ABSTRACT

LEE, JU SANG. Performance Based Evaluation of Asphalt Surface Treatments Using Third Scale Model Mobile Loading Simulator. (Under the direction of Dr. Y. Richard Kim).

This dissertation presents the research efforts to evaluate the characteristics of asphalt surface treatment (AST) performance including aggregate retention, bleeding, and skid performance using the third-scale Model Mobile Loading Simulator (MMLS3). A new test protocol is developed that uses the MMLS3 and incorporates the digital image processing technique and British Pendulum Test (BPT) for the performance evaluation of ASTs.

In this study, the new MMLS3 AST performance test method is applied to evaluate the effects of fines content, aggregate gradation, and aggregate type (i.e., granite vs. lightweight) on aggregate retention performance. It is confirmed that aggregate retention performance is improved as the fines content decreases and the gradation becomes more uniform. Moreover, it is found that the aggregate gradation factor plays a critical role in the aggregate retention performance regardless of the type of aggregate. This research also develops a performance-based uniformity coefficient as an AST performance indicator.

A methodology is developed to determine the optimum application rate based on AST performance in laboratory tests; this methodology is then extended to the field application. Based on the characteristics of AST performance determined by MMLS3 tests with various AST application rates, the AST design equation as a function of the voids at the loose aggregate state is developed.

This research also develops a correlation that converts skid resistance laboratory results to field results. The ability of the MMLS3 test to simulate the texture of ASTs in the field is confirmed by finding the same trends in skid resistance characteristics of the two aggregate types for both laboratory and field results.
PERFORMANCE BASED EVALUATION OF ASPHALT SURFACE TREATMENT USING THIRD SCALE MODEL MOBILE LOADING SIMULATOR

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A dissertation submitted to the Graduate Faculty of North Carolina State University in partial fulfillment of the requirement for the degree of Doctor of Philosophy

Civil Engineering

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DEDICATION

To

my parents,

Huchul Lee and Sunam Park,

who made all of this possible,

for their endless love, unfailing support, and encouragement.
BIOGRAPHY

Ju Sang Lee was born in Seoul, Korea on May 10, 1971, the son of Huchul Lee and Sunam Park. He received his Bachelor’s degree in Civil Engineering in 1997 from Seoul National University of Technology, Seoul, Korea and Master’s degree in Civil Engineering in 1999 from Dangook University, Seoul, Korea. During that time, he specialized in geotechnical engineering. He went to University of North Carolina State University, joined the Ph.D. program in transportation engineering at the department of Civil Engineering.
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# TABLE OF CONTENTS

LIST OF TABLES .......................................................................................................................... vii
LIST OF FIGURES ........................................................................................................................ viii

1. INTRODUCTION .................................................................................................................. 1
1.1 Research Needs and Significance .................................................................................. 1
1.2 Research Objectives ..................................................................................................... 4
1.3 Dissertation Organization ............................................................................................ 4

2. LITERATURE REVIEW ........................................................................................................ 6
2.1 General ............................................................................................................................ 6
2.2 Effects of Aggregate Characteristics on Asphalt Surface Treatment Performance 10
   2.2.1 Aggregate Retention Test Methods ..................................................................... 10
   2.2.2 Effects of Application Rate ............................................................................... 13
   2.2.3 Effects of Fines Content ................................................................................... 13
   2.2.4 Effects of Gradation ......................................................................................... 16
2.3 Asphalt Surface Treatment Design Methods ............................................................ 17
   2.3.1 Hanson Design Method ...................................................................................... 19
   2.3.2 2004 New Zealand Method .............................................................................. 20
   2.3.3 McLeod Method ................................................................................................ 20
   2.3.4 Kearby Method .................................................................................................. 22
   2.3.5 Modified Kearby Method (Texas) .................................................................... 22
   2.3.6 Multiple AST Designs ....................................................................................... 24
   2.3.7 Other Design Methods ..................................................................................... 25
   2.3.8 Reference Voids for AST Design ..................................................................... 26
2.4 Skid Resistance of Asphalt Surface Treatments ....................................................... 30
   2.4.1 Textural Measurement ....................................................................................... 31
   2.4.2 Friction Measurement ....................................................................................... 33
2.5 Material Selection of Asphalt Surface Treatments .................................................. 36

3. MATERIALS AND TESTING METHODS ........................................................................ 38
3.1 Material Selection ......................................................................................................... 38
3.2 Component Material Properties .................................................................................. 38
   3.2.1 Gradation of Aggregate Particle Size .............................................................. 38
   3.2.2 Flakiness Index ................................................................................................. 40
   3.2.3 Average Least Dimension ............................................................................... 42
   3.2.4 Bulk Specific Gravity ....................................................................................... 42
   3.2.5 Loose Unit Weight of Aggregate .................................................................... 44
   3.2.6 Aggregate Absorption ...................................................................................... 45
   3.2.7 Residual Asphalt Content ............................................................................... 46
3.3 Specimen Preparation ................................................................................................... 47
3.4 Flip-Over Test ................................................................................................................ 49
3.5 MMLS3 Performance Test Procedure ...................................................................... 49
3.6 Bleeding (or Flushing) Measurement .......................................................................... 54
   3.6.1 Bleeding Calculation ......................................................................................... 56
4. EVALUATION OF AGGREGATE GRADATION ON AGGREGATE RETENTION PERFORMANCE
4.1 Selection of Optimum Aggregate and Emulsion Application Rates
4.2 Effects of Aggregate and Emulsion Application Rates
4.3 Effects of Fines Content
4.4 Effects of Gradation
4.5 Effects of Aggregate Type
4.6 Analytical Evaluation of Effects of Aggregate Gradation on AST Performance
4.7 Performance-Based Uniformity Coefficient

5. DEVELOPMENT OF ASPHALT SURFACE TREATMENT DESIGN METHOD
5.1 Comparison of AST Design Rates Using Various Design Methods
5.2 Development of Performance-Based Asphalt Surface Treatment Design Method
5.2.1 Experimental Program
5.2.2 Aggregate Loss Performance
5.2.3 Bleeding Performance of Asphalt Surface Treatments
5.2.4 Design Methods Comparison for AST Performance
5.2.5 Determination of Optimum AST Application Rate
5.2.6 Development of AST Design Equation under MMLS3 Testing

