC 15P (P- Koch)	1.12	0.71	63.06	2.8	40.08	25.27

*F – Field Asphalt, P- Plant Asphalt
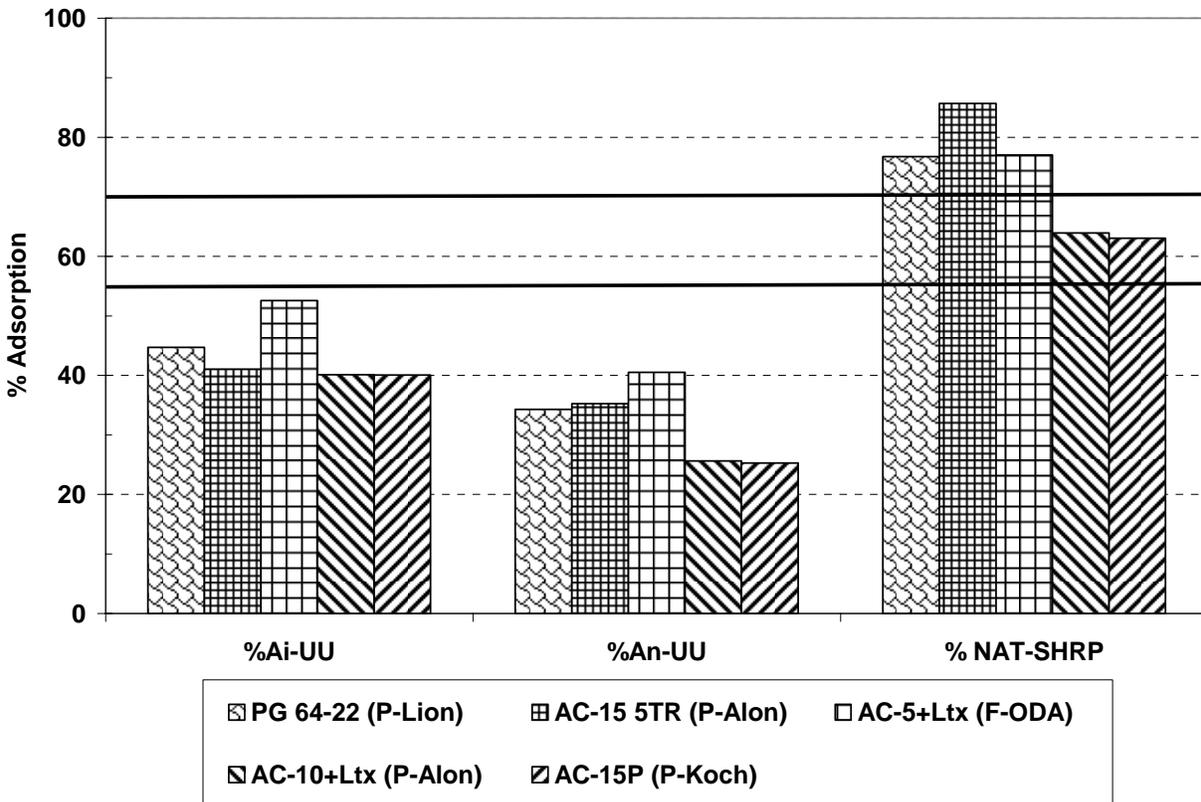
Figure 4-7 Adhesion and Stripping Potential between Siliceous Aggregate and Seal Coat Binders

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Figure 4-7 shows that siliceous gravel aggregate has the lowest initial adsorption values of all aggregates tested. This indicates that the siliceous gravel is not effectively wetted by the asphalt cements. The percent initial adsorption was at least 40 percent for all binders, but only one binder showed an initial adsorption in excess of 50 percent (i.e. AC-5 with latex from the Odessa district field sample). It was interesting to note that from a stripping resistance standpoint, three binders scored high marks with a 'good' rating' (PG 64-22, AC-15 5TR from Alon USA plant, and AC-5 with latex from the Odessa field sample). The other two binders (AC-10 with latex and AC-15P) showed marginal stripping resistance with the siliceous gravel aggregate tested. The siliceous gravel did not attract much asphalt onto its surface initially, but it held onto the one that it attracted fairly well. This was a somewhat unexpected finding. In general, siliceous gravel is considered hydrophilic because of the negative surface charges on the aggregate surface. The acidic nature of this aggregate causes it to have a higher affinity to form hydrogen bonding with water. Studies show that very few basic ingredients are found in naturally occurring asphalts and hence hinder the formation of long-lasting bonds at the polar sites. However, these results go to show that a good testing protocol that can evaluate both the affinity of an aggregate to asphalt as well as its stripping potential is necessary.

Tables 4-10 and 4-11 summarize the percent initial adsorption (% Ai) and the percent net adsorption (%A_n) values, sorted from highest to lowest for all the aggregate-binder combinations tested. A closer look at Table 4-10 shows some interesting and somewhat expected results. For percent initial adsorption which shows the affinity of an asphalt for an aggregate, the values for each aggregate type when averaged for the 5 binder sources tested shows 61, 56, 56, 50 and 44 percent for limestone, rhyolite, lightweight, sandstone and siliceous gravel respectively. This appears to show expected trends based on the acid-base interactions between aggregate and binder as well as the acidity (or basicity) of the aggregate. When the percent initial adsorption results are averaged by the binder grade for the 5 aggregate types tested, the results show 58, 58, 55, 52 and 44 percent for AC-15 5TR, AC-5 (with latex), AC-15P, PG 64-22 and AC-10 (with latex) respectively. Statewide performance of respective binder grades in seal coats appears to be in agreement with these results.

It is important to note here that the validity of these results is limited to the material samples tested and the specific test conditions. The high variability of aggregate types and binder grades from one source to the next calls for testing of specific aggregate-binder combinations a district plans to use.

Table 4-11 shows the percent net adsorption values calculated using the SHRP protocol, for all the aggregate-binder combinations tested. The bold lines demarcate the SHRP classification limits for good, marginal and poor performance of aggregate-binder bond for stripping potential (see Table 4-4). It can be seen that approximately 60 percent of all combinations tested were rated as having 'good' stripping resistance. Only one aggregate-binder combination (Limestone and PG 64-22) was rated as having 'poor' stripping resistance.

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Table 4-10 Percent Initial Adsorption Values for All Aggregate-Binder Combinations

Aggregate Type	Binder Grade/Type	% Initial Adsorption (%Ai-UU)
Limestone	AC-15 5TR (P-Alon)	79
Lightweight	AC-15P (P-Koch)	71
Rhyolite	AC-5+Ltx (F-ODA)	66
Rhyolite	AC-15 5TR (P-Alon)	64
Limestone	AC-15P (P-Koch)	64
Rhyolite	AC-5+Ltx (P-Alon)	62
Lightweight	AC-5+Ltx (F-ODA)	61
Limestone	AC-15 5TR (P-Trumbull)	61
Limestone	AC-5+Ltx (F-ODA)	58
Sandstone	AC-15 5TR (P-Alon)	57
Limestone	PG 64-22 (P-Lion)	55
Limestone	AC-15 5TR (F-LFK)	55
Rhyolite	PG 64-22 (P-Lion)	54
Lightweight	PG 64-22 (P-Lion)	54
Rhyolite	AC-15P (P-Koch)	54
Rhyolite	AC-10+Ltx (F-LBB)	53
Siliceous Gravel	AC-5+Ltx (F-ODA)	53
Sandstone	AC-5+Ltx (F-ODA)	52
Sandstone	PG 64-22 (P-Lion)	52
Lightweight	AC-15 5TR (P-Alon)	52
Limestone	AC-10+Ltx (P-Alon)	50
Limestone	AC-15P (F-ODA)	50
Sandstone	AC-15P (P-Koch)	49
Siliceous Gravel	PG 64-22 (P-Lion)	45
Rhyolite	AC-10+Ltx (P-Alon)	44
Lightweight	AC-10+Ltx (P-Alon)	44
Siliceous Gravel	AC-15 5TR (P-Alon)	41
Sandstone	AC-10+Ltx (P-Alon)	41
Siliceous Gravel	AC-10+Ltx (P-Alon)	40
Siliceous Gravel	AC-15P (P-Koch)	40

When the percent net adsorption values are averaged for the five aggregate types tested, the values were 77, 75, 73, 68 and 65 percent for rhyolite, sandstone, siliceous gravel, lightweight aggregate and limestone respectively. These results are somewhat unexpected, but when one considers the complex chemical and physical nature of the aggregate and binder involved, it has to be taken seriously. One possible reason for the unexpectedly low value for limestone is the low percent net adsorption value for the Limestone-PG 64-22 combination which had 45 percent net adsorption. All the other NAT values for limestone were in the 64 to 82 percent range. Due to this unexpectedness of the result, this combination was tested 3 times (with 3 replicates) instead of the usual one.

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Table 4-11 Percent Net Adsorption Values for All Aggregate-Binder Combinations

Aggregate Type	Binder Grade/Type	% Net Adsorption (% NAT-SHRP)
Rhyolite	AC-10+Ltx (F-LBB)	86
Siliceous Gravel	AC-15 5TR (P-Alon)	86
Rhyolite	AC-5+Ltx (F-ODA)	84
Rhyolite	AC-10+Ltx (P-Alon)	84
Limestone	AC-15 5TR (P-Alon)	82
Limestone	AC-15 5TR (P-Trumbull)	81
Lightweight	AC-5+Ltx (F-ODA)	81
Sandstone	AC-5+Ltx (F-ODA)	81
Limestone	AC-15 5TR (F-LFK)	79
Sandstone	PG 64-22 (P-Lion)	77
Siliceous Gravel	AC-5+Ltx (F-ODA)	77
Siliceous Gravel	PG 64-22 (P-Lion)	77
Rhyolite	AC-15P (P-Koch)	75
Rhyolite	AC-15 5TR (P-Alon)	75
Sandstone	AC-15 5TR (P-Alon)	74
Sandstone	AC-10+Ltx (P-Alon)	72
Sandstone	AC-15P (P-Koch)	71
Lightweight	AC-10+Ltx (P-Alon)	69
Limestone	AC-5+Ltx (F-ODA)	68
Limestone	AC-10+Ltx (P-Alon)	68
Limestone	AC-15P (F-ODA)	68
Rhyolite	PG 64-22 (P-Lion)	68
Lightweight	AC-15P (P-Koch)	66
Lightweight	AC-15 5TR (P-Alon)	66
Limestone	AC-15P (P-Koch)	64
Siliceous Gravel	AC-10+Ltx (P-Alon)	64
Siliceous Gravel	AC-15P (P-Koch)	63
Lightweight	PG 64-22 (P-Lion)	59
Rhyolite	AC-5+Ltx (P-Alon)	58
Limestone	PG 64-22 (P-Lion)	45

The percent net adsorption values when averaged by the binder grade were 78, 76, 71, 68 and 65 percent for AC-5 (with latex), AC-15 5TR, AC-10 (with latex), AC-15P and PG 64-22 respectively. It is difficult to make any judgments on these results due to the high variability in performance for the same binder grade from different sources.

It is important to note that in terms of the performance of aggregate-binder bond, either the percent initial adsorption or the percent net adsorption alone cannot provide an accurate prediction of the performance of the material combination. However, if these two parameters are combined, a reasonable assessment of the effectiveness of the bond can be made. The researchers combined the values for these two test parameters of both the aggregate type and the

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binder grade, and the following rankings ensued.

- For the five aggregate types, the rankings (from best to worst) were Rhyolite, Limestone, Lightweight aggregate, Sandstone and Siliceous Gravel.
- For the five binder grades, the rankings (from best to worst) were AC-5 (with latex), AC-15 5TR, AC-15P, PG 64-22 and AC-10 (with latex)

These rankings appear to follow the general pattern of field performance observed for seal coats around the state. It is important that these rankings *not* be generalized to evaluate the effectiveness of specific material combinations involving one specific aggregate source with one specific binder source. For such an evaluation, this test can be conducted to evaluate the specific material combination involved. Furthermore, this limited list of rankings can be used to explore ways to improve the performance of seal coats that use locally available materials, particularly the aggregates. One example of such improvements would be the use of antistripping agents in seal coats.

CHAPTER 5

PERFORMANCE-BASED SEAL COAT
AGGREGATE-BINDER COMPATIBILITY TEST

5.1   Development of 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.

A performance-based testing protocol to determine the effectiveness of bond between seal coat aggregate and binder must have the following components. It is a challenging task to incorporate all these characteristics into a test procedure. It is even more daunting when the testing protocol has to be very inexpensive in line with the funds available for seal coat applications. The researchers looked at several test methods that have been developed in the past, but quickly determined that they do not incorporate the performance conditions of seal coats. This research team focused on ways in which equipment already available in a typical TxDOT district lab could be incorporated to this new testing protocol, therefore leveraging the investments TxDOT has already made to their material test program. A list of pertinent test conditions to be incorporated in the new testing protocol is given below:

- Use of different aggregate types and grades
- Use of different binder types and grades
- Use of precoated of aggregate
- Simulation of aggregate conditions at the point of use
- Simulation of binder application, aggregate application and rolling
- Simulation of different construction conditions
    - Pavement temperature
    - Binder application temperature
    - Time delay between binder spray and aggregate spread
    - Time delay between binder application and rolling
- Simulation of pavement exposure conditions including freeze-thaw cycles and wetting-drying cycles
- A mechanical device to check the effectiveness of aggregate-binder bond

A number of preliminary tests were carried out to check their feasibility to incorporate the test conditions outlined above. After several months of such preliminary tests, the following framework for the testing protocol was identified.

- Use of a 'dry' micro-deval test run to simulate the dust generation potential of aggregate when it is handled in the field using front-end loader
- Prepare seal coat samples in the laboratory using 6 inch square metal plates

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- The specimens are to be prepared on a hot plate that will allow the simulation of different pavement temperatures at the time of seal coat application
- The aggregate application on the steel plate has to simulate the aggregate spreader
- Once prepared, seal coat specimens to be rolled using a small tennis court roller covered with a stiff rubber padding to simulate the effect of pneumatic roller
- Use a combination of freeze-thaw and wet-dry cycles to simulate the field conditions to which a seal coat is subjected to

With this basic testing framework in mind, the researchers set about the goal of perfecting the procedural details of the testing protocol. This was an exciting endeavor because a comprehensive performance-based test method incorporating performance conditions is not available at this time.

5.1.1   Specimen Preparation

Aluminum plates similar to the steel plates used to collect field specimens were prepared for laboratory preparation of seal coat specimens. In order to avoid spilling of asphalt binder, the plates were dammed along its perimeter using thick tape. The binder was first heated to the desired temperature and poured to the sample plate placed on top of a weighing scale. This allowed the placing of desired binder weight. The amount of binder needed to achieve the desired asphalt spray rate was calculated using the following procedure.

For a binder rate of 0.40 gal/yd^2 and plate dimensions of 6 in. x 6 in.,

Binder thickness on the plate =	0.40 gal/yd^2 x 0.4527	=	0.1811 cm
The volume of the binder =	15 cm x 15 cm x 0.1811 cm =		40.75 cm^3

For an asphalt specific gravity of 1.02,
The mass of asphalt needed   =   40.75 cm^3 x 1.02 x 1 g/cm^3 =   41.56 g

Therefore, in order to achieve a binder rate of 0.40 gal/yd^2 on a 6 inch square plate, 41.56 grams of binder is needed.

The constructability review showed that in the construction process, the binder temperatures 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 ¼ in thick rubber sheet (Figure 5-1) that simulated the rolling of a lightweight roller.

5.1.2   Impact Testing

A certain amount of mechanical energy is needed to initiate debonding failure between aggregate and binder in the plate specimens. Instead of using a steel ball to impart this mechanical energy on the specimen plate as the Vialet test does, a Modified Proctor Hammer (MPH) was selected for this purpose (Figure 5-2). The MPH was selected because it is readily

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available in district laboratories. The hammer has an 18 in. (45 cm) vertical drop of a 10 lb (4.5 kg) weight that could be dropped on the inverted specimen plate. Preliminary trials indicated that it is best to apply mechanical energy on the specimens at low temperatures to achieve a satisfactory level of reproducibility of data. Tests indicated that at such temperatures, the energy provided by the MPH directly on the specimen plate may cause failure between the specimen plate and the binder instead of at the aggregate-binder interface as desired, particularly when the specimens are tested at lower temperatures and when stiffer binders are tested. Therefore, the inverted specimen was placed on a rubber padding strip glued to the steel frame along its perimeter (see Figure 5-3) and a ¼ in. thick steel plate was placed on the top of the inverted specimen to evenly distribute the impact energy over the specimen plate and reduce its likelihood of bending under the load. In order to provide even support between the inverted specimen plate and the steel frame, the aggregates were placed at least ¼ in far from each side of the steel plate.

Figure 5-1 Specimen Compaction Using the Laboratory Roller

Figure 5-2 Impact Test Apparatus

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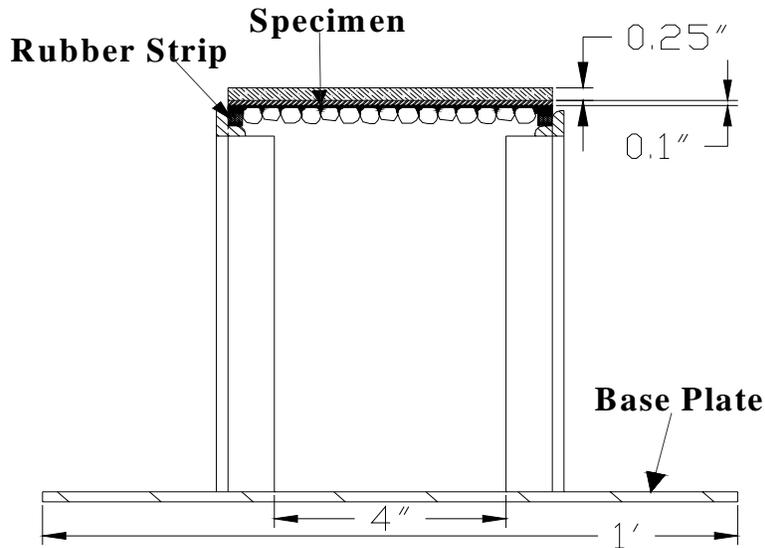
Figure 5-3 Schematic of Steel Frame with Seal Coat Specimen

The MPH was 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 number of aggregate particles in the original specimen prior to the test.

5.1.3   Effect of Specimen Conditioning

After the specimens were prepared, they were subjected to various conditioning regimens to select and simulate the field conditions. Several freeze-thaw-soak and boil-soak protocols were tried out. It was found that successive freeze-soak-thaw cycles and extended soaking were the most effective ways to identify weak debonding at the aggregate-binder interface.

First, the effect of soaking was investigated. Table 5-1 shows the data indicating the effect of different soaking durations. Identical specimens were tested after 0, 24, 48, 72, 96, and 120 hours of soaking at 25 °C (77 °F). In this series of tests, the specimens were tested on the impact test apparatus at the end of the soaking period. However, if the soaking period was less than 120 hours, the specimens were tested after each 24 hour intervals, and continually soaked until 120 hours. For example, specimen #3 was first tested after 48 hours of soaking, and the number of aggregates lost due to debonding after 48 hours of soaking was counted and used to calculate the 'percent initial loss'. After testing at 48 hours, the specimen was returned to the soaking bath and tested at 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, the total number of aggregate particles 'lost' was obtained and divided by the initial number of aggregates in the specimen prior to conditioning. This ratio was termed 'percent cumulative loss'. For specimen #6, the initial loss was equal to cumulative loss because it was tested just once (after 120 hours of soaking).

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Table 5-1 Effect of Soaking Duration on Aggregate Loss

Specimen	# of Aggregates in Specimen Before Impact Test	# of Aggregates Lost at						Total # of Aggregates Lost	% Initial Loss	% Cumulative Loss
		0 hrs	24 hrs	48 hrs	72 hrs	96 hrs	120 hrs			
1	51	**1**	6	8	4	3	5	27	2.0	52.9
2	54		**3**	11	4	6	3	27	5.6	50.0
3	55			**3**	6	2	5	16	5.5	29.1
4	54				**2**	5	8	15	3.7	27.8
5	50					**5**	16	21	10.0	42.0
6	52						**23**	23	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 soaking duration and the number of times the specimens were 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 a soaking period of 120 hours could give a reasonable assessment of aggregate loss for different aggregate-binder combinations. The objective of these trials was not to pick the conditioning regimen that provided the highest percent aggregate loss, but rather to pick a regimen that is sensitive to the extremes of 'good' and 'bad' aggregate-binder combinations.

In this series of tests, only one replicate was tested for each condition. However, at this preliminary feasibility phase of testing, they provided sufficient evidence to conclude that soaking and its duration has an effect on aggregate retention. Moreover, the impact loading also affects the percent aggregate by initiating the debonding at the aggregate-binder interface as projected.

After the soaking feasibility study outlined above was completed, other conditioning methods including wetting/drying, freezing/soaking and boiling were tried to simulate field conditions in the lab. Different durations of conditioning such as 16 hours freezing+8 hours soaking, and 2 hours of boiling were also used. Based on these trials, 16 hours of soaking followed by 8 hours of freezing was found to be the most effective.

Freeze-soak cycles start with the soaking of seal coat specimen under water for 16 hours at room temperature 25° C (77° F), followed by freezing at -25 °C (-13 °F) for 8 hours. This 24-hour period formed one 'cycle' of conditioning. The durations of 16 hours and 8 hours were chosen due to its effectiveness as well as the practicality of conducting the test during a typical working day.

When the freeze-soak process is followed by soaking, it showed significant sensitivity on the 'aggregates lost' for various aggregate-binder combinations. Table 5-2 shows the results for specimens tested under different freeze-thaw and soaking conditions. Four identical specimens were prepared using the AC-5 (with 2% latex) binder and a fifth specimen was prepared using the AC-15 5TR binder. All five specimens had the same aggregate. As shown in Table 5-2, four

different conditioning regimens were applied to these specimens for different combinations of number of freeze-thaw cycles and the soaking duration.

