OPTIMIZING GRADATIONS FOR SURFACE TREATMENTS

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Submitted by

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16. Abstract

This report presents the research effort to evaluate the aggregate retention and skid performance of various asphalt surface treatments (ASTs). The study includes two types of aggregate for the aggregate retention test and 14 field AST sections for the skid resistance tests. A new test protocol for the performance evaluation of ASTs was developed using the third-scale Model Mobile Loading Simulator (MMLS3). The AST performance characteristics that can be covered by this protocol include aggregate retention, bleeding, skid resistance, aggregate embedment depth, cracking, and rutting. In this study, the new MMLS3 AST performance test method was applied to evaluate the effects of various mix parameters on aggregate retention and bleeding; these parameters include aggregate and emulsion application rates, fine content, aggregate gradation, and aggregate type (i.e., granite vs. light-weight). It was confirmed that the amount of aggregate loss decreases as the aggregate application rate decreases, the emulsion application rate increases, the fine content decreases, and the gradation becomes more uniform. The aggregate gradation factor, among the others, plays a critical role in the aggregate retention performance regardless of the type of aggregate. The skid resistance test results from the test sections show an adequate skid resistance performance for these sections. The correlation between British Pendulum Numbers and Skid Numbers is found relatively strong, which allows the prediction of the Skid Number from the laboratory British Pendulum.

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1. INTRODUCTION

1.1 Research Needs and Significance

Asphalt surface treatments (ASTs) are among the most frequently used pavement management treatments for flexible pavements. ASTs provide a nonstructural but durable and functional pavement surface that serves as a highly economical highway maintenance option when constructed properly. Typically, an AST consists of a thin layer of asphalt concrete (less than one inch thick) formed by the application of emulsified asphalt and aggregate. They are used with the purpose of sealing the existing pavement's surface cracks, improving ride quality, and protecting the surface against aging or oxidation. Furthermore, the surface treatment seals the existing pavement against water and air, restores its weathered and raveled surface, provides a skid-resistant surface, and improves night visibility of lane demarcations.

Application of ASTs is a common treatment in the NCDOT's pavement preservation program. ASTs in North Carolina cover approximately 50% of paved road miles, but only approximately 8% of the road maintenance budget. These numbers illustrate the effectiveness of ASTs for road maintenance. However, the AST still requires a significant degree of "art" in its design and installation (Gransberg 2005). The performance life of ASTs in North Carolina is typical of that in other states, but about half of that in Australia or New Zealand. That is to say, despite a certain degree of success, AST design can be significantly improved in order to enhance the NCDOT's pavement preservation program in the future and optimize overall road surface quality.

The principal failure modes in ASTs include loss of cover aggregate, streaking, debonding between the existing surface and the new AST, and flushing or bleeding. Generally, the greatest aggregate loss occurs during the initial trafficking and typically is caused by the effects of weather, poor construction, and inadequate AST application design and material selection (McLeod 1996, Shuler 1990). Too much aggregate or not enough asphalt can cause the roller or traffic to grind the excess aggregate into the seated aggregate particles and dislodge them (North Carolina Division of Highways State Road Maintenance Unit 2000). However, not enough aggregate or too much asphalt can cause bleeding.

AST performance, evaluated primarily in terms of aggregate loss, has been studied using various test methods. However, these methods apply different forms of mechanical energy to access the aggregate-emulsion interaction instead of applying a mechanical force that simulates traffic wheels. The third-scale Model Mobile Loading Simulator (MMLS3) is a scaled-down accelerated pavement testing (APT) device and has been used successfully for evaluating the performance of hot-mix asphalt pavements (Lee 2004). This device is used in this study to evaluate the performance of the AST under realistic loading conditions. Moreover, a new comprehensive AST performance test procedure has been developed using the MMLS3 at North Carolina State University (NCSU). Although various performance characteristics may be evaluated using this procedure, this project highlights the effects of fine content and gradation on the aggregate retention performance of surface treatments. Two types of aggregate (i.e., granite and expanded slate light-weight aggregate) were selected for the aggregate retention evaluation because they are both commonly used in North Carolina.

Another performance characteristic that is evaluated in this study is skid resistance. There are several skid resistance measurement devices, including the British Pendulum

Tester (BPT), Locked Wheel Skid Tester (LWST), and GripTester (GT). Among these methods, the BPT is the only test method that can be used in the laboratory, whereas the test method recommended in the specification is the LWST. Therefore, the focus of this study is to develop a relationship between the British Pendulum Number (BPN) from the BPT and the Skid Number (SN) from the LWST. This relationship allows the BPT to be used in the laboratory and allows the conversion of the BPN to the SN so that these numbers may be checked against the specification guidelines.

The following chapters present the laboratory and field research efforts to evaluate the aggregate retention and skid performance of ASTs.

1.2 Research Objectives

The primary objectives of the research are:

1. to develop a performance-based test method that can be used in evaluating various performance characteristics of ASTs;
2. to evaluate the effects of aggregate gradation, fine content, and aggregate type on the AST aggregate retention performance;
3. to evaluate the skid resistance of selected AST roads in North Carolina by different test methods and to develop the relationships among their friction numbers.

1.3 Report Organization

This report is composed of five chapters. Chapter 1 presents the research needs and objectives. Chapter 2 summarizes the literature review of AST aggregate retention performance test methods and the effects of various mix factors on aggregate retention. Skid resistance measurement methods are also presented in this chapter. Chapter 3 describes physical characteristics of selected materials in this study. It also discusses the specimen fabrication methods and the protocols for the flip-over test and the MMLS3 test used in this study to evaluate aggregate retention performance. In Chapter 4, a discussion of the application rate design of ASTs using the McLeod method and the modified Kearby method is followed by a discussion on the effects of AST application rate, fine content, gradation, and aggregate type on aggregate retention performance. Chapter 5 reports the results of skid resistance performance obtained from the three different test methods and their correlations. Conclusions from this research and future research recommendations are given in Chapter 6.

2. LITERATURE REVIEW

2.1 General

As a result of the continued commitment by state highway agencies (SHAs) to pavement preservation, the use of surface treatments has been steadily increasing. Thus, it becomes imperative for the agencies to optimize the use of those treatments in terms of prolonged service life, decreased life cycle costs, increased operational efficiency, and enhancement of safety. In a recent study (Ksaibati et al. 1996) aimed at evaluating the use of surface treatment practices in the United States, twenty-five SHAs rated their ASTs as good, seven (including the NCDOT) rated them as average, while three rated them as fair. Not a single SHA believed its AST operation was excellent. Several agencies, including those in Minnesota, Virginia, South Dakota, Wyoming, and Saskatchewan in Canada recognized the need to improve overall pavement performance and consequently invested in an evaluation of their AST operations (Ksaibati et al. 1996, Alaska DOT and Public Facilities 2001, Roque et al. 1991, Shuler 1986).

In their early days, ASTs were predominantly used as wearing courses in the construction of low traffic volume roads and have evolved into a maintenance treatment that can be successful on both low and high traffic volume pavements. ASTs in North Carolina are applied to roads that have an average daily traffic (ADT) of less than 2000 vehicles (Gransberg 2005).

ASTs are not meant to enhance the structural capacity of the pavement section and, therefore, should not be applied to roads that exhibit severe distresses. There are several triggers that initiate the selection of an AST, however, such as surface wearing, skid resistance, oxidation, and water infiltration. In North America, the evidence of distress and the prevention of water infiltration constitutes the most common trigger for the necessity of ASTs (Gransberg 2005).

The principal failure modes in ASTs are streaking, debonding between the existing surface and the new AST, flushing or bleeding, and loss of cover aggregate. Streaking is due to the failure to apply asphalt uniformly inch by inch across the road surface, as shown in Figure 2-1. Streaking is generally caused by the asphalt sprayer's nozzles being clogged or perhaps set at the wrong setting or some other functional problem.

A new AST may fail to establish a good bond with an existing surface for several reasons, including the presence of a layer of dust or dirt on the existing surface, the existing surface being wet or too cold, or the asphalt being too hard. Normally, this failure to establish a good bond with an existing surface only causes a problem on a small area of only a few square inches or a few square feet. Occasionally, however, a few square yards and sometimes even an entire AST can fail for this reason (McLeod 1969). A typical debonding failure of AST is shown in Figure 2-2.

Another major long-term distress that appears in AST roads is bleeding or flushing (Figure 2-3). This failure is usually caused by the application of too much asphalt, which causes the excess binder to ooze out of the cover aggregate onto the surface. Flushing or bleeding may also result from the loss of a portion of the cover aggregate for any number of reasons, such as a rainfall shortly after construction, asphalt that is too hard and fails to develop adequate adhesion with the cover aggregate, and use of cover stone that is too dirty or too wet to establish good adhesion to the asphalt (McLeod 1969, Gransberg 2005).

Other major distresses in the AST include aggregate loss and loss of skid resistance. Since these are the two distresses evaluated in this study, their causes and measurement methods are described in more detail in the following subsections.

Figure 2-1 Typical streaking of AST

Figure 2-2 Typical debonding failure of AST

Figure 2-3 Partial bleeding failure of AST

2.2 Retention Performance on Aggregate Characteristics

Aggregate loss is one of the critical AST failure modes. Generally, the most aggregate loss occurs during the initial traffic passes once a road is newly opened to traffic. Other major causes of aggregate loss include unexpected cold and/or wet weather, excessive aggregate, inadequate traffic control during construction, inadequate embedment of the stone particles in the emulsion, inadequate aggregate characteristics, and dusty or dirty aggregate (Shuler 1990, Gransberg 2005). The aggregate loss due to construction faults occurs within a few months, and an AST with this type of problem should be repaired rather than resealed because just a reseal will not cannot normally last the expected AST life (Transit New Zealand 2005). The aggregate properties in the AST, such as gradation, shape, moisture condition, and dust play a major role in the aggregate retention.

2.2.1 Aggregate Retention Test Methods

Aggregate retention performance can be evaluated using various test methods, including the Aggregate Retention Test (ART) (Tex-216-F), Vacuum Test, Vialit Test (Kandhal 1987, Hank and Brown 1949, Benson and Gallaway 1954, Barnat 2001, Yazgan 2004), Pennsylvania Aggregate Retention Test (PART), and the Sweep Test (ASTM D7000). However, each of these methods applies a different form of mechanical energy to assess the aggregate-binder bond interaction instead of applying a mechanical force that simulates traffic wheels. Table 2-1 provides the name of each test, the agency that developed each test, and the loading characteristic of each test method.

Table 2-1 Aggregate-binder compatibility tests

Name of Test	Agency	Characteristic
Aggregate Retention Test	Texas DOT, Tex-216-F	Light Sweep Test
Vialit Test	French Public Works	Inverted Tray, Ball Impact
Pennsylvania Retention Test	Pennsylvania DOT	Inverted Tray, Sieve Shaker
AST Sweep Test	ASTM WK139	Replicates Sweeping
Macrosurfacing Sweep Test	Koch Materials TM101	Replicates Sweeping

Hank and Brown (1949) developed an Aggregate Retention Test according to a TxDOT standard test method, Tex-216-F. They produced a uniform aggregate distribution by applying asphalt on a 23 in. by 15 in. tray and lightly brushing away loose materials at an angle of 75°. They researched the effect of asphalt, rolling, aggregate gradation, and temperature on aggregate retention. Five years later, Benson and Gallaway (1954) developed a vacuum pull-off test to determine the tenacity with which the cover aggregate is held. The AST was constructed by applying the asphalt with a small laboratory sprayer using standard distributor nozzles and distributing the cover aggregate. The ART is performed first on the specimen, followed by the vacuum test.

The Vialit Test was developed by the French Public Works Research Group and standardized in BS EN 12272-3. A stainless steel ball (1.1 lb.) is dropped three times from a height of 19.7 in. onto inverted AST trays. The standard curing time for this test is 48 hours at 140°F, and sample conditioning takes place in a freezer at -7.6°F for 30 minutes. The percentage of aggregate loss after three ball drops is used for evaluation. The impact on the sample simulates the brooming procedure after 30 minutes of curing and also the rolling procedure after 10 minutes of curing. This test procedure has been evaluated using the AST sample fabricated in the field and examined for the effects of aggregate gradation on different binders by Davis et al. (1991).

PART was developed by the National Center for Asphalt Technology at Auburn University to evaluate the adhesion of precoated aggregate in ASTs. This test uses the Mary Ann sieve shaker's shaking and tapping action on an inverted AST tray for five minutes to evaluate AST performance.

The Sweep Test measures the curing performance characteristics of bituminous emulsion and aggregates by using a brush to sweep the surface treatment constructed in the laboratory. This test, standardized in ASTM D7000, uses a gyration mixer with a steel brush to sweep the AST which is fabricated on a felt disk. This test simulates the brooming procedure in AST construction and measures the aggregate loss. The macrosurfacing test, modified from Abrasion Cohesion Test Esso (ACTE), is very similar to the Sweep Test, but a hose replaces the brush.

2.2.2 Effect of Application Rate

The most common deviation from proper practice during AST construction appears to be the application of an excessive amount of aggregate. In doing so, materials are wasted, and excess aggregate may be thrown by rapidly moving traffic. An incorrect assumption often made regarding the application of too much aggregate is that excess aggregate can simply be swept off the surface, leaving the correct application quantity in place. However, when this practice is exercised, at least two major forms of distress result: vehicular distress and pavement distress.

The pavement distress occurs when more than one aggregate thickness is present and the excess aggregate on the surface is pushed into the layer below. This action causes dislodgement of the first layer, thus leading to loss of aggregate and changes in grading. Crushing of aggregate can also occur; this can be offset somewhat by the inclusion of hard, durable particles, but some dislodgement nonetheless occurs, creating early aggregate loss and the potential for flushing (Shuler 1990). When larger quantities of aggregate are applied, the small stones adhere and the large stones are brushed off (Benson and Gallaway 1953). It has been reported that a considerable excess of cover material is often more detrimental than a slight shortage of cover material, in that with an excess of cover material the amount of fines applied is also increased (Kearby 1952).

2.2.3 Effect of Fine Content

Clean aggregate is extremely important. Dusty or dirty aggregate causes aggregate loss because the asphalt may not stick to the aggregate properly. If the particles are dusty or coated with silt or clay, the asphalt may not stick because the dust produces a film which prevents adhesion to the aggregate. That is, good results cannot be assured with dusty or dirty aggregate (The Asphalt Institute 1967).

Washing and drying the aggregate by mechanical means before application solves this problem almost entirely (Kandhal and Motter 1992). It is recommended that the aggregate be sprayed with water a couple of days prior to the start of the project. Washing chip seal aggregate with clean, potable water prior to application may assist in removing fine particles that prevent adhesion with the binder (Gransberg 2005). High float emulsion and polymer-modified emulsion can be successfully used with somewhat dusty aggregate because they permit a thicker and tackier asphalt film on the aggregate (Alaska DOT Public Facilities 2001).

Table 2-2 Specification of maximum percentage of fine content (Kandhal 1987)

State	Maximum Percentage Passing No. 200
Alabama	1.0
Florida	3.75
Indiana	2.0
Kansas	2.0
Maryland	1.0
North Carolina	1.5
North Dakota	4.0
Ohio	3.0
Pennsylvania	2.0
South Carolina	0.0
South Dakota	2.0
Tennessee	1.0
Average	1.9

Dust is normally defined as the percentage of fine aggregate that passes the No. 200 sieve. To improve the quality of the material in ASTs, the percentage of fine aggregate passing the No. 200 sieve has been specified in many states as a maximum of 2% at the time of manufacture, and some states require 0.5% or less passing the No. 200 sieve (Kandhal 1987, Alaska DOT Public Facilities 2001). The maximum allowable fine contents for different states are summarized in Table 2-2.

The effect of fine on aggregate retention has been studied using various test methods, including the ART (Tex-216-F), Vacuum Test, PART, and Vialit Test. Benson and Gallaway (1953) conducted the ART (Tex-216-F) and the vacuum test and found that the presence of dust even in relatively small quantities can cause a reduction in aggregate retention.

Kandhal (1987) developed PART and found that the rate of increase in aggregate loss with increasing fine contents becomes significantly greater after about a 3% dust content in most cases. Therefore, he considers 3% a threshold value. Since most states specify a maximum of 2% dust for unwashed aggregates, Kandhal reports that 2% seems to be reasonable for low volume roads, particularly if the cost of washing or precoating is very high.

Yazgan (2005) modified the Vialit test by applying more mechanical impact energy to assess the aggregate-binder bond. He found that fine content affects the aggregate retention independently of the embedment depth.