6. EVALUATION OF SKID RESISTANCE TEST PERFORMANCE OF ASPHALT SURFACE TREATMENTS
6.1 Skid Resistance Tests and Results
6.2 Effect of Aggregate Types on Skid Resistance
6.3 Correlations of Skid Resistance Test Methods
6.4 Skid Resistance Performance Evaluation using MMLS3

7. CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER RESEARCH
7.1 Conclusions
7.1.1 Effects of Fines Content and Gradation
7.1.2 Development of Performance-Based Asphalt Surface Treatment Design
7.1.3 Evaluation of Skid Resistance
7.2 Recommendations for Further Research
7.2.1 Evaluation of Performance-Based Uniformity Coefficient (PUC)
7.2.2 Development of Asphalt Surface Treatment Design Method

CITED REFERENCES
LIST OF TABLES

Table 2-1 Aggregate-Asphalt Compatibility Tests ................................................................. 11
Table 2-2 Specification of Maximum Percentage of Fines Content (Kandhal 1987)............. 15
Table 2-3 Summary of Design Methods ................................................................................. 18
Table 2-4 Recommended EAR and AAR (NCDOT Standard Specifications for Roads and Structures 2002) .............................................................................................................. 25
Table 2-5 PIARC Texture Definitions (Kuttesch 2004).......................................................... 31
Table 2-6 Proposed Minimum Skid Number (Henry 2000) ................................................... 34
Table 2-7 Friction Level Classifications for Runway Pavement Surfaces ............................... 35
Table 2-8 Types of Materials used for ASTs in North Carolina ............................................. 37
Table 3-1 Summary of the Sieve Analysis ............................................................................. 39
Table 3-2 Slot Sizes Required for Different Fractions of Aggregate Size ............................... 41
Table 3-3 Flakiness Index of Aggregates .............................................................................. 41
Table 3-4 Average Least Dimension of Lightweight and Granite Aggregate ......................... 42
Table 3-5 Bulk Specific Gravity (BSG) of Aggregates .......................................................... 43
Table 3-6 Loose Unit Weight Test Results ............................................................................. 45
Table 3-7 Asphalt Absorption Test Results .......................................................................... 46
Table 4-1 Design Input Parameters for McLeod and Modified Kearby Methods ................... 61
Table 4-2 Application Rates in Optimum Mix Design Study .................................................. 63
Table 4-3 Generated Optimum Gradation in 78M Specification Range ................................. 89
Table 4-4 Summary of Example Gradations .......................................................................... 93
Table 4-5 Data Summary of Three Different Gradations ...................................................... 97
Table 4-6 Summary of AST Performance Expectations Using Gradations ........................... 98
Table 5-1 Summary of Designed AST Rates ........................................................................ 100
Table 5-2 Experimental Program ...................................................................................... 105
Table 5-3 Summary of Threshold Intensities for Bleeding Measurement .............................. 109
Table 5-4 Volume Ratios of RER to RAR in AAR and EAR Combinations ......................... 114
Table 5-5 AST Aggregate Loss and Bleeding Failure with AST Rate Combinations ...... 118
Table 5-6 Summary of Optimum AST Rates and Design Factors ....................................... 122
Table 5-7 Voids Comparison between Loose Aggregate State and Compacted Aggregate State by MMLS3 ........................................................................................................... 122
Table 6-1 Summary of Information for Skid Resistance Test Sections ................................. 127
Table 6-2 Average Skid Numbers for Test Sections ............................................................. 132
LIST OF FIGURES