Table 5-2 Effect of Freeze-Thaw Cycles and Soaking Duration

	Conditioning Regimen				
	1 F-T Cycle + 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 Temperature at Aggregate Spreading (°F)	125.0	125.0	125.0	125.0	125.0
Binder Temperature at Rolling (°F)	105-110	105-110	105-110	105-110	105-110
% Aggregate Loss	26.8	59.5	76.4	88.5	50.0

These conditioning regimens 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 applied to the two different binders tested. At the end of each conditioning regimen, the specimens were subjected to three impact loadings from the MPH. The 'percent aggregate loss' was calculated for the five cases using the number of aggregates as the basis. The results showed percent aggregate loss in the range of 26.8 to 88.5, and the 3 F-T cycles + 48 hours of soaking resulting in the highest percent aggregate loss. As it was indicated previously, the objective of these trials was not to pick the conditioning regimen that provided the highest percent aggregate loss, but rather to pick a regimen that is sensitive to different levels of field performance. Based on the testing trials, a testing protocol of 3 F-T cycles + 48 hours of soaking followed by impact loading with MPH was selected to proceed to the validation phase of the testing protocol.

5.1.4 Effect of Aggregate Type

At this phase of the study, specimens using the same binder (AC-5 + 2% Latex), but with four commonly used seal coat aggregate types were subjected to the conditioning protocol selected above (3 freeze-thaw cycles followed by 48 hours of soaking and impact loading). Results of these tests are shown in Table 5-3. Similar to the previous series of trials, in order to isolate one factor, i.e. aggregate type, other factors such as binder type, binder rate, binder temperature at the time of rolling and spreading the aggregate were kept the same. It can be seen from Table 5-3 that the testing protocol is responsive to different types of aggregate. The

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lightweight aggregate performed the best (22.7% loss) compared to siliceous gravel aggregate which lost approximately 3.5 times more aggregate.

Table 5-3 Sensitivity of Testing Protocol to Aggregate Type

	Aggregate Type			
	Lightweight	Limestone	Rhyolite	Siliceous River Gravel
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 Temperature when Aggregate is Spread (°F)	125.0	125.0	125.0	125.0
Binder Temperature at Rolling (°F)	105-110	105-110	105-110	105-110
% Aggregate Loss	22.7	34.2	45.0	78.3

5.1.5   Verification of Testing Protocol using Field Specimens

In the final phase of test protocol development, specimens brought from field test sections (see Chapter 3) were tested. Since the surface of the seal coat specimens brought from site visits were covered with aggregate over the full 36 square inches of surface area, the aggregate particles along the outer ¼ inches wide perimeter were removed by hand to ensure proper seating of the specimen on the test frame.

Three replicate specimens from 15 test projects were tested and the results are shown in Table 5-4. Specimens from 9 test sections had 0% aggregate loss. Abilene test section lost 11.82% of its aggregate, thus confirming its aggregate-binder compatibility problem observed during the field monitoring follow-up study. San Antonio and Lubbock field specimens showed percent losses of 6.56% and 4.28% respectively. All these district sample test data were in agreement with the actual field performance observed during the field monitoring follow-up study presented in Chapter 3. Fort Worth district specimens showed a negligible amount of aggregate loss.

The aggregate-binder incompatibility problem observed in the Lufkin test section could not be verified using this test protocol. However, according to the district personnel, the aggregate loss in that test section was caused by a heavy thunderstorm occurred the day after construction. Since the specimens collected from that test section were brought back to Lubbock the same day and were stored in the laboratory until testing, the specimens did not experience conditions similar to what occurred soon after construction. It is likely that the problem in Lufkin was not cause by an incompatibility of the materials, but was rather caused by adverse conditions immediately after construction. The remaining test sections that showed excellent performance (Odessa, El Paso, Tyler, Ft Worth, Childress, San Angelo, Paris, Pharr, Brownwood I, II, and III) were correctly verified by the test protocol except for Brownwood I and III.

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Table 5-4 Performance of Field Specimens under Proposed Test Protocol

District	% Aggregate Loss			Average
	Replicate Specimen #			
	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 test results for Brownwood III showed 6.34% aggregate loss while Brownwood I showed a loss of 2.85%. Even though the evaluation criteria for the testing protocol have not established at this point, it is likely that an aggregate loss up to 3% may be acceptable 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 may be due to the effect of traffic because both these sections have considerably high traffic levels compared to most of the others. Traffic has a significant effect on seal coat performance. Furthermore, field observations in Brownwood I test section showed flushing and bleeding, which suggests that the aggregate on the road is highly embedded into the binder.

When the data from Tables 5-2, 5-3 and 5-4 are compared, it can be observed that laboratory prepared specimens showed higher aggregate losses compared to specimens brought from the field. It should be noted that lab prepared samples were subjected to the conditioning regimen 24 hours after specimen preparation, whereas specimens brought from the field were not subjected to conditions for several months because they were stored in the laboratory waiting for the final testing protocol to be developed. This could be the reason for the difference in performance between lab-prepared and field specimens.

In summary, it could be said that the proposed specimen conditioning protocol of 3 freeze-thaw cycles followed by 24 hours of soaking provided very good agreement between

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performance of field test sections and their corresponding field-obtained specimen test results using the proposed testing protocol.

5.2   Testing Procedure

5.2.1   Specimen Preparation

After the testing protocol was developed and the preliminary testing was completed, an experimental plan was developed to further verify the new testing protocol under aggregate-binder combinations that show different levels of performance in the field. Several experimental factors, identified as causes for incompatibility during the constructability review, were incorporated in this plan. Specimens with different aggregate-binder combinations were prepared and tested using the new testing protocol. The experimental plan had three major components depending on the materials used; hot applied asphalt, asphalt emulsion, precoated aggregate.

Due to the potential corrosion of steel plates during the conditioning phase, it was decided to use aluminum plates of similar size (6 in. square) instead of steel plates. The preparation of specimens was different from that adopted during the test protocol development. It was decided to concurrently prepare 3 replicate specimens for each test condition. 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 done using the thickness of binder film applied. The three replicate specimen plates were lined up to form a rectangle with dimensions of 6 inches by 18 inches to ensure that they are prepared under similar conditions. A 'sweeping' blade, the height of which from the top of the sample plate(s) is adjustable, was placed on two parallel rails located on either side of the three sample plates. The clearance between the top of sample plates and the sweeping 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 form a perfect binder film. Since the blade could slide on the rails, the process was easy, smooth and consistently left a uniform binder coating. Figure 5-4 shows the specimen preparation setup. The sweeping blade can be initially placed at one end of the sample plates. The binder, heated in the oven to the desired temperature (350° F for all hot asphalts and 170° F for emulsified asphalts) is poured in front of the sweeping blade and the blade is slid towards the other end of the three sample plates in one smooth motion, creating a uniform binder film.

Two different surface temperatures, 70 °F and 140 °F, were used to prepare laboratory specimens to mimic cool and hot weather seal coating. For hot asphalt seals, both the above temperatures were used, and for emulsified asphalt seals, the surface temperature was kept at 70 °F. For the testing segment with precoated aggregate, the surface temperature was kept at 140 °F for hot asphalts and 70 °F for emulsions. The surface temperature was varied by placing the specimen plates on a hot plate that was long enough to accommodate all three specimen plates (i.e. 18 inches). The hot plate had a temperature regulator to vary the temperature (Figure 5-5).

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Figure 5-4 Specimen Preparation Setup

Figure 5-5 Hot Plate

5.2.2   Test Factorial

After the binder was poured and spread at the required rate of application, the aggregate was spread on the binder film when the binder temperature dropped to the surface temperature. The binder temperature was monitored using a non-contact infrared thermometer. TxDOT grade 4S aggregate was used for all experiments. The time elapsed after the spreading of the aggregate was one of the experimental factors. At the appropriate time, specimens were taken out of the hot plate, placed on the floor, and rolled for 5 passes using the roller shown in Figure 5-1. The factorials for the testing programs involving asphalt cement, emulsified asphalt and precoated aggregate are presented in Tables 5-5, 5-6 and 5-7 respectively. The experimental factors used

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in these tables are discussed in Section 5.2.3. For each experimental condition, three specimen replicates were used.

Table 5-5 Testing Factorial for Hot Asphalt

Embedment	Roller Delay (min)	Dust Condition					
		No Dust (0-Min. Micro-Deval)		Medium Dust (2 Min. Micro-Deval)		High Dust (5 Min. Micro-Deval)	
		70 °F	140 °F	70 °F	140 °F	70 °F	140 °F
15%	3	X	X	X		X	X
15%	10	X					
35%	3	X	X	X		X	X
35%	10	X		X		X	
50%	3	X	X	X		X	X
50%	10	X					

Table 5-6 Testing Factorial for Emulsified Asphalt

			Aggregate Type		
			Siliceous Gravel	Limestone	Lightweight
Air-Dry	B/Break	3hr cure	X	X	X
		10hr cure	X	X	X
	A/Break	3hr cure	X	X	X
		10hr cure	X	X	X
SSD	B/Break	3hr cure		X	X
		10hr cure		X	X
	A/Break	3hr cure		X	X
		10hr cure		X	X

Table 5-7 Testing Factorial for Precoated Aggregates

	Lightweight Precoated with CSS-1h			Limestone Precoated with PG 64-22		
	No Precoat	50% Precoat	90% Precoat	No Precoat	50% Precoat	90% Precoat
AC-15 5TR	X	X	X	X	X	X
AC-5+2% Latex	X	X	X	X	X	X
CRS-2P	X	X	X	X	X	X

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5.2.3 Experimental Factors

5.2.3.1 Embedment Depth

A seal coat is a surface treatment where a single layer of aggregate is partially covered with the asphalt binder. The height of this partial asphalt coating of the aggregates is often referred to as 'embedment depth'. The ratio between embedment depth and the average least dimension of the aggregate, when expressed as a percentage, is referred to as the 'percent embedment'. The percent embedment typically dictates the mechanical component of the bond.

The primary factors that affect seal coat embedment depth include the aggregate and binder application rates, condition of the existing pavement surface, aggregate grade and the roller compaction efficiency. Figure 5-6 illustrates the interrelationships between these factors in a tree-diagram. 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 in the literature. On the other hand, the fourth factor, 'efficiency of compaction' has often been neglected, and therefore needs to be investigated more than the others.
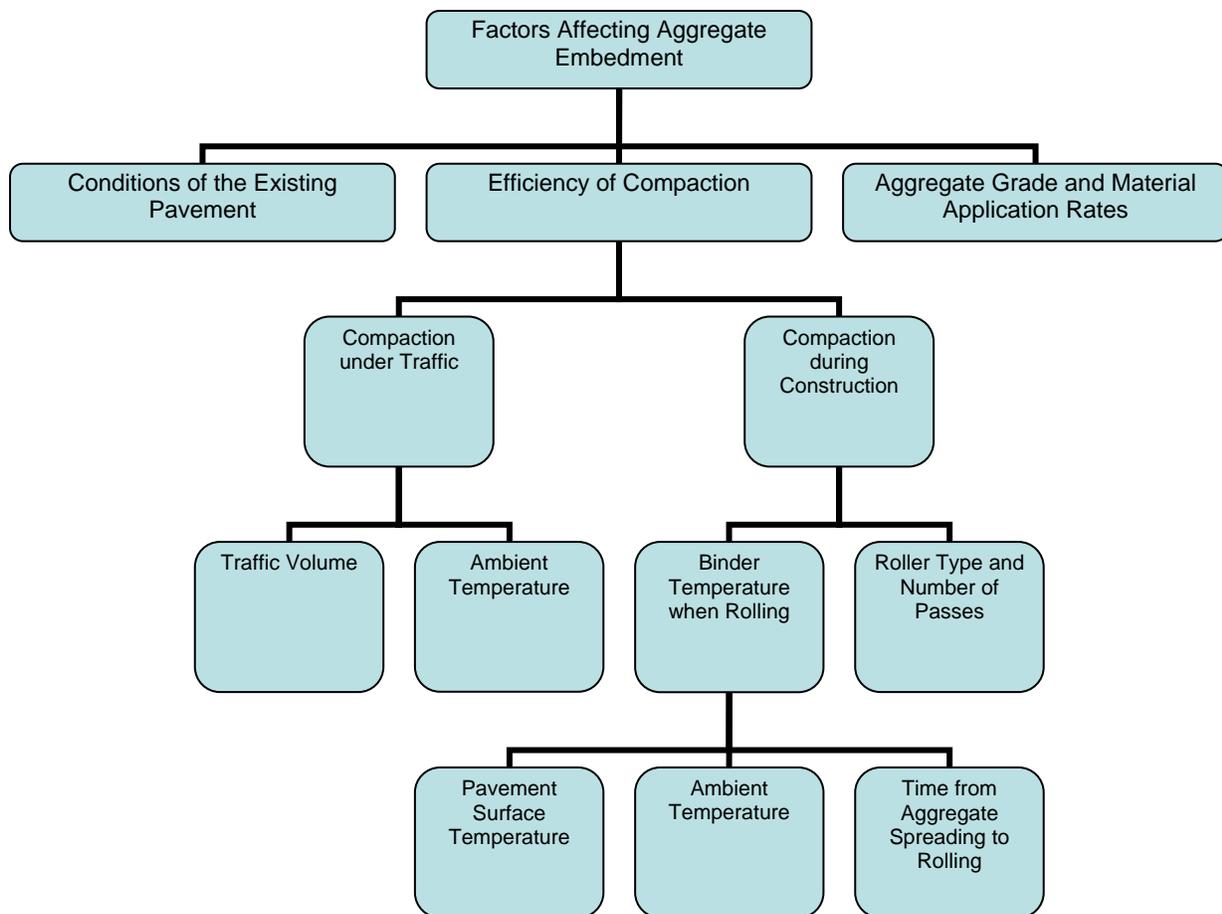
Figure 5-6 Factors Affecting Aggregate Embedment in Seal Coats

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In Figure 5-6, the 'efficiency of compaction' is divided into two parts; compaction under traffic, compaction during construction. It is well known that traffic helps to compact seal coat aggregates. The only situations where traffic may cause loss of aggregate are when braking and turning movements are involved. Furthermore, the constructability review showed that loss of aggregate typically does not occur in the wheel paths. This can be explained by the compaction under traffic, which is accentuated by the ambient temperature and volume of traffic.

On the other hand, compaction during construction is more complex because of the factors affecting it. Equipment-related factors such as roller type (or weight) and the number of roller passes is somewhat self-explanatory, but the factor 'temperature of binder at the time of rolling' has to be explained further. The stiffness of binder is directly related to its temperature. In other words, if the binder is hot, it will be less stiff and the rollers can more effectively set the aggregates at their most stable position. The three primary factors that influence heat loss from the binder after it is sprayed from the asphalt distributor are pavement surface temperature, ambient temperature and time elapsed between the application of binder and rolling. Due to the interaction of these factors, embedment depth becomes a complex phenomenon and it can be difficult to control both in the lab and field environment.

Moreover, the same amount of binder and aggregate will not result in the same embedment depth because of these other factors that affect embedment depth. As a result, the seal coat specimens for the asphalt cement test program were prepared to achieve three different embedment depths; 15%, 35%, and 50% using TxDOT design criteria. The adopted calculation procedure is illustrated for limestone aggregate from the limestone from Vulcan Materials Brownwood Pit.

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

Where:

A = Asphalt application rate (gal/yd^2)
G = Dry bulk specific gravity of the aggregate (measured at 2.67)

$$E = ed$$

e = design embedment (either 15%, 35% or 50%)

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

Q = Quantity of aggregate from board test, 22.04 lbs/yd^2
W = Dry loose unit weight of aggregate, 90.42 lbs/ft^3

Therefore,

d = 1.33 x 22.045/90.42 = 0.32426 inches

For 15% embedment, E = 0.04864 in
For 35% embedment, E = 0.1135 in
For 50% embedment, E = 0.1621 in

For 15% embedment, A = 5.61 x 0.04864 x (1-(90.42/(62.4 x 2.67))) = 0.125 gal/yd^2
For 30% embedment, A = 0.291 gal/yd^2
For 50% embedment, A = 0.416 gal/yd^2

Results from similar calculations for siliceous gravel aggregate from Fordyce Company, Showers Pit and for the lightweight aggregate from TXI Streetman plant are presented below.

For siliceous river gravel,
Q = Quantity of aggregate from board test, 19.69 lbs/yd^2
W = Dry loose unit weight of aggregate, 87.96 lbs/ft^3
G = Dry bulk specific gravity of the aggregate, 2.61
A = 0.115 gal/yd^2 (15% embedment)
A = 0.269 gal/yd^2 (35% embedment)
A = 0.384 gal/yd^2 (50% embedment).

For lightweight aggregate,
Q = Quantity of aggregate (board test), 8.34 lbs/yd^2
W = Dry loose unit weight of aggregate, 46.15 lbs/ft^3
G = Dry bulk specific gravity of the aggregate, 1.46
A = 0.110 gal/yd^2 (15% embedment)
A = 0.233 gal/yd^2 (35% embedment)
A = 0.333 gal/yd^2 (50% embedment).

In order to use these design values with emulsified asphalt, the binder rates were adjusted based on the percent residual asphalt in the emulsion. For example, the application rates to achieve 35% embedment depth for an emulsion with 70% asphalt and 30% water, the following calculations will apply.

Limestone: 0.291/0.70 = 0.416 gal/yd^2
Gravel: 0.269/0.70 = 0.384 gal/yd^2
Lightweight: 0.233/0.70 = 0.333 gal/yd^2

5.2.3.2 Dust Content on Aggregate Surface

Dust on seal coat aggregate is one of the most common factors that contribute to aggregate loss (raveling). Although the aggregates are washed when they are produced at the source, significant amount of dust may be generated during transportation and handling, particularly with crushed limestone and lightweight aggregate. In order to investigate the effect of dust on aggregate-binder bond, it was decided to prepare seal coat test specimens using aggregates with three different dust levels; no dust, medium dust and high dust.

Instead of artificially adding dust to the aggregate, it was decided to allow the aggregate to generate its own dust under conditions somewhat similar field conditions (i.e. handling). This

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was done in the laboratory using the Micro-Deval test equipment. This test is typically used to evaluate the abrasion resistance of aggregates. In the standard test procedure, 5000 grams of aggregate and 1500 grams of 1 in diameter steel balls are both put into a steel drum and closed with a lid. This cylindrical drum is placed on the Micro-Deval device and rotated for a specified duration. Preliminary tests showed that too much dust is generated when steel balls are used, and also it does not accurately mimic field dust generation condition. Therefore, the Micro-Deval equipment was used without the steel balls. The duration of the drum rotation affects the amount of dust generated. Figure 5-7 shows the amount of dust generated with 2, 5 and 15 minutes of drum rotation. The trend line of these 3 points turned out to be linear. After these preliminary trials, the dust contents were evaluated and 0, 2, and 5 minutes of drum rotation were selected to represent the conditions of no dust, medium dust and high dust contents. It was also observed that from the total amount 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 5-7 as 'percent dust retained'.

Before putting them in the Micro-Deval equipment, all aggregate were washed and then dried in the oven for 24 hours. The aggregate was then stored in sealed bags until seal coat specimens were prepared. As expected, the siliceous gravel did not generate any dust when subjected to this procedure, and therefore, dust content was not considered as an experimental factor for this aggregate.
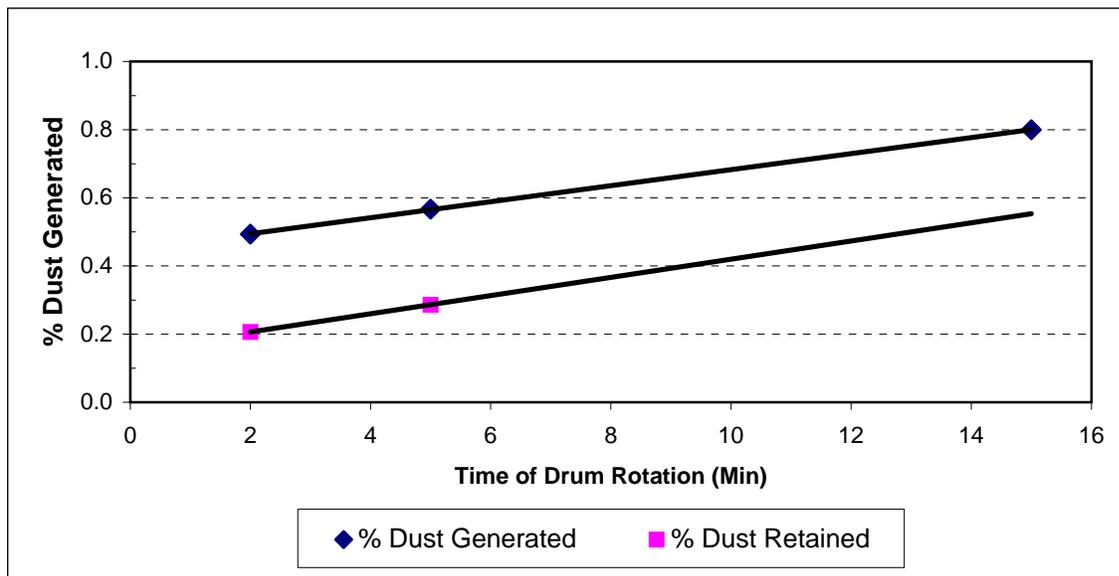
Figure 5-7 Dust Generation and Retention from Micro-Deval Test for Limestone

5.2.3.3 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

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spreading of aggregate and the rolling was selected as 3 minutes and 10 minutes respectively, based on the cooling rate of hot asphalts. These durations were chosen after careful consideration of the field data collected during the constructability review. Binder temperature at the time of rolling was measured using an infrared non-contact thermometer and recorded for each run.