2.2.4 Effect of Gradation

The aggregate gradation plays a key role in the design, construction, and performance of chip seals. The specified gradation should be such that the texture of the seal is consistent. Tight gradation bands, which ensure a uniformly graded aggregate with minimal fines and dust, are necessary for a quality project. The literature and surveys show a consensus that single-sized aggregate with less than 2% passing the No. 200 sieve is considered ideal (Gransberg 2005). One of the most important advantages of using a one size cover aggregate in a surfacing operation is that maximum contact is obtained between the tire and the surface. This increases the frictional area, and thus, there is better skid resistance as long as the correct quantity of binder is used (Herrin et al. 1968).

The aggregate should be as close to uniform size as is economically practical so that the surface treatment has only one layer of aggregate. If there is a significant difference between the largest and the smallest sized particles, the asphalt film may completely cover the smaller sizes and prevent proper embedding of the larger particles. Generally, the largest size for a surface treatment aggregate should be no more than twice the smallest size, with a reasonable tolerance for oversize and undersize to allow for economical production (The Asphalt Institute 1967). As the magnitude of the tolerance is increased, it is believed that performance quality is sacrificed. Therefore, from the viewpoint of overall economy, it may be preferable to have higher initial costs to obtain close to one size aggregate that performs well than to have lower initial costs and higher annual maintenance expenses (McLeod 1960).

Hank and Brown (1949) found that aggregate gradation is an important factor in the quantity of stone retained in ASTs. The gradation effect is significant when asphalt cements are used; however, the influence of fines, including aggregate passing the No. 10 sieve, is not very noticeable when emulsion is used.

Benson and Gallaway (1959) found that an increase in the fine content from 0 to 30% of the aggregate causes a 10% reduction in aggregate retention. Therefore, in order to obtain the most cover stone that adheres for a given maximum size, it is desirable to have cover aggregate that is nearly uniform in gradation. This issue is also tied in with economical considerations because aggregate costs must necessarily increase as the gradation requirements become more restrictive. However, if two aggregates are otherwise the same in price and quality, the aggregate that has the uniform gradation is the preferred one.

Kandhal (1991) also reports that the aggregate retention loss is greater with the use of graded cover stones. These graded stones contain additional smaller particles which tend to fill the voids between large particles and, thus, may not become effectively embedded into the applied binder.

2.3 Skid Resistance of Asphalt Surface Treatments

Pavement characteristics comprise only one element in the multiple component system of a skid accident that involves the driver, roadway, the vehicle, and the weather. Road surface conditions that are indicative of potential safety hazards include bleeding, polished aggregate with a smooth microtexture, a smooth macrotexture, rutting, and an inadequate cross slope (Huang 1993).

In North America, loss of skid resistance is one of the common road conditions that indicates the need for an AST; thus, one of the major advantages of surface treatments is the increase in skid resistance (Gransberg 2005). The textural depth created by such treatments allows for better contact and adhesion between a vehicle’s tires and the road surface, thereby increasing road safety.

Most SHAs have a specified cycle on which skid resistance is measured as a part of their pavement management system, which is invaluable to deciding which roads to chip seal. However, there is no evidence that a single public highway agency has used skid numbers to directly evaluate the performance of chip seals. (Gransberg 2005).

Skid resistance changes over time. Typically, it increases in the first two years following construction as the asphalt binder is worn away by traffic, then decreases over the remaining pavement life as aggregates become more polished. Skid resistance tends to increase in winter when wet and cold weather create a gritty detritus that roughens the surface. In drier summer conditions the surface detritus is dusty, and the dust polishes the surface, resulting in a reduction in skid resistance. This seasonal variation is quite significant and can severely skew skid resistance data if not properly taken into consideration. The winter recovery may not be sufficient to balance the summer polishing (Jayawickrama and Thomas 1998, Hunter 2000).

There are two different testing subsets for determining a pavement's skid resistance, textural and drag or friction testing. These test methods are explained in the following subsections.

2.3.1 Textural Measurement

Road pavement texture is categorized into four levels by the World Road Association (formerly known as the Permanent International Association of Road Congress or PIARC). These levels and their corresponding texture wavelengths are presented in Table 2-3. Microtexture and macrotexture are the two levels of pavement texture that affect the friction between the pavement and the tire. If both microtexture and macrotexture are maintained at high levels, they can provide good resistance to skidding on wet pavement (Henry 2000).

Henry (2000) reports that no practical procedure for the direct measurement of microtexture profiles in traffic currently exists. The portions of the pavement surface that make contact with the tires are polished by traffic, and it is the microtexture of the surface of the exposed aggregate that comes into contact with the tire that influences the friction. Wet pavement friction at low speeds is primarily influenced by the microtexture.

Table 2-3 PIARC texture definitions (Kuttesch 2004)

Texture Level	Wavelength (?)
Microtexture	? < 0.5 mm
Macrotexture	0.5 mm < ? < 50 mm
Megatexture	50 mm < ? < 0.5 m
Roughness	0.5 m < ? < 50 m

Ergun et al. (2005) researched the development of microtexture measurement methods with an image analysis technique that can precisely measure the road surface microtexture under laboratory conditions. Also, a new friction coefficient prediction model was developed and correlated using macrotexture and microtexture.

Meyer (1991) states that there are three common methods for measuring pavement macrotexture: profilometers, volumetric, and outflow. Profilometers typically use lasers to generate a two-dimensional assessment of the pavement macrotexture. The volumetric measurement technique, commonly called the *sand patch* method and specified in ASTM E 965 Standard Measuring Pavement Macrotexture Depth Using a Volumetric Technique, involves spreading a known volume of a single-sized material in a circle on the pavement surface. The volume divided by the area is reported as mean texture depth (MTD).

Roque et al. (1991) studied the performance and the prediction of chip seal life. MTD, as measured by the sand patch test, is used to characterize the surface texture and to evaluate the in-service performance of the seal coat. The MTD measurement may also be used to estimate the remaining life of the chip seal.

The outflow method measures the time for a known volume of water to flow from a cylinder placed on the pavement surface. The time is reported as the outflow time (OFT). The OFT is highly correlated with MTD for nonporous pavements (Henry 2000).

Fulop et al. (2000) found that macrotexture has a direct effect on skid resistance; the better the macrotexture, the smaller the slope of the friction coefficient speed function.

2.3.2 Friction Measurement

Seneviratne (1994) studied the safety effects of chip seals using the skid resistance number in an effort to determine countermeasures to potential accidents. Although the average accident rate seems to have decreased after chip seal treatment, a definite relationship between the skid number (SN) and accident rate in the road sections that underwent chip seal treatment is not evident.

Pavement friction is measured most frequently in accordance with the locked-wheel method, as specified in ASTM E 274 Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire. The locked-wheel friction testers usually operate at speeds between 40 and 60 mph. Once the target test speed has been attained, a film of water is sprayed onto the pavement 10 to 18 inches in front of the test tire. This water film has a nominal thickness of 0.02 in. (0.5 mm). At this point, the wheel is locked for a period of 1 sec, and the frictional force is measured and averaged over that period of time. The SN from this test is calculated by dividing the horizontal force by the vertical load and then multiplying by 100 to obtain a whole number, which could theoretically range from 0-100.

A SN below 40 in North Carolina indicates roads that need further study or corrective action to improve skid resistance. Most SHAs have established their own minimum skid number requirements, as shown in Table 2-4.

Table 2-4 Proposed minimum skid number (Henry 2000)

State	**Minimum Skid Number (SN at speed 40 mph)**
Idaho	30
Illinois	30
Kentucky	28
New York	32
South Carolina	41
Texas	30
Utah	35
Washington	30
Wyoming	35

GripTester (GT), another surface friction (skid resistance) tester, was also developed for field-testing and has been supplied to highway and airport authorities since 1987. The GT is drawn behind a vehicle; a recording wheel in the middle of the apparatus is "gripped" by a set of gears as it is being dragged. The GT is defined in BS 7941-2 Surface friction of pavements – Part 2: Test method for measurement of surface skid resistance using the GripTester braked wheel fixed slip device. The GT measures the longitudinal friction coefficient (LFC) between the pavement and a measuring wheel, which is a specific, designated tire. A sliding rate that generates the grip force is obtained by a mechanical drive between the two carrying wheels and the measuring wheel. The measuring wheel axis is equipped with a pressure gauge system that permits the measurement of reactions, that is, the vertical force (FV) and the horizontal force (FH). The LFC measured by the GT, named the grip number (GN), is proportional to the ratio FH over FV. Table 2-5 explains the friction values of the GT for the Federal Aviation Administration (FAA) classification levels, qualified at a 40 mph test speed.

Table 2-5 Friction level classifications for runway pavement surfaces

40 mph			**60 mph**		
Minimum	Maintenance Planning	New Design /Construction	Minimum	Maintenance Planning	New Design /Construction
0.43	0.53	0.74	0.24	0.36	0.64

The most widely accepted method for laboratory drag testing of skid resistance is the British Pendulum Test (BPT) (ASTM E 303). This test method utilizes the Pendulum Skid Resistance Tester, a relatively small device weighing less than 10 pounds. The device's pendulum swings across a wet section of the pavement, and the amount of retardation (drag) caused by the pavement is measured by a dial on the pendulum tester. The British Pendulum Tester is an easy device to use, and provides repeatable results and a good measurement of skid resistance. The Tester is fitted with scales that measure the recovered height of the pendulum in terms of a British Pendulum Number (BPN) over a range of 0 to 140. The typical slip speed for the BPT is commonly assumed to be about 6 mph (10 km/h). The BPN is used mainly as a substitute for the microtexture test.

Corley-Lay (1998) performed skid resistance tests on HMA pavements with various surface course mixtures. She concludes that neither the BPT nor the sand patch test can be used to predict the friction number from a LWST with sufficient accuracy. However, it must be noted that her conclusions are based on the test results from HMA pavements, not ASTs.

2.4 Material Selection of Asphalt Surface Treatments

AST material selection is generally dependent upon climatic conditions, aggregate quality, and product availability. Aggregate selection is a function of geological availability and transportation distance of aggregate. The pavement's surface, size and gradation of aggregate, and local climate are considered for the binder selection process (Gransberg 2005).

ASTs in North Carolina are specified with No. 78M for the aggregate size and CRS-2 or RS-2 for the asphalt type (NCDOT Standard Specifications for Roads and Structures 2002). However, the light-weight aggregate size has not been specified in North Carolina. The most common size of aggregate for a straight seal is usually a 3/8 in. (10 mm) Nominal Maximum Size of Aggregate (NMSA) (Gransberg 2005).

According to the North Carolina statewide survey conducted in 2003, No. 78M for granite and 5/16 in. NMSA for light-weight aggregate are the most common sizes in a straight seal, split seal, and triple seal, as well as the blotting sand and screening aggregate for the top layer of a triple seal. The types of emulsion used in North Carolina are CRS-2 and CRS-2P. Table 2-6 summarizes the aggregate and asphalt types used in North Carolina for ASTs.

Table 2-6 Types of materials used for ASTs in North Carolina

	Layer	Aggregate	Emulsion
Mat & Seal	Top	Sand, 78M, or Screenings	CRS-2 or CRS-2P
	Third	78M or Light-weight 5/16 in.	
	Second		
	Bottom	No. 5, No.57, No.6, or No.67	
Straight Seal		78M or Light-weight 5/16 in.	
Split Seal	Top	Sand, 78M, Light-weight 5/16 in., or Screenings	
	Bottom	78M or Light-weight 5/16 in.	
Triple Seal	Top	Sand, 78M, Light-weight 5/16 in., or Screenings	
	Middle	78M or Light-weight 5/16 in.	
	Bottom		

3. MATERIALS AND TESTING METHODS

This chapter presents the materials, testing equipment, and the flip-over and MMLS3 test methods used for AST performance evaluation.

3.1 Materials

This section includes the materials selected for testing and the component materials' physical properties.

3.1.1 Material Selection

In this project, two types of aggregate were selected to be used with the CRS-2 emulsion: expanded slate (light-weight aggregate) with a 5/16 in. NMSA and granite No. 78M. The granite aggregate comes from the Garner quarry; the light-weight aggregate is produced by the Carolina Stalite Company using a rotary kiln expanded slate light-weight aggregate. The CRS-2 emulsion is obtained from SEACO in Columbia, South Carolina.

3.1.2 Component Material Properties

3.1.2.1 Gradation of Aggregate Particle Size

Dry and wet sieve analyses were performed on both the aggregates in accordance with ASTM C 117. Table 3-1 presents the percentage of aggregate passing through each sieve, averaged over the weights of each sample. Sieve analysis results for the individual samples are shown in Figure 3-1, plotted on the 0.45 power chart, and the median particle size (M) of each sample is summarized at the bottom of Table 3-1.

The sieved granite was close to the upper limit recommended by the No. 78M specification. The uniformity coefficient is the ratio of the particle size that is 60% finer by weight to the particle size that is 10% finer by weight on the grain size distribution curve

(Das 1993). The uniformity coefficient is a measure of how well or uniformly the aggregate is distributed. The closer this number is to one, the more uniformly the aggregate is graded. The uniformity coefficient of the tested aggregate is 1.72 and 2.44 for the light-weight aggregate and the granite, respectively. Therefore, the light-weight aggregate has a more uniform aggregate particle size than the granite.

Table 3-1 Summary of the sieve analysis

Sieve Size	**Percent Passing**	
	Light-weight	**Granite**
1/2 in.	100.0	100.0
3/8 in.	100.0	95.4
1/4 in.	66.0	64.8
No. 4	23.7	40.1
No. 8	4.9	7.4
No. 16	3.5	2.4
No. 50	2.6	0.8
No. 100	2.2	0.7
No. 200	1.6	0.4
Median size, M	0.21 in. (5.31 mm)	0.23 in. (5.70 mm)

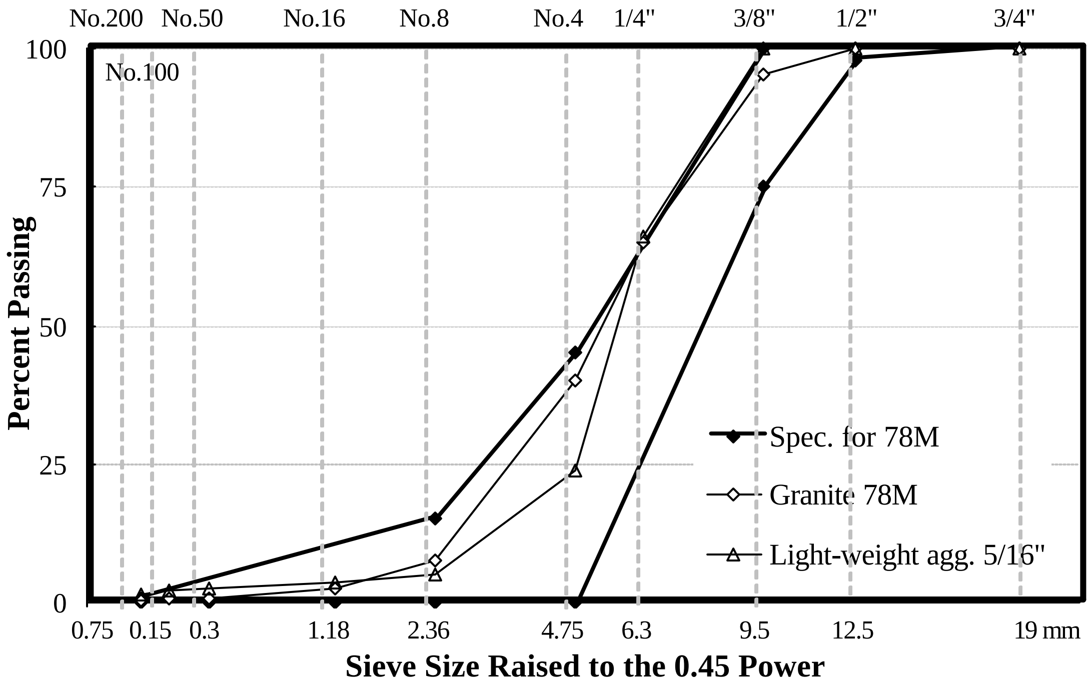


Figure 3-1 Aggregate particle size gradations

3.1.2.2 Flakiness Index

The Flakiness Index (FI) is a measure of the percentage, by weight, of flat particles. It is determined by testing a small sample of aggregate particles for their ability to fit through a slotted plate (Figure 3-2). There are five slots for five different size fractions of the aggregate (Table 3-2). If the aggregate particles fit through the slotted plate, they are considered to be flat. If not, they are considered to be cubical.