Figure 2-1 Typical streaking of AST ........................................................................................ 8
Figure 2-2 Typical debonding failure of AST ................................................................. 9
Figure 2-3 Partial bleeding failure of AST ........................................................................... 9
Figure 2-4 States of embedment of AST in asphalt .......................................................... 19
Figure 2-5 Reference voids for AAR determination with various design methods ........ 29
Figure 2-6 Reference voids for EAR determination with various design methods .......... 29
Figure 3-1 Aggregate particle size gradations ................................................................. 40
Figure 3-2 Flakiness Index plate gauge ............................................................................. 41
Figure 3-3 Volume comparison of 0.22 lb (100 g) granite (left) and lightweight (right) aggregate ................................................................. 43
Figure 3-4 Loose unit weight test ....................................................................................... 44
Figure 3-5 Rice vacuum saturation test ............................................................................. 46
Figure 3-6 AST specimen fabrication procedure: (a) emulsion application gun; (b) applied CRS-2 emulsion on the felt disk; (c) AST specimen in template; (d) hand steel compactor ................................................................. 48
Figure 3-7 Weight changes of CRS-2 emulsion due to curing .......................................... 49
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 ........................................................................... 53
Figure 3-9 DIP procedure of AST specimen: (a) actual specimen; (b) image acquisition using scanner (grayscale); (c) image processing; (d) data file ........................................................................... 55
Figure 3-10 Photos of AST surface: (a) digital image of AST; (b) mapped bleeding area .. 59
Figure 3-11 Determination of critical bleeding GIV ........................................................................... 59
Figure 4-1 Top view of lightweight aggregate on 1/2 yd² board ........................................... 62
Figure 4-2 Aggregate loss measured from the flip-over test: (a) lightweight aggregate; (b) granite aggregate ........................................................................... 66
Figure 4-3 Aggregate loss measured from the MMLS3 test: (a) lightweight aggregate; (b) granite aggregate ........................................................................... 67
Figure 4-4 Surface texture change after the bleeding test of AST specimens with 9 lb/yd² of lightweight aggregate and EARs of: (a) 0.26 gal/yd²; (b) 0.35 gal/yd²; (c) 0.4 gal/yd². 68
Figure 4-5 Distribution of fines: (a) lightweight aggregate; (b) granite aggregate .......... 70
Figure 4-6 Effect of fines content and gradation on aggregate loss performance: (a) lightweight aggregate; (b) granite ........................................................................... 73
Figure 4-7 Normalized aggregate loss as a function of fines content at 12,800 MMLS3 wheel loads ........................................................................... 74
Figure 4-8 Aggregate loss as a function of fines content after the flip-over test (FOT) and at 12,800 MMLS3 wheel loads ........................................................................... 74
Figure 4-9 Gradation change after eliminating aggregate retained on No. 8 sieve: (a) lightweight; (b) granite aggregate ........................................................................... 77
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 .......... 78
Figure 4-11 Surface textures of lightweight AST specimens before and after the aggregate retention test: (a) original gradation with 2% fines; (b) retained on No. 8 sieve .......... 79
Figure 4-12 Large aggregate sitting on smaller aggregates (granite aggregate) .................. 80
Figure 4-13 Comparison of aggregate type ....................................................................... 81
Figure 4-14 Surface textures of AST specimens before and after the aggregate retention test: (a) original gradation of lightweight aggregate with 2% fines; (b) granite with the lightweight aggregate gradation ................................................................. 82
Figure 4-15 Scheme of McLeod’s AST failure criteria .......................................................... 83
Figure 4-16 Performance prediction of 78M specification .................................................. 86
Figure 4-17 AST performance in terms of median sizes .................................................... 87
Figure 4-18 AST performance in terms of gradations ....................................................... 87
Figure 4-19 The most uniform gradations within 78M specification range ......................... 88
Figure 4-20 Determination of median size showing the best AST performance ................ 88
Figure 4-21 Performance prediction of lightweight aggregate gradation ............................ 90
Figure 4-22 Performance prediction of granite aggregate gradation .................................. 90
Figure 4-23 Aggregate particle setting comparison in ASTs: (a) lightweight aggregate with 9 lb/yd2 and 0.25 gal/yd2; (b) granite aggregate with 14 lb/yd2 and 0.2 gal/yd2 ....... 91
Figure 4-24 Examples of gradation uniformity .................................................................. 93
Figure 4-25 Three created gradations with the same median for performance comparison... 97
Figure 5-1 AST application rates using different methods: lightweight aggregate ............. 101
Figure 5-2 AST application rates using different methods: granite aggregate ................. 101
Figure 5-3 Procedure for developing performance-based AST design ............................... 104
Figure 5-4 Aggregate loss performances with various AAR and EAR combinations: (a) lightweight aggregate; and (b) granite aggregate .................................................. 107
Figure 5-5 Section views of ASTs: (a) uniform-sized aggregate; and (b) graded aggregate ......................................................................................................................... 108
Figure 5-6 Residual and applied rates on AST specimens in MMLS3 test: (a) lightweight aggregate; and (b) granite aggregate ............................................................... 108
Figure 5-7 Critical GIVs in GIV histograms: (a) lightweight aggregate; (b) granite aggregate ......................................................................................................................... 110
Figure 5-8 Scheme of AST bleeding performance ............................................................. 112
Figure 5-9 Bleeding performances after MMLS3 bleeding test: (a) lightweight aggregate with AARs; (b) lightweight aggregate with RARs; (c) granite aggregate with AARs; (d) granite aggregate with RARs ......................................................... 113
Figure 5-10 Aggregate loss and bleeding performances with various design methods: (a) lightweight aggregate; and (b) granite aggregate .......................................................... 116
Figure 5-11 Voids reduction of lightweight and granite aggregate by MMLS3 .................. 123
Figure 5-12 Voids relationship between loose aggregate and compacted aggregate by MMLS3 .................................................................................................................. 123
Figure 6-1 British Pendulum Test (BPT) on AST ................................................................. 128
Figure 6-2 Histogram of British Pendulum Numbers (BPNs) .............................................. 128
Figure 6-3 Locked Wheel Skid Test (LWST) .................................................................... 129
Figure 6-4 Histogram of Skid Numbers (SNs) .................................................................. 129
Figure 6-5 Towing vehicle and Grip Tester (GT) .............................................................. 130
Figure 6-6 Histogram of Grip Numbers (GNs) .................................................................. 130
Figure 6-7 Skid resistance comparison among different types of aggregate ................. 133
Figure 6-8 Correlation between average BPN and average SN............................................. 134
Figure 6-9 Correlation between average SN and average GN.............................................. 135
Figure 6-10 Correlation between average BPN and average GN......................................... 135
Figure 6-11 BPN and SN of the granite and the lightweight aggregate on the straight seal 137
Figure 7-1 Digital image of AST cut surface of lightweight aggregate ................................. 142
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. ASTs are used to seal the existing pavement’s surface cracks, improve ride quality, and protect 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 North Carolina Department of Transportation’s (NCDOT’s) pavement preservation program. ASTs in North Carolina cover approximately 50% of paved road miles, but comprise 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 and construction 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.

The vast majority of agencies use quantities of emulsion and aggregate as determined by experience and/or precedence (Roberts et al. 1996). This lack of sound design methodology may result in ASTs that have poor performance characteristics. Although typical aggregate and emulsion application rates (AARs and EARs) are available in specifications such as ASTM D 1369-84 and the NCDOT Standard Specifications for Roads and Structures, it is recommended that a design method be used that computes the optimum AAR and EAR for each given job condition. The development of AST design methodology essentially ceased in 1970 in North America with the introduction of the McLeod method (1969), which was subsequently adopted by the Asphalt Institute (Gransberg 2005).

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 assess 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 to evaluate 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 that incorporates the digital image processing (DIP) technique and the British
Pendulum Test (BPT).

The AST MMLS3 performance test developed in this study is applied to: 1) an
evaluation of the effects of fines content and gradation on the aggregate retention
performance and 2) an investigation of the characteristics of aggregate retention and bleeding
performance using various AST application rates, thus leading to the development of an AST
design equation under MMLS3 test conditions in terms of voids at the loose aggregate state.
Two types of aggregate (i.e., granite and expanded slate lightweight 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.
Several skid resistance measurement tests are available, including the British Pendulum Test
(BPT), Locked Wheel Skid Test (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 NCDOT specifications is the LWST. Therefore, the focus of this study
is to develop a relationship between the British Pendulum Number (BPN) obtained from the
BPT and the Skid Number (SN) obtained 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.
1.2 Research Objectives

The primary objectives of the research are:

1. to develop a performance-based test method that can be used to evaluate various performance characteristics of ASTs;
2. to evaluate the effects of fines content and aggregate gradation on the AST aggregate retention performance;
3. to investigate the characteristics of AST performance with various AST application rates and evaluate the current AST design methods;
4. to develop a design tool for ASTs under MMLS3 loading conditions as a function of voids at the loose aggregate state; and
5. to evaluate the skid resistance of selected ASTs in North Carolina using different test methods and to develop the relationships among their friction numbers.