5.3   Results for Asphalt Cement Test Program

Based on the 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 performance of the widest variety of material combinations could be verified. The two combinations selected were:

- AC-5 (with 2% Latex) + Siliceous Gravel
- AC-15 5TR + Crushed Limestone

Seal coat specimens using these two combinations of materials were prepared in the laboratory to incorporate four experimental factors; embedment depth, dust content, surface temperature, time elapsed between the spreading of aggregate and rolling. These four experimental factors were selected based on the constructability review (see Chapter 3) and their effect on aggregate-binder bond.

After the specimens were prepared, they were subjected to the conditioning protocol developed previously (i.e. 3 Freeze-Thaw cycles followed by 48 hours of soaking). After that, the impact testing was conducted by dropping the Modified Proctor Hammer (MPH) on the inverted specimen three times. The number of aggregate particles in each specimen was counted before conditioning, after conditioning and after impact loading. Percent loss for each specimen replicate and the average percent loss for the three replicates were calculated. The results for AC-15 5TR - Limestone combination are given in Table 5-8.

The percent aggregate loss shown in Table 5-8 are the averages for 3 replicates. The aggregate dust condition represents the dust generated from 0, 2 and 5 minutes in the Micro-Deval equipment without the steel charge.   These three states correspond to 0, 0.2 and 0.3 percent dust respectively, expresses as a percentage of the weight of aggregate. Pavement surface (hot plate) temperatures of 70 °F and 140 °F were included in the test along with 3 and 10-minute time delays between aggregate spread and rolling.

The data presented in Table 5-8 is graphically represented in Figures 5-9, 5-10 and 5-11. In these figures, the percent aggregate loss (PAL) is plotted in the vertical (z-) axis while two other factors are plotted in the other two orthogonal (x- and y-) axes. In Figure 5-8, the PAL is plotted against aggregate dust content and percent aggregate embedment for 70 °F pavement (surface) temperature and a 3-minute delay between aggregate spreading and rolling. The 70°F pavement temperature is typical of the lower end of the spectrum where seal coating is allowed in specifications, and the 3-minute time delay is typical of the delay between aggregate spreading and rolling based on field observations made by researchers.

As for percent embedment, most districts use 35-40 percent, but the current trend appears to be to use higher percentages of embedment (as much as 50-55%), particularly with Grade 3 aggregates and tire rubber asphalt. The 15% embedment lost as much as 94% of its aggregate with high dust content. The percent loss decreased to 70% for no dust aggregate. The difference

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between these two is approximately 24%. The specimens prepared with 35% embedment lost as much as 52% with high dust content aggregate and when no dust was present, the PAL reduced to 23%. The difference of approximately 29% between these two conditions is comparable to the 24% aggregate retention difference for 15% embedment. This data strongly suggests that dust content has a significant effect on aggregate retention somewhat independently from the embedment depth. The data clearly shows the effect of these factors on seal coat bonding performance, and more importantly, the testing protocol shows excellent sensitivity to conditions that we know to affect field performance.

Table 5-8 Percent Aggregate Loss for AC-15 5TR + Limestone

Percent Embedment	Time from Aggregate Spread to Rolling (min)	No Dust		Medium Dust (2-Min. Micro-Deval)		High Dust (5-Min. Micro-Deval)	
		70 °F Pavement Temp.	140 °F Pavement Temp.	70 °F Pavement Temp.	140 °F Pavement Temp.	70 °F Pavement Temp.	140 °F Pavement Temp.
15%	3	69.6	-	78.3	57.6	94.4	71.7
	10	-	-	-	60.2	-	-
35%	3	22.8	9.3	40.6	19.7	-	-
	10	-	-	-	18.0	-	-
50%	3	11.9	-	36.6	5.4	-	7.5
	10	-	-	-	6.6	-	12.5
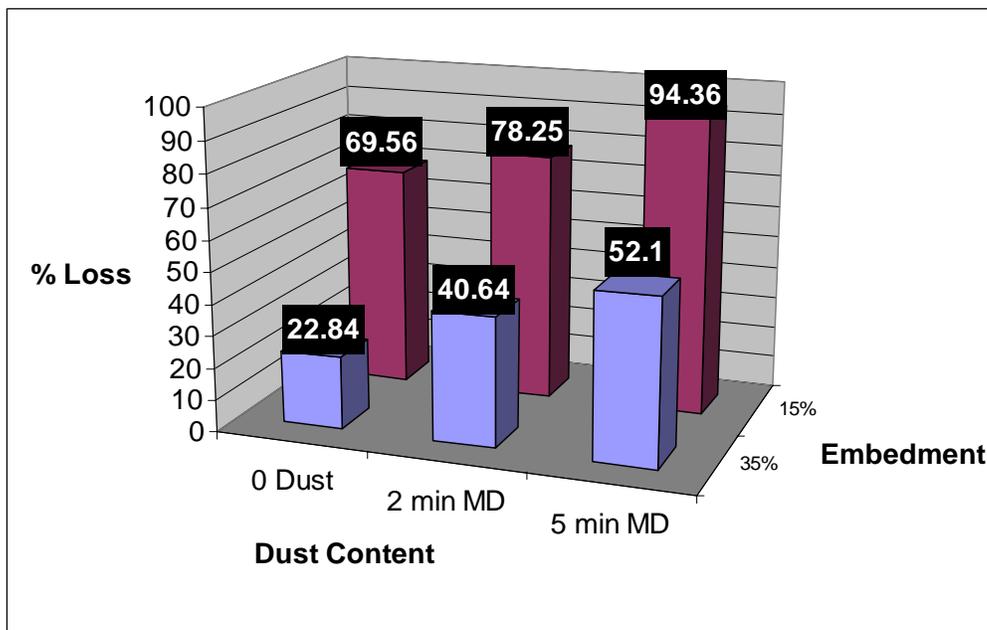
Figure 5-8 Effect of Aggregate Dust Content and Embedment on % Aggregate Loss at 70 °F Surface Temperature and 3-minute Roller Delay for AC-15 5TR + Limestone

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In Figure 5-9, the PAL is plotted against percent embedment and time delay between aggregate spread and rolling for 140 °F surface temperature and medium aggregate surface dust. The very high difference in PAL between 15% and 35% embedment values is quite remarkable. It shows that aggregate retention improves exponentially with increasing embedment depth for the dust conditions indicated (approximately 0.2% of aggregate weight) for the 140 °F binder temperature when aggregate was spread. In addition, for the test conditions of 140 °F surface temperature and medium surface dust, the rolling delay from 3 to 10 minutes did not make a significant difference for all three values of percent embedment.
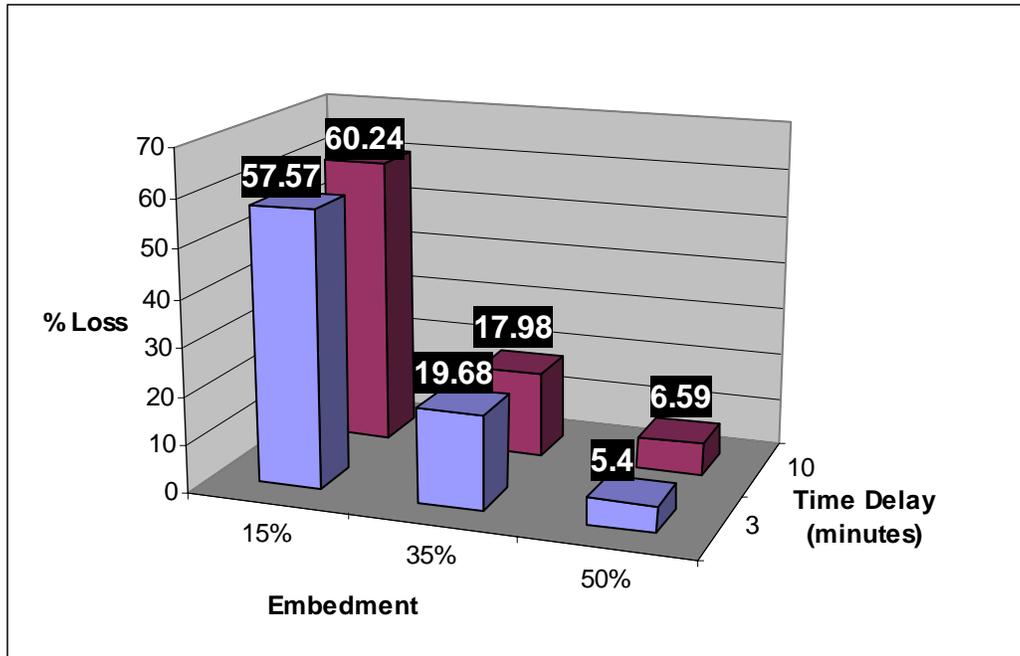
Figure 5-9 Effect of Embedment and Roller Delay on % Aggregate Loss at 140 °F Surface Temperature and Medium Aggregate Dust for AC-15 5TR + Limestone

Figure 5-10 shows a chart where PAL is plotted against percent embedment and surface temperature for medium dust and 3 minutes of rolling delay. Embedment depths of 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 (5.4%) at the surface temperature of 140 °F and 50% embedment, and at this surface temperature, there was significant benefit gained by increasing percent embedment. However, at 70 °F surface temperature, increasing percent embedment from 15 to 35 percent helped with aggregate retention, but not to the same extent as with 140°F. The aggregate-binder bond showed high sensitivity to surface temperature at all three levels of percent embedment. This leads to the conclusion that bonding between the aggregate and binder depends on a combination of percent embedment and pavement surface temperature, and therefore, field inspectors must take special care to roll the aggregate immediately, particularly at low pavement temperatures, to ensure that hot asphalt seal coats attain good bonding. This test protocol may be used by TxDOT to revisit its seal coat specifications.

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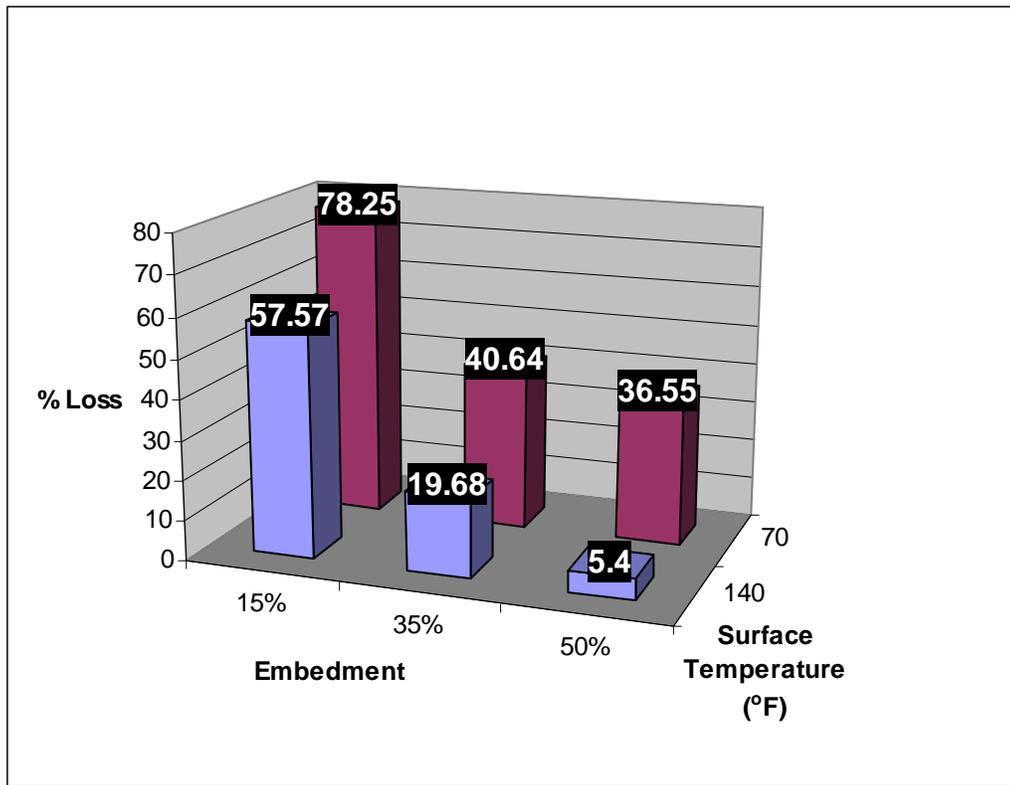
Figure 5-10 Effect of Embedment and Surface Temperature on % Aggregate Loss at Medium Aggregate Dust and 3-Minure Rolling Delay for AC-15 5TR + Limestone

The results for the AC-5 (with 2% latex) - Siliceous Gravel combination are given in Table 5-9. All tests for this material combination were conducted at medium aggregate dust level because it appeared to best represent conditions in the field for non-precoated aggregates. Dust in this case was artificially applied to the aggregate because of low dust generation from siliceous river gravel aggregate. The lowest PAL for this material combination is 33.4% and the highest is 88.2%. Comparison of these results with the previous combination suggests that the protocol was successful in identifying the good and the poor material combinations.

Table 5-9 Percent Aggregate Loss for AC-5 (with 2% latex) + Siliceous Gravel

% Embedment	Time from Aggregate Spread to Rolling (min)	70 °F Pavement Temp.	140 °F Pavement Temp.
15%	3	88.2	83.3
	10	-	80.3
35%	3	86.8	58.3
	10	-	68.2
50%	3	56.4	35.1
	10	-	33.4

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Data from Table 5-9 are illustrated in Figure 5-11. The effect of both percent embedment of aggregate and the surface temperature on percent aggregate loss (PAL) are significant, and the magnitudes of losses are higher when compared to the AC-15 5TR – Limestone combination. In addition, data from Table 5-9 shows that rolling delay did not have much influence on aggregate retention for 140 °F surface temperature, and this trend was similar to the AC-15 5TR-Limestone material combination.
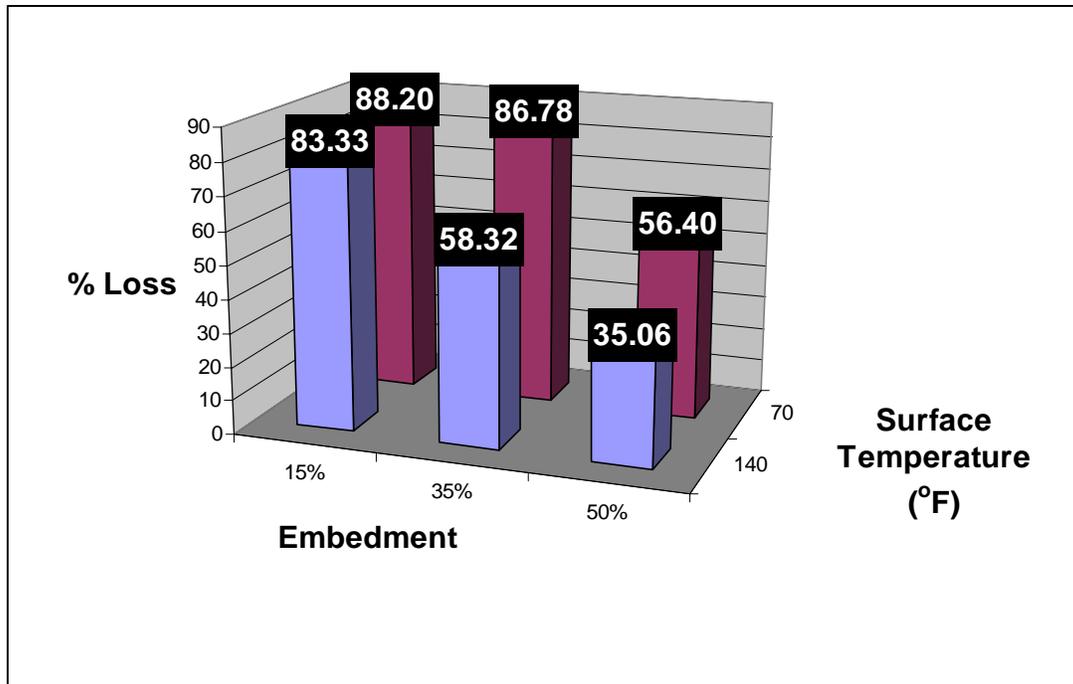
Figure 5-11 Effect of Embedment and Surface Temperature on % Aggregate Loss at Medium Aggregate Dust and 3-Minure Rolling Delay for AC-5 (with 2% latex) + Siliceous Gravel

5.4   Results for Emulsified Asphalt Test Program

In this test program for emulsified asphalt, laboratory specimens were prepared using the CRS-2P emulsified asphalt, which is the predominant summer seal coat emulsion used by TxDOT. The source of the emulsified asphalt was Koch Materials, Brownwood plant. It was tested with three aggregates; siliceous gravel, crushed limestone and lightweight aggregate. From the constructability review, it was identified that aggregate moisture state and the breaking/curing processes affect emulsion-aggregate bond. Therefore, the following experimental parameters and their states were used in this test program.

- Air-dry and saturated-surface-dry (SSD) aggregate moisture states
- Aggregate application prior to and after the emulsion break
- Siliceous gravel, limestone and lightweight aggregate

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Curing time of emulsified asphalt was also included in the experimental program. However, since the specimen conditioning starts with soaking, some emulsions dissolved in water during the soaking, causing total failure of specimens in a few hours. Due to this the testing protocol was adjusted to start testing 3 days after samples were prepared.

The SSD aggregate was prepared by first soaking the aggregate for 24 hours, and immediately before the seal coat specimen preparation, it was taken out of the water bath and dried with a towel to remove the free surface moisture. Breaking process of the emulsion was tracked by time. The aggregate was applied immediately after the binder to simulate the 'before breaking' scenario. The aggregate was applied after 75 minutes from binder application time to simulate the 'after breaking' scenario. In the field, the emulsion breaks much earlier than 75 minutes, but the laboratory environment seemed to slow down the breaking process, and therefore, the 75 minute duration was selected. For emulsion specimens the testing procedure started 72 hours after the specimens were prepared.

Siliceous gravel was not tested in SSD condition because it had a very low absorption capacity. The surface temperature for specimen preparation was 70 °F for all specimens and a percent embedment of 35% was used. The asphalt-to-water ratio in the emulsion was 70% to 30%, and adjustments were made to calculate the binder rate accordingly. The test results for the emulsified asphalt seal coat specimens are shown in Table 5-10.

Table 5-10 Percent Aggregate Loss for CRS-2P Emulsified Asphalt Seal Coat Specimens used with Siliceous Gravel, Limestone, and Lightweight Aggregate

Moisture Content	Timing of Aggregate Application	Siliceous Gravel	Limestone	Lightweight
Air Dry	Before Breaking	0.00	0.00	0.85
Air Dry	After Breaking	4.75	19.31	0.00
SSD	Before Breaking		0.00	1.21
SSD	After Breaking		0.67	0.97

In general, the performance of emulsified asphalt specimens was better than asphalt cement seal coat specimens. 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 aggregate loss. Lightweight aggregate is very porous and the emulsion, which has a low viscosity at the time of application combined with the presence of water in the medium, can penetrate easily into these pores. This often leads to more effective bonding, as is evident from the test results. One of the most commonly used aggregates with emulsified asphalt is uncoated lightweight aggregate, and historically this material combination has performed well

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when applied under right conditions. 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 in the SSD moisture state performed better than those in the air-dry state. Field observations and the constructability review suggested that when emulsified asphalt is used, SSD aggregate bond better with the binder. In these experiments, for the SSD moisture condition, limestone and lightweight aggregate did not lose significant amounts of aggregate in both before breaking and after breaking aggregate application.

5.5   Results for Precoated Aggregate Test Program

In the third phase of the performance-based laboratory testing program, seal coat specimens prepared using precoated aggregate were tested with the new testing protocol. It is well known that the dust on seal coat aggregate is one of the common causes of debonding between the aggregate and binder. The aggregates that generate significant amounts of dust during transportation and handling have to be precoated to either eliminate or minimize incompatibility problems. In addition to improved bonding, some districts prefer to use precoated aggregate for seal coats because it provides improved visibility of pavement markings that provide high contrast on darker precoated aggregates.

The constructability review showed that TxDOT 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, these two binders were used to precoat aggregates in the lab for this phase of the testing program. In addition, seal coat specimens were prepared using aggregate with two different precoating levels, up to 50% and more than 90% of the aggregate surfaces coated. The three binders AC-15 5TR, AC-15P, and CRS-2P were used as the seal coat binder. The precoating levels were checked based on visual observations. The binder requirement, in the form of binder-to-aggregate (B-A) ratio, for the two precoating levels was determined by trial and error. For limestone aggregate, the B-A ratios were 0.2% and 0.5% by weight for 50% and 90% precoating levels respectively. For lightweight aggregate, the B-A ratios were 1.7% and 4.5% for 50% and 90% precoating levels, respectively. The aggregates were washed and oven-dried prior to precoating so that there was no dust left on the surface of the particles. Since AC-15P is not typically used with precoated lightweight aggregate in field practice, that combination was excluded.

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

The results from the precoated aggregate testing program are presented in Table 5-11. The overall performance of precoated aggregates seal coats was far better than that of uncoated aggregates. The new testing protocol successfully identified the influence of precoating level for AC-15P. For the other binders, there was no significant difference between the two precoating levels since all of them did not lose more than 0.01% of their aggregate. This clearly illustrates how precoating improves the performance of seal coats if it is used with the appropriate

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preacoating 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 5-11 Percent Aggregate Loss for Seal Coat Specimens using Precoated Aggregate with 35% Embedment and Rolling Delay of 3 Minutes

Seal Coat Binder	Uncoated Lightweight Aggregate	Uncoated Limestone	Emulsion Precoat (Css-1h)		Hot Asphalt Precoat (PG 64-22)	
			50% Precoated Area	90% Precoated Area	50% Precoated Area	90% Precoated Area
AC-15 5TR	11.25	19.68	0.00	0.00	0.01	0.00
AC-15P	-	57.07	-	-	9.53	3.27
CRS-2P	0.85	0.00	0.00	0.00	0.00	0.00

Note: The surface (pavement) temperature for hot asphalt seals was 140 °F and for emulsified asphalt seals was 70 °F.