The weight of materials passing all of the slots is divided by the total weight of the sample to give the flat particles' percentage, by weight, or Flakiness Index. Table 3-3 shows the FI of each aggregate type. It can be seen that the granite aggregate from the Garner quarry is much flatter than the light-weight aggregate.

Table 3-2 Slot sizes required for different fractions

Size of Aggregate		Slot width, in.
Passing	Retaining	
1 in.	3/4 in.	0.532
3/4 in.	1/2 in.	0.384
1/2 in.	3/8 in.	0.258
3/8 in.	1/4 in.	0.184
1/4 in.	No. 4	0.123

Table 3-3 Flakiness Index of aggregates

Aggregate Type	Light-weight	Granite
Flakiness Index (%)	8.28	24.01

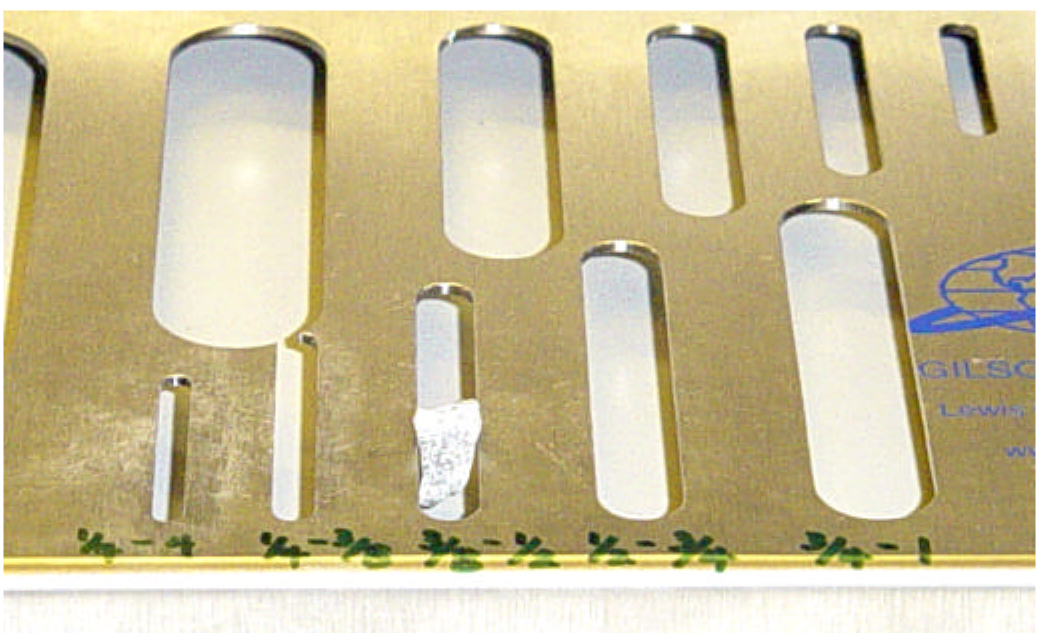


Figure 3-2 Flakiness Index plate gauge

3.1.2.3 Bulk Specific Gravity

Bulk specific gravity (BSG) tests were performed on all aggregates. The aggregates were divided into three different sizes: the aggregate retained on the No. 8 sieve; passing the No. 8 sieve and retained on the No. 200 sieve, and passing the No. 200 sieve. These tests were conducted in accordance with the standard test methods, and the results are summarized in Table 3-4. The bulk specific gravity of the light-weight aggregate is 0.6 times that of the granite aggregate, as Figure 3-3 shows.

Table 3-4 Bulk specific gravities of aggregates

Aggregate Type	**Light-weight**		**Granite**	
	Standard	**BSG**	**Standard**	**BSG**
Retained on No. 8 Sieve	Tex-433A	1.62	ASTM C 127	2.65
Passing No.8 and Retained on No. 200 Sieve	ASTM C 128	1.94	ASTM C 128	2.63
Passing No. 200 Sieve	ASTM D 854	2.55	ASTM D 854	2.52
Total Bulk Specific Gravity		1.66		2.64

Figure 3-3 Volume comparison of 0.22 lb (100 g) granite (left) and light-weight (right) aggregate

Figure 3-4 Loose unit weight test

3.1.2.4 Loose Unit Weight of the Cover Aggregate

The loose unit weight (W) is determined by ASTM C 29 (Figure 3-4) and is needed to calculate the voids in the aggregate in a loose condition. The design requirements for the quantities of cover aggregates to be applied per square yard for the AST are based on the ASTM bulk-specific gravity of the cover stone and on the fraction of voids in its loose weight condition. The fraction of voids (V) is calculated from the following equation, and the results are shown in Table 3-5:

$$V = 1 - \frac{W}{62.4G}$$

where:

V = voids in the loose aggregate, in percentage, expressed as a decimal;

W = loose unit weight of the cover aggregate, ASTM C 29, lbs/ft^3; and

G = bulk specific gravity of the aggregate.

Table 3-5 Loose unit weight test results

Aggregate Type	**Light-weight**	**Granite**
Loose Unit Weight (lbs/ft^3)	49.48	96.0
Voids in the Loose Aggregate (V)	0.51	0.42

3.1.2.5 Aggregate Absorption

Since the light-weight aggregate is the rotary kiln expanded slate aggregate, it is expected to have higher surface voids and, thus, greater absorption. The research team checked their asphalt absorption values by using Rice's vacuum saturation method, ASTM D 2041. PG 70-22 asphalt was mixed with aggregate at 329°F using a 6% asphalt content. Then, the mixtures were vacuumed (Figure 3-5) and weighed in water to measure the maximum specific gravity used for calculating the asphalt absorption. The light-weight aggregate has a higher asphalt absorption value than the granite, as shown in Table 3-6.

Figure 3-5 Rice vacuum saturation test

Table 3-6 Asphalt absorption test results

Aggregate Type	Light-weight	Granite
Effective Sp.G.	1.64	2.67
Asphalt Absorption (%)	0.48	0.38

3.1.2.6 Residual Asphalt Content

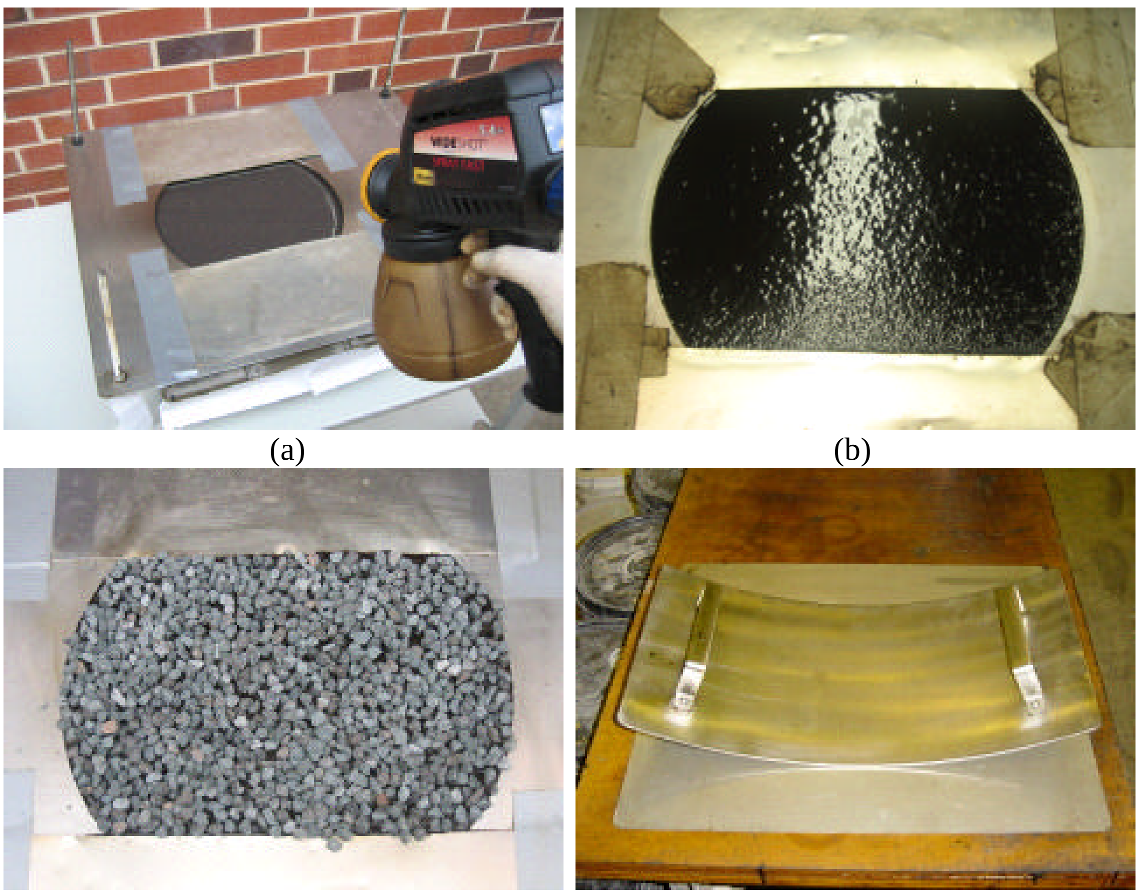
Asphalt emulsion is comprised of asphalt binder and water that evaporates as the binder cures. Therefore, in designing the AST it is important to know the residual asphalt content of the binder. The CRS-2 used in this project has a 68-70% residual asphalt content, according to the test results provided by the NCDOT Materials and Tests Unit.

3.2 Specimen preparation

With the exception of specimen shape and curing time, the MMLS3 specimen fabrication procedure complies with the sweep test procedure that is specified by the ASTM D7000. The AST specimen used in the MMLS3 testing has a rectangular shape with curved ends (Figure 3-8 (b)) whereas the sweep test specimen is circular. The modification of the circular sweep test specimen to a rectangular specimen was necessary because, during the aggregate retention test some dislodged aggregate particles landed on untrafficked areas of the circular specimen and, therefore, were counted as retained aggregate.

The wheel path under wandering MMLS3 loading is 7.1 in. wide, yielding a total area of 72.3 in^2. Fabrication required a felt disk with a diameter of 11.8 in. and a template with a rectangular hole. The felt disk was placed on a balance, and the template was placed and centered over the felt disk. The emulsion, heated to 158°F, was sprayed onto the felt desk according to the design application rate, and the preweighted aggregate was immediately applied to the emulsion (Figure 3-6(a) and Figure 3-6(b)). Once the aggregate had been placed on the emulsion, the aggregate particles were compacted using the half-circle hand kneading compactor for three half-cycles along the wheel pass direction of the specimen (Figure 3-6 (c)). The time required for the fabrication was approximately 25 minutes.

The curing of the AST specimen was studied before developing the specimen fabrication procedure. The sweep test specimens were prepared by using a 0.35 gal/yd^2 binder application rate on the felt disks and placed in the forced mechanical convection oven at 95°F and 30 ± 3% relative humidity (RH). The degree of curing was determined by measuring the weight change of the CRS-2 in terms of time. In Figure 3-7, the weight of the CRS-2 residue is normalized by the initial weight and expressed as a percentage decrease due to the increasing curing time. Most of the water in the CRS-2 evaporated within 12 hours, and the final residue percentage was slightly over 70%, which was obtained from the emulsion residue test by evaporation. In this project, all the specimens were cured at 95°F and 30 ± 3% RH for 24 hours to fully break and cure the emulsion for the AST performance test.



(a) (b) (c) (d)

Figure 3-6 AST specimen fabrication procedure: (a) emulsion application gun; (b) AST specimen in template; (c) applied CRS-2 on the felt disk; (d) hand steel compactor

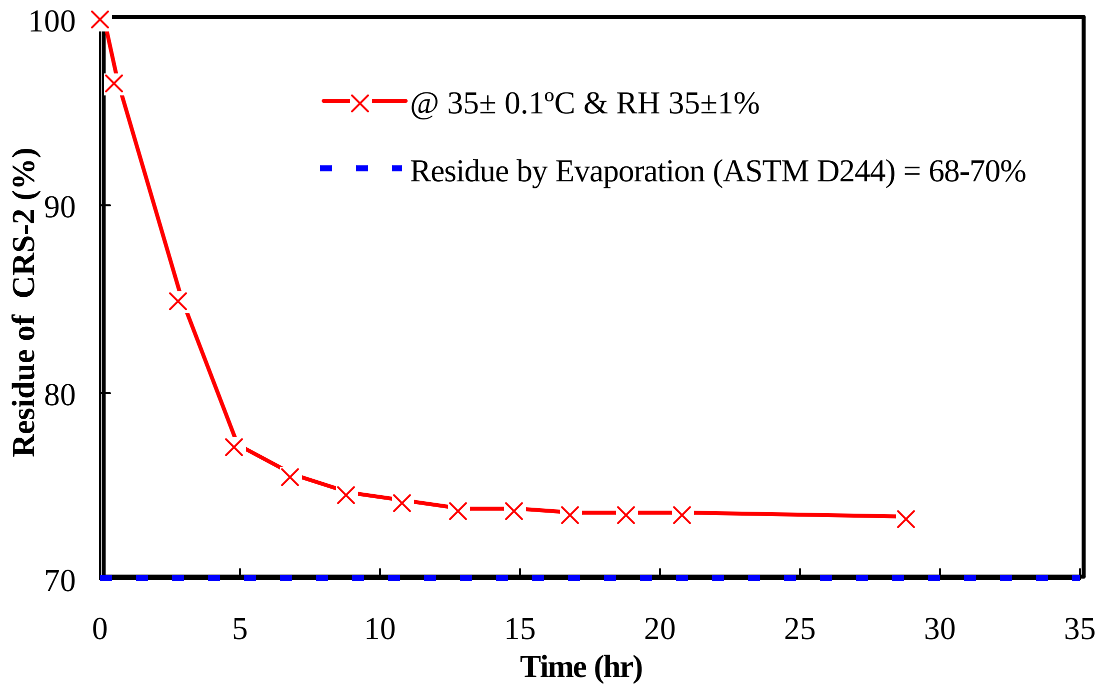


Figure 3-7 Weight changes of CRS-2 due to curing

3.3 Flip-Over Test Procedure

The flip-over test (FOT) is the part of the sweep test procedure (ASTM D7000) that measures the amount of excess aggregate on the specimen. At the end of the curing time, the specimen was turned vertically and any loose aggregate was removed by lightly brushing the specimen. The specimen was weighed before and after the FOT to determine the amount of excess aggregate on the specimen.

3.4 MMLS3 Performance Test Procedure

The MMLS3 is a third-scale unidirectional vehicle load simulator that uses a continuous loop for trafficking. It is comprised of four bogies with only one wheel per bogie. These wheels are pneumatic tires that are 11.8 inches in diameter, approximately one-third the diameter of a standard truck tire. The wheels travel at a speed of about 5,500 wheel applications per hour, which corresponds to a dynamic loading of 3.3 Hz on the pavement surface. This loading consists of a 0.3 sec haversine loading time and a rest period of 0.3 sec. The dynamic load on the pavement surface by the MMLS3 in motion was measured by a Flexiforce® pressure sensor. The mean value of maximum dynamic loads from the four wheels was approximately 802.6 lbf. The contact area was measured to be approximately 5.27 in.^{2} from the footprint of one MMLS3 wheel inflated to 101.5 psi, thus resulting in a surface contact stress of approximately 152.1 psi (Lee 2004).

The major steps in the MMLS3 test preparation are shown in Figure 3-8. For AST testing under the MMLS3, specimens were attached to thin steel plates that were fastened to a steel base plate, as illustrated in Figure 3-8 (c). MMLS3 loading was applied after a 3-hour temperature preconditioning period at 77°F. The weight of the specimen attached to the steel plate was measured before and after the MMLS3 loading to determine the aggregate loss. Also, the AST was examined visually during the MMLS3 testing to determine the amount of bleeding.

The aggregate loss during the initial traffic loading in the field (normally occurring within half a day) was measured after one wandering cycle of the MMLS3 loading. Then, MMLS3 loading was applied and the weight measurements were taken periodically over a 2-hour period (equivalent to 11,820 wheel loads) to evaluate the aggregate retention performance of the AST under traffic. The percentage of aggregate loss was calculated by the following equation:

$$\text{Aggregate Loss (\%)} = \frac{W_{Before} - W_{After}}{W_{Before}} \times 100$$

where

W_{before} = weight of aggregate on AST specimen before any loading and

W_{After} = weight of aggregate on AST specimen after MMLS3 loading.

The percentage of aggregate loss in this study was calculated based on the weight of the aggregate in the wheel path area. Therefore, the percentage of aggregate loss reported in this paper would be much higher than the field values if the field values were based on the weight of aggregate in the entire lane width.