1.3 Dissertation Organization

This dissertation is composed of seven chapters. Chapter 1 introduces the research and presents the research needs and objectives. Chapter 2 summarizes the literature review of AST aggregate retention performance test methods, the effects of various factors on AST performance, and the current AST design methods. 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 (FOT), the MMLS3 test, and the digital image processing (DIP) technique used in this study to evaluate aggregate retention and bleeding 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 of the effects of AST application rates, fines content and gradation on aggregate retention performance. Chapter 5 discusses the characteristics of aggregate retention and bleeding performances with the various AST application rates, an evaluation of current design methods, and the effects of gradation and loose aggregate voids on the optimum application rate. Chapter 6 discusses the results of skid resistance performance tests obtained from the three different test methods and their correlations. The application of the BPN obtained from the laboratory to the SN expected in the field is introduced in this chapter. Conclusions from this research and future research recommendations are given in Chapter 7.
2. LITERATURE REVIEW

2.1 General

Several similar terms for asphalt surface treatment (AST) exist in the literature, including chip seal, seal coat, surface treatment, bituminous surface treatment, sprayed seal (Austria), and surface dressing (United Kingdom). The official term used in NCDOT specifications is asphalt surface treatment (AST) (NCDOT Standard Specifications for Roads and Structures 2002).

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 is 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 enhanced 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 operations were 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).

Originally, ASTs were used predominantly as wearing courses in the construction of low traffic volume roads, but they 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) count 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 wear, skid resistance, oxidation, and water infiltration. In North America, the evidence of distress and the prevention of water infiltration constitute the most common triggers 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 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 an 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 asphalt 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 ASTs include loss of aggregate and loss of skid resistance. Because these are the two distresses evaluated in this study, their causes and measurement methods are described in more detail in the following subsections.
Figure 2-2 Typical debonding failure of AST

Figure 2-3 Partial bleeding failure of AST
2.2 Effects of Aggregate Characteristics on Asphalt Surface Treatment Performance

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 into the asphalt, 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 a reseal alone will not normally last the expected life of the AST (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. Also, the McLeod procedure recognizes that some of the cover aggregate will be thrown to the side of the roadway by passing vehicles as the fresh seal coat is curing. The amount of aggregate that is “whipped off” in this manner is related to the speed and number of vehicles on the new seal coat. To account for this occurrence, a traffic whip-off factor is included in the aggregate design equation. Reasonable values to assume are 5% for low volume residential type of traffic and 10% for higher speed roadways, such as county roads (Alaska DOT and Public Facilities 2001).

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-asphalt 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-Asphalt Compatibility Tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Agency</th>
<th>Characteristic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate Retention Test</td>
<td>Texas DOT, Tex-216-F</td>
<td>Light Sweep</td>
</tr>
<tr>
<td>Vialit Test</td>
<td>French Public Works</td>
<td>Inverted Tray, Ball Impact</td>
</tr>
<tr>
<td>Pennsylvania Retention Test</td>
<td>Pennsylvania DOT</td>
<td>Inverted Tray, Sieve Shaker</td>
</tr>
<tr>
<td>AST Sweep Test</td>
<td>ASTM WK139</td>
<td>Replicates Sweeping</td>
</tr>
<tr>
<td>Macrosurfacing Sweep Test</td>
<td>Koch Materials TM101</td>
<td>Replicates Sweeping</td>
</tr>
</tbody>
</table>

Hank and Brown (1949) developed the ART 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 effects 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 adheres. Benson and Gallaway constructed the AST by applying the asphalt with a small laboratory sprayer using standard distributor nozzles and distributing the cover aggregate. The ART was 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 a sweeping procedure after 30 minutes of curing and also a rolling procedure after 10 minutes of curing. This test procedure has been evaluated using an AST sample fabricated in the field and examined for the effects of aggregate gradation on different asphalts by Davis et al. (1991).

The 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 asphalt 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 sweeping procedure in AST construction and measures the aggregate loss. The macrosurfacing test, modified from the Abrasion Cohesion Test Esso (ACTE), is very similar to the sweep test, but a hose replaces the brush.
2.2.2 Effects 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 applying too much aggregate, materials are wasted and excess aggregate may be whipped off 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: pavement distress and vehicular distress.

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 Effects of Fines 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 cannot stick properly because the dust produces a film
that 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 AST aggregate with clean, potable water prior to application may assist in removing fine particles that prevent adhesion with the asphalt (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).

Dust is normally defined as the percentage of fines that passes the No. 200 sieve. To improve the quality of the material in ASTs, the percentage of fines 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 fines contents for various states are summarized in Table 2-2.

The effect of fines 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 the PART and found that the rate of increase in aggregate loss with increasing fines content becomes significantly greater above about 3% dust content in most cases. Therefore, Kandhal considers 3% a threshold value. Because most states
specify a maximum of 2% dust for unwashed aggregates, Kandhal reports that 2% seems to
be reasonable for low volume traffic 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-asphalt bond. He found that fines content affects the aggregate
retention independently of the embedment depth.

Table 2-2 Specification of Maximum Percentage of Fines Content (Kandhal 1987)

<table>
<thead>
<tr>
<th>State</th>
<th>Maximum Percentage Passing No. 200</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alabama</td>
<td>1.0</td>
</tr>
<tr>
<td>Florida</td>
<td>3.75</td>
</tr>
<tr>
<td>Indiana</td>
<td>2.0</td>
</tr>
<tr>
<td>Kansas</td>
<td>2.0</td>
</tr>
<tr>
<td>Maryland</td>
<td>1.0</td>
</tr>
<tr>
<td>North Carolina</td>
<td>1.5</td>
</tr>
<tr>
<td>North Dakota</td>
<td>4.0</td>
</tr>
<tr>
<td>Ohio</td>
<td>3.0</td>
</tr>
<tr>
<td>Pennsylvania</td>
<td>2.0</td>
</tr>
<tr>
<td>South Carolina</td>
<td>0.0</td>
</tr>
<tr>
<td>South Dakota</td>
<td>2.0</td>
</tr>
<tr>
<td>Tennessee</td>
<td>1.0</td>
</tr>
<tr>
<td>Average</td>
<td>1.9</td>
</tr>
</tbody>
</table>
2.2.4 Effects of Gradation

Aggregate gradation plays a key role in the design, construction, and performance of ASTs. 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. Such contact increases the frictional area and, thus, there is better skid resistance as long as the correct quantity of asphalt is used (Herrin et al. 1968).