5.6   Suggested Evaluation Criteria for the Testing Protocol

Table 5-12 presents the results of a simple statistical analysis of results for the AC-15 5TR + Limestone combination. The 5-10% range of aggregate loss is the most frequently occurring data range. The medium dust condition (2 min MD) with 50% embedment at 140 °F lost 5.40% and 6.59% aggregate at 3 minutes and 10 minutes of rolling delay, respectively. A loss of 7.50% occurred for high dust condition (5 min MD) at 140 °F for 3 minutes of rolling delay. A loss of 9.26% occurred for the no dust condition, 35% embedment at 140 °F and 3 minutes of rolling delay. These four instances are considered to be acceptable. Therefore 0 to 10% loss range was considered as 'Very Compatible'. Two data points fell in the 10 to 15% range; 11.88% loss for 50% embedment at 70 °F for no dust and 3 minutes rolling delay and 12.49% for high dust (5 min MD), 50% embedment at 140 °F with 10 minutes rolling delay. These percent aggregate loss numbers may indicate some incompatibility triggered by low surface temperature in one and the high dust content and delayed rolling in the other. The losses of 18.0% and 19.7% resulted from medium dust (2 min MD), 35% embedment at 140 °F with 3 minutes rolling delay and 10 minutes rolling delay, respectively. These combinations are reasonably performing combinations. Therefore, the 10 to 20% aggregate loss range is considered as 'reasonably compatible'. Aggregate loss of 22.8% was the result of 35% embedment, no dust at 70 °F, 36.5% was the result of 50% embedment medium dust (2 min MD) at 70 °F and 40.6% was the result of 35% embedment, medium dust (2 min MD) at 70 °F. The last two combinations are highly susceptible to debonding, but the first one may still perform adequately, because of no dust condition and 50% embedment. Therefore, 20 to 30% is rated as 'Marginally Compatible'. Any percent loss beyond 30% can be classified as 'incompatible' when tests are conducted under laboratory conditions that simulate local conditions. In this case, 'local conditions' would include the surface temperature as well. A district can use suitable pavement surface temperatures for the testing of their material combinations. The evaluation criteria established above are summarized in Table 5-13. It should be noted that these evaluation criteria are based on the performance of lab prepared specimens. They should only be used for

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field prepared specimens if the specimens were tested as per the testing protocol at the appropriate age.

Table 5-12 Frequency of Occurrence for % Aggregate Loss Data for AC-15 5TR - Limestone Combinations

Aggregate Loss Range (%)	Frequency	% Loss Recorded within Range
0 to 5	0	-
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 5-13 Suggested Evaluation Criteria for Testing Protocol

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

5.7   Conclusions

After the work leading up to the development of the new testing protocol was 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. This 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 (with 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 it is not a common problem with siliceous gravel aggregate.

The new testing protocol identified levels for the different experimental factors that cause incompatibility. According to the test results, AC-15 5TR + Limestone performed better than AC-5 (with 2% Latex) + Gravel as expected. These results indicate that the new testing protocol can successfully discriminate between 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 this laboratory testing activity, emulsified asphalt seal coats were investigated. Specimens prepared with CRS-2P emulsion and 3 different types of aggregates (limestone, lightweight and siliceous gravel) were prepared and tested using the newly developed testing protocol. It was decided to use only CRS-2P emulsion because it was by far the predominant seal coat binder used by the TxDOT summer seal coat program. In addition to testing the suitability of material combinations, the effect of aggregate moisture state and breaking process of emulsion on aggregate-binder bonding were also investigated. The specimens prepared using aggregate in the SSD moisture condition performed better than those with air-dry condition. This is in agreement with the field experience with emulsions. On the other hand, the specimens in which aggregate was placed after the 'breaking' of emulsion performed worse than those in which aggregate was placed before the emulsion break. This was true for both limestone and siliceous gravel aggregate. These results confirm the ability of the new testing protocol recognizing the time-tested factors that cause incompatibility.

In the third phase of this laboratory testing activity, 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. Effect of precoating binder coverage of aggregate surface 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.

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CHAPTER 6

ADDITIONAL TESTS

In addition to the two primary test protocols developed in this research that were presented in Chapters 4 and 5, two more tests were conducted for additional investigation of the aggregate-binder bond in seal coats. These tests were the Interfacial Bonding Test (IBT) and the Modified Australian Pull-out Test.

6.1   Interfacial Bonding Test

6.1.1   Test Procedure and Experimental Plan

This test involved the pulling out of an aggregate partially embedded in an asphalt binder. In order to have a known aggregate surface area, the aggregate particles were cut into a cubic shape using a precision electric saw. Before embedding the aggregate particle into the binder, the dimensions of each aggregate particle were measured using a digital caliper. Later, these dimensions were used to calculate the aggregate-binder contact surface area, tensile stress and percent embedment. A motorized loading frame with displacement control capability 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. After a certain time was elapsed, the aggregate particle was pulled out from the binder at a specific displacement rate and the load-displacement data was recorded in the computer using a RS-232 machine interface. Figure 6-1 shows the test equipment and the aggregate being pulled out of asphalt. The displacement rate used for the tensile test was 2 in/min for all IBT experiments. After the aggregate particle was pulled out of the asphalt binder, the embedment depth of aggregate in the asphalt and the length of the binder string at the time of maximum load were recorded. Figure 6-2 shows a typical force-displacement curve for a typical experiment. Similar to the experiments described in Chapter 5, tests were conducted using both hot applied asphalt and asphalt emulsion.

6.1.2   Test Results

6.1.2.1 Hot Applied Asphalt

Specimens were prepared for the two material combinations AC-15 5TR + Limestone and AC-5 (with 2% latex) + Rhyolite Gravel. Binder temperature at the time of aggregate placement on binder (100 °F and 140 °F) and aggregate dust content were the experimental factors investigated in this phase. The two dust contents used were 'clean' and 'dusty'. Dust was introduced on the aggregate surface by immersing it in a container with the dust. Even though no accurate control could be enforced on the dust content adsorbed to the aggregate surface, care was taken to keep the aggregate surface in a dry condition when it was immersed in the dust. Tests were conducted with the aggregate particles placed in two positions, one where the aggregate particle is embedded into the binder to a predetermined embedment depth, and the other where just the bottom of the aggregate particle was touching the binder (i.e. 0% embedment). These two embedment scenarios were selected because the difference in

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performance between them will provide information on how embedment depth improves performance of the interfacial bond.
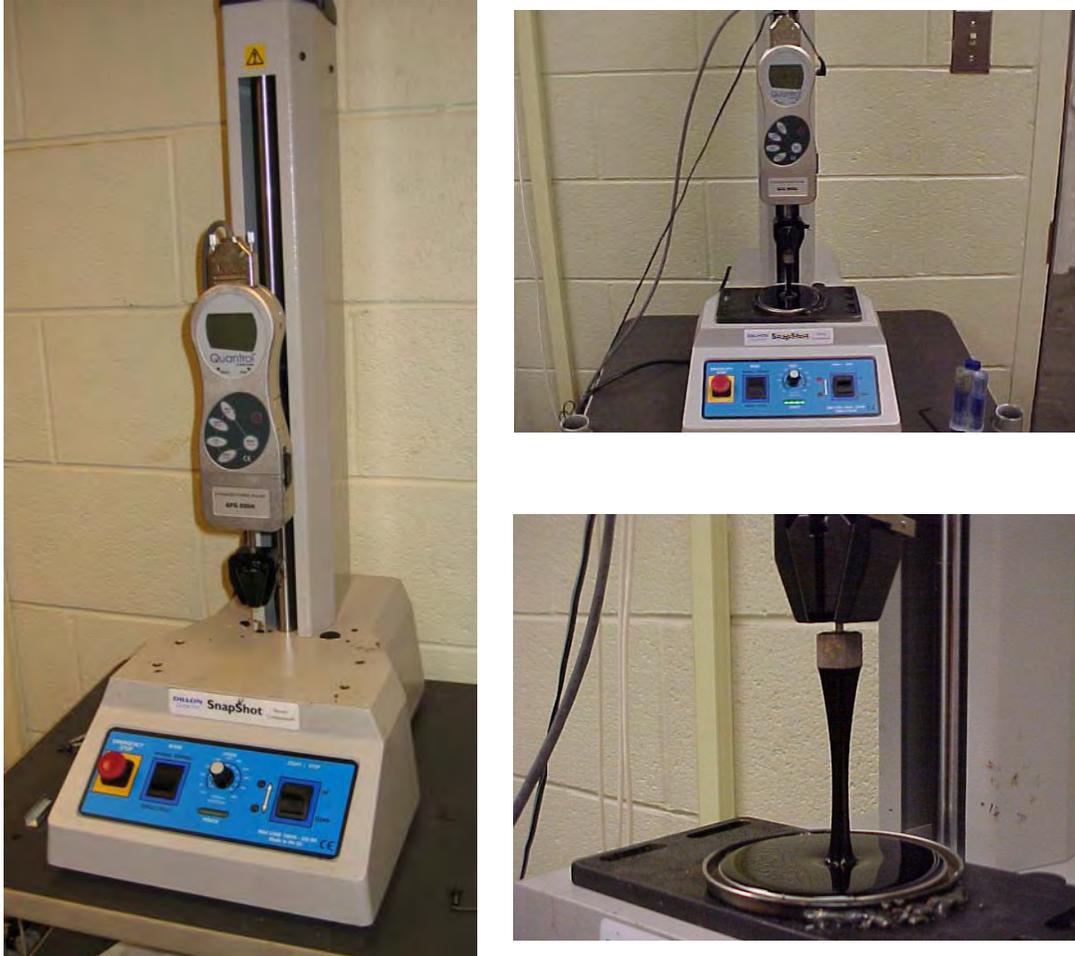
Figure 6-1 Aggregate Pull-Out Equipment used in the Interfacial Bonding Test

Table 6-1 shows the results for AC-5 (with 2% latex) + Rhyolite Gravel. The first two columns indicate the level of dust on the aggregate surface, 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 (i.e. 0% embedment). The fourth column has the peak pull-out stress measured when the aggregate is in maximum embedment position. In the fifth column, the percent embedment of each aggregate particle is given, and the sixth column shows the pull-out stress calculated per one percent embedment by dividing the peak stress value of the embedded aggregate by its percent embedment. The last column is the peak pull-out stress calculated from the difference between the two embedment positions (maximum embedment 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.

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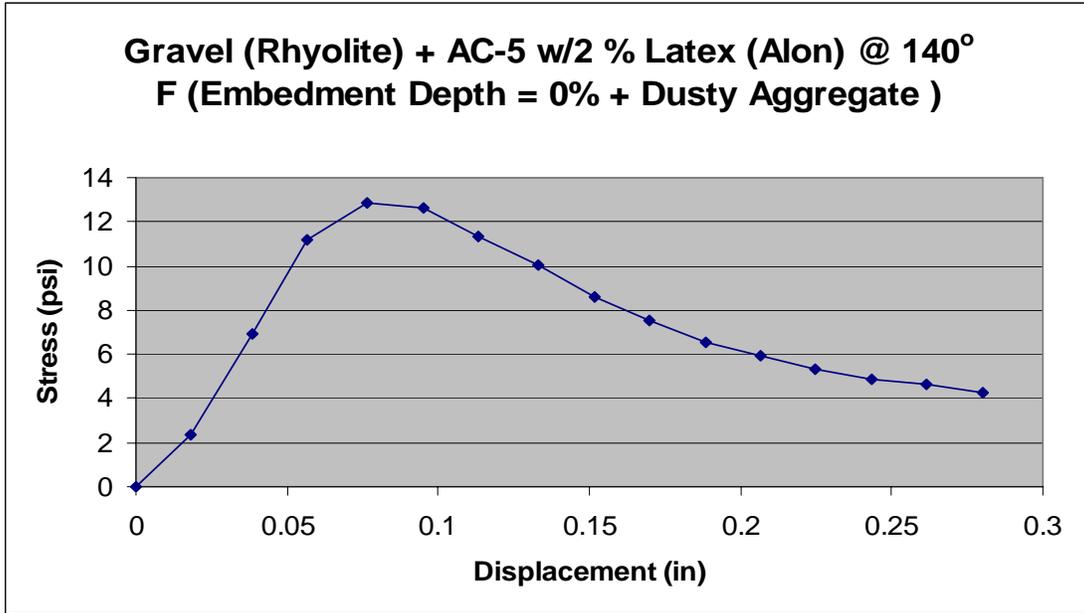
Figure 6-2 A Typical Stress-Displacement Curve for IBT Pull-Out Test

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 in the binder at 140 °F performed better than those placed at 100 °F. Based on 1% embedment comparison between the maximum embedment and the zero embedment position, the peak stress values follow this trend. As expected, based on 1% embedment comparison, the highest pull-out stress was for the clean aggregate applied at 140 °F, and the lowest stress comes from the dusty aggregate applied at 100 °F. Figure 6-3 shows the peak pull-out stress difference for 1% embedment plotted against dust content and binder temperature. The consistency of the results can easily be seen within the factors plotted.

Table 6-1 IBT Test Data for AC-5 (with 2% Latex) + Rhyolite Gravel

Aggregate Dust Content	Surface Temp (°F)	Peak Stress for 0% Embedment (psi)	Peak Stress for Max. Embedment (psi)	% Embedment	Peak Stress per 1% Embedment (psi)	Peak Stress Difference per 1% Embedment (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

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Table 6-2 IBT Test Data for AC-15 5TR + Limestone

Aggregate Dust Content	Surface Temp (°F)	Peak Stress for 0% Embedment (psi)	Peak Stress for Max. Embedment (psi)	% Embedment	Peak Stress per 1% Embedment (psi)	Peak Stress Difference per 1% Embedment (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
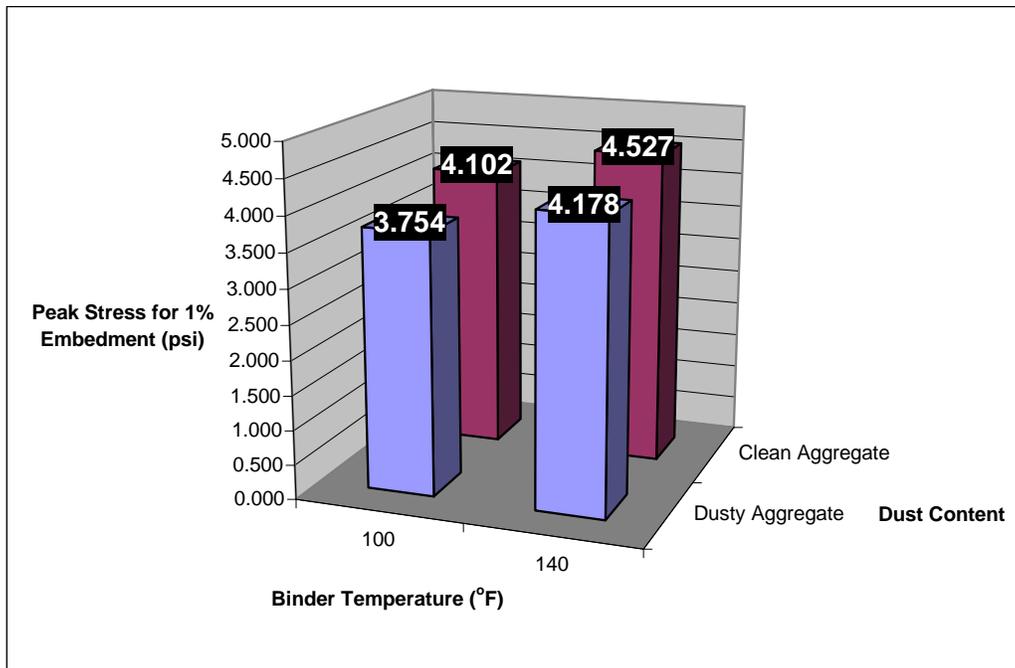
Figure 6-3 Peak Pull-out Stress Difference versus Dust Content and Binder Temperature at the time of Aggregate Application for AC-5 (with 2% Latex) + Rhyolite Gravel

Table 6-2 shows similar data for the AC-15 5TR + Limestone material combination. Based on data for peak stress per 1% embedment, the clean aggregates showed higher stress values than for AC-5 (with 2% latex) + Rhyolite Gravel material combination. On the other hand, dusty aggregate showed lower stress values than AC-5 (with 2% latex) + Gravel combination. This can be explained by the fact that the gravels do not naturally have dust while the limestone does. Another possible explanation is, during the preparation of dusty aggregate,

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there was no 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 limestone aggregate adsorbed more dust due to its surface texture. The binder was not able to wet the aggregate when the aggregate is dusty and in 0% embedment condition. Therefore, the strength of the bond was not measured at those conditions. Similar to the previous material combination, for this combination, the highest and the lowest peak stress values were for 140 °F clean aggregate and 100 °F dusty aggregate, respectively. The peak stress values decrease significantly when the aggregate surface is covered with dust.

6.1.2.2 Asphalt Emulsions

Specimens prepared using CRS-2P emulsified asphalt with lightweight aggregate and siliceous gravel, were tested using the IBT procedure. Experimental factors included in the test program were emulsion breaking time, curing time and aggregate moisture content. Two moisture states, air-dry and SSD, were used for the aggregate. Curing times of 3 hours and 10 hours after the application of aggregate were used. The emulsion breaking process was controlled by applying aggregate immediately after the application of the emulsion (before breaking) and 75 minutes after application (after breaking). Unlike the hot asphalt experiments, only the maximum embedment condition was tested.

Peak pull-out stress results measured are presented in Table 6-3. Since all aggregates had maximum embedment, only the peak stress per 1% embedment was calculated. The peak stress values were much lower than the stress values for hot asphalt. This is probably due to emulsion still curing under laboratory conditions when the pull-out test was conducted. Peak stress values for the SSD aggregate were lower than for dry aggregate specimen. These results do not match with those obtained for performance-based testing protocol results presented in Chapter 5. The curing times for the IBT specimens were 3 hours and 10 hours while the specimens for the performance-based test described in Chapter 5 were cured for 30 hours. This is an indication of the effect water in the emulsion has on the aggregate-emulsion bond. The bond gains its full strength when there is no water in the system. This also suggests that the emulsions are more sensitive to the climatic conditions soon after construction that can alter its curing process. The peak stress values for SSD aggregate applied after the emulsion break were lower than for the aggregate applied before breaking. However, the air dry aggregates did not follow this trend.

6.1.3 Conclusions

In the *Interfacial Bonding Test (IBT)*, the bond between a cube-shaped aggregate and binders were investigated using a controlled pull-out test procedure. It has been reported in the literature that this interfacial bond is one of the key factors that affect aggregate retention performance of seal coats. However, there are also various other factors that influence bonding. The experimental factors used for hot asphalt were dust content and the binder temperature at the time of aggregate application. For emulsified asphalt the curing time, breaking time, and aggregate moisture content were investigated.

Table 6-3 IBT Peak Stress Results for CRS-2P with Limestone and Siliceous Gravel

Moisture Content	Curing Time (Hours)	Before Break						After Break					
		Sil. Gravel Stress (psi)	% Embedment	Stress per 1% Embedment (psi)	Limestone Stress (psi)	% Embedment	Stress per 1% Embedment (psi)	Sil. Aggregate Stress (psi)	% Embedment	Stress per 1% Embedment (psi)	Limestone Stress (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 of binder temperature at the time of aggregate application was clearly evident with the IBT. Both material combinations tested responded to this factor but AC-15 5TR + Limestone combination performed much better than AC-5 (with 2% Latex) + Rhyolite Gravel combination. This may be due to the better surface texture of limestone aggregate. The binder AC-5 (with 2% Latex) is less viscous than the AC-15 5TR at 100 °F and 140 °F. One could assume that AC-5 will more effectively penetrate into the pores of the aggregate due to its low viscosity and provide better bonding, but the results did not confirm this hypothesis. Differences in aggregate surface texture and in the chemical interactions may be the likely causes of these apparent inconsistencies.

The IBT also recognized differences in the bond due to aggregate dust content. Due to the lack of control over how much dust was applied, the results do not show any clear trend for 0% embedment. On the other hand, results for the maximum embedment case suggest that bond strength decreases significantly when dust is present, particularly for the limestone.

The comparison of the two material combinations shows that AC-15 5TR + Limestone combination is the better of the two when no dust is present on the aggregate surface. The performance of this material combination deteriorated 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 of 10 hours. The IBT was successful in identifying the before/after break effect for SSD aggregates, while it was unsuccessful for air dry aggregates. Moreover, the IBT distinguished the difference in performance between two curing times.

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

6.2   Modified Australian Pull-out Test

6.2.1   Background

The objective of this modified Australian pull-out test is similar to that of the original T-238 test procedure developed in Australia, and that is to evaluate the initial adhesion between aggregate and binder. The two test methods have differences in both sample preparation and testing procedures. Due to environmental concerns, cutback binders are not commonly used in Texas seal coats. Instead of the 'cutback' approach to liquefy the binder, it is heated up to approximately 320 °F to facilitate the pouring of binder. Such a modification does not require the use of water in the preparation of sample or during testing because some preliminary tests showed the evidence of total loss of bonding when water is present as required in T-238. The binder heating approach is used to better simulate the field condition in the state of Texas. In order to replicate the field construction operation, asphalt was heated to 320 °F until it became fluid and emulsion was heated to 170 °F before pouring onto a metal tray to prepare the sample.