The complete MMLS3 test procedure involves the following steps:

1. curing the specimens in the forced mechanical convection oven for 24 hours at 95°F and 30 ± 3% RH, as specified by the ASTM D7000;
2. measuring the initial specimen weight;
3. conditioning specimens at 77°F for 3 hours for the aggregate retention test;
4. MMLS3 loading for 10 minutes, which is the time required for the MMLS3 to complete one wandering cycle, and then measuring the specimen weight;
5. MMLS3 loading for 2 hours with periodic measurements of the specimen weight;
6. conducting a visual survey and other performance tests, such as the British Pendulum Test (BPT), sand patch test, transverse profiling, etc. before bleeding test;
7. conditioning at 122°F for 3 hours for the bleeding test;
8. MMLS3 loading for 4 hours at 122°F;
9. measuring the final specimen weight; and
10. conducting a visual survey and other performance tests after bleeding test, again.

Figure 3-8 MMLS3 test preparation: (a) MMLS3 test specimen; (b) specimen curing at 95°F; (c) installation of specimens on a steel base; (d) side view of MMLS3; (e) positioning MMLS3 in the temperature chamber; (f) complete MMLS3 test setup for AST testing

4. EVALUATION OF AGGREGATE RETENTION PERFORMANCE OF ASPHALT SURFACE TEATMENTS

4.1 Selection of Optimum Aggregate and Emulsion Application Rates

The application rates for the aggregate and emulsion are major factors that affect the AST aggregate loss. Therefore, in order to evaluate the effect of aggregate gradation on the aggregate loss, it is imperative to test AST specimens with varying aggregate characteristics at the optimum application rates for aggregate and emulsion. Two approaches are used in this study to determine the optimum application rates; one incorporates existing design methods and the other is based on the experience of NCDOT field engineers.

McLeod developed a surface treatment design procedure in which the aggregate application rate depends on the aggregate gradation, shape, and specific gravity. The emulsion application rate depends on the gradation, absorption, and shape of the aggregate, as well as traffic volume, existing pavement conditions, and the residual asphalt content of the liquefied asphalt. ASTs of granite and light-weight aggregate were designed using the McLeod design method which resulted in the aggregate application rates of 15.03 and 11.15 lb/yd^2 and the emulsion rates of 0.14-0.19 and 0.19-0.26 gal/yd^2 for granite and light-weight aggregates, respectively. Table 4-1 summarizes all input parameters for the McLeod design and the modified Kearby method. A visual observation of the AST specimen surface suggests that the application rates for the granite aggregate are reasonable, whereas a visual observation and preliminary MMLS3 testing of the AST specimens with light-weight aggregate reveal that the mixture was far too dry. This trend is explained by the fact that the equations in the McLeod design were developed for conventional aggregate and are not applicable to light-weight aggregate.

Table 4-1 Design input parameters for McLeod and Modified Kearby method

Type of Aggregate	Light-weight	Granite
Median Particle Size, in.	0.23	0.21
Flakiness Index, %	8.28	24.01
Average Least Dimension, in.	0.18	0.15
Voids in the Loose Aggregate	0.51	0.48
Bulk-specific Gravity	1.66	2.64
Asphalt Absorption (%)	0.48	0.38
Existing Pavement Texture	Smooth, nonporous	
Percentage Waste Allowed for	0	
Residual Asphalt Content of Emulsion	0.7	
Construction Season for Field Adjustment	Summer	

The modified Kearby method was used to design ASTs with light-weight aggregate. This design method was developed to cover both conventional and light-weight aggregates by using a laboratory board test method. The test was conducted by placing one layer of aggregate on a 1/2 yd^2 area board, as shown in Figure 4-1, and determining the aggregate application rate by converting the aggregate weight into a unit of lb/yd^2 of roadway. The emulsion application rate was determined from the dry loose unit weight and the dry bulk specific gravity of the aggregate, average depth of the aggregate particles, traffic volume, existing pavement conditions, residual asphalt content, and field seasonal adjustment factors. The relationship used in this method for light-weight aggregate results in approximately 30% more embedment than the one used for the conventional aggregate for the same mat thickness (Gransberg 2005). The aggregate and emulsion application rates for light-weight aggregate used in this study were found to be 8.4 lb/yd^2 and 0.28-0.32 gal/yd^2.

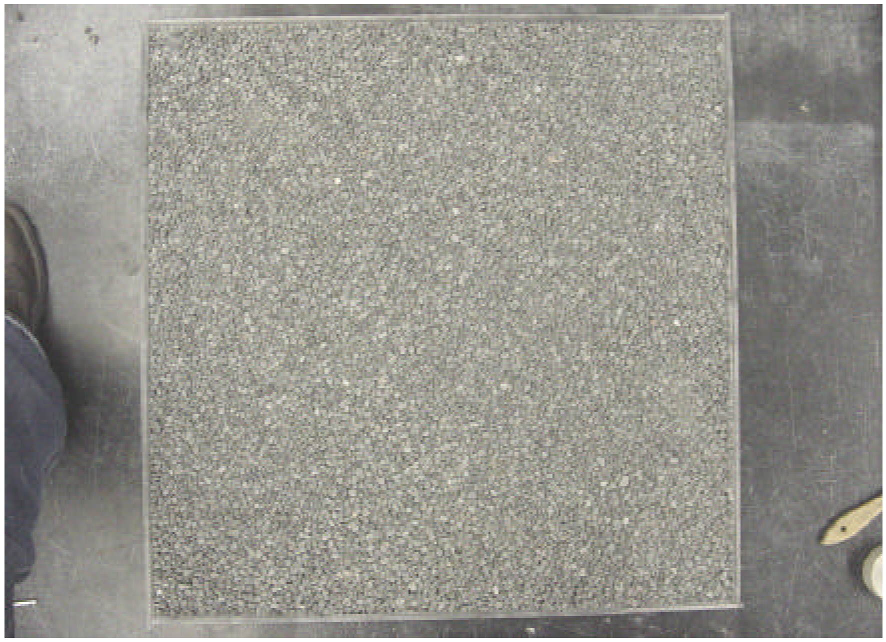


Figure 4-1 Top view of light-weight aggregate on 1/2 yd^2 board

To confirm these design rates, eight NCDOT Bituminous Supervisors and Road Maintenance Unit engineers were asked to participate in a blind test. First, the typical application rates used in North Carolina were determined by a statewide survey among seven different Divisions. Based on this survey, a total of 20 AST designs were selected for performance testing. These designs are shown in Table 4-2. For each application rate combination, three AST specimens were fabricated. One specimen was not subjected to any form of testing and was used to represent the AST surface condition immediately after construction, but before trafficking. The second specimen was subjected to the FOT to determine the amount of excess aggregate. The third specimen was subjected to the MMLS3 aggregate retention test to simulate the AST surface condition after sufficient trafficking.

Table 4-2 Application rates in optimum mix design study

Type of Aggregate	**No.**	**Aggregate lb/yd^2**	**Emulsion gal/yd^2**
Light-weight	1	8	0.26
	2		0.35
	3		0.40
	4	9	0.26
	5		0.35
	6		0.40
	7	10	0.26
	8		0.35
	9		0.40
	10	11	0.26
	11		0.35
	12		0.40
Granite	13	14	0.20
	14		0.25
	15		0.30
	16	16	0.20
	17		0.25
	18		0.35
	19	18	0.30
	20	20	0.20

The NCDOT personnel examined the surfaces of two specimens for each design, one before flip-over and trafficking, and the other after trafficking. The personnel were not made aware of the test results prior to this examination. They were asked to select one AST design from each of the light-weight and granite aggregates, based on their field experience. The optimum application rates were selected as 9 lb/yd^2-0.26 gal/yd^2 for the light-weight aggregate and 14 lb/yd^2-0.20 gal/yd^2 for the granite. These designs were chosen unanimously.

It was encouraging to find that the McLeod and modified Kearby design application rates for granite and light-weight mixtures, respectively, are close to the values selected visually by the NCDOT engineers. For the remainder of this report, the application rates chosen by the NCDOT engineers are used as the optimum application rates.

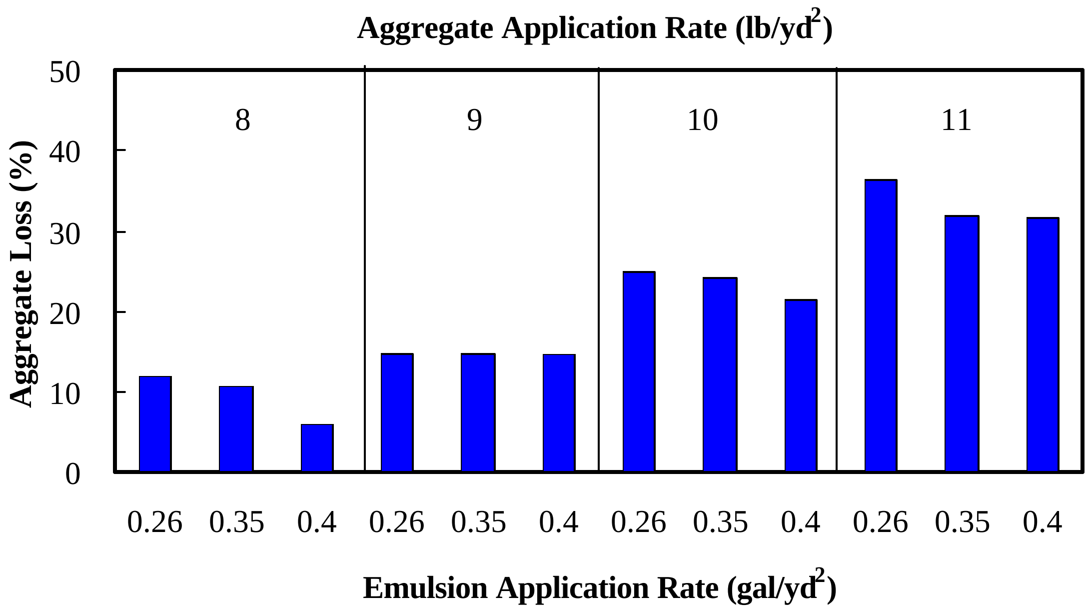
4.2 Effect of Aggregate and Emulsion Application Rates

In this section, the results from the 20 AST designs using the FOT and the MMLS3 performance test are presented to discuss the effects of aggregate and emulsion application rates on aggregate retention and bleeding. Figure 4-2 shows the FOT results for the two aggregate types at multiple rates. As expected, more aggregate was lost as the aggregate application rate increased and the emulsion application rate decreased.

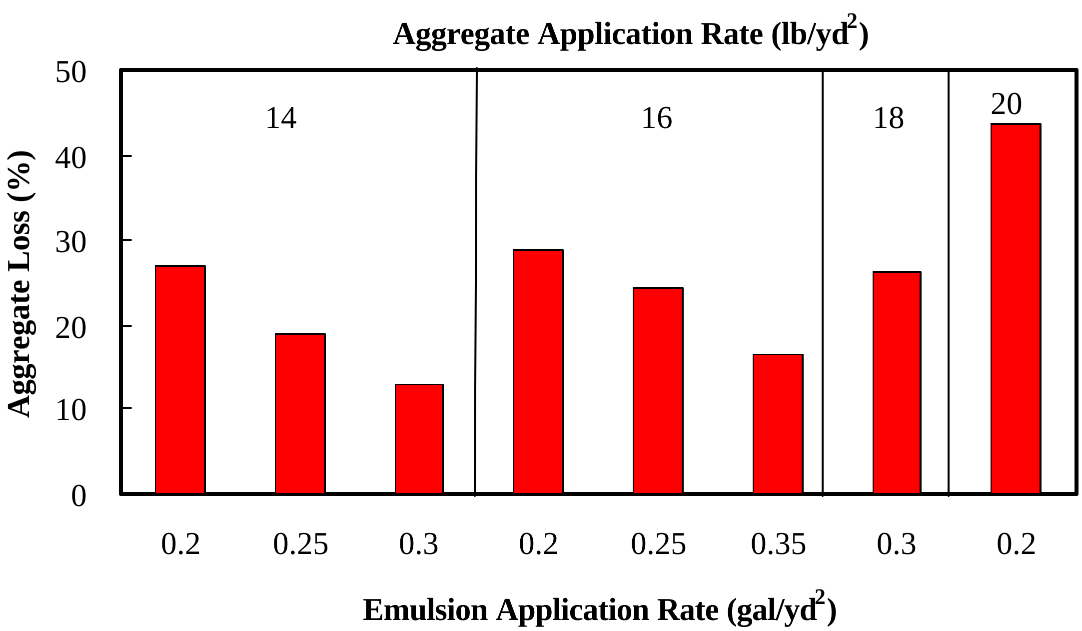
Figure 4-3 shows the percentage of aggregate loss during the 10-minute initial trafficking (after 980 wheel passes) and during the 2-hour aggregate retention test (after 11,820 wheel passes) for the light-weight and the granite aggregates. A similar trend is observed from these figures to those from the FOT; that is, as the aggregate application rate increases and the emulsion rate decreases, the percentage of aggregate loss increases. These results show only one side of the AST performance, however, meaning that too low an aggregate application rate and/or too high an emulsion rate will yield a lower aggregate loss, but may cause bleeding. For a comprehensive evaluation of AST performance, the bleeding performance should also be taken into consideration. Figure 4-4 presents the changes in the surface textures of the light-weight AST specimens after the bleeding test as the emulsion application rate changes. A visual examination of the surface textures, as presented in Figure 4-4, confirms that more bleeding occurs as the emulsion application rate increases. It is expected that a digital image analysis of these textures could yield a more definite criterion to avoid bleeding.

The results of the FOT show a greater aggregate loss than those of the MMLS3 test due to the effect of wheel compaction on excess or unbonded aggregate. This observation can be extended to claim that the conventional aggregate retention tests, which determine the aggregate loss before significant trafficking, are conservative test methods for determining aggregate retention performance.

In general, when comparing the types of aggregate, the light-weight aggregate has better retention than the granite due to its uniform size and low FI. For the light-weight aggregate, most of the aggregate loss occurs during the initial trafficking, whereas for granite, continuous aggregate loss occurs after the initial trafficking.