The aggregate should be as close to uniform size as is economically practical so that the AST 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 ones 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 of 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 fines content from 0 to 30% of the aggregate causes a 10% reduction in aggregate retention. Therefore, in order to retain 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 gradation 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 preferred.

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

2.3 Asphalt Surface Treatment Design Methods

The earliest design procedure for ASTs was developed by Hanson (1934/35) in New Zealand. His design methodology is incorporated in all major AST design methods in current practice. The most recent AST design method that evolved from the Hanson method is the 2004 Chipseal Design in New Zealand (hereinafter called 2004 New Zealand method). The modified Kearby method and McLeod method are the most popular AST design methods in North America (Gransberg 2005). The characteristics of the Hanson, McLeod, 2004 New Zealand, Kearby, and modified Kearby design methods are provided in the following subsections, and the modified Kearby method, McLeod method, and 2004 New Zealand method are summarized in Table 2-3.
### Table 2-3 Summary of Design Methods

<table>
<thead>
<tr>
<th>Factors for aggregate application rate (AAR)</th>
<th>Modified Kearby</th>
<th>McLeod</th>
<th>2004 New Zealand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Board test</td>
<td>Aggregate gradation</td>
<td>Aggregate gradation</td>
<td>Aggregate gradation</td>
</tr>
<tr>
<td>Aggregate gradation</td>
<td>Flakiness index</td>
<td>Flakiness index</td>
<td>Flakiness index</td>
</tr>
<tr>
<td>Bulk specific gravity of aggregate</td>
<td>Traffic correction</td>
<td>Bulk specific gravity of aggregate</td>
<td>Bulk specific gravity of aggregate</td>
</tr>
<tr>
<td>Loose unit weight of aggregate</td>
<td>Aggregate absorption</td>
<td>Loose unit weight of aggregate</td>
<td>Wastage</td>
</tr>
<tr>
<td>Traffic correction</td>
<td>Percentage of residual asphalt in emulsion</td>
<td>Surface condition</td>
<td>Aggregate absorption</td>
</tr>
<tr>
<td>Surface condition correction</td>
<td>Traffic volumes</td>
<td>Aggregate absorption</td>
<td>Percentage of residual asphalt in emulsion</td>
</tr>
<tr>
<td>Seasonal adjustment</td>
<td></td>
<td>Traffic volumes</td>
<td>Traffic volumes</td>
</tr>
<tr>
<td>Percentage of residual asphalt in emulsion</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Factors for emulsion application rate (EAR)</th>
<th>Modified Kearby</th>
<th>McLeod</th>
<th>2004 New Zealand</th>
</tr>
</thead>
<tbody>
<tr>
<td>AAR</td>
<td>Aggregate gradation</td>
<td>Aggregate gradation</td>
<td>Aggregate gradation</td>
</tr>
<tr>
<td>Bulk specific gravity of aggregate</td>
<td>Flakiness index</td>
<td>Flakiness index</td>
<td>Flakiness index</td>
</tr>
<tr>
<td>Loose unit weight of aggregate</td>
<td>Traffic correction</td>
<td>Traffic correction</td>
<td>ADT</td>
</tr>
<tr>
<td>Traffic correction</td>
<td>Bulk specific gravity of aggregate</td>
<td>Bulk specific gravity of aggregate</td>
<td>Percentage of heavy commercial vehicles per day</td>
</tr>
<tr>
<td>Surface condition correction</td>
<td>Loose unit weight of aggregate</td>
<td>Loose unit weight of aggregate</td>
<td>Texture depth</td>
</tr>
<tr>
<td>Seasonal adjustment</td>
<td>Surface condition</td>
<td>Surface condition</td>
<td>Soft substrate</td>
</tr>
<tr>
<td>Percentage of residual asphalt in emulsion</td>
<td>Aggregate absorption</td>
<td>Aggregate absorption</td>
<td>Absorptive surfaces</td>
</tr>
<tr>
<td>Aggregate gradation</td>
<td>Percentage of residual asphalt in emulsion</td>
<td>Traffic volumes</td>
<td>Aggregate shape</td>
</tr>
<tr>
<td>Traffic volumes</td>
<td>Traffic volumes</td>
<td>Traffic volumes</td>
<td>Traffic volumes</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Reference voids for AAR</th>
<th>Voids at the board test condition, approximately 50%</th>
<th>Voids at ultimate compacted AST state, 20%</th>
<th>Voids at two-year light traffic volumes, approximately 40%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference voids for EAR</td>
<td>Voids at the board test condition, approximately 50%</td>
<td>Voids at ultimate compacted AST state, 20%</td>
<td>Voids at the first major frost day, normally higher than 40%</td>
</tr>
<tr>
<td>Embedment depth (%)</td>
<td>Variable in terms of AST mat thickness and aggregate type</td>
<td>65-80</td>
<td>35</td>
</tr>
<tr>
<td>Synthetic aggregate</td>
<td>Considered in EAR</td>
<td>Not considered</td>
<td>Not considered</td>
</tr>
<tr>
<td>Multilayer</td>
<td>N.A.</td>
<td>Available with empirical guideline</td>
<td>Available with empirical guideline</td>
</tr>
</tbody>
</table>
2.3.1  *Hanson Design Method*

The Hanson design method was developed primarily for liquid asphalt, particularly cutback asphalt, and it is based on the average least dimension (ALD) of the cover aggregate spread on the pavement. Hanson calculated the ALD by manually callipering a representative aggregate sample to obtain the smallest value for the ALD that represents the rolled cover aggregate layer. He observed that when cover aggregate is dropped from an aggregate spreader onto asphalt, the voids between the aggregate particles are approximately 50%. He theorized that when the layer is rolled, this value is reduced to 30%, and it is further reduced to 20% when the cover aggregate is compacted by traffic, as shown in Figure 2-4. Hanson specified the percentage of voids to be filled by residual asphalt to be between 60% and 75%, depending on the type of aggregate and traffic level (Hanson 1934/35).