The modified test procedure involved preparing a specimen with 72g of asphalt on a 6 inch x 6 inch square metal tray at a surface temperature of 100 °F with a target binder thickness of approximately 3 mm, which is the same as in the original T-238 test procedure. This temperature represents a typical pavement surface temperature during seal coat construction season in most parts of Texas. Although other aggregate surface conditions were also tested as part of these experiments, clean and dry aggregate was used in this test. As indicated earlier, curing was not done under water but in a dry state at room temperature. Typically, about 30 aggregate particles were used per sample-plate. In order to minimize the number of sample plates prepared, and also to reduce the variability among different material combination, two types of aggregates were used in a single plate, with half of the plate for each. Selected aggregates were glued to the metal nails facilitating the pull-out operation as shown in Figure 6-4. Such aggregates with the nail glued to its top were pushed into the asphalt binder at the same temperature and the metal tray with aggregates was kept aside 24 hours for curing at room temperature. Styrofoam bracings were used to keep the nails straight while curing. After 24 hrs of curing, the sample is evaluated for various testing parameters as discussed below. The results, were used to evaluate the bonding strength and binding compatibility of the combination is determined.

The original T-238 test procedure measures only the stripped area to calculate the percentage of contact area between aggregate and binder for evaluation of aggregate-binder compatibility. The modified version of the test uses a similar measure but in terms of percent coated area. In addition to this subjective measure, other parameters were also tried in this modified test protocol to evaluate the adhesion between aggregate and binder. Aggregates are pulled out with a handheld force gage attached to the nails by a specially made device shown in Figure 6-4. Another device without the nail was also used (see Figure 6-5). These methods provide quantitative results, and therefore should have less subjectivity involved. In reality,

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however, it was very difficult to keep the nails aligned vertically during the pull-out operation. The other device (Figure 6-5) also did not provide a good grip on the aggregate.

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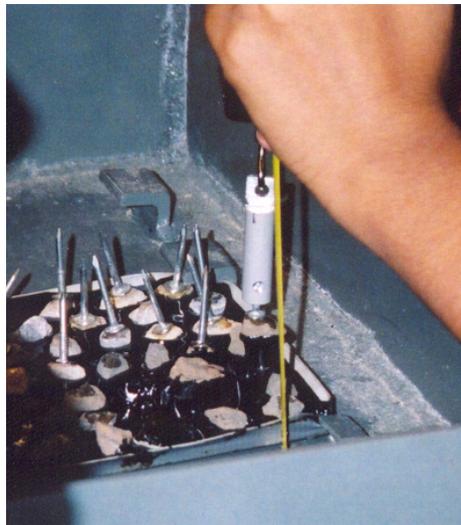
Figure 6-4 Pull-Out Test Aggregates with Glued Nail Attached

The parameters measured under the new protocol are the percent aggregate area coated with asphalt, peak tensile stress, elongation length of coated asphalt film, and energy required to pull the aggregate out. The tests were performed at various temperatures to simulate field conditions. A brief description of these parameters is given below.

1. Coated Area – The percentage of embedded (contact) area of the pulled-out aggregate that is covered with asphalt is visually estimated. Each aggregate is rated as 100%, 50% and 0% covered depending on the following categories:
   a. 100% rating – More than 75% area covered with asphalt.
   b. 50% rating – 25 to 75% of contact area retains asphalt after pulling out.
   c. 0% rating – Less than 25% area covered with asphalt.
   An aggregate with 100% rating is counted as one, 50% rating as one-half and 0% as zero. The sum of such counts is expressed as a percentage of total number of test aggregates to represent the material combination tested.

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Figure 6-5 Aggregate Pull-Out using Force Gage

2. Peak Tensile Stress – Aggregates were pulled out of the binder with a hand-held force gage attached to the glued nails mentioned above. Tensile stress is calculated using the peak force before the bond breaks and the projected area of contact surface as shown in Figure 6-6. The strength of the aggregate-asphalt bond was evaluated based on the tensile stress.
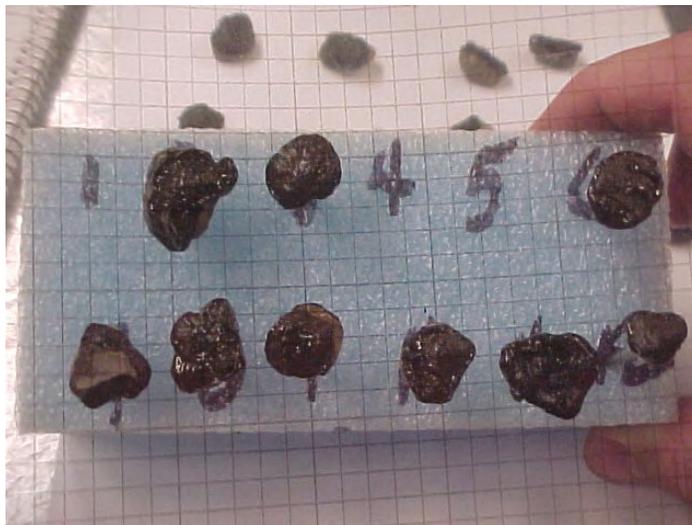
Figure 6-6 Measurement of Projected Contact area after Aggregate Pull-Out

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3. Elongation Length – The longest asphalt string generated during pull-out, as shown in Figure 6-7, is measured as elongation length in inches. This parameter is supposed to measure the potential for returning of the aggregate to its original position after distress.
4. Pseudo Energy – It is assumed that both the bond strength as measured by the peak tensile stress and elastic rebound as measured by the elongation length contribute to the resistance against aggregate loss in seal-coats. Therefore, in an attempt to combine these two factors, 'pseudo energy' is calculated as the product of these two measurements.

Figure 6-7 Elongation due to Aggregate Pull-Out

Experimental Plan

Five different aggregate types and five different asphalt types from different sources were selected for the modified Australian pull-out test. In addition, several other material combinations were selected to incorporate the environmental factors and variations in samples preparation. A summary of this experimental plan is presented in Table 6-4. Most aggregates used in Texas seal coats are pre-coated. Therefore, evaluation of pre-coating and the effect of dust were also included in the plan. At least one emulsion was also included in the experimental plan. Where possible, the special aggregate-binder combinations were matched with the actual field sections.

Samples were prepared under different conditions. The normal test condition was washed, cleaned and dried aggregate. Some variations in aggregate surface condition such as dusty, saturated surface dry (SSD), and saturated surface wet conditions were also evaluated. The condition during the pull-out process was also varied. Although the usual practice is to test under the dry condition, some were tested under soaked (in water) conditions at room temperature and some of the samples were tested at 140 °F water temperature after 30 minutes of soaking. During the seal-coat sample preparation, surface temperature was also varied. In general, the surface temperature during sample preparation was kept at 100 °F. This temperature

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was selected to match with typical field conditions. In order to accommodate the variation in the surface temperature, other samples were prepared at 120 °F and 140 °F, and also at room temperature. Some combinations of asphalt and aggregate were tested after freeze-thaw conditioning to check the adverse effect of environment on the bonding between binder and aggregate.

Table 6-4 Summary of Experimental Plan for Pull-out Test

Standard TechMRT Pull-out Test with Uncoated Aggregate					
Aggregates	Hot Applied Binders				
	PG 64-22	AC 15-5TR	AC 15 P	AC 10 (2L)	AC 5 (2L)
Limestone	√	√	√	√	√
Light Weight	√	√	√	√	√
Rhyolite	√	√	√	√	√
Gravel (Siliceous)	Not Performed	√	√	√	√
Sandstone	√	√	√	√	√

Investigation of Factors Influencing Sample Preparation		
Aggregates	Binders	Comments
Limestone	AC 15-5TR	Factors investigated:
Light Weight	AC 15-5TR	1. Surface temperature 67^0 F and 140^0 F, 2. Sample preparation under water, 3. Curing under water, 4. Saturated surface wet aggregate.

Comparison of Pre-coated vs Uncoated Aggregate		
Aggregates	Binders	Comments
Gravel	AC 15-5TR	Pharr district field section.
Gravel	AC 5 (2L)	Childress district field section.
Limestone	AC 10 (2L)	Lufkin district field section.
Limestone	AC 15-P	San Antonio district field section.

Evaluation of Environmental Effects		
Aggregates	Binders	Comments
Pre-coated LS	AC 15-5TR	Abilene district field section.
Limestone (dusty)	AC 15-5TR	Vulcan Materials, Brownwood, TX.
Gravel (siliceous)	AC 5 (2L)	

Evaluation of Emulsion as Seal-coat Binder		
Aggregates	Binders	Comments
Limestone	CRS-2P	Pre-coated limestone is form Brownwood district field section. Sources for other materials are mentioned in Table 1.
Light Weight	CRS-2P	
Rhyolite	CRS-2P	
Gravel (siliceous)	CRS-2P	
Sandstone	CRS-2P	
Pre-coated LS	CRS-2P	

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<u>6.2.3   Results and Discussion</u>

All tests were conducted at the asphalt laboratory of Texas Tech University. In addition to the 25 material combinations in Table 6-4, other aggregate-binder combinations were also tested to investigate field conditions using field-sampled asphalt. The results of these pull-out tests are presented below.

The first set of tests were done on the selected five aggregates and five binders indicated earlier. All but one of the possible combinations (24 in this case), were tested under the following general conditions:

- Sample preparation temperatures was 100 °F to 110 °F (surface temperature)
- Aggregate particles were clean (washed and dried)
- Sample preparation, curing and testing were done at dry condition. Some initial tests clearly showed that the presence of water during sample preparation resulted in complete de-bonding of aggregate after sample preparation.

The results are presented in Tables 6-5 and 6-6 for % coated area, peak tensile stress, elongation length, and pseudo-energy. The general observation showed best performance from AC 15-5TR asphalt whereas PG 64-22 and AC 15-P performed the worst of the binders selected. There were no consistent trends observed for other parameters.

6.2.3.1 Investigation of the Effect of Sample Preparation Conditions

The sample preparation according to the Australian standard test method (T-238) involves water both during aggregate lay-down and curing operation. A similar approach was tried in the modified pull-out test for the limestone + AC 15-5TR combination. Although this combination was expected to show the best initial bonding, a complete lack of bonding was observed when the aggregate was placed in the asphalt (under water) at room temperature. A similar lack of bonding was also observed with the saturated-surface-wet aggregates even when the aggregate was placed without water over the asphalt. This may well be due to the fact that the T-238 test was developed for cutback asphalt, which is in a more liquefied form than asphalt cement used for these tests.

Another experimental factor of interest was surface temperature of the binder during aggregate placement. It was possible to place the aggregates at room temperature in the original T-238 test because of the cutback asphalt that has lower viscosity which facilitated the bonding between aggregate and binder. Since cutback asphalt was not used in this modified testing protocol, it was necessary to establish a suitable temperature for aggregate placement. Since a higher temperature will provide better wetting of the aggregate by the binder, to simulate field conditions, a temperature in the range of 100-110 °F was selected for aggregate placement. From the field constructability review, it was observed that pavement temperature during aggregate placement varied between 70 °F to 140 °F with 100-110 °F being the average. Although average temperature was used for most of the testing under the modified protocol, a few combinations were also tested at two extreme temperatures (70 °F and 140 °F). As mentioned earlier, the lower bound temperature (approximately 70 °F) did not show any bonding between aggregate and binder even with the best aggregate-binder combination.

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Table 6-5 Percent Coated Area from Pull-out Test.

Percent (%) Coated Area for PG 64-22					
Aggregate	< 25%	25 to 50%	50 to 75%	> 75%	Total (%)	Average
Limestone	35.8	0	48.86	15.34	39.77	
Light Weight	0	16.59	44.95	38.46	69.23	
Rhyolite	66.66	0	11.11	22.22	27.775	47.83
Gravel (Siliceous)	*Not Tested*					
Sandstone	9.09	9.1	63.63	18.18	54.545	

Percent (%) Coated Area for AC-15 5TR					
Aggregate	< 25%	25 to 50%	50 to 75%	> 75%	Total (%)	Average
Limestone	0	0	15.4	84.6	92.3	
Light Weight	37.5	0	25	37.5	50	
Rhyolite	0	0	30	70	85	73.46
Gravel (Siliceous)	0	10	50	40	70	
Sandstone	0	0	40	50	70	

Percent (%) Coated Area for AC-5+2% latex					
Aggregate	< 25%	25 to 50%	50 to 75%	> 75%	Total (%)	Average
Limestone	16.16	0	50	41.67	66.67	
Light Weight	0	0	50	50	75	
Rhyolite	0	58.34	25	16.66	58.33	64.52
Gravel (Siliceous)	0	12.5	50	37.5	68.75	
Sandstone	0	53.85	38.46	7.69	53.845	

Percent (%) Coated Area for AC-10+2% latex					
Aggregate	< 25%	25 to 50%	50 to 75%	> 75%	Total (%)	Average
Limestone	0	16.67	50	33.33	66.665	
Light Weight	0	16.67	83.33	0	50	
Rhyolite	0	25	50	25	62.5	58.08
Gravel (Siliceous)	0	10	30	60	80	
Sandstone	37.5	25	37.5	0	31.25	

Percent (%) Coated Area for AC-15P					
Aggregate	< 25%	25 to 50%	50 to 75%	> 75%	Total (%)	Average
Limestone	81.81	9.19	9	0	9.095	
Light Weight	60	30	10	0	20	
Rhyolite	10	20	70	20	65	37.82
Gravel (Siliceous)	0	28.6	71.4	0	50	
Sandstone	50	20	30	20	45	

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Table 6-6 Pull-out Test Results (additional parameters).

Test Parameters	Results for PG 64-22 Combined with Different Aggregates					
	Limestone	Light Weight	Rhyolite	Gravel (Siliceous)	Sandstone	Average
Peak Tensile Stress (psi)	31.02	56.32	86.06	n/a	117.02	72.61
Elongation, inch	--	--	0.28	n/a	2.97	1.63
Pseudo Energy (lb-ft/sq. in)	1.29	2.3	2.03	n/a	0.29	1.48

Test Parameters	Results for AC-15 5TR Combined with Different Aggregates					
	Limestone	Light Weight	Rhyolite	Gravel (Siliceous)	Sandstone	Average
Peak Tensile Stress (psi)	62.13	45.27	38.5	39.61	50.63	47.23
Elongation, inch	1.375	2.94	3.07	3.22	3.13	2.75
Pseudo Energy (lb-ft/sq. in)	7.05	9.1	9.86	10.6	13.25	9.97

Test Parameters	Results for AC-5+2% latex Combined with Different Aggregates					
	Limestone	Light Weight	Rhyolite	Gravel (Siliceous)	Sandstone	Average
Peak Tensile Stress (psi)	56.46	47.14	56.46	35.22	47.14	48.48
Elongation, inch	1.75	1.62	1.75	3.34	1.62	2.02
Pseudo Energy (lb-ft/sq. in)	7.6	6.78	7.6	9.73	6.78	7.70

Test Parameters	Results for AC 10+2% latex Combined with Different Aggregates					
	Limestone	Light Weight	Rhyolite	Gravel (Siliceous)	Sandstone	Average
Peak Tensile Stress (psi)	59.92	41.12	58.56	30.25	74.69	52.91
Elongation, inch	2	2.375	2.1	7.88	1.38	3.15
Pseudo Energy (lb-ft/sq. in)	10.11	8.18	7.52	3.65	8.35	7.56

Test Parameters	Results for AC-15P Combined with Different Aggregates					
	Limestone	Light Weight	Rhyolite	Gravel (Siliceous)	Sandstone	Average
Peak Tensile Stress (psi)	53.99	51.83	42.31	32.75	49.52	46.08
Elongation, inch	1.8	1.46	3	3.5	2.85	2.52
Pseudo Energy (lb-ft/sq. in)	9.095	20	65	50	45	37.82

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The effect of upper bound temperature (140 °F) was evaluated with two material combinations that were expected to perform the worst (PG 64-22 binder with limestone and lightweight aggregates). The results from these tests are presented in Figure 6-8. It is evident that higher construction temperature gives better initial bonding even with the worst aggregate-binder combination. This is true for all four parameters measured in the pull-out test. Therefore, it can be concluded that construction temperature is a factor that is significant for seal coat performance.
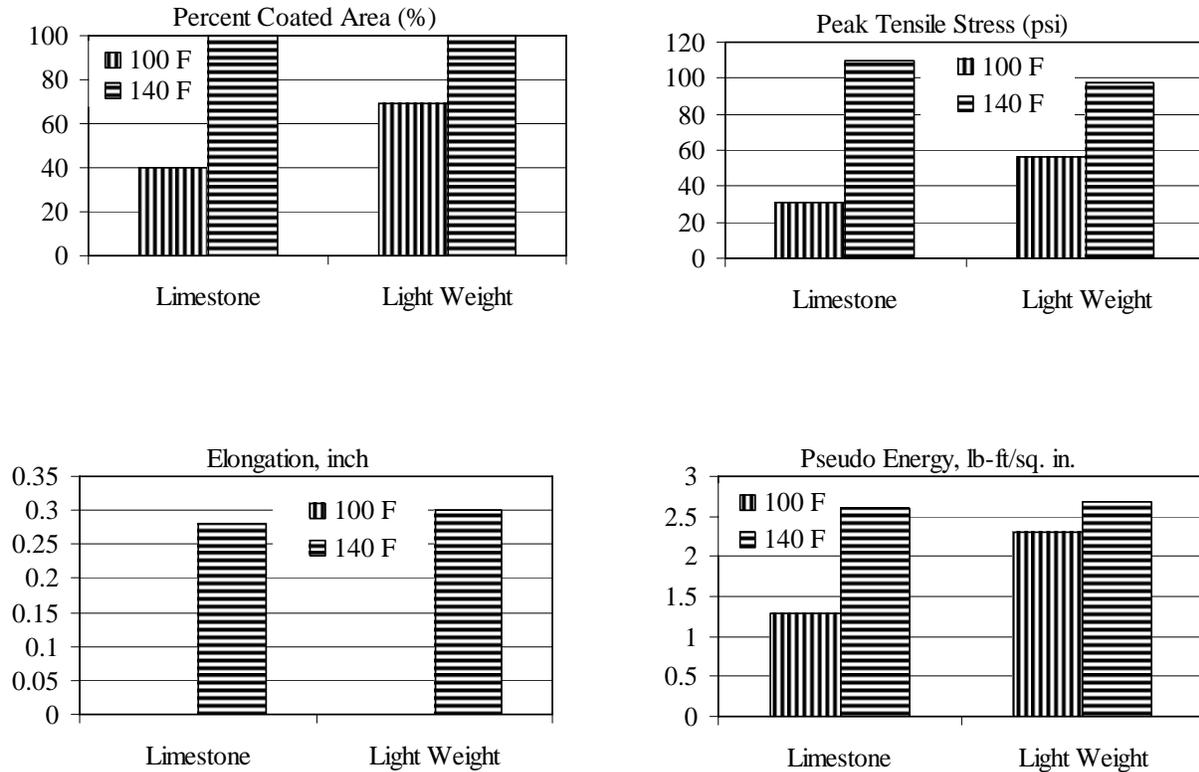
Figure 6-8 Effect of Temperature on Pull-Out Sample Preparation

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6.2.3.2 Pre-coated vs. Uncoated Aggregate

Use of pre-coated aggregate is very common in seal coat operations, especially when limestone is used because of its dustiness. The effects of precoating on seal coat performance were evaluated with the modified pull-out test in the laboratory. Four binder-aggregate combinations, matched with four field test sections, were selected for pull-out test. The binders used for this purpose were AC 5+2% latex, AC-15 5TR, AC 10+2% latex and AC-15P. Aggregates were obtained from two gravel and two limestone sources. The precoated gravels were from field test sections in Childress and Pharr districts, and pre-coated limestone aggregates were from field test sections in San Antonio and Lufkin districts. Aggregates from the same sources, but uncoated, were also tested for comparison. The results of these pull-out tests are presented in the Table 6-7. There appear to be no significant improvements in terms of any test parameters for pre-coated gravel for both projects, even though such improvements have been observed in the field. Even precoated limestone aggregate showed only marginal improvement in some test results. Peak tensile stress, elongation and pseudo energy did not show an improvement for pre-coated over uncoated limestone. In both limestone projects, the only test parameter which showed some improvement is percent coated area. The uncoated limestone may have had surface dust on them that can hinder the proper development of bonding between aggregate and binder. On the other hand, gravels usually do not have any dust on them. It is important to note here that a lot of subjectivity is involved in the estimation of percent area of the aggregate coated by asphalt.

Table 6-7 Pull-out Test Results Pre-coated vs Uncoated Aggregates.

Aggregate	Siliceous Gravel 1		Siliceous Gravel 2	
Binder	AC-15 5TR		AC-5+2% latex	
Test Parameters	Uncoated	Pre-coated	Uncoated	Pre-coated
Percent Coated Area (%)	70	68.75	68.75	75
Peak Tensile Stress, psi	39.61	31.03	35.22	23.04
Elongation, inch	3.22	2.34	3.34	3.85
Pseudo Energy, lb-ft/sq. in.	10.6	5.89	9.73	7.74
Aggregate	Limestone 1		Limestone 2	
Binder	AC-10+2% latex		AC-15P	
Test Parameters	Uncoated	Pre-coated	Uncoated	Pre-coated
Percent Coated Area (%)	66.665	93.75	9.095	83.33
Peak Tensile Stress, psi	59.92	25.75	53.99	32.13
Elongation, inch	2	16.41	1.8	10.83
Pseudo Energy, lb-ft/sq. in.	10.11	7.3	7.88	4.25

6.2.3.3 Evaluation of Environmental Effects

The pull-out test is intended to measure the development of initial bond between the aggregate and binder. In order to evaluate the effects of environmental factors on the bonding between aggregate and binder, laboratory prepared seal coat samples were subjected to freez-

thaw conditions described in Chapter 5 of this research report. The following three aggregate binder-combinations were selected for the investigation:

1. AC-15 5TR + Pre-coated LS (Abilene)
2. AC-15 5TR + Limestone (Dusty)
3. AC-5(+2% latex) + Gravel (Siliceous)

Field experience suggests that combination #1 should be the best performer, and combination # 3, the worst. Combination #2 was selected to show the adverse effect of dust on the aggregate-binder bond. Three samples were prepared and a few aggregates were pulled out before freeze-thaw cycling to estimate the initial bond strength. The remainder of the sample was subjected to three freeze-thaw cycles, and then subjected to the pull-out test. The results are compared in Figure 6-9, where only two parameters, percent coated area and pseudo-energy, are compared. Loss of bonding and coated area due to freeze-thaw was obvious in all three cases. Thus, this test procedure confirmed the effect of environment and aggregate surface dust on bond strength.