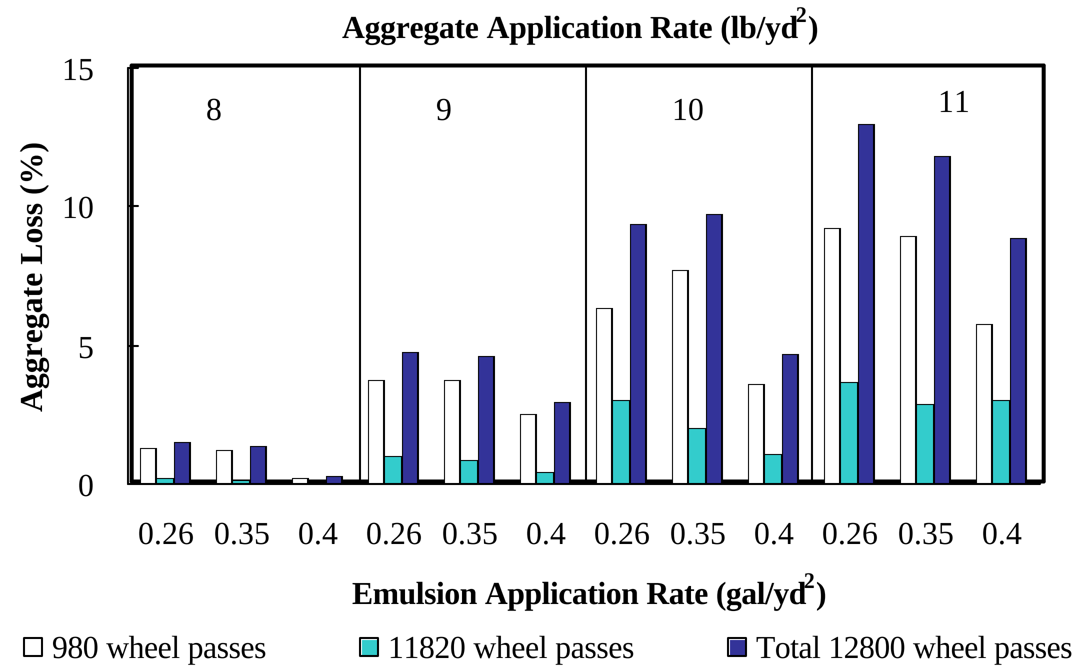


(a)

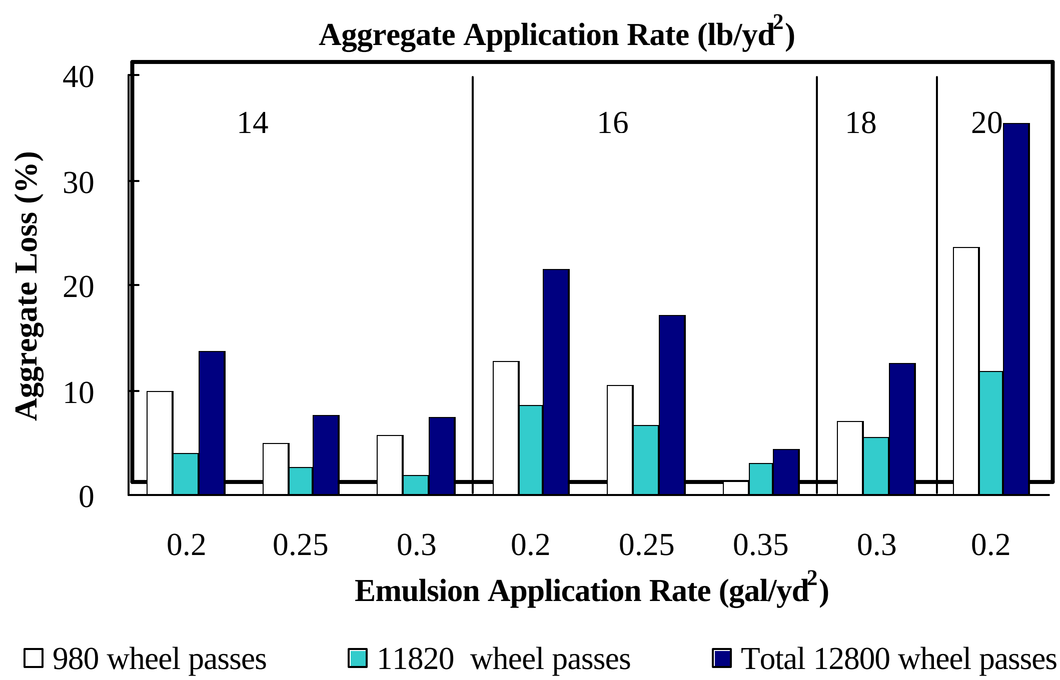


(b)

Figure 4-2 Aggregate loss measured from the flip-over test: (a) light-weight aggregate; (b) granite



(a)



(b)

Figure 4-3 Aggregate loss measured from the MMLS3 test: (a) light-weight aggregate; (b) granite

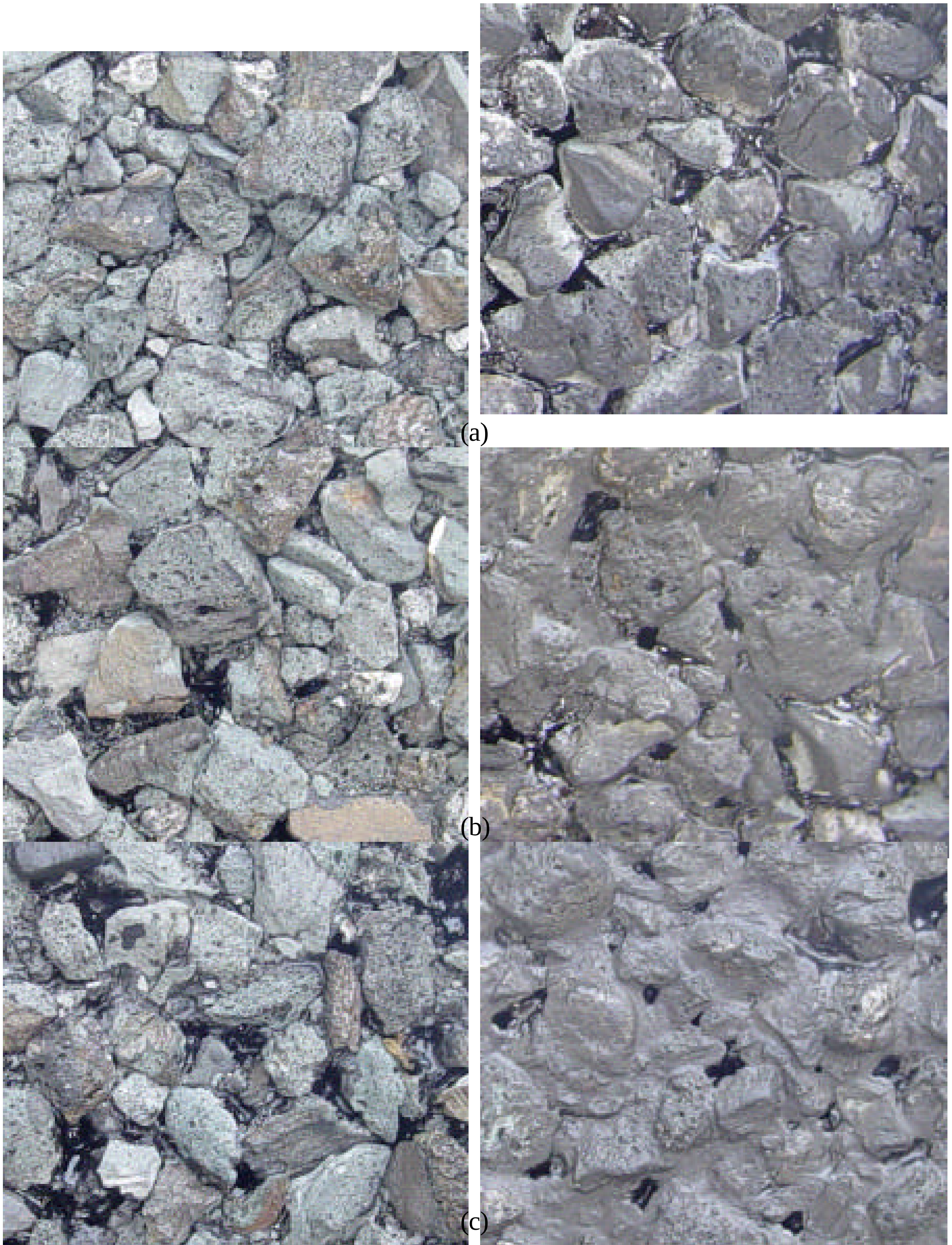
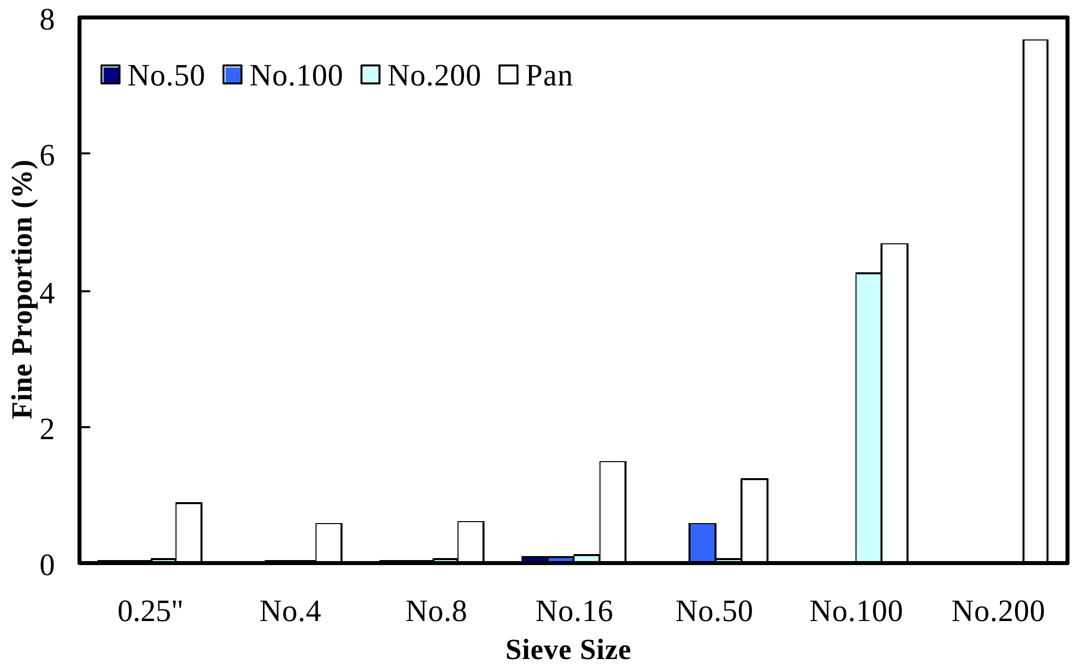


Figure 4-4 Surface texture change after the bleeding test of BST specimens with 9 lb/yd^2 of light-weight aggregate and the emulsion rates of: (a) 0.26 gal/yd^2; (b) 0.35 gal/yd^2; (c) 0.4 gal/yd^2

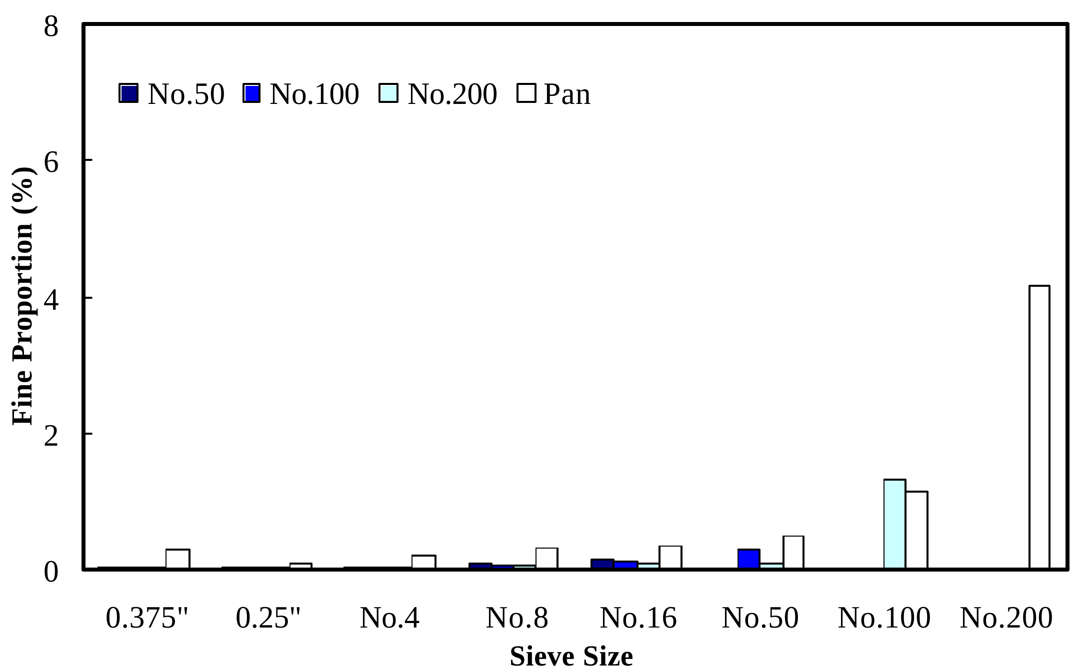
4.3 Effect of Fine Content

Since the cleanliness of the aggregate is a critical factor that influences the aggregate loss performance of ASTs, the amount of fines on coarse aggregate was measured to obtain the distribution of fines in the aggregate. Dry and wet sieve analyses with the light-weight aggregate and the granite were conducted (modified ASTM C 117). The dry-sieved aggregate was washed through the three sieves (No. 50, No. 100, and No. 200) to determine the fine particle size distribution in the aggregate.

Figure 4-5 displays two types of information: (1) the fine contents on the surface of different sized particles; and (2) gradations of fines attached to different sized aggregate particles. It can be seen from these figures that, in general, the smaller particles hold more fines. Also, it was found that the majority of fine particles attached to aggregate are the aggregate particles passing the No. 200 sieve. This finding corresponds to the definition of fines (i.e., materials passing the No. 200 sieve) specified in most agencies.



(a)



(b)

Figure 4-5 Distribution of fines: (a) light-weight aggregate; (b) granite aggregate

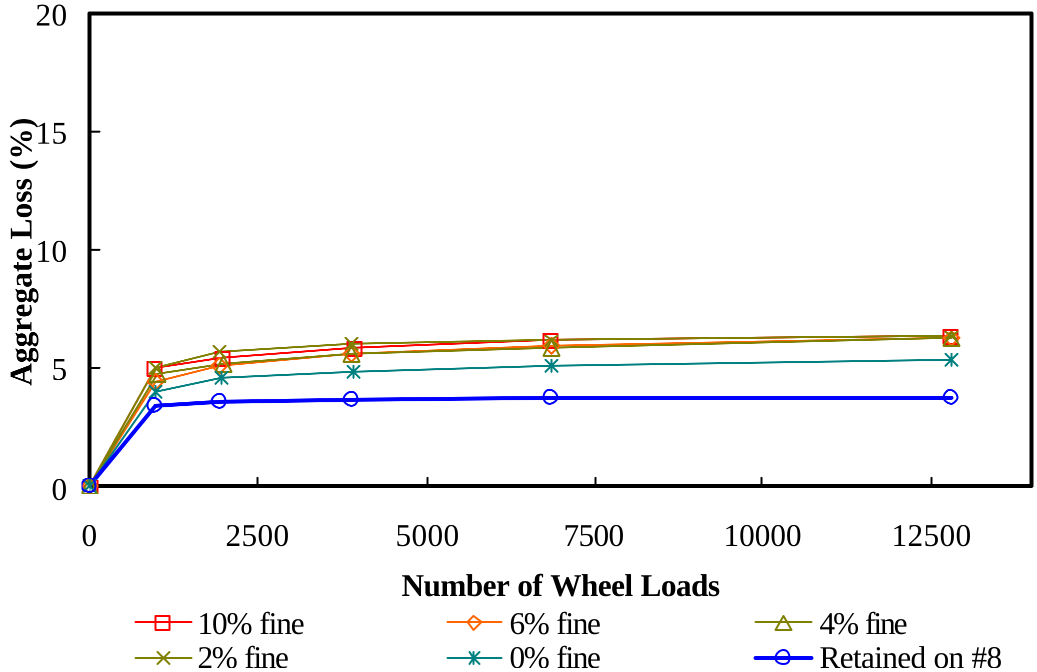
To determine the effect of the amount of fines on the AST's ability to retain aggregate, the FOT and the MMLS3 AST performance test methods were used. The aggregate and emulsion application rates were kept the same for the different fine contents. In other words, the overall aggregate application rate was kept constant as the amount of fines was increased. Therefore, a higher fine content means a lower amount of coarse aggregate. This method was expected to yield a less significant effect of fines on aggregate loss because the effect of more fines and less coarse aggregate available for the loss may cancel out each other. The decision to use this method was based on the fact that most field engineers would use the rates that they use when they begin the AST construction. Then, the engineers adjust the aggregate application rates according to the appearance of the AST surface. Therefore, the critical fine content to be determined from this approach (i.e., a constant aggregate application rate with varying fine contents) represents the maximum allowable amount of fines that does not affect the aggregate retention performance when no adjustments are made in the field.

Five fine contents were selected for testing: 0, 2, 4, 6, and 10% of the total aggregate weight. Three replicate AST specimens were fabricated and tested by both the FOT and MMLS3 test methods.