![Figure 2-4 States of embedment of AST in asphalt](image-url)
2.3.2 2004 New Zealand Method

The Hanson method has evolved into the 2004 New Zealand. Potter et al. (1976) and Patrick (1999) indicate that the total volume of voids is significantly higher than 20% in a compacted seal, and that voids continue to decrease with further compaction under traffic. This finding was adopted in the 2004 New Zealand. This method was developed as a performance-based AST design method that considers the aggregate loss during the first winter as well as the AST voids reduction model (Transit New Zealand 2005). One of the major difficulties involved in the design of material application rates is non-uniformity of the substrate. The 2004 New Zealand employs a substrate correction factor using the sand circle (sand patch) test for the texture depth of the substrate and the ball penetration test for soft substrate.

2.3.3 McLeod Method

Throughout the 1960s, McLeod (1969) developed an AST design procedure based partly on Hanson’s previous work and also on empirical relationships and observations. His method covers both single and multiple applications of surface treatments and, like any pavement design method, it determines the quantity of aggregate, quantity and type of asphalt, and rate of asphalt application. These quantities are determined based on several equations McLeod developed (1969).

Quantity of Aggregate

The equations used to determine the quantity of aggregate needed for a given surface treatment course are based on the following assumptions:
- 80% of the aggregate will ultimately be embedded into the pavement;
- the aggregate is one size (the equation is slightly modified for graded aggregate); and
- the aggregate will ultimately arrange itself so that the thickness of the layer is equal to the ALD of the aggregate.

Additional consideration must be given to the type of aggregate, the type of supporting layer, climatic variations, etc.

**Quantity of Asphalt**

The equation used to determine the quantity of asphalt is also based on several assumptions:

- 20% of the total surface treatment will ultimately be asphalt (80% embedment of aggregate);
- the aggregate is one size (the equation is slightly modified for graded aggregate); and
- the temperature during measurement is 60°F (if different than 60°F the value must be corrected).

The appropriate asphalt type and grade depend on the aggregate size and surface temperature at the time of application, and are determined by a chart developed by McLeod. The Asphalt Emulsion Manufacturers Association and the Asphalt Institute have adapted and furthered McLeod’s work by providing recommendations for asphalt types and grades for various aggregate gradations, and correction factors to the asphalt application rate based on existing surface conditions.
2.3.4 Kearby Method

One of the first efforts in the United States toward AST mix design was made by Jerome P. Kearby (1953). Kearby developed a design method to determine the amounts and types of asphalt and aggregate for one-course ASTs. Kearby’s work resulted in the development of a monograph that provides an asphalt cement application rate in gallons per square yard for the input data of average thickness, percentage of aggregate embedment, and percentage of voids (Kearby 1953). Kearby recommends the use of a uniformly graded aggregate by outlining eight grades of aggregate based on gradation and associated average spread ratios. He also recommends that the combined flat and elongated particle content not exceed 10% of any aggregate gradation requirement. The Kearby method accounts for the effects of existing pavement conditions and traffic volume on the optimum aggregate embedment depth. The percentage of embedment should be increased for hard aggregates and reduced for soft aggregates in the case of ASTs on an existing hard surface. For ASTs under heavy traffic, the percentage of embedment should be reduced, along with the use of larger-sized aggregates; and under low volume traffic, the percentage of embedment should be increased, with the use of medium-sized aggregates.

2.3.5 Modified Kearby Method (Texas)

In 1974, Epps and his associates proposed a further change to the design curve developed by Kearby for use in ASTs by incorporating the use of synthetic aggregates (Epps et al. 1974). Based on the high porosity of synthetic aggregates, Epps et al. proposed a curve showing approximately 30% more embedment than the Benson–Gallaway curve (Benson and Gallaway 1953). The rationale for this increase was that high friction lightweight aggregate
may turn over and subsequently ravel under traffic. In a separate research effort, Epps et al. (1980) continued the work done in Texas by Kearby (1953) and Benson and Gallaway (1953) by undertaking a research program to conduct a field validation of Kearby’s design method. Data from before and after the construction of 80 different projects were gathered and analyzed for this purpose (Holmgreen et al. 1985). It was observed that the Kearby design method predicted lower asphalt application rates than those used in the Texas practice, and so the Epps study proposed two changes to the design procedures. The first one was a correction to the asphalt application rates based on level of traffic and existing pavement conditions. The second change justified the shift of the original design curve proposed by the Kearby and Benson-Gallaway methods, as suggested for lightweight aggregates (Epps et al. 1974). Since then, practitioners and researchers have labeled this design approach as the Modified Kearby Method.

In this method, the AAR is determined using the laboratory board test method where only one aggregate layer is placed in a ½ yd² area. The dry loose unit weight and the bulk specific gravity of the aggregate are determined and used to convert the amount of aggregate to cover the ½ yd² area to an AAR in the field. The test board is made of plywood or masonite with sides framed by 12 mm (1/2 in.) molding strips. The asphalt application rate is determined by an equation that includes the traffic level (vehicles per day per lane), the existing surface conditions, the residual quantity of asphalt in the emulsion or cutback, and the field factor based on field experience.

According to the study done by Epps (1974) on ASTs with lightweight aggregate, the modified Kearby method appears to be the best methodology for the prediction of the AAR; and the Lovering method (Lovering 1954) and the Texas Highway Department method are
the best for the EAR prediction. In the Epps study (1974), an unusually high AAR from the Hanson method was reported for the lightweight aggregate, which may be due in part to the difference in the aggregate bulk specific gravity of the lightweight aggregate and the conventional aggregate.

2.3.6 Multiple AST Designs

Multiple ASTs consist of two or three successive alternate applications of asphalt and aggregate. The official names for such double ASTs and triple ASTs in North Carolina are split seal and triple seal, respectively.