6.2.3.4 Evaluation of Emulsion

One emulsion binder (CRS-2P from Koch Materials, Brownwood plant) was evaluated with the modified pull-out test. In addition to a precoated limestone, all five sources of aggregate shown in Table 6-8 were used in this part of the experiment. The precoated limestone was from Brownwood district field test section. The results are presented in Table 6-8. All four test parameters show better performance for precoated limestone as observed in the field.

Table 6-8 Pull-out Test Results for Emulsion (CRS-2P).

| Aggregate | Percent (%) Coated Area | | | | | Peak Tensile Stress (psi) | Elongation inch | Pseudo Energy (lb-ft/sq. in.) |
	< 25%	25-50%	50-75%	>75%	Total (%)			
Limestone	87.5	0	12.5	0	6.25	25.94	1.53	3.78
Light Weight	42.5	43.22	14.28	0	28.75	20.7	1.92	2.9
Rhyolite	0	0	50	50	75	32.25	2.5	9.17
Gravel (Siliceous)	63.36	0	11.18	9.09	14.68	29.81	4.94	11.35
Sandstone	33.33	22.23	0	44.44	55.55	40.1	6.49	22.08
Pre-coated Limestone	0	0	0	100	100	34.12	7.74	22.8

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Percent Coated Area (%)

Pseudo Energy (lb-ft/sq. in.)
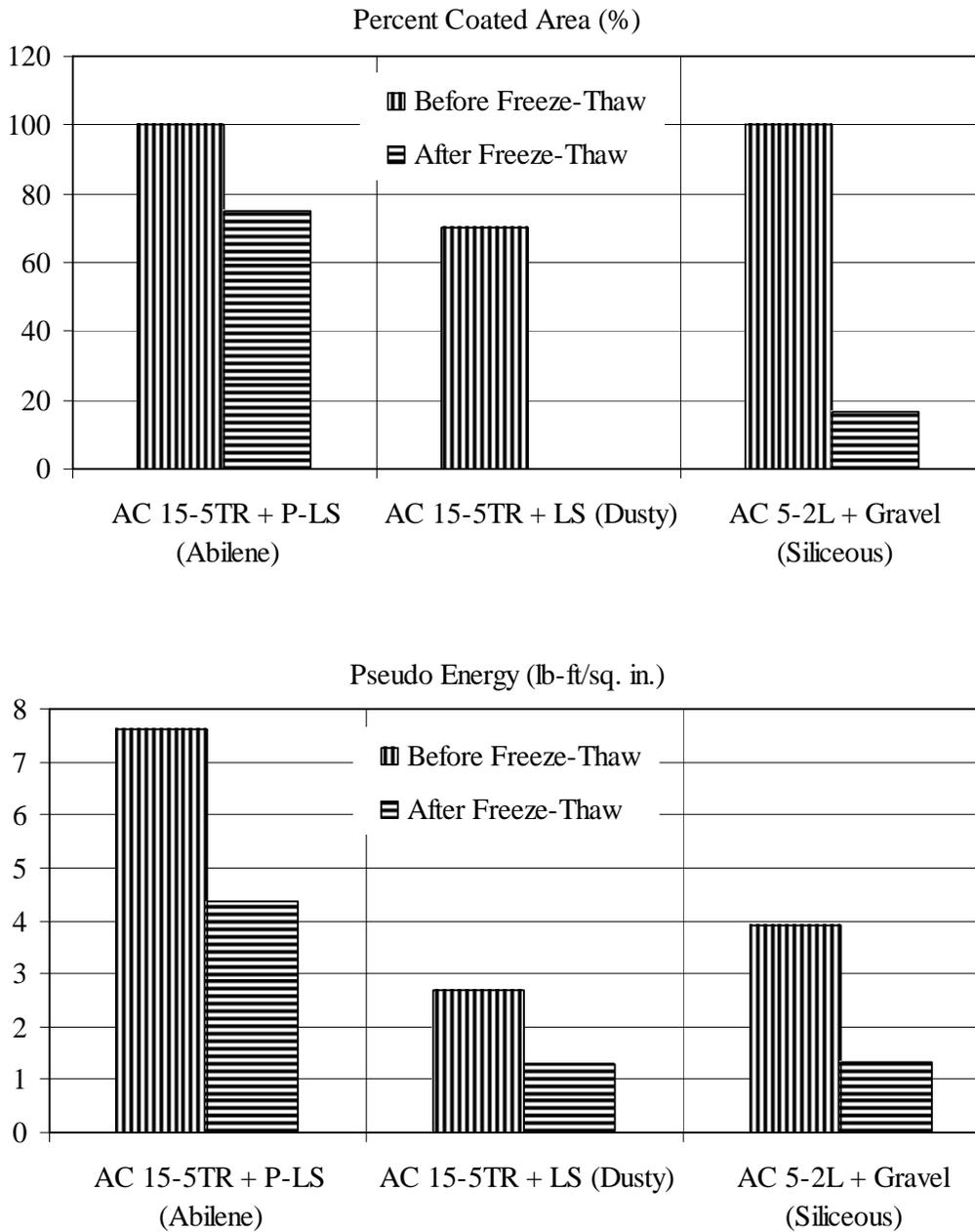
Figure 6-9 Effect of Freeze-Thaw and Dust on Pull-Out Test Performance.

6.2.4   Conclusions

Several commonly used aggregate-binder combinations were evaluated using the
Modified Australian Pull-out Test. No consistent trends were observed in terms of performance
measured by either peak tensile stress, asphalt elongation or pseudo energy. In general, AC-15

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5TR showed higher initial adhesion with all five aggregates indicated by the percent asphalt coated area. The same parameter also showed poor adhesion for PG 64-22 and AC-15P.

Introduction of any surface moisture or the presence of water during sample preparation showed a complete lack of bond between binder and aggregate when hot applied binder was used. Sample preparation temperature which represents the field construction temperature, showed a significant influence on initial bonding between aggregate and binder. Higher temperatures provided better bonding even for the supposedly 'poor' material combination. Pre-coating of dusty aggregate such as limestone was found to be beneficial to initial adhesion between aggregate and binder. Once again, percent coated area is the only parameter that showed this trend.

Although the pull-out test is supposed to measure only the initial adhesion between aggregate and binder, it was possible to use this testing protocol to investigate the effects of environmental factors during construction. A drop in percent coated area and pseudo-energy was observed after freeze-thaw cycling of several pull-out test samples. Introduction of some dust to the aggregate surface also had the same effect after freeze-thaw cycling. The pull-out test showed promise, but difficulties in the reproducibility of measurements appear to be a hindrance to its implementation.

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CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

State highway agencies such as TxDOT are responsible for the effective management of large pavement networks. For these agencies, preventive maintenance treatments such as seal coats (chip seals) are an effective pavement management tool. At any given moment, the majority of highway lane miles managed by TxDOT have seal coats as the wearing surface. Therefore, good performance of seal coats is vital for TxDOT to satisfy the needs of the traveling public and to make effective use of tax dollars.

The performance of seal coats depends a great deal on the effectiveness of aggregate-binder bond. Adequate bond between the aggregate and binder will minimize seal coat distresses such as loss of aggregate (raveling). In addition, better aggregate-binder bond will enable designers to optimize the use of material quantities, which, in turn, can reduce binder cost and also the likelihood of bleeding/flushing problems.

Highway agencies such as TxDOT are in the process of implementing performance-based materials specifications. These specifications must be developed based on test methods that better simulate field conditions they are exposed to, both during construction and in 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 aggregate-binder combinations. TxDOT uses a number of tests to determine, separately, the suitability of seal coat aggregate and binder for seal coats. Therefore, a need exists for a performance-based testing protocol that simulates the field conditions in the lab in order to assess seal coat aggregate-binder compatibility. This research project was commissioned by TxDOT to develop such a testing protocol that incorporates not only material factors, but also construction and performance conditions.

The researchers conducted several key research tasks. The first important task was to conduct a constructability review to identify factors that need to be incorporated in the proposed testing protocol. The constructability review included interviewing TxDOT personnel as well as visiting construction projects and monitoring them to track their performance.

As a part of the field constructability review, the research team visited 15 of the 25 TxDOT districts and test sections were selected from 14 of those districts. The constructability review showed that seal coat performance is 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 for hot seal coat asphalt in the distributor is typically 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 towards the existing pavement temperature within a 2-3 minutes 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 aggregate application 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. For the test sections that were visited by researchers, the ambient

temperature at the time of construction varied between 79 °F and 100 °F, and the pavement surface temperature varied between 84 °F and 142 °F. Emulsified asphalt (CRS-2P) was used as the seal coat binder in 3 of the 14 test sections, while the remaining 11 used hot asphalt.

A second phase of the constructability review was carried out to monitor the performance of seal coated test sections approximately one year after they were constructed. One general observation made in this 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 during 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. 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 strength at the aggregate-binder interface. In general, most of the test sections visited by the researchers performed very well. Four district test sections experienced aggregate retention problems, and efforts were made in the laboratory testing phase to verify such problems. In one of those four sections, aggregate retention problems were attributed to a heavy thunderstorm that occurred the night after the construction was completed.

The next phase of the research was to conduct preparatory research to develop the new testing protocol that is both effective and economical to be used for seal coat applications. After about a year of investigative research involving trial-and-error procedures, two testing protocols were selected. The first of these two protocols is the modified Net Adsorption Test (NAT), based on the work done by the Strategic Highway Research Program (SHRP) and by the National Roads Authority of Ireland.

Moisture poses a major threat to the satisfactory functioning of asphalt pavements, and there are only a very few laboratory test methods available that can accurately predict the effects of moisture on asphalt-aggregate combinations. In the late 1980's, The Strategic Highway Research Program (SHRP) developed a test method to evaluate the moisture susceptibility of hot mix asphalt mixes. This new testing protocol was released as *SHRP Standard Test Method 1013: Measurement of Initial Asphalt Adsorption and Desorption in the presence of Moisture* (Curtis et al. 1990). This test method produced evidence to suggest that adhesion failure is primarily caused by undesirable aggregate characteristics and to a lesser extent by the asphalt binder characteristics. Based on research conducted at the University of Ulster in Ireland, Woodside et al (1993) presented a modified version of the SHRP test. Walsh et al. (1995) modified the test method of Woodside et al. even further to extend its applicability to seal coats (i.e. chip seals). This modified net adsorption test (NAT) developed by Walsh et al., and later implemented by the National Roads Authority of Ireland, was used in this research to evaluate the effectiveness of aggregate-bitumen bond in seal coats.

They recommended the modified NAT to identify suitable hot asphalt-aggregate combinations. This research project conducted an extensive evaluation of the modified NAT using seal coat aggregates and binders commonly used in TxDOT highways. This evaluation concluded that modified NAT is suitable to select good aggregate-binder

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combinations when uncoated aggregates are used with hot asphalt binders, and also to select good precoating binder-aggregate combinations.

One of the limitations of the Net Adsorption Test protocol is that it cannot be used for emulsified asphalt seal coats and the seal coats that use precoated aggregate. The researchers explored the viability of a number of tests for emulsified asphalt-aggregate bond, and the only test method that had some promise to evaluate the effectiveness of adhesion bond was the zeta-potential test. However, this test uses finely ground aggregate particles, and the practical validity of its use in day-to-day TxDOT seal coat work was questionable. The National Roads Authority of Ireland used the modified NAT to also assess the effectiveness of emulsified asphalt-aggregate combinations. However, since emulsified asphalt could not be used directly in the test procedure, its base asphalt residue was extracted and used. This research team does not recommend this approach for TxDOT. However, since a large majority of TxDOT seal coat binders comprises of hot asphalts, the modified NAT can still play a vital role in identifying effective aggregate-binder combinations. The researchers recommend that for these two types of seal coat materials, (i.e. when emulsified asphalt and precoated aggregates are used in seal coats) the *Performance-Based Seal Coat Aggregate-Binder Compatibility Test* developed in this research shall be used. This performance-based test method is also recommended for the materials tested using the modified NAT because of the ability of that test method to simulate a number of field conditions that influence seal coat performance.

The experimental design for tests using the modified Net Adsorption Test used two factors; asphalt binder and aggregate. Using the test results, three parameters representing initial adsorption, final adsorption and percent net adsorption were calculated and reported. The percent initial adsorption provides a measure of the effectiveness with which the asphalt cement can wet the aggregate. The percent net adsorption on the other hand provides a measure of the resistance of aggregate-asphalt bond to stripping. Therefore, it is important that both these parameters are used to investigate the performance of aggregate-binder bond. Asphalt cement and aggregate, each from five different sources, were selected for the first round of tests. Five additional asphalt cement-aggregate combinations were also tested for material samples taken from field projects.

A closer look at the percent initial adsorption and percent net adsorption data for different aggregates and binders tested in this research show some interesting observations. In the case of percent initial adsorption, which shows the affinity of an asphalt for an aggregate, the values for each aggregate type, when averaged for the 5 binder sources tested shows 61, 56, 56, 50 and 44 percent for limestone, rhyolite, lightweight, sandstone and siliceous gravel respectively. This appears to show expected trends based on the acid-base interactions between aggregate and binder as well as the acidity (or basicity) of the aggregate. When the percent initial adsorption results are averaged by binder grade for the 5 aggregate types tested, the results show 58, 58, 55, 52 and 44 percent for AC-15 5TR, AC-5 (with latex), AC-15P, PG 64-22 and AC-10 (with latex) respectively. Statewide performance of respective binder grades in seal coats appears to be in agreement with these results.

It is important to note here that the validity of these results is limited to the material samples tested and the specific test conditions. Due to the high variability of

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aggregate types and binder grades from one source to the next calls for testing of specific aggregate-binder combinations a district plans to use.

As for percent net adsorption results, it can be seen that approximately 60 percent of all combinations tested were rated as having 'good' stripping resistance according to SHRP evaluation criteria. Only one aggregate-binder combination (Limestone and PG 64-22) was rated as having 'poor' stripping resistance. When the percent net adsorption values are averaged for the five aggregate types tested, the values were 77, 75, 73, 68 and 65 percent for rhyolite, sandstone, siliceous gravel, lightweight aggregate and limestone respectively. These results are somewhat unexpected, but when one considers the complex chemical and physical nature of the aggregate and binder involved, it deserves serious consideration. One possible reason for the unexpectedly low value for limestone is the low percent net adsorption value for the Limestone-PG 64-22 combination which had 45 percent net adsorption. All the other NAT values for limestone were in the 64 to 82 percent range. Due to the unexpectedness of this result, this combination (limestone+PG 64-22) was tested 3 times (with 3 replicates) instead of the usual one time.

The percent net adsorption values when averaged by the binder grade were 78, 76, 71, 68 and 65 percent for AC-5 (with latex), AC-15 5TR, AC-10 (with latex), AC-15P and PG 64-22 respectively. It is difficult to make any judgments on these results due to the high variability in performance for the same binder grade from different sources.

It is important to note that in terms of the performance of aggregate-binder bond, either the percent initial adsorption or the percent net adsorption alone cannot provide an accurate prediction of the performance of the material combination. However, when these two parameters are combined, a reasonable assessment of the bonding effectiveness can be made. The researchers combined the values for these two test parameters by both the aggregate type and the binder grade, and the following rankings ensued.

- For the five aggregate types, the rankings (from best to worst) were Rhyolite, Limestone, Lightweight aggregate, Sandstone and Siliceous Gravel.
- For the five binder grades, the rankings (from best to worst) were AC-5 (with latex), AC-15 5TR, AC-15P, PG 64-22 and AC-10 (with latex)

These rankings appear to follow the general pattern of field performance observed for seal coats around the state. It is important that these rankings *not* be generalized to evaluate the effectiveness of specific material combinations involving one specific aggregate source with one specific binder source. For such an evaluation, this test can be conducted to evaluate the specific material combination involved. Furthermore, this limited list of rankings can be used to explore ways to improve the performance of seal coats that use locally available materials, particularly the aggregates. One example of such improvements would be the use of antistripping agents in seal coats.

The second testing protocol recommended to TxDOT is the *Performance-Based Seal Coat Aggregate-Binder Compatibility Test*, which is the primary testing protocol developed in this research project. As the title implies, this is a truly performance-based testing protocol that can be used to evaluate the compatibility of various seal coat aggregate-binder combinations under a variety of construction situations and performance conditions. Since the success of a seal coat is determined to a large extent

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on the conditions under which it is placed and performed, this would be a very attractive testing option for TxDOT to evaluate their seal coat material compatibility under local construction and performance conditions. Some of these local conditions are pavement temperature at the time of sealing, time delay between binder application and aggregate spread, time delay between aggregate spread and rolling (for hot asphalt), aggregate dust content, percent embedment of aggregate and climatic conditions soon after the seal coat is placed.

This protocol is easy to perform and the equipment needed can be assembled at a reasonable cost, and the test can be easily performed in a TxDOT district lab with its available equipment. The process of developing the new protocol started with a literature review on the subject, which revealed that available test methods are unable to predict performance of the bonding primarily because they do not effectively simulate the field conditions. Seal coat specimens were prepared on 6 in. by 6 in. square aluminum plates and were subjected to environmental conditioning to simulate, in an accelerated manner, the field conditions to which it is subjected to. This was done by subjecting the samples to wetting-drying and freezing-thawing cycles. The wetting/drying and freezing/thawing processes had to be effectively reproduced in the laboratory for the test procedure to be effective in predicting field performance. The findings from the constructability review were very important to the test protocol development in two ways. It helped the researchers identify scenarios of incompatibility both from the standpoints of construction activities and performance conditions. Furthermore, researchers were able to collect field seal coat specimens from actual projects. This was achieved by dropping 6 inches square steel plates that were picked up after the seal coat process was complete. Then, they were brought back to the Texas Tech Asphalt Laboratory for testing with the newly developed test protocol. A year later, a follow-up performance evaluation was conducted for the test sections.

When developing the testing protocol, various specimen conditioning methods and durations were tried but 3 cycles of freezing-soaking followed by 48 hours of soaking at room temperature was found to be the most effective specimen conditioning protocol. Each freeze-soak cycle involved 16-hours soaking in water at room temperature followed by 8 hours of freezing at -25° C (-13° F). After this conditioning process, the specimens were subjected to impact loading (3 drops) with the Modified Proctor Hammer (MPH). Once the testing protocol was finalized, the specimens brought from the field visits were tested with this protocol. Comparison of findings from field test section performance evaluation and the results from testing of field specimens using the new laboratory testing protocol provided excellent agreement between field observations and results from the new testing protocol, thus enhancing the credibility of the new testing protocol.

The results from field specimens for 13 test sections matched with the follow-up study results, while 2 sections did not. One of these two sections experienced a severe thunderstorm soon after the construction, which probably caused the loss of aggregate experienced in the field. The field specimens from this site that were brought back to Lubbock did not experience the adverse weather conditions, and did not show aggregate loss. This is the likely cause of discrepancy between the field performance and lab results. In the other test section where the field performance did not match with field specimen data, the field section performed well, but the specimens indicated significant aggregate loss. Follow-up field observations indicated that in this section, the traffic

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levels were significant, and flushing was observed. The specimens brought from the field were not tested for a year, and no additional compaction was done on them in the lab. The researchers believe that these two factors contributed to the section not showing aggregate loss.

After the testing protocol was perfected, the laboratory testing using that protocol 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. One of the key aspects of developing a testing protocol such as this one is to ensure that it can accurately distinguish between aggregate-binder combinations with different performance characteristics. To do this, 'good' and 'poor' performing aggregate-binder combinations (one from each category) were tested using the new testing protocol. The 'good' combination was AC-15 5TR + Limestone, and the 'poor' combination was AC-5 (with 2% Latex) + Siliceous Gravel.

The dust content was included as an experimental factor to simulate the dust generation potential of a non-precoated aggregate during transportation, handling and storage. It was only used in the testing program for crushed limestone because dust is not a significant problem with siliceous gravel. Simulation of three dust levels (no dust, medium dust and high dust) was achieved using a customized procedure using the Micro-Deval (MD) test apparatus. Washed and oven dried aggregate was used for the 'no dust' state, while 2 minutes and 5 minutes of spinning in the Micro-Deval apparatus (without the steel charge) were used to represent 'medium dust' and 'high dust' states respectively. Results from the new test method showed that it can effectively identify between seal coat aggregates with different surface dust contents. In addition, the test procedure also clearly demonstrated the difference in performance between seal coats with different percent aggregate embedment, roller time delay and pavement (surface) temperature at the time of sealing.

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 specimens. 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 (i.e. if aggregates are spread before or after the emulsion break) on bonding. The aggregates with two different moisture contents (i.e. air-dry and SSD), were also used in preparing specimens. The SSD aggregates performed well with emulsions thus confirming the general experience from field projects.

The precoated aggregate phase included testing the effects of precoating binder type and precoating level for different aggregate-binder combinations. Limestone precoated with PG 64-22 and Lightweight precoated with CSS-1h were the precoating combinations tested. The seal coat binders used with precoated aggregate included CRS-2P, AC-15 5TR, and AC-15P. Two precoating coverage levels (50% and 90% of the aggregate surface covered) were used in the experiment, and the coverage was assessed from visual evaluation. The performances of precoated aggregates indicate that the bonding problems associated with dust content can be solved with precoating. The

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uncoated aggregates showed significant aggregate loss. For the seal coat specimens prepared with AC-15P, the test identified the effect of different precoating levels. Both emulsion and AC precoated aggregates performed very well.

The results for different material combinations under various experimental conditions suggest that limestone should be precoated with hot asphalt. The use of AC-5 with siliceous gravel is not recommended due to high probability of failure. On the other hand, use of CRS-2P with non-precoated lightweight aggregate is recommended. When siliceous gravel is used with CRS-2P, steps must be taken to spread the aggregate before the emulsion breaks. When AC-15P is used with limestone, the aggregate should be precoated, preferably to at least 90% precoating level.