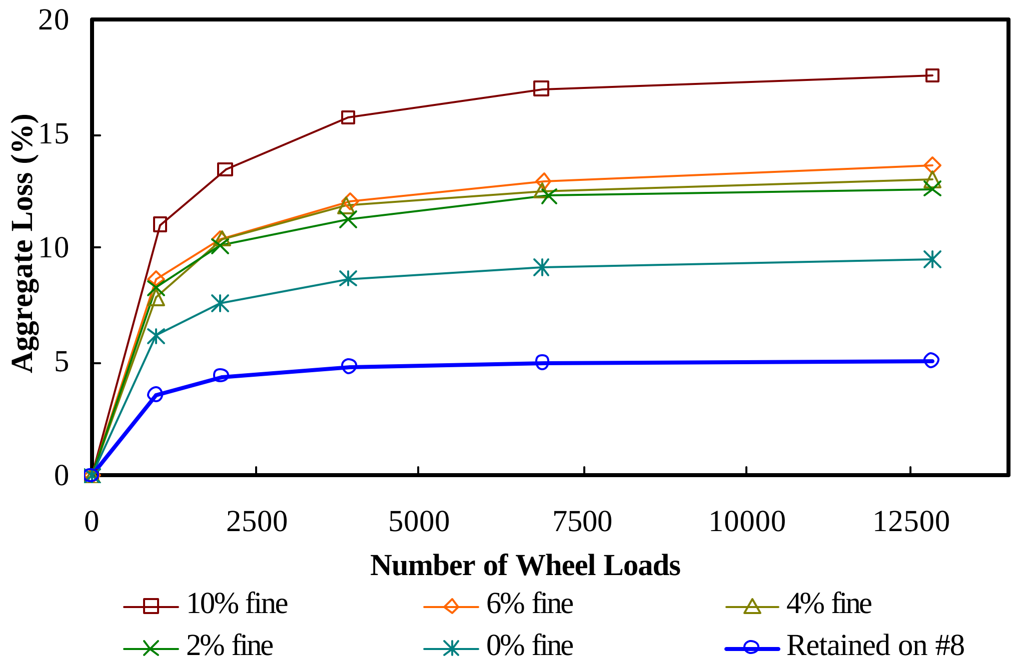
Figure 4-6 presents the MMLS3 test results for different fine contents for both aggregate types. In general, the aggregate loss increases as the number of wheel loads increases and as the fine content increases. As can be seen in this figure, most of the light-weight aggregate loss occurs during the initial trafficking, whereas continuous aggregate loss is shown for the granite. In general, the light-weight aggregate retains better than the granite.

Moreover, the effect of additional fines on aggregate loss is much less significant with the light-weight than the granite aggregate.

Figure 4-7 presents the normalized aggregate loss, which is a ratio of aggregate loss at a specific fine content to the value at 0% fine content, as a function of fine content after the aggregate retention test. The rate of increase of the aggregate loss with increasing fine content becomes significantly greater between 0% and 2% fine content in both aggregate types, and after a 6% fine content in the granite. According to Figure 4-7, the specified fine content of 2% by Kandhal and Motter (1987) yields the normalized aggregate losses of 1.32 and 1.16 for granite and light-weight aggregates, respectively, which correspond to 12.6% and 6.1% of aggregate loss. In general, a similar trend was also observed from the FOT results, seen in Figure 4-8, although the aggregate loss from the FOT is greater than that from the MMLS3 test. This difference can be explained by the compaction effect of wheel loading on the AST specimens during the MMLS3 test.



(a)



(b)

Figure 4-6 Aggregate loss performance on effect of fine content and gradation: (a) light-weight aggregate; (b) granite

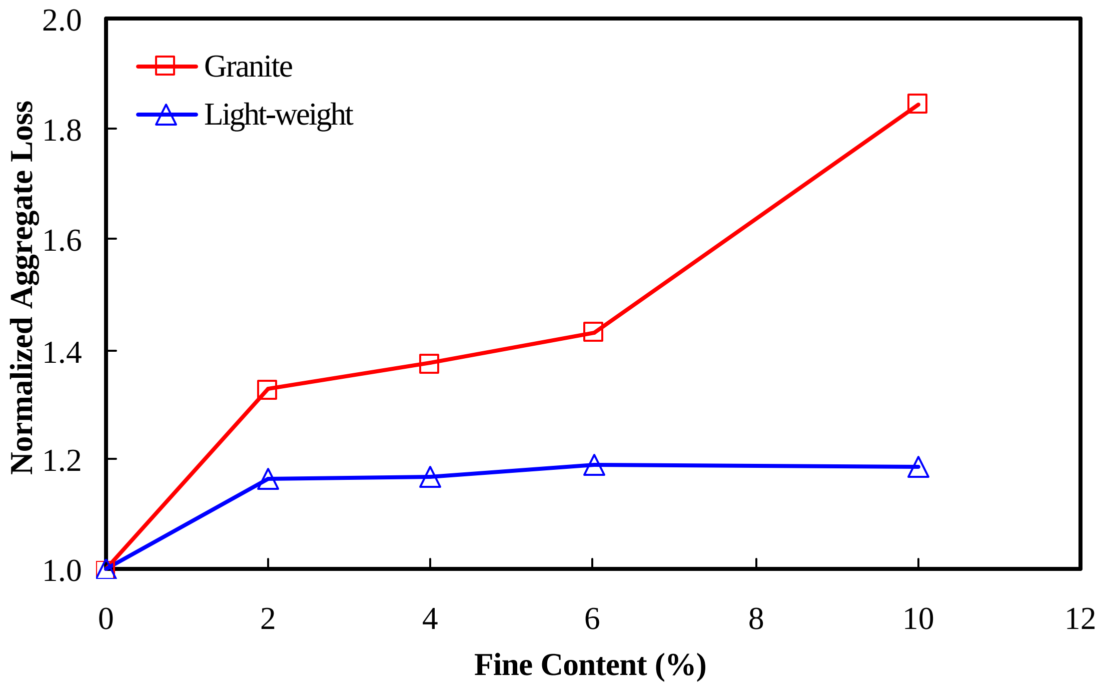


Figure 4-7 Normalized aggregate loss as a function of fine content at 12,800 MMLS3 wheel loads

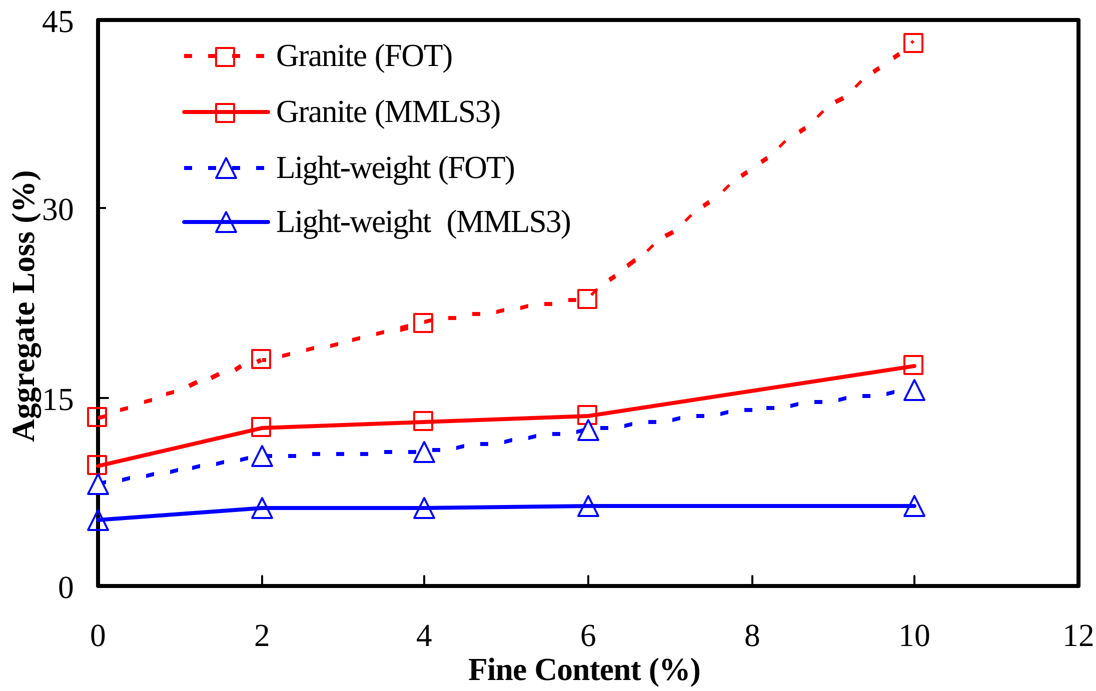


Figure 4-8 Aggregate loss as a function of fine content after the flip-over test (FOT) and at 12,800 MMLS3 wheel loads

4.4 Effect of Gradation

The effect of gradation on AST performance was evaluated by the MMLS3 test and the FOT. The majority of the aggregate particle sizes were between 1/4 in. and the No. 8 sieve for the granite and between 3/8 in. and No. 8 for the light-weight aggregate. The aggregate gradations in both aggregates were changed by removing the aggregate passing the No. 8 sieve, as shown in Figure 4-9. The uniformity coefficient was changed from 2.4 to 2.0 and from 1.7 to 1.6 for the granite and the light-weight aggregate, respectively. Due to the changed gradation, the aggregate application rates were redesigned using the board test, which is part of the modified Kearby Method. The ratio of the aggregate application rate before the gradation modification to the rate after the modification was used to determine the emulsion application rate after the modification. The aggregate application rates of 12.6 lb/yd^2 and 8.5 lb/yd^2 and the emulsion application rates of 0.18 gal/yd^2 and 0.25 gal/yd^2 were determined for the granite and the light-weight aggregate, respectively.

The effect of the aggregate gradation on the aggregate loss in the MMLS3 test is displayed in Figure 4-6. This figure shows that the removal of aggregate passing the No. 8 sieve causes a reduction in aggregate loss for both aggregates, with a much more significant effect on the granite. The reason for this reduction in aggregate loss is that the smaller particles filled in between the larger particles in the graded aggregate, and therefore, may not become effectively embedded into the applied emulsion. A similar trend is also seen from the results of the FOT. Another important observation to be made from this figure is that the aggregate loss from the modified gradation is about the same for both the granite AST and light-weight AST. Therefore, it can be concluded that the lower aggregate loss, shown in Figure 4-6 for the light-weight AST with the original gradation, has much more to do with the uniform gradation in the light-weight aggregate than the fact that it is light-weight, that it has a higher FI, or that it has a higher absorption value.

Figure 4-10 and Figure 4-11 show the surface texture changes due to the MMLS3 loading during the aggregate retention test. The surface texture changes are most evident in Figure 4-10 (a) where the granite AST specimen with the unmodified gradation is shown. It can be seen that the effect of removing the passing No. 8 sieve aggregate is much greater in the granite ASTs (Figure 4-10) than in the light-weight ASTs (Figure 4-11), supporting the quantitative observation made from the MMLS3 test in Figure 4-6.

Additional visual observation reveals that the loose coarse aggregate sits mostly on small aggregate particles embedded in the emulsion (Figure 4-12). These coarse aggregate particles that sit atop the small aggregate particles can be easily crushed or lost during compaction and trafficking.

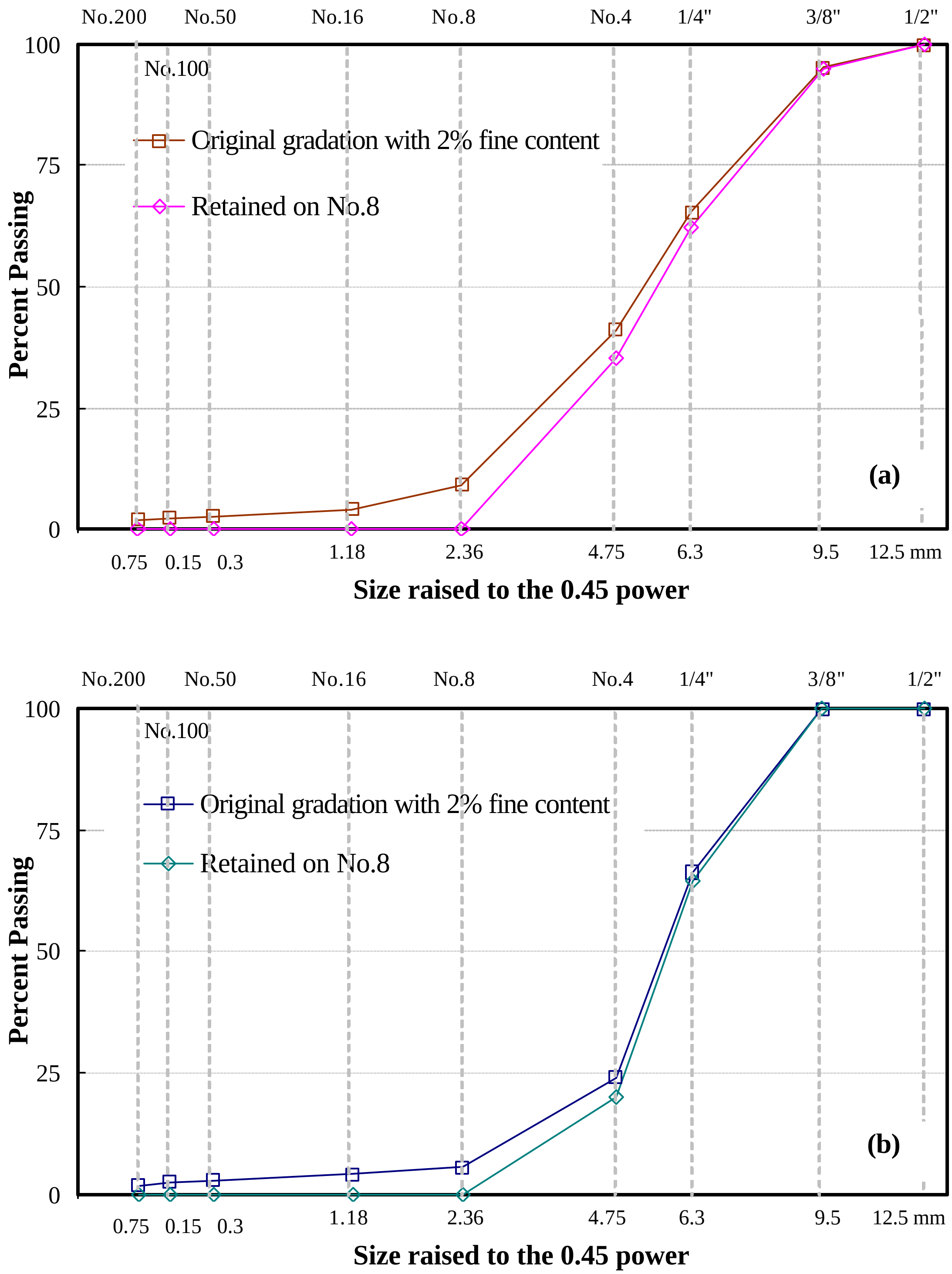


Figure 4-9 Gradation change after eliminating aggregate retained on No. 8 sieve: (a) granite; (b) light-weight aggregate

Before After

(a)

Before After

(b)

Figure 4-10 Surface textures of granite AST specimens before and after the aggregate retention test: (a) original gradation with 2% fines; (b) retained on No. 8 sieve

Before After

(a)

Before After

(b)

Figure 4-11 Surface textures of light-weight AST specimens before and after the aggregate retention test: (a) original gradation with 2% fines; (b) retained on No. 8 sieve

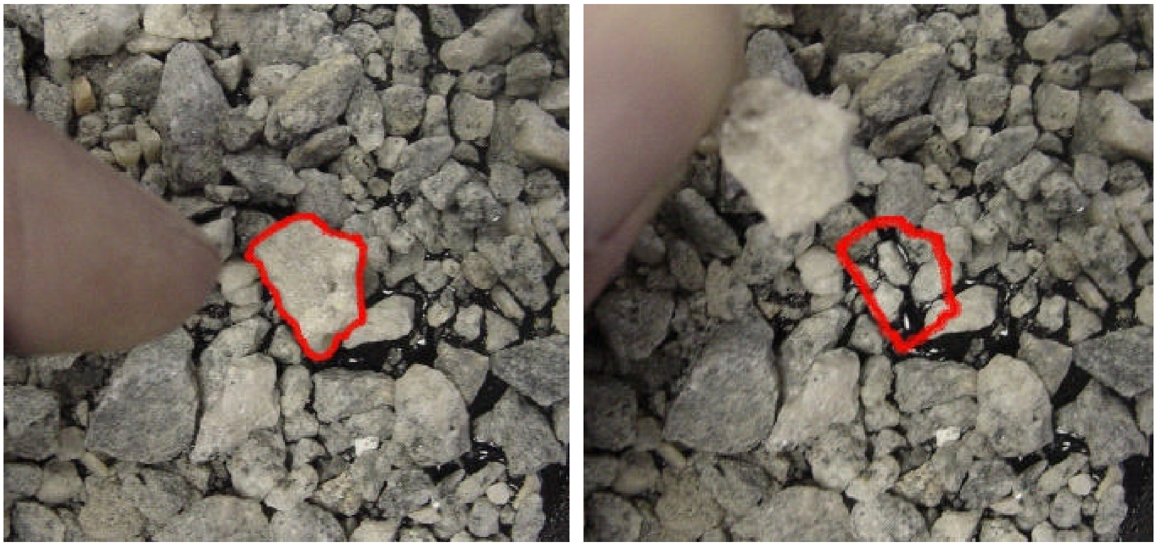


Figure 4-12 Large aggregate sitting on smaller aggregates (granite aggregate)

4.5 Aggregate Type Comparison

The effects of aggregate type on the retention performance of ASTs were studied by using the same gradation for granite and light-weight aggregate. The granite aggregate was sieved and reassembled to fit the gradation of light-weight aggregate containing 2% fine.