The McLeod method and the 2004 New Zealand method present both single and multiple AST design methods. These design methods apply the same fundamental design concept regardless of the number of AST layers. The reason for this is based on the assumption that the asphalt and aggregate required for each layer of a multiple AST are identical, with minor adjustments, to the EAR and the AAR that would be applied if each layer were to serve as an isolated single application AST. It is noted that in the McLeod method, the aggregate size of the second layer should be one-half the size of the aggregate in the first layer. This change in aggregate size is recommended because 1) using coarse aggregate for the bottom layer can support heavier traffic volumes and 2) using smaller aggregate for the top layer can reduce windshield damage and tire noise.

The design procedure used to determine AARs and EARs for multiple ASTs is as follows:

1. Design for each layer’s AAR and EAR, as if it were to be the only layer.
2. Make no allowance for wastage because the excessive amount of aggregate on the
first layer can form a sparsely coated aggregate layer that can cause debonding of the layers and aggregate loss due to the lack of top layer aggregate embedment.

3. Except for the first course, make no correction for the underlying surface texture. Add together the EARs determined for each layer to obtain a total asphalt requirement.

For the split seal, the total EAR is divided into each layer of the bottom and top courses with a specified proportion (60-40 or 50-50 or 40-60), depending on the design method. The 40-60 split is used in New Zealand (Transit New Zealand 2005). For the triple seal, the recommended split ratio is 40-40-20 or 30-40-30 for the first, second, and third applications (McLeod 1969). The recommended AARs and EARs in the NCDOT specifications are shown in Table 2-4.

Table 2-4 Recommended EAR and AAR (NCDOT Standard Specifications for Roads and Structures 2002)

<table>
<thead>
<tr>
<th>Type of AST</th>
<th>Total EAR (gal/yd²)</th>
<th>Aggregate size</th>
<th>Total AAR (lb/yd³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straight seal</td>
<td>0.35-0.40</td>
<td>No. 78M</td>
<td>17-22</td>
</tr>
<tr>
<td>Split seal</td>
<td>0.45-0.50</td>
<td>No. 78M</td>
<td>30-35</td>
</tr>
<tr>
<td>Triple seal</td>
<td>0.60-0.75</td>
<td>No. 78M</td>
<td>45-51</td>
</tr>
</tbody>
</table>

2.3.7 Other Design Methods

The AST design procedures that existed prior to 1968 are summarized by Herrin et al. (1968). These include the following design methods: (a) Hanson, (b) California, (c) Nevitt, (d) modified Kearby, (e) Lovering spread modulus, (f) European, (g) McLeod, (h)
Mackintosh, (i) American Bitumen, and (j) Asphalt Institute. Although the procedures in the various design methods are not the same in all details, a number of factors are common to all (Gransberg 2005). Some of these design methods, including the Hanson, California, modified Kearby, Lovering spread modulus, and McLeod were evaluated using the MMLS3 test and are discussed in Chapter 5.3.

2.3.8 Reference Voids for AST Design

Most international AST design methods use the voids concept for their designs. The various voids reduction behaviors are defined according to the different design methods. Furthermore, the different design methods use different reference voids for their AAR and EAR designs.

In the modified Kearby method, the initial voids before roller compaction are used as the design reference voids, which are the voids at the loose aggregate state, normally around 40%-50%. This method does not include a wastage factor which can provide a better aggregate structure in the ASTs and reduce the possibility of an uncovered aggregate area of existing pavement caused by the uneven aggregate distribution. The AAR is determined by the board test, and the EAR is determined based on the amount of emulsion that fills the loose state voids in an AST that shows a good AST performance after substantial traffic.

The McLeod method uses Hanson’s voids concept, correcting for the difference between the loose aggregate voids measurement in the two methods. The voids concept assumption found in the McLeod and Hanson methods is that the AST has 20% voids after ultimate compaction by traffic; this percentage is used as the reference void. McLeod’s voids reduction behavior is based on the assumption that the aggregate settles into a stable state
after approximately one year, as shown in Figure 2-5. Based on 20% voids reduction, 80% of
AST volume is comprised of the aggregate, and a certain embedment depth percentage of
20% voids is applied for the EAR determination. The embedment depth percentage equals
the volume of emulsion percentage in the volume of AST voids.

The Hanson method (1934/35) has evolved into the 2004 New Zealand method.
According to this method, the total volume of voids is significantly higher than 20% in a
compacted AST, and the percentage of voids continues to decrease with further compaction
under traffic, as shown in following equation:

\[ Voids(\%) = 0.83 - 0.07 \log_{10}(elv) \]  

(1)

where

\[ elv = \] cumulative number of equivalent light vehicles, based on the assumption that
one heavy commercial vehicle (HCV) is equivalent to ten cars.

ASTs in North Carolina are applied to roads that have an average daily traffic (ADT)
of less than 2000 vehicles. The AST routine service life in North America is 5.76 years
(Gransberg 2005). Based on the above information, the \( elv \) is calculated as 7,989,120 with
10% HCV. The voids reduction rate is shown in Figure 2-5.

Reference voids constitute the major factor for determining the AAR and EAR. The
reference voids for AAR determination are approximately 50% (voids in loose aggregate) for
the modified Kearby method, 20% for the McLeod method, and approximately 40% (after
two-year light traffic) for the 2004 New Zealand method, as shown in Figure 2-5.
For the EAR determination, the modified Kearby method uses 50% reference voids with the embedment depth showing a good AST. The McLeod method uses 20% reference voids and 65-80% embedment depth for the EAR determination. For the 2004 New Zealand method, 44% reference voids and 100 days between the onset of AST construction and the first frost day are used to determine the EAR; this determination is made in light of the assumption that if the aggregates do not dislodge during the first winter, there is a low risk that premature low-temperature aggregate loss will occur later. The effect of winter on aggregate retention is also considered with the assumption that aggregate loss would occur with the first cold snap if the binder does not raise the aggregate by 35% to fill the voids. The importance of reference voids in AST design is presented in Chapter 5.1.2.
Figure 2-5 Reference voids for AAR determination with various design methods

Figure 2-6 Reference voids for EAR determination with various design methods
2.4 Skid Resistance of Asphalt Surface Treatments

In North America, loss of skid resistance is one of the common road conditions that indicate the need for an AST; thus, one of the major advantages of surface treatments is the increase in skid resistance (Gransberg 2005). 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).