Two additional test methods were also used in this research to evaluate aggregate-binder bonding characteristics. They are the *Interfacial Bonding Test (IBT)* and the *Modified Australian Pull-Out Test*.

In the *Interfacial Bonding Test (IBT)*, the bond between a cube-shaped aggregate and binders were investigated using a controlled pull-out test procedure. It has been reported in the literature that this bonding is one of the key factors that affect the aggregate retention performance of the seal coats. However, there are also various other factors that influence bonding. The experimental factors used for hot asphalt were dust content and the binder temperature at the time of aggregate application. For emulsified asphalt, emulsion curing time, aggregate application with respect to breaking, and aggregate moisture content were investigated.

The effect of binder temperature at the time of aggregate application on bonding was well recognized with the IBT. Both the material combinations tested responded to this factor but AC-15 5TR + Limestone combination performed much better than the AC-5 (with 2% Latex) + Rhyolite Gravel combination. This may be due to the better surface texture of limestone aggregate. The binder AC-5 (with 2% Latex) is less viscous than the AC-15 5TR at 100 °F and 140 °F. One could assume that AC-5 will more effectively penetrate into the pores of the aggregate due to its low viscosity and provide better bonding, but the results did not confirm this hypothesis. Differences in aggregate surface texture and in chemical interactions may be the likely causes of these apparent inconsistencies.

The IBT also recognized differences in the bond due to aggregate dust content. Due to the lack of control over how much dust was applied, the results do not show any clear trend for 0% embedment. On the other hand, results for the embedded aggregate suggest that bond strength decreases significantly when dust is present, particularly for the limestone.

Comparison of the two material combinations shows that AC-15 5TR + Limestone combination is the better of the two when no dust is present on the aggregate surface. The performance of this material combination deteriorated 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 of 10 hours. The IBT was successful in identifying the before/after break effect for SSD aggregates, while it was unsuccessful for air dry aggregates. Moreover, the IBT distinguished the difference in performance between two curing times.

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The results discussed above indicate that 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.

Several commonly used aggregate-binder combinations were evaluated using the *Modified Australian Pull-out Test*. No consistent trends were observed in terms of performance measured by either the peak tensile stress, asphalt elongation or the pseudo-energy. In general, AC-15 5TR has shown higher initial adhesion with all five aggregates as measured by the percent of coated area. The same parameter also showed poor adhesion for PG 64-22 and AC-15P.

Introduction of any surface moisture or the presence of water during sample preparation showed a complete loss of bond between binder and aggregate when hot asphalt binder was used. Sample preparation temperature which represents field construction temperature showed a significant effect on the initial bonding between aggregate and binder. Higher temperatures provided better bonding even for the supposedly 'poor' material combination.

Pre-coating of dusty aggregate such as limestone was found to be beneficial to the initial adhesion between aggregate and binder. Again, percent pre-coated area is the only parameter that showed this trend.

Although the pull-out test is supposed to measure only the initial adhesion between aggregate and binder, it was possible to use this testing protocol to include the effects of environment. A drop in percent coated area and pseudo-energy was observed after freezing and thawing of several pull-out test samples. Introduction of some dust to the aggregate surface also had the same effect after several freeze-thaw cycles. The pull-out test showed promise, but the difficulties in the measurement and lack of reproducibility of results appear to be a hindrance to its implementation.

Even though these two test methods (IBT and Pull-Out) provided useful information, they are not a part of the testing protocol recommended for implementation by TxDOT. The following two products have been developed for implementation in this research, and they are included in Appendices A and B of this report.

1. Draft Test Procedure(s) for Aggregate-Binder Compatibility, including Evaluation Protocol for Seal Coat Aggregate and Binder Selection
2. Draft Updated Specifications for Seal Coats and Surface Treatments including New Acceptance Criteria

The primary objective of this research project was to develop a testing protocol that combines sound technical principles with practicality and economy. The researchers placed a heavy emphasis to incorporate experimental parameters that represent construction and performance conditions of seal coats. This is particularly important for seal coats whose performance is significantly affected by such factors. The researchers feel comfortable about the new testing protocol, and it is a product that districts can use to come up with aggregate-binder combinations that work with local conditions such as climate and aggregate availability.

As outlined in this report, extensive testing was conducted during this research project in all its phases. However, these tests were done with the purpose of demonstrating the validity of the test protocol to predict field performance and its ability

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to distinguish between 'good' and 'poor' material combinations. More tests must be conducted and verified with field performance data before this test protocol is considered for implementation. Furthermore, the testing protocol can be used to evaluate aggregate-binder combinations for districts by taking into consideration the local conditions of the area, including climate and availability of aggregates. The researchers believe that guidelines can be developed using these two testing protocols to provide information for TxDOT districts to update specifications and general notes used with seal coats and surface treatments.

Draft Test Procedures for Seal Coat Aggregate-Binder Compatibility

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Seal Coat Binder-Aggregate Compatibility

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Tex-XXX-X   Compatibility Test and Evaluation Protocol for Aggregate and Precoating Hot Asphalt Binder

Overview:

This test method is almost identical to "The Net Adsorption Test for Chip Sealing Aggregates and Binders" developed by Walsh et al. (1995). This test method developed by Walsh et al. to evaluate the compatibility between seal coat aggregates and binders was based on a similar method developed for asphalt-aggregate systems in general, by Curtis et al. (1993). Both test methods indicated above include an asphalt *adsorption* phase (on the aggregate surface) from an asphalt-toluene solution and an asphalt *desorption* phase from the aggregate surface in the presence of water (i.e. stripping), which is considered as a measure of the strength of bond between the two materials. It is suggested that the *percent net adsorption* calculated as proposed by Curtis et al. (1993) be used to evaluate the appropriateness of an asphalt to precoat a particular surface treatment aggregate.

Apparatus:

1. Mechanical Shaker Table (Figure 1): Equipped with 8 holders for 500 ml Erlenmeyer flasks.
2. Spectrophotometer (Figure 2): Capable of providing a continuous 410 nm wavelength with an accuracy of +/- 2nm, holding standard 10 mm path length cuvettes.
3. Spectrophotometer cuvettes: Capable of 4.5 ml and 10 mm path length.
4. Erlenmeyer flasks with the capacity of 500 ml.
5. Volumetric flasks with capacities of 25 ml and 1000ml.
6. Filter paper: Whatman No. 42, 125 mm in diameter.
7. 250 ml graduated glass cylinder.
8. 10 ml pipettes
9. Analytical balance with precision of up to 0.001 grams.
10. Aggregate drying oven capable of maintaining 135 °C.

Reagents:

1. Toluene: UV/Spectroanalyzed grade
2. Distilled water

Test Procedure:

The test takes nearly 24 hours to complete and the test procedure can be divided into the seven steps outlined below:

1. Calibration of the Spectrophotometer to Measure Light Absorbance

The spectrophotometer must be calibrated using the procedure recommended by its manufacturer. This procedure should result in a calibration curve for each asphalt binder to be tested.

2.   Preparation of Aggregate Samples

For each aggregate-asphalt combination, prepare four 50-gram aggregate samples graded according to the standard grading recommended by the National Roads Association of Ireland (NRA) shown in Table 1. The aggregate must be dried uncovered in an oven for approximately 15 hours at a temperature of 135 °C. The aggregate samples must be removed from the oven at least 15 minutes prior to the beginning of the test.

Table 1. Recommended Aggregate Gradation for NRA Test Method (Walsh et al. 1995)

Sieve Size	Percent Retained	Wt. Retained (g)
2.36 mm	8.0	4.3
1.18 mm	25.0	13.5
600 μ m	17.0	9.1
300 μ m	23.0	12.4
150 μ m	14.0	7.5
75 μ m	6.0	3.2
		Total 50

3.   Preparation of Stock Solution

Measure approximately 0.6-grams (±0.001g) of asphalt binder and dissolve it in 600 ml of toluene in a 1000 ml volumetric flask. This will produce an asphalt-toluene stock solution with an approximate concentration of 1g/liter.

4.   Measure Initial Light Absorbance of Stock Solution

Take four milliliters of the stock solution prepared in Step 3. Place this solution sample in a spectrophotometer cuvette and measure the initial light absorbance of the solution using the spectrophotometer at a wavelength of 410 nm. Use the calibration curve(s) developed in Step 1 to obtain the corresponding concentration (A_1) of the solution. With some spectrophotometers, the solution may have to be diluted to a known concentration before the measurement can be taken. Use a clean 25 ml volumetric flask for this purpose when needed.

5.   Preparation of Test Samples and Control Sample in Erlenmeyer Flasks

Place each of the four aggregate samples in a separate 500 ml Erlenmeyer flask. Of the four Erlenmeyer flasks, three contain test samples and the fourth contains the control sample. Add 140 ml of freshly-prepared asphalt-toluene stock solution to each of the three flasks containing the test samples, and add 140 ml of pure toluene to the flask containing the control sample. Place all four Erlenmeyer flasks on the mechanical shaker.

6.   Adsorption Phase

Shake the four flasks for six hours at 300 rpm. At the end of the shaking period, take four milliliters of the solution from each of the four Erlenmeyer flasks. Place this solution sample in a spectrophotometer cuvette and measure the light absorbance of the solution after the adsorption phase using the spectrophotometer at a wavelength of 410 nm. Use the calibration curve to obtain the corresponding concentration (A_2) of the solution. With some spectrophotometers, the solution may have to be diluted to a known concentration before the measurement can be taken. Use a clean 25 ml volumetric flask for this purpose when needed.

7.   Desorption Phase

Add 2 ml of distilled water to each Erlenmeyer flask and shake for a further 15-17 hours. At the end of this period, take four milliliters of the solution from each of the four Erlenmeyer flasks. Place this solution sample in a spectrophotometer cuvette and measure the light absorbance of the solution after the desorption phase using the spectrophotometer at a wavelength of 410 nm. Use the calibration curve to obtain the corresponding concentration (A_3) of the solution. With some spectrophotometers, the solution may have to be diluted to a known concentration before the measurement can be taken. Use a clean 25 ml volumetric flask for this purpose when needed.

Calculation Procedure

This proposed calculation procedure is similar to that recommended by Curtis et al. (1995).

Initial Adsorption (A_i), i.e. the amount of bitumen initially adsorbed onto the aggregate surface;

$$A_i = VC*(A_1-A_2)/ (WA_1)$$

Net Adsorption (A_n), i.e. the amount of bitumen remaining on the aggregate after water is added;

$$A_n= V_rC (A_1-A_3)/ (WA_1)$$

% Net Adsorption (%A_n), i.e. the percentage of initially adsorbed bitumen remaining on the aggregate after the desorption phase;

$$\%A_n = (A_n / A_i) * 100$$

Where:
A_i = Initial adsorption, mg / g
V = volume of solution in the flask, 140 ml
W = weight of aggregate, in grams
C = Initial concentration of bitumen in solution, 1g / l
A_1 = Initial absorbance reading
A_2 = Absorbance reading after 6 hours
A_3 = Absorbance reading after 16-17 hours
A_n = Net adsorption, mg/g
V_r = Volume of solution in the flask at the time A_3 is obtained, 136 ml.

Evaluation Protocol for Aggregate-Binder Compatibility:

The criteria selected by SHRP to evaluate the performance of aggregate-binder adhesion based on percent Net Adsorption results ($\%A_n$) are recommended for this test (Table 2).

Table 2. Evaluation Criteria for Aggregate-Binder Adhesion (based on Walsh et al. 1995)

Percent Net Adsorption ($\%A_n$)	Expected Performance of Aggregate-Binder Bond
>70	Good
55-70	Acceptable
<55	Poor

References:

G. Walsh, I. L. Jamieson and M. O'Mahoney, "The Net Adsorption Test for Chip Sealing Aggregates and Binders", *RC 372*, National Roads Authority, Ireland, November 1995.

C. W. Curtis, K. Ensley and J. Epps, "Fundamental Properties of Asphalt-Aggregate Interactions Including Adhesion and Absorption", *Research Report SHRP-A-341*, Strategic Highway Research Program, National Research Council, Washington, DC, 1993.

Figure 1. Mechanical Shaker

Figure 2. Spectrophotometer

0-4362 P1 – Part 2

Tex-YYY-Y   Performance-Based Compatibility Test and Evaluation Protocol for Aggregates and Binders used in Seal Coats and Surface Treatments

Overview:

This test method is recommended to predict the bonding compatibility of aggregate and bituminous binder used in seal coats and surface treatments. This is a performance-based test that incorporates field condition of materials, construction conditions and performance conditions during the first year after the seal coat or surface treatment is applied.

Apparatus:

The following apparatus is required:

1. 6 in. x 6 in. x 1/8 in. thick (15 cm x 15 cm x 3 mm thick) Aluminum plates: It is recommended that for each binder-aggregate combination to be tested, a minimum of three replicate specimens be used. Therefore, a sufficient number of Aluminum plates need to be secured. The plates must of uniform thickness and should be free of warping. The thickness of asphalt film could be as little as 1.25 mm, and therefore, the specimen plates must be flat and level to an accuracy of 0.1 mm.
2. 6 in. x 6 in. x ¼ in. thick (15 cm x 15 cm x 6 mm thick) steel plate
3. Asphalt oven capable of maintaining a temperature of up to 200 °C (400 °F)
4. Hot plate capable of maintaining a temperature of up to 65°C (150 °F)
5. Weighing balance readable to 0.1g
6. Adjustable blade and rails to apply the binder film of uniform thickness
7. Automated chip spreader (Figure 1). The aggregate may also be spread by hand.
8. Micro-Deval Test Equipment (ASTM D 6928 o AASHTO T 327) for dust generation in aggregates
9. Water tank to soak laboratory prepared specimens: The size of tank depends on the number of specimens soaked at one time. If several specimens are soaked at the same time, it is recommended that a rack be used to keep specimens in the tank so that they do not rest on each other.
10. Freezer capable of maintaining -25 °C (-13 °F)
11. Modified Proctor hammer used in soil compaction test Tex-113-E
12. Specimen mounting apparatus (Figures 2-5)
13. Steel drum roller padded with ¼ in. thick rubber sheet (Figure 6), and capable of producing a contact pressure of 45 psi on 6 in. wide seal coat specimen, assuming a contact width of 2 inches.
14. Non-contact thermometer: This thermometer must be calibrated according to guidelines given by the manufacturer.
15. Miscellaneous items such as brush, small scoop, small can, heat resistant gloves, etc.

Materials:

The following materials are needed for the test.

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1. Bituminous binder
2. Aggregate (precoated or uncoated)
3. Antistripping agent (if needed for laboratory precoating of aggregate)

Specimen Preparation:

This step consists of the following activities:
- Preparation of aggregate
- Preparation of test specimens

Preparation of Aggregate

1. Secure a representative sample of aggregate (cover stone) for the seal coat.
2. Sieve the aggregate to obtain a sample of Grade 4S or other approved grade.
3. Take 1500 g of aggregate and soak it in water for 6 hours.
4. Wash the aggregate in running water for 5 minutes and oven-dry at 110°C for 24 hours.
5. Put the oven-dried aggregate to the Micro-Deval test machine and operate the machine for 2 minutes without the steel charge (aggregate only). If the aggregate is sampled directly from a field stockpile, it may be used as is, if desired.
6. The aggregate sample is now ready for specimen preparation. If precoating is desired, aggregate can be precoated at this stage using a laboratory mixer.

Preparation of Test Specimens

Follow these steps to prepare material and equipment for this procedure.

1. Place an appropriate quantity of binder into a small can for easy handling.
2. Heat the binder in the oven to the desired seal coat application temperature.
3. Calculate the amount of binder needed for each specimen plate using the following equation:

   For a binder application rate of B gal/yd^2,
   The corresponding thickness of the asphalt film T, in millimeters, can be calculated by the equation:

   $$T = 4.527 \cdot B \qquad\qquad \text{Eq. 1}$$

   The mass of binder M_B, in grams, needed for one specimen plate at the application rate of B can be calculated by the equation:

   $$M_B = 105.198 \cdot B \, G_b \qquad\qquad \text{Eq. 2}$$

   G_b is the specific gravity of asphalt binder.
4. Adjust the asphalt sweeper blade height to provide the film thickness T calculated using Eq. 1 above.

5. Place sticker tape to the four sides of the specimen plate so that the tape rises to a height of ¼ inch above the specimen plate. This tape will serve as an overflow barrier for asphalt in the specimen plate. Measure the mass of plate and tape and record it as M_P.

6. Place the aluminum plate on the hot plate and adjust the hot plate temperature dial until the aluminum plates attain the desired temperature. This temperature is related to the desired pavement temperature at which seal coat is applied in the field.

7. Pour M_B grams of binder to the end of specimen plate and run the sweeper blade across the specimen plate to provide an asphalt film of uniform thickness T. Steps 4, 5 and 6 may be replaced by an appropriate binder spray mechanism that is capable of providing an asphalt film of uniform thickness.

8. Weigh the plate with the binder in it and record as M_{PB}. If the exact binder content needed to get the desired asphalt rate is used,

    $$M_{PB} = M_B + M_P$$

9. Measure the temperature of binder in the plate at intervals of 15 seconds using a properly calibrated non-contact thermometer.

10. Place the aggregate to the binder film when the binder cools down to the desired aggregate application temperature. It is recommended that this temperature be set as the temperature of the hot plate. The quantity of aggregate used should be determined based on the aggregate application rate used in the field. This can be calculated using the procedure outlined below:

    If the aggregate application rate is 1:A (i.e. 1yd^3 of aggregate used over A yd^2 of pavement),

    Loose volume of aggregate needed for one specimen $= 0.520833/A$ ft^3

    If the unit weight of aggregate is U lb/ft^3,

    Mass of aggregate sample needed for one specimen $= (0.520833/A)\ U$

    Do not place any aggregate within ½ inch from the four edges of the plate. The aggregate application may be done either by hand, or by automated means. The total time taken to apply aggregate must be within 15 seconds. It is recommended that aggregate particles be dropped onto the binder from a height of approximately 12 inches from the specimen plate. This may be of particular significance for emulsified asphalt seals.

11. Move the specimen from the hot plate to the rolling area.

12. Measure the temperature of the binder at 15-second intervals using a properly calibrated non-contact thermometer.

13. Roll the seal coat specimen with the roller after the desired time has elapsed between aggregate application and rolling. It is recommended that five roller passes be used to roll the specimen. One forward pass or one backward pass is treated as one roller pass. The roller should have a contact pressure of approximately 45 psi.

14. Count the number of aggregate particles in the specimen and record it as C_A.

15. Weigh the plate + binder + aggregate and record as M_T. The mass of aggregate (M_A) in the specimen can be calculated by,

$$M_A = M_T - M_P - M_B$$

16. Store the specimens under room temperature for 96 hours until specimen conditioning begins.

Specimen Conditioning:

Follow these steps for specimen conditioning

1 Once the specimens are cured for 96 hours, soak the specimens in water at room temperature for 16 hours.
2 At the end of 16 hours of soaking, transport the specimens to the freezer and freeze at the desired temperature for 8 hours. The freezing temperature must be selected to represent the coldest temperature experienced in the region.
3 Repeat steps 1 and 2 for two more cycles such that a total of three soak-freeze cycles are achieved.
4 After three soak-freeze cycles, continuously soak the specimen in a water bath at room temperature for 64 hours. This completes the specimen conditioning process.

Impact Testing:

The complete testing apparatus used to determine aggregate retention is shown in Figure 7.

1 When the conditioning process is finished, place the inverted specimen (with the aggregate facing down) on the impact testing apparatus as shown in Figure 3. Care should be taken not to have aggregate particles along the boundary where the inverted specimen rests on the impact testing apparatus. This way, the specimen plate is resting on the rubber pads along the perimeter of impact loading apparatus.
2 Place the 6 inches x 6 inches x 1/4 in thick steel plate on to the inverted specimen plate.
3 Drop the modified proctor hammer 3 times onto the ¼ in. thick steel plate over its full drop height of 18 inches.
4 Count the number of aggregates retained on the specimen and record as C_R.
5 Weigh the specimen after testing and record as M_F.

Table 1 shows the recommended test schedule.

Table 1. A Recommended Specimen Preparation, Conditioning and Testing Schedule

Day of Week	Time	Activity
Friday		Prepare specimens
Tuesday	4pm	Begin 16-hour soak
Wednesday	8am	End 16-hour soak and begin 8-hour freeze
	4pm	End 8-hour freeze and begin 16-hour soak

Thursday	8am	End 16-hour soak and begin 8-hour freeze
	4pm	End 8-hour freeze and begin 16-hour soak
Friday	8am	End 16-hour soak and begin 8-hour freeze
	4pm	End 8-hour freeze and begin 64-hour soak
Monday	8am	End 64-hour soak and specimens ready for impact testing

Calculations:

Use the following calculations to determine percent loss.