The board test used in the modified Kearby method was performed to adjust the application rate for the granite aggregate with the light-weight aggregate gradation. The ratio of the aggregate application rate before the gradation modification to the rate after the modification was used for the emulsion application rate. The aggregate and emulsion application rates for the granite with the light-weight aggregate gradation were found to be 13.5 lb/yd^2 and 0.19 gal/yd^2, respectively.

A comparison of MMLS3 aggregate retention performance between the two aggregates is presented in Figure 4-13. Both aggregates show essentially the same aggregate loss performance. In this comparison, the effects of gradation and fine content differences are eliminated by matching the gradations from the two aggregates. Although these aggregates have different aggregate shapes, different levels of electrical interactions between aggregate and emulsion, and different asphalt absorption levels, their effect on the aggregate loss performance seems to be minor. Therefore, it is concluded that the aggregate gradation is the major factor affecting aggregate retention performance. The FOT results yield a similar conclusion, except with higher percentages of aggregate loss than those from the MMLS3 testing.

A visual observation of the AST surface texture is presented in Figure 4-14. It was found that the light-weight aggregate has a more uniform texture than the granite, with the light-weight aggregate gradation due to the more cubical shape of the light-weight aggregate (i.e., high FI).

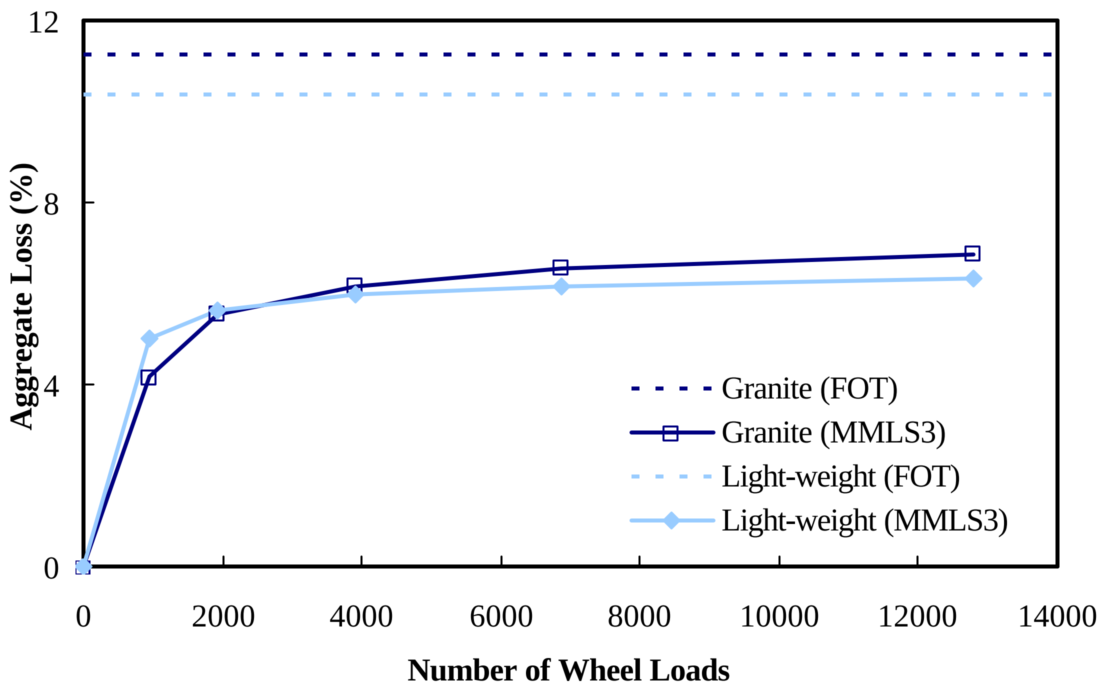


Figure 4-13 Aggregate type comparison on MMLS3 test

Before After

(a)

Before After

(b)

Figure 4-14 Surface textures of AST specimens before and after the aggregate retention test: (a) original gradation of light-weight aggregate with 2% fines; (b) granite with the light-weight aggregate gradation

5. EVALUATION OF SKID RESISTANCE TEST PERFORMANCE OF ASPHALT SURFACE TREATMENTS

One of the performance characteristics that is important in well-performing ASTs is the skid resistance. As was discussed in Chapter 2, several test methods are available for the measurement of skid resistance of ASTs, including the British Pendulum Test (BPT), the Locked Wheel Skid Test (LWST), and the GripTest (GT). Among these methods, the BPT is the only test method that can be used in the laboratory, although the test method proposed in the specification is the LWST. Since one of the objectives of this research is to develop a laboratory performance test method for ASTs, it is important to develop a relationship between the BPN and the SN. This relationship will allow the use of the BPT in the laboratory to convert the BPN to the SN in order to check against the specification. In this research, a field study was performed to find this relationship.

Three different tests, including the BPT, the LWST, and the GT, were performed on 14 AST sections in four counties of North Carolina. The characteristics of these 14 sections are summarized in Table 5-1. They were constructed in 2003, and the skid resistance was measured between November 2004 and January 2005. The materials used on the selected roads were granite (GNT), light-weight aggregate (L.W.), and their screenings (SCN). The tests were performed in both traffic directions.

5.1 Skid Resistance Tests and Results

The BPTs, as shown in Figure 5-1, were performed at four selected locations on the inner wheel path of each traffic direction, in accordance with ASTM E 303-93. The surface and air temperatures during the BPT ranged from 38°F to 87°F and from 40°F to 73°F, respectively. The average BPNs for each road are shown within a range from 71 to 90 in Table 5-1. Figure 5-2 shows the histogram of BPNs which range from 53 to 100. The mean of the BPNs for all divisions is 82.6, and the standard deviation is 8.15. Approximately 90% of the sections have a BPN value over 70, implying a high skid resistance of the pavement sections tested.

The LWST (Figure 5-3) was performed on all the selected AST roads, in accordance with ASTM E 274: Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire. The skid resistance obtained from the LWST was measured at a constant speed of 40 mph after a 0.02 in. (0.5 mm) thin layer of water was sprayed onto the inner wheel path of each traffic direction. The results from the LWST are reported as a SN and summarized in Table 5-1.

Figure 5-4 shows a histogram generated to investigate the LWST results. The distribution of the results from the skid tests ranges from 35.2 to 68.2. The mean of the SNs for all Divisions is 57.0. The standard deviation is 5.7. Approximately 88% of the SNs are over 50, and only 2% of the total SN frequency is less than 40. Most state transportation departments have established their own minimum SN requirements, usually between 28 and 41. The NCDOT's current practice is that a pavement is considered failed regarding skid resistance when the friction number measured by the LWST moving at 40 mph is below 40. The LWST results indicate that the pavement surfaces of the test sections have a higher skid resistance than is required.

The GT, shown in Figure 5-5, was also used to test the 12 AST roads, in accordance with BS 7941-2 *Surface friction of pavements – Part 2: Test method for measurement of surface skid resistance using the Grip tester braked wheel fixed slip device*. The GT is attached to the center of the towing vehicle and measures the skid resistance at the center of the lane, not the wheel path. The LWST measures the skid resistance at the wheel path. In order to test the same location using the GT and LWST, the GT towing vehicle moved from the center of the lane to the wheel path area. The friction data were collected every 1 in. at a speed of 40 mph and then averaged for a 15-foot section of pavement. Thus, the GNs reported are for the 15-foot segment. This method also utilizes a 0.02 in. (0.5 mm) thin layer of water sprayed onto the ASTs.

According to the histogram generated by the GT results in Figure 5-6, the range of values is from 0.25 to 0.99, and the standard deviation of all data is 0.14. The average GN collected is 0.78. In order to investigate the measured GN for evaluating the surface friction, evaluation and maintenance guidelines are used based on the friction level classified in Table 2-5. Most of the measured values from the field are greater than 0.43, the minimum required for 40 mph testing, except one section. From the GNs, there appears to be good skid resistance performance because 75% of all the data is greater than 0.8.

It can be concluded that, based on the above results from three different test methods, the selected ASTs in the four Divisions have good friction values.

Table 5-1 Summary of information for the skid resistance testing sites

County	SR	SN	BPN	GN	Aggregate Type		AST Type
					Bottom Layer	Top Layer	
Granville	1150	58.7	89	0.85	GNT	L.W.& GNT SCN	Triple
	1515	58.1	84	-	L.W.	GNT SCN	Split
	1122	60.7	90	0.82	GNT	GNT & GNT SCN	Triple
	1151	62.1	89	-	GNT	L.W.& GNT SCN	Triple
	1193	58.6	89	0.82	GNT	GNT SCN	Split
Alamance	1576	57.8	81	0.79	GNT	GNT & L.W. SCN	Triple
	1902	48.6	71	0.78	GNT	GNT & L.W. SCN	Triple
	1580	60.7	83	0.89	GNT	GNT & L.W. SCN	Triple
Lee	1546	61.8	85	0.80	GNT	L.W.	Split
	1313	57.5	85	0.80	GNT	L.W.	Split
Chatham	1900	57.8	79	0.70	GNT	GNT	Split
	1747	52.3	74	0.75	GNT	GNT	Split
	1733	49.8	75	0.65	GNT	GNT	Split
	1108	56.5	84	0.69	GNT	L.W.	Split

Figure 5-1 British Pendulum Test (BPT) on AST

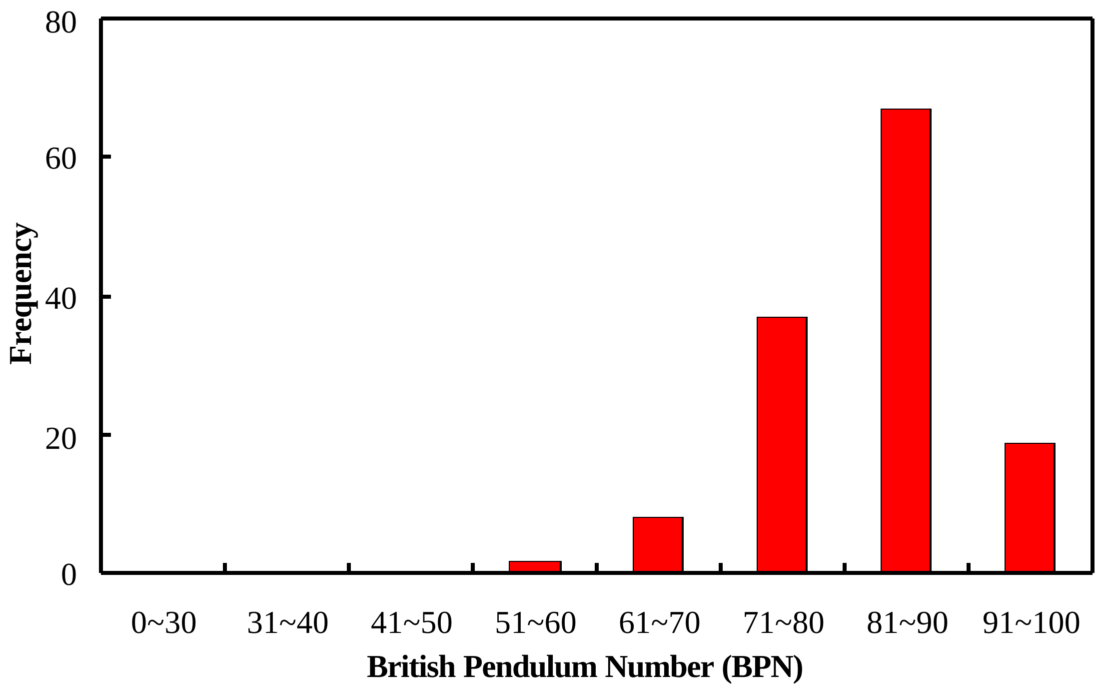


Figure 5-2 Histogram of British Pendulum Numbers

Figure 5-3 Locked Wheel Skid Tester (LWST)

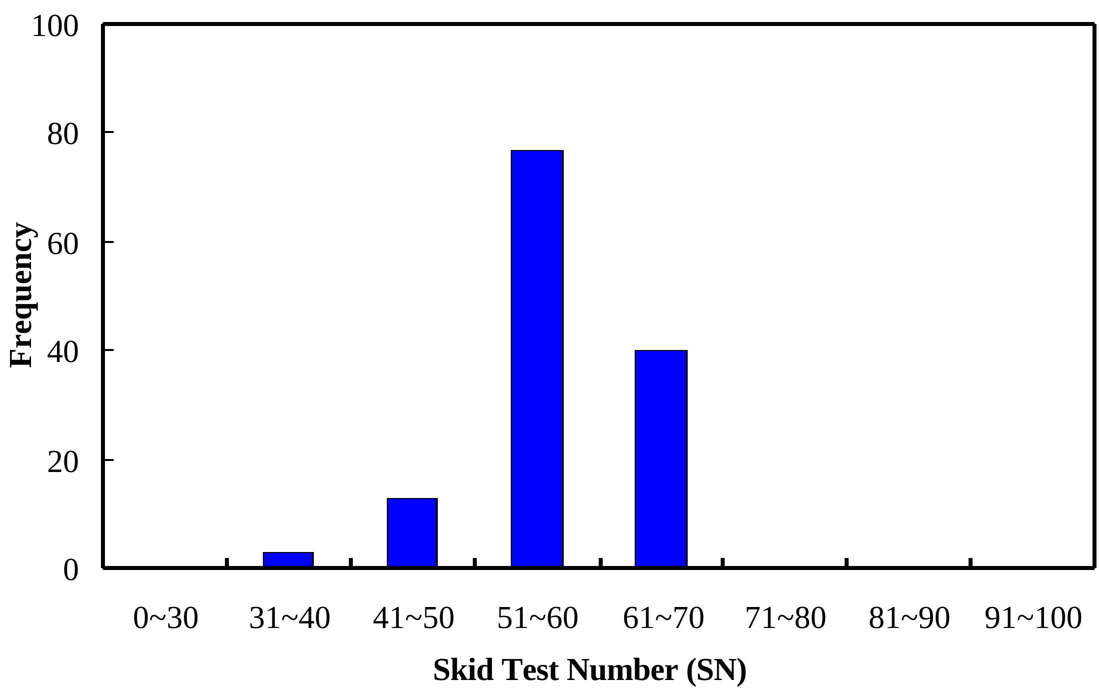


Figure 5-4 Histogram of Skid Numbers

Figure 5-5 Grip Tester (GT)

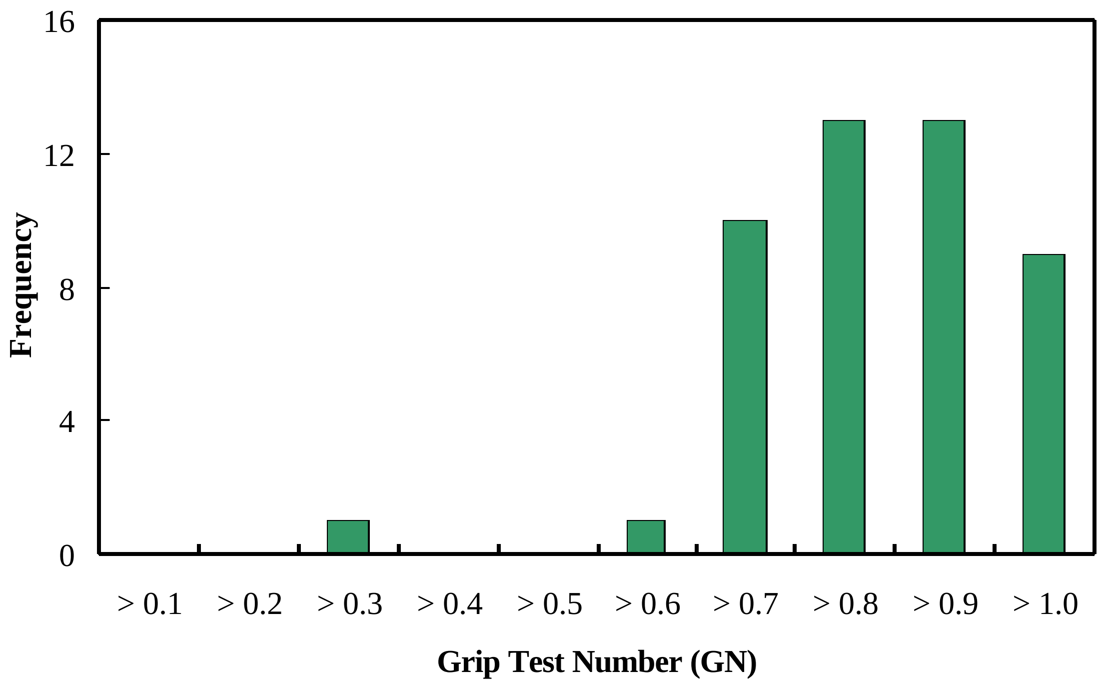


Figure 5-6 Histogram of Grip Test Numbers

5.2 *Effect of Aggregate Types on Skid Resistance*

The type of AST cover aggregate is one of the factors that may affect skid resistance performance. The skid resistance usually depends on the top layer of aggregate, but in the case of screening (SCN) aggregate as the top layer, skid resistance depends on the second layer of aggregate. As shown in Table 5-2, the top layer ASTs tested were composed of the light-weight (L.W.) 5/16 in. aggregate, the granite (GNT) 78M aggregate, and their screening aggregates. In Figure 5-7, all the test results from the three test methods are compared in terms of different aggregate types. It is noted that the GN is presented on the right side of the y-axis, and the BPN and SN are presented on the left side of the y-axis.