If the evaluation is based on aggregate count,

$$\% \; Loss \; by \; Aggregate \; Count = \frac{C_A - C_R}{C_A} \; x \, 100$$

Where:

C_A = Number of aggregates on the specimen before conditioning starts
C_R = Number of aggregates retained on the specimen after impact loading

If the evaluation is based on weight,

$$\% \; Loss \; by \; Aggregate \; Mass = \frac{M_T - M_F}{M_T - M_{PB}} \; x \, 100$$

Where:

M_F = Mass of the specimen after impact loading
M_T = Mass of the plate + aggregate + binder before impact testing
M_{PB} = Mass of the plate + binder

Evaluation:

Percent Aggregate Loss Based on Aggregate Count	Compatibility Evaluation
0 to 10	Very Good
10 to 20	Good
20 to 30	Marginally Compatible
30 to 100	Incompatible

Figure 1. Automated Chip Spreader used in Research Project 0-4362

TOP VIEW (Without Specimen)
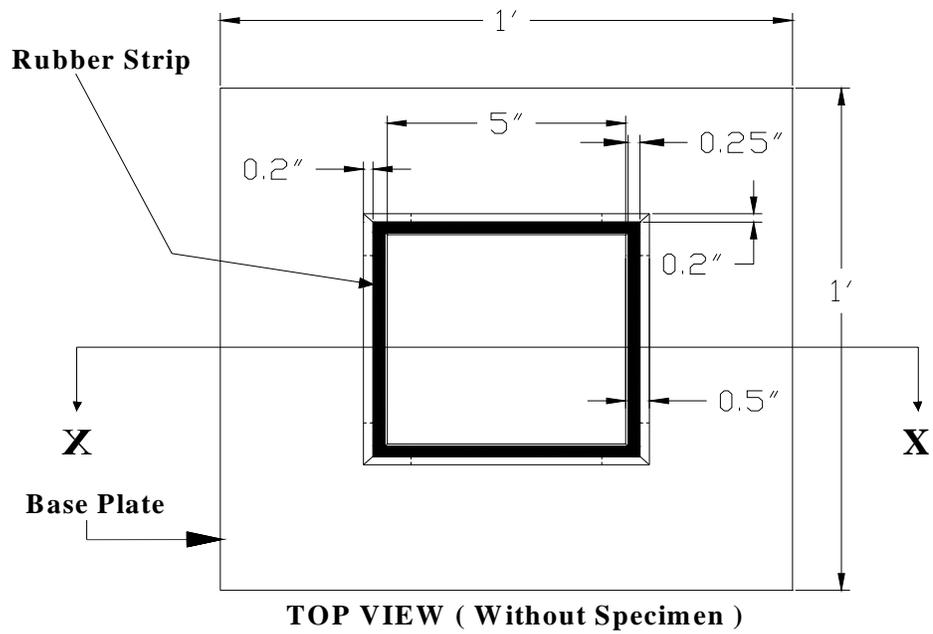
Figure 2. Top View of Specimen Mounting Unit (without Test Specimen)

Cross Section X-X (Without Specimen)
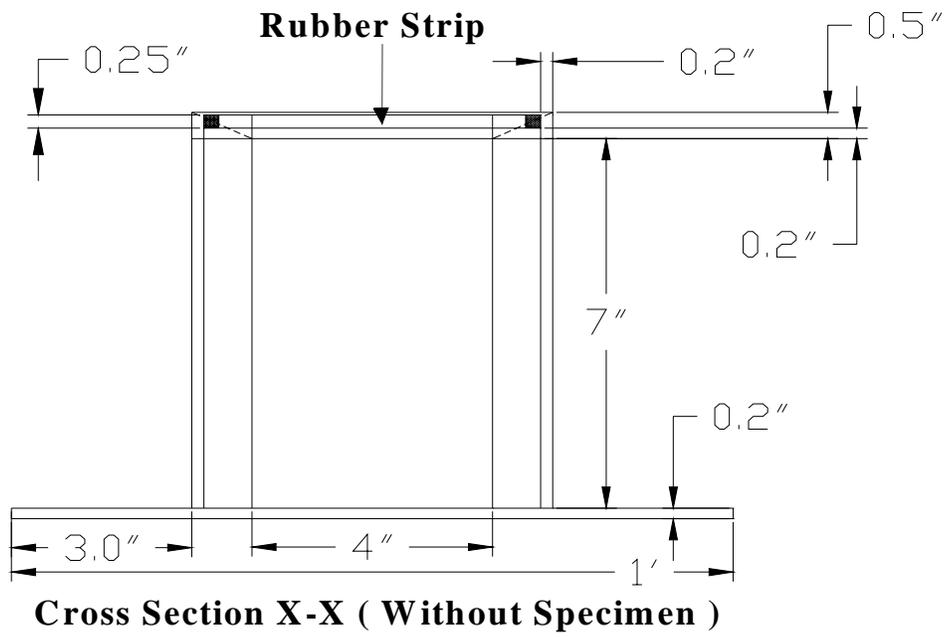
Figure 3. Cross Section View X-X of the Specimen Mounting Unit (without Test Specimen)
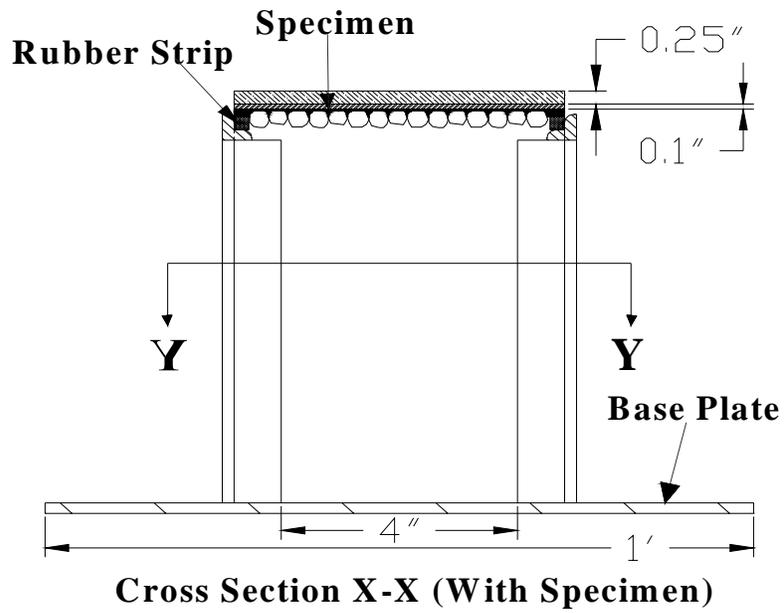
Figure 4. Cross Section View X-X of the Specimen Mounting Unit (with Test Specimen)

TOP VIEW at Cross Section Y-Y
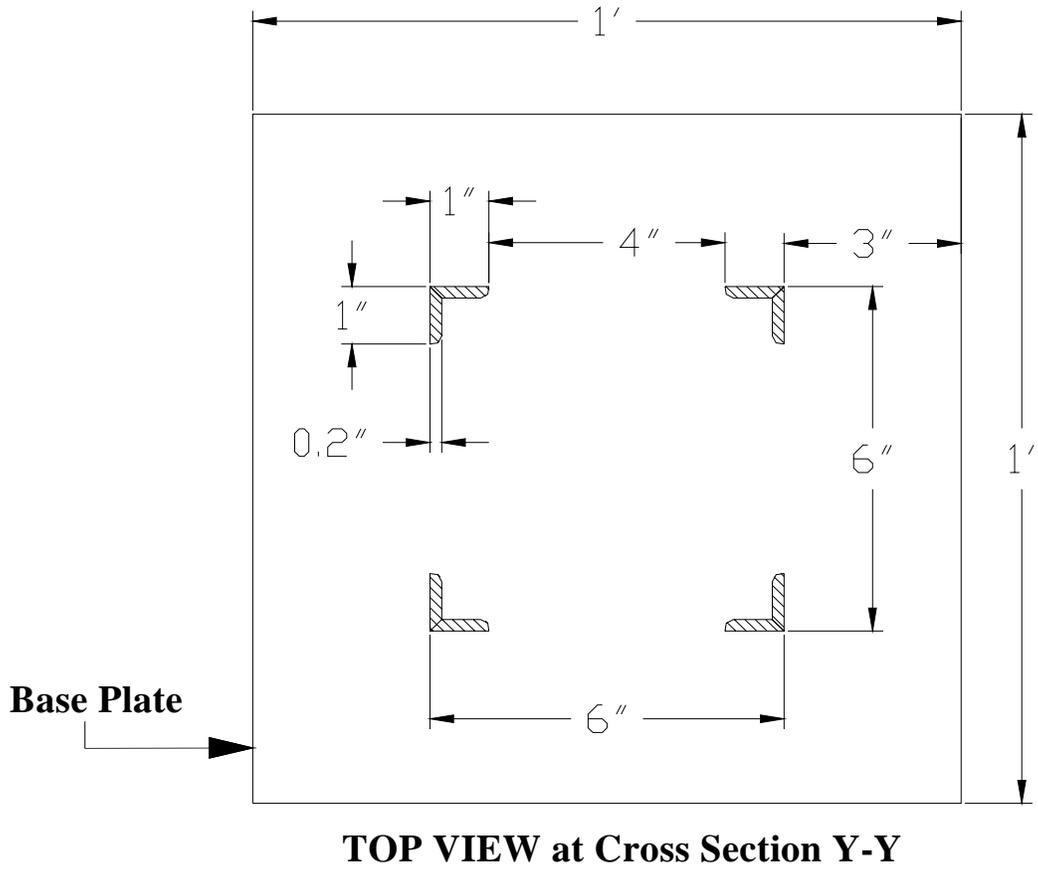
Figure 5. Cross Section View Y-Y of the Specimen Mounting Unit
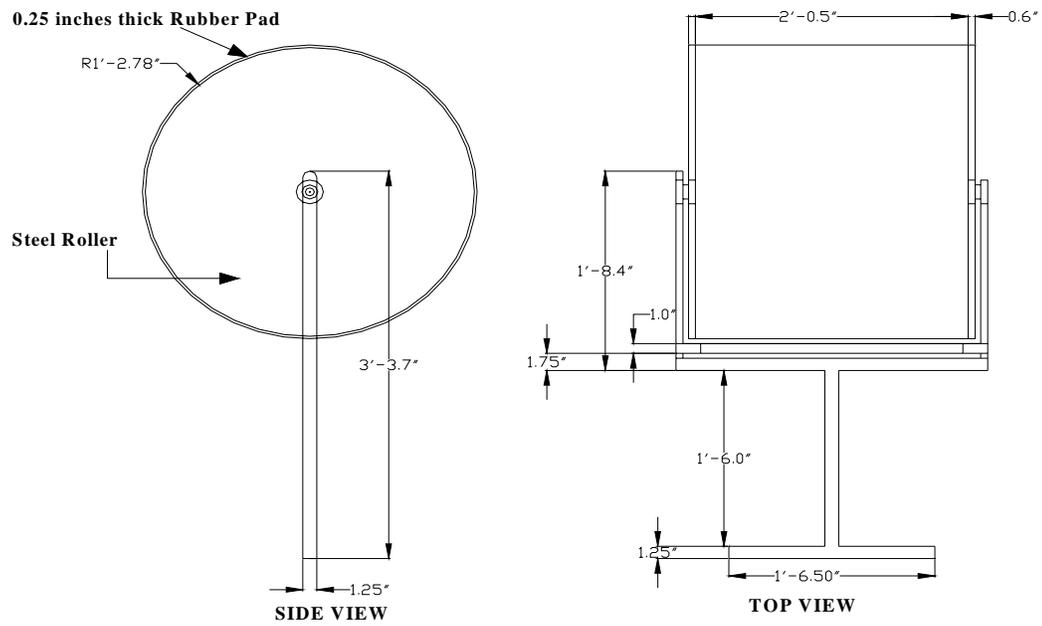
(a)

(b)

Figure 6. Steel Drum Roller (a) Shop Drawing (b) Roller used in Research Project 0-4362

Figure 7. Aggregate-Binder Compatibility Test Apparatus.

Draft Updated Specifications for Seal Coats and Surface Treatments Including New Acceptance Criteria

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Baris Yazgan and Subrata Das

Project Number: 0-4362
Project Title: Develop a Testing and Evaluation Protocol to Assess
Seal Coat Binder-Aggregate Compatibility
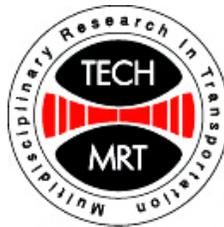
Department of Civil Engineering
Texas Tech University

Center for Multidisciplinary Research in Transportation

Submitted to
Texas Department of Transportation

Product 0-4362-P3   September 2005

ITEM 316

SURFACE TREATMENTS

316.1. Description. Construct a surface treatment consisting of 1 or more applications of a single layer of asphalt material covered with a single layer of aggregate.

316.2. Materials. Furnish materials of the type and grade shown on the plans in accordance with the following:

• Item 300, "Asphalts, Oils, and Emulsions"
• Item 302, "Aggregates for Surface Treatments."

The aggregate and binder used in a surface treatment must be compatible with each other to the extent specified in the plans. This compatibility is applicable to the following situations:

• Compatibility between the aggregate and its precoating binder
• Compatibility between the surface treatment aggregate (precoated or non-precoated) and the surface treatment binder

The compatibility of aggregate and binder in each of the above scenarios can be evaluated using the following standard test procedures.

• Tex-XXX-X: Compatibility Test and Evaluation Protocol for Aggregate and Precoating Hot Asphalt Binder
• Tex-YYY-Y: Performance-Based Compatibility Test and Evaluation Protocol for Aggregates and Binders used in Seal Coats and Surface Treatments

For final surfaces, unless otherwise shown on the plans, furnish aggregate with a surface aggregate classification of "B" or better.

316.3. Equipment.

A. Distributor. Furnish a distributor that will apply the asphalt material uniformly at the specified rate or as directed.

 1. Transverse Variance Rate. When a transverse variance rate is shown on the plans, ensure that the nozzles outside the wheel paths will output a predetermined percentage more of asphalt material by volume than the nozzles over the wheel paths.

 2. Calibration.

 a. Transverse Distribution. Furnish a distributor test report, no more than 1 yr. old, documenting that the variation in output for individual nozzles of the same size does not exceed 10% when tested at the greatest shot width in accordance with Tex-922-K, Part III.

Include the following documentation on the test report:
- the serial number of the distributor,
- a method that identifies the actual nozzle set used in the test, and
- the fan width of the nozzle set at a 12-in. bar height.

When a transverse variance rate is required, perform the test using the type and grade of asphalt material to be used on the project. The Engineer may verify the transverse rate and distribution at any time. If verification does not meet the requirements, correct deficiencies and furnish a new test report.

b. Tank Volume. Furnish a volumetric calibration and strap stick for the distributor tank in accordance with Tex-922-K,

Part I.
Calibrate the distributor within the previous 5 yr. of the date first used on the project. The Engineer may verify calibration accuracy in accordance with Tex-922-K, Part II.

3. Computerized Distributor. When paying for asphalt material by weight, the Engineer may allow use of the computerized distributor display to verify application rates. Verify application rate accuracy at a frequency acceptable to the Engineer.

B. Aggregate Spreader. Use a continuous-feed, self-propelled spreader to apply aggregate uniformly at the specified rate or as directed.

C. Rollers. Unless otherwise shown on the plans, furnish light pneumatic tire rollers in accordance with Item 210, "Rolling."

D. Broom. Furnish rotary, self-propelled brooms.

E. Asphalt Storage and Handling Equipment. When the plans or the Engineer allows storage tanks, furnish a thermometer in each tank to indicate the asphalt temperature continuously.

Keep equipment clean and free of leaks. Keep asphalt material free of contamination.

F. Aggregate Haul Trucks. Unless otherwise authorized, use trucks of uniform capacity to deliver the aggregate. Provide documentation showing measurements and calculation in cubic yards. Clearly mark the calibrated level. Truck size may be limited when shown on the plans.

G. Digital Measuring Instrument. Furnish a vehicle with a calibrated digital-measuring instrument accurate to ±6 ft. per mile.

316.4. Construction.

A. General. Asphalt application season will be as shown on the plans. Asphalt and aggregate rates shown on the plans are for estimating purposes only. The Engineer will adjust the rates for the existing conditions.

B. Temporary Aggregate Stockpiles. The Engineer will approve the location of temporary aggregate stockpiles on the right of way before delivery. Place stockpiles in a manner that will not:
• obstruct traffic or sight distance,
• interfere with the access from abutting property, or
• interfere with roadway drainage.

Locate stockpiles a minimum of 30 ft. from roadway when possible. Sign and barricade as shown on the plans.

C. Aggregate Furnished by the Department. When shown on the plans, the Department will furnish aggregate to the Contractor without cost. Stockpile locations are shown on the plans.

D. Adverse Weather Conditions. Do not place surface treatments when, in the Engineer's opinion, general weather conditions are unsuitable. Meet the requirements for air and surface temperature shown below.

1. Standard Temperature Limitations. Apply surface treatment when air temperature is above 50°F and rising. Do not apply surface treatment when air temperature is 60°F and falling. In all cases, do not apply surface treatment when surface temperature is below 60°F.

2. Polymer-Modified Asphalt Cement Temperature Limitations.
When using materials described in Section 300.2.B, "Polymer Modified Asphalt Cement," apply surface treatment when air temperature is above 70°F and rising. Do not apply surface treatment when air temperature is 80°F and falling. In all cases, do not apply surface treatment when surface temperature is below 70°F.

3. Asphalt Material Designed for Winter Use. When winter asphalt application is allowed, the Engineer will approve the air and surface temperature for asphalt material application. Apply surface treatment at air and surface temperatures as directed.

E. Surface Preparation. Remove existing raised pavement markers. Repair any damage incurred by removal as directed. Remove dirt, dust, or other harmful material before sealing. When shown on the plans, remove vegetation and blade pavement edges.

F. Rock Land and Shot.

1. Definitions.
• A "rock land" is the area covered at the aggregate rate directed with 1 truckload of aggregate.
• A "shot" is the area covered by 1 distributor load of asphalt material.

2. Setting Lengths. Calculate the lengths of both rock land and shot. Adjust shot length to be an even multiple of the rock land. Verify that the distributor has enough asphalt material

to complete the entire shot length. Mark shot length before applying asphalt. When directed, mark length of each rock land to verify the aggregate rate.

G. Asphalt Placement.

1. General. The maximum shot width is the width of the current transverse distribution test required under Section 316.3.A.2, "Transverse Distribution," or the width of the aggregate spreader box, whichever is less. Adjust the shot width so operations do not encroach on traffic or interfere with the traffic control plan, as directed. Use paper or other approved material at the beginning and end of each shot to construct a straight transverse joint and to prevent overlapping of the asphalt. Unless otherwise approved, match longitudinal joints with the lane lines. The Engineer may require a string line if necessary to keep joints straight with no overlapping. Use sufficient pressure to flare the nozzles fully. Select an application temperature, as approved, in accordance with Item 300, "Asphalts, Oils, and Emulsions." Uniformly apply the asphalt material at the rate directed, within 15°F of the approved temperature, and not above the maximum allowable temperature.

2. Limitations. Do not apply asphalt to the roadway until:
• traffic control methods and devices are in place as shown on the plans or as directed,
• the loaded aggregate spreader is in position and ready to begin,
• haul trucks are loaded with enough aggregate to cover the shot area, and
• haul trucks are in place behind the spreader box.

3. Nonuniform Application. Stop application if it is not uniform due to streaking, ridging, puddling, or flowing off the roadway surface. Verify equipment condition, operating procedures, application temperature, and material properties. Determine and correct the cause of nonuniform application. If the cause is high or low emulsion viscosity, replace emulsion with material that corrects the problem.

4. Test Strips. The Engineer may stop asphalt application and require construction of test strips at the Contractor's expense if any of the following occurs:
• nonuniformity of application continues after corrective action;
• on 3 consecutive shots, application rate differs by more than 0.03 gal. per square yard from the rate directed; or
• any shot differs by more than 0.05 gal. per square yard from the rate directed.

The Engineer will approve the test strip location. The Engineer may require additional test strips until surface treatment application meets specification requirements.

H. Aggregate Placement. As soon as possible, apply aggregate uniformly at the rate directed without causing the rock to roll over.

I. Rolling. Start rolling operation on each shot as soon as aggregate is applied. Use sufficient rollers to cover the entire mat width in 1 pass, i.e., 1 direction. Roll in a staggered pattern. Unless otherwise shown on the plans, make a minimum of:

• 5 passes or
• 3 passes when the asphalt material is an emulsion.

If rollers are unable to keep up with the spreader box, stop application until rollers have caught up, or furnish additional rollers. Keep roller tires asphalt-free.

J. Patching. Before rolling, repair spots where coverage is incomplete. Repair can be made by hand spotting or other approved method. When necessary, apply additional asphalt material to embed aggregate.

K. Brooming. After rolling, sweep as soon as aggregate has sufficiently bonded to remove excess.

L. Final Acceptance. Maintain surface treatment until the Engineer accepts the work. Repair any surface failures. Before final project acceptance, remove all temporary stockpiles and restore the area to the original contour and grade.

316.5. Measurement.

A. Asphalt Material. Unless otherwise shown on the plans, asphalt material will be measured by one of the following methods:

1. Volume. Asphalt material will be measured at the applied temperature by strapping the tank before and after road application and determining the net volume in gallons from the distributor's calibrated strap stick. The quantity to be measured for payment will be the number of gallons used, as directed, in the accepted surface treatment.

2. Weight. Asphalt material will be measured in tons using certified scales meeting the requirements of Item 520, "Weighing and Measuring Equipment," unless otherwise approved. The transporting truck must have a seal attached to the draining device and other openings. The Engineer may require random checking on public scales at the Contractor's expense to verify weight accuracy. Upon work completion or temporary suspension, any remaining asphalt material will be weighed by a certified public weigher, or measured by volume in a calibrated distributor or tank and the quantity converted to tons at the measured temperature. The quantity to be measured will be the number of tons received minus the number of tons remaining after all directed work is complete and minus the amount used for other items.

B. Aggregate. Aggregate will be measured by the cubic yard in the trucks as applied on the road. The Engineer may require loaded aggregate to be struck off for accurate measurement.

C. Loading, Hauling, and Distributing Aggregate. When the Department furnishes the aggregate, the loading, hauling, and distributing will be measured by the cubic yard in the trucks as applied on the road.

316.6. Payment. The work performed and materials furnished in accordance with this Item and measured as provided under "Measurement" will be paid for at the unit prices bid for "Asphalt," "Aggregate," and "Loading, Hauling, and Distributing Aggregate" of the types–grades specified. These prices are full compensation for surface preparation; furnishing, preparing, hauling, and placing materials; removing existing pavement markers and excess aggregate; rolling; cleaning up stockpiles; and equipment, labor, tools, and incidentals.