Table 5-2 Average skid numbers for the test sections

County	**SR**	**Average**			**Aggregate Type for Top Layer**	**AST Type**
		SN	**BPN**	**GN**		
Granville	1150	60.4	89	0.85	L.W. & GNT SCN	Triple
Granville	1151					
Granville	1122	60.7	90	0.82	GNT & GNT SCN	Triple
Alamance	1576	55.7	78.3	0.82	GNT & L.W. SCN	Triple
Alamance	1902					
Alamance	1580					
Granville	1515	58.1	84	-	L.W. & GNT SCN	Split
Granville	1193	58.6	89	0.82	GNT & GNT SCN	Split
Lee	1546	58.6	84.7	0.76	L.W.	Split
Lee	1313					
Chatham	1108					
Chatham	1900	53.3	76	0.7	GNT	Split
Chatham	1747					
Chatham	1733					

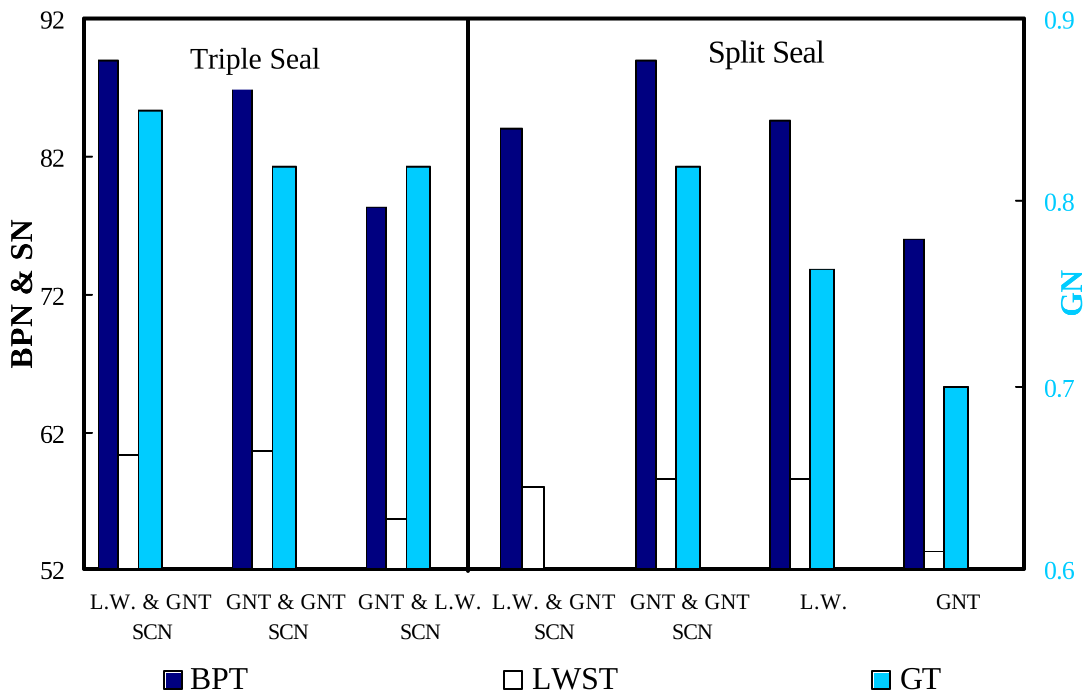


Figure 5-7 Skid resistance comparison on different types of aggregate

The first observation to be made from Figure 5-7 is that the trends from the three test methods are similar. The effect of screenings on skid resistance can be evaluated only in the split seal. The effect of screenings is quite evident in the granite and granite screenings, whereas in the lightweight aggregate with granite screenings, the screenings effect is nonexistent. Due to the limited number of sections where this type of comparison can be made fairly, it is difficult to make a firm conclusion on the effect of screenings on the skid resistance of ASTs. Also, it is difficult to draw consistent conclusions on the effects of the aggregate types because the different test methods yield slightly different rankings.

5.3 Correlations of Skid Resistance Test Methods

To develop the relationships among the results from different skid resistance test methods, the average friction numbers in each section, shown in Table 5-1, were used. It is noted that the three tests on each road were not conducted on the same day or at the same time of day, which means that the testing conditions (such as temperature) for the different tests are not the same. Such differences may have affected the skid resistance measurements.

The BPNs and SNs are plotted in Figure 5-8. A linear regression of the BPNs versus SNs results in a fairly good trend with a R^2 value of 0.74. This relationship allows the BPN from the laboratory to be converted to the SN.

The correlations between the SNs and GNs and between the BPNs and GNs are presented in Figure 5-9 and Figure 5-10, respectively. Since the mechanical systems of these two methods for measuring skid resistance are relatively similar, as opposed to that of the BPT, a better correlation result was expected. However, a linear regression of them shows a low R^2 value of 0.34. The primary reason for this low R^2 value is the variability in the GN values. Compared to the SN and BPN, the GN values are relatively insensitive from one section to another, as can be seen in Figure 5-9 in the flat slope between the GN and SN relationship. Another reason for this poor correlation is that some GT tests were conducted during different seasons than the LWST testing.

The BPNs and GNs were also used for performing the regression analysis. From Figure 5-10, the general trend also presents a low R^2 value of 0.28. The same reasons for the poor GN vs. SN correlation can explain the poor relationship between the BPN and GN.

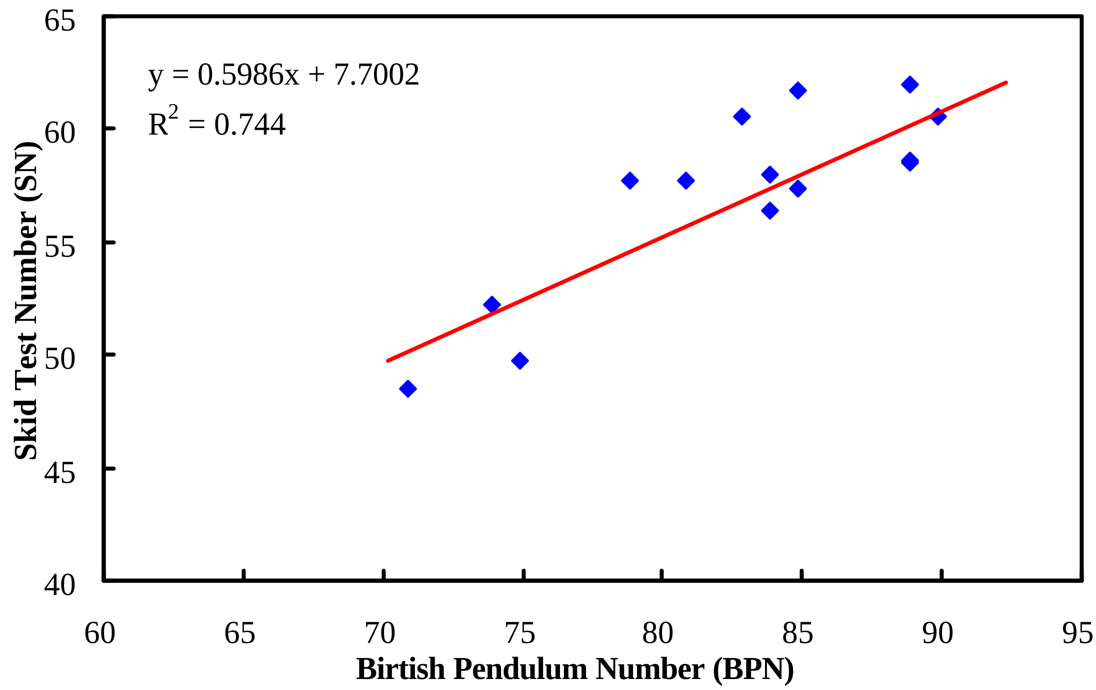


Figure 5-8 Correlation between average BPN and average SN

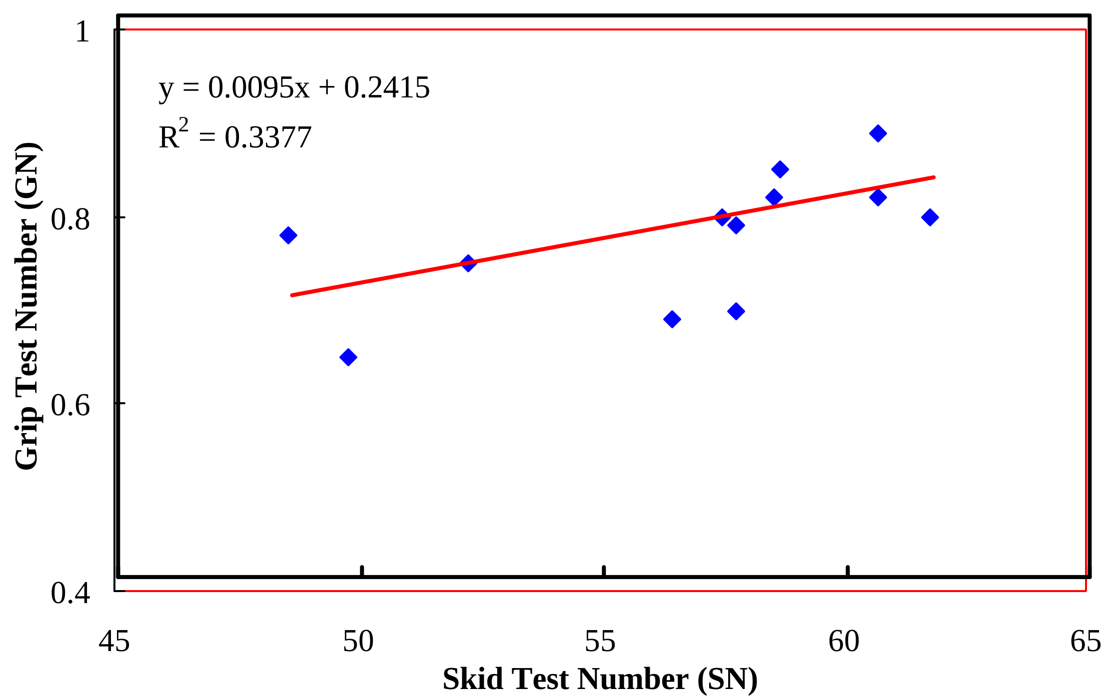


Figure 5-9 Correlation between SN and GN

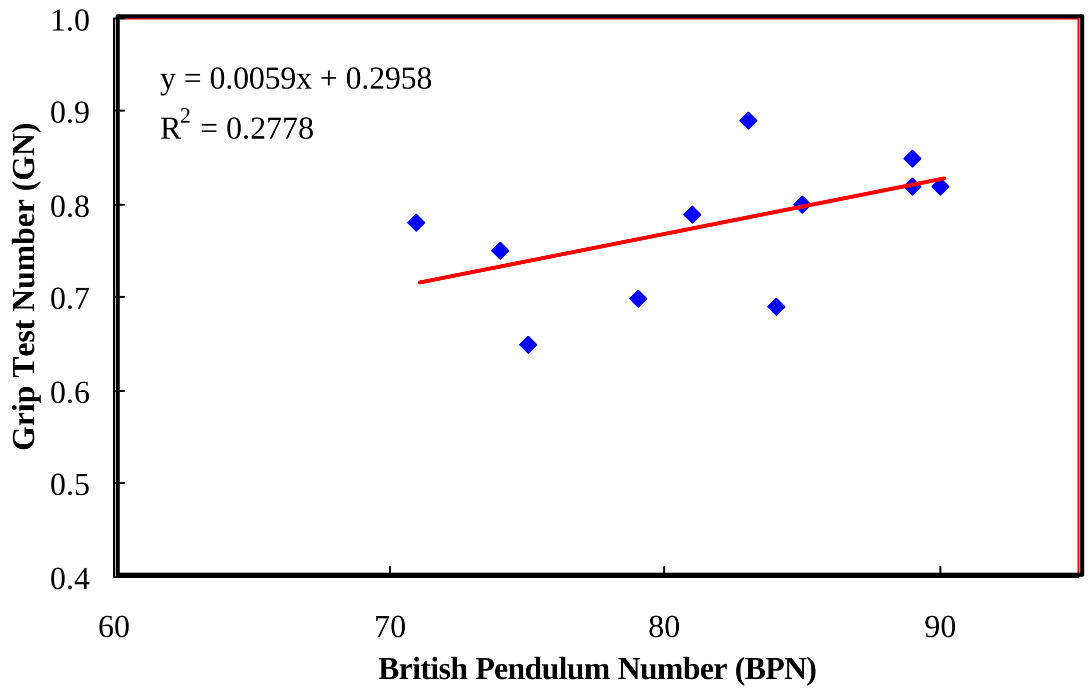


Figure 5-10 Correlation between average BPN and average GN

6. CONCLUSIONS AND FUTURE RESEARCH RECOMMENDATIONS

This report presents an experimental study on the aggregate retention and skid resistance performance of asphalt surface treatments (ASTs). Two typical AST cover aggregates used in North Carolina, granite and light-weight, were tested for their aggregate retention performance. Also, 14 selected AST roads were investigated for their skid resistance performance.

A new AST performance test method was developed using the MMLS3. It was found that this test method is an excellent means of evaluating aggregate loss due to various mixture factors. The following conclusions are drawn based on the MMLS3 AST performance test and the flip-over test (FOT) presented in this report:

1. The results from the FOT and the MMLS3 aggregate retention tests confirm that the amount of aggregate loss decreases as the aggregate application rate decreases, the emulsion application rate increases, the fine content decreases, and the gradation becomes more uniform.
2. The light-weight aggregate with a 5/16 in. NMSA, having a more cubical shape and uniform gradation, shows a better aggregate retention performance than the granite 78M shows.
3. The amount of fine has much less of an effect on aggregate loss in the light-weight AST with a more uniform gradation than in the granite AST with a less uniform gradation.

4. The aggregate gradation plays a critical role in the aggregate retention performance regardless of the type of aggregate. Regardless of aggregate type, the most critical factor in minimizing the aggregate loss in AST is uniform gradation.

Based on the findings from this study, the following two recommendations are made:

1. Maintain the maximum allowable fine content at the current specification level, i.e., 1.5%. The literature review and the national survey on the maximum allowable fine content support this recommendation. The enforcement of this specification becomes increasingly important the more the aggregate deviates from the uniform gradation.
2. Use only the materials retained on the No. 8 sieve for the AST construction. Although this recommendation may not be feasible to implement due to economic and practical constraints with regard to quarries, some Divisions may have certain local situations (such as poor performance of ASTs, a cooperative relationship with quarries, etc.) that may make the implementation of this recommendation possible. The research team believes that the impact of implementing this recommendation would be significantly positive on the performance of ASTs in North Carolina.

Skid resistance was evaluated on the 14 selected ASTs using three different test methods: the British Pendulum Tester, the Locked Wheel Skid Tester, and the GripTester. The following conclusions can be drawn:

1. The respective friction test results from the BPT, LWST, and GT for the test sections show an adequate skid resistance performance.

2. The correlation between BPNs and SNs is relatively strong with a R^2 value of 0.74. This finding indicates that the BPN can be utilized for predicting the SN.

The following recommendations are made for future research:

1. The lab-to-field correlation functions should be developed for predicting the AST aggregate retention and bleeding performance in the field.
2. A digital image analysis of AST textures for quantifying its bleeding level could yield a more definite criterion for bleeding performance.
3. A new AST design method should be developed that utilizes performance-based test methods and more advanced testing techniques (e.g., digital imaging and the laser method).
4. A field device should be developed to evaluate quality control and quality assurance of ASTs based on a mechanism similar to the MMLS3.

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