Most SHAs have a specified cycle in which skid resistance is measured as a part of their pavement management system. The skid resistance measurements are invaluable to the decision-making as to which roads require surface treatment. However, there is no evidence that a single public highway agency has used skid numbers to directly evaluate the performance of ASTs (Gransberg 2005).

Skid resistance changes over time. Typically, it increases in the first two years following construction as the asphalt 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 creates a gritty detritus that roughens the surface. In drier summer conditions, this 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).

Two different testing subsets are used to determine a pavement’s skid resistance –
textural and drag or friction testing. These test methods are explained in the following
subsections.

### 2.4.1 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 2-5.
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 affects the friction. Wet
pavement friction at low speeds is primarily affected by the microtexture.

---

<table>
<thead>
<tr>
<th>Texture Level</th>
<th>Wavelength ($\lambda$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Microtexture</td>
<td>$\lambda &lt; 0.5$ mm</td>
</tr>
<tr>
<td>Macrotexture</td>
<td>$0.5$ mm $&lt; \lambda &lt; 50$ mm</td>
</tr>
<tr>
<td>Megatexture</td>
<td>$50$ mm $&lt; \lambda &lt; 0.5$ m</td>
</tr>
<tr>
<td>Roughness</td>
<td>$0.5$ m $&lt; \lambda &lt; 50$ m</td>
</tr>
</tbody>
</table>
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, they developed and correlated a new friction coefficient prediction model 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 Macrotecture 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 the mean texture depth (MTD).

Roque et al. (1991) studied the performance and the prediction of AST life. The 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 AST.

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 the 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.
2.4.2 **Friction Measurement**

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

Pavement friction is measured most frequently in accordance with the locked wheel method (the LWST), as specified in the 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 SN requirements, as shown in Table 2-6.

The 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 is referred to as the grip number (GN) and is proportional to the ratio FH over FV. Table 2-7 explains the friction values of the GT for the Federal Aviation Administration (FAA) classification levels, qualified at a 40 mph test speed.

Table 2-6 Proposed Minimum Skid Number (Henry 2000)

<table>
<thead>
<tr>
<th>State</th>
<th>Minimum Skid Number (SN at speed 40 mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Idaho</td>
<td>30</td>
</tr>
<tr>
<td>Illinois</td>
<td>30</td>
</tr>
<tr>
<td>Kentucky</td>
<td>28</td>
</tr>
<tr>
<td>New York</td>
<td>32</td>
</tr>
<tr>
<td>South Carolina</td>
<td>41</td>
</tr>
<tr>
<td>Texas</td>
<td>30</td>
</tr>
<tr>
<td>Utah</td>
<td>35</td>
</tr>
<tr>
<td>Washington</td>
<td>30</td>
</tr>
<tr>
<td>Wyoming</td>
<td>35</td>
</tr>
</tbody>
</table>

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
employs a pendulum that 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 hot-mix asphalt (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.

Table 2-7 Friction Level Classifications for Runway Pavement Surfaces

<table>
<thead>
<tr>
<th></th>
<th>40 mph</th>
<th>60 mph</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>0.43</td>
<td>0.24</td>
</tr>
<tr>
<td>Maintenance</td>
<td>0.53</td>
<td>0.36</td>
</tr>
<tr>
<td>Planning</td>
<td>0.74</td>
<td>0.64</td>
</tr>
<tr>
<td>New Design / Construction</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
2.5 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 the aggregate. The pavement’s surface, size and gradation of aggregate, and local climate are considered for the asphalt 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). 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). The lightweight aggregate has been popularly used as AST material in North Carolina. This material in ASTs provides a very skid-resistant surface, provides good color contrast which improves visibility in daylight and at night, provides a surface on which paint striping maintenance is reduced, and eliminates glass damage caused by flying stone (Epps et al. 1974). However, the lightweight aggregate size has not been specified in North Carolina.

According to the North Carolina statewide survey conducted in 2003, No. 78M for granite and 5/16 in. NMSA for lightweight aggregate are the most common sizes in a straight seal, split seal, and triple seal. Moreover, these aggregate sizes serve as the blotting sand and screening aggregate for the top layer of a triple seal. The types of asphalt used in North Carolina are CRS-2 and CRS-2P. Table 2-8 summarizes the aggregate and asphalt types used in North Carolina for ASTs.
Table 2-8 Types of Materials used for ASTs in North Carolina

<table>
<thead>
<tr>
<th>Layer</th>
<th>Aggregate</th>
<th>Asphalt</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mat and Seal</td>
<td>Top</td>
<td>Sand, 78M, or Screenings</td>
</tr>
<tr>
<td></td>
<td>Third</td>
<td>78M or Lightweight 5/16 in.</td>
</tr>
<tr>
<td></td>
<td>Second</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>No. 5, No.57, No.6, or No.67</td>
</tr>
<tr>
<td>Straight Seal</td>
<td>Top</td>
<td>78M or Lightweight 5/16 in.</td>
</tr>
<tr>
<td>Split Seal</td>
<td>Top</td>
<td>Sand, 78M, Lightweight 5/16 in., or Screenings</td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>78M or Lightweight 5/16 in.</td>
</tr>
<tr>
<td>Triple Seal</td>
<td>Top</td>
<td>Sand, 78M, Lightweight 5/16 in., or Screenings</td>
</tr>
<tr>
<td></td>
<td>Middle</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Bottom</td>
<td>78M or Lightweight 5/16 in.</td>
</tr>
</tbody>
</table>
3. MATERIALS AND TESTING METHODS

3.1 Material Selection

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

3.2 Component Material Properties

3.2.1 Gradation of Aggregate Particle Size

Dry and wet sieve analyses were performed on both 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 median size is the aggregate particle size at 50% passing in sieve analysis curve. The sieved granite is close to the upper limit recommended by the No. 78M specification. The uniformity coefficient (UC) 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 UC 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 UC of the tested aggregate is 2.02 and 2.35 for the lightweight
aggregate and the granite aggregate, respectively. Therefore, the lightweight aggregate has a more uniform aggregate particle size than the granite aggregate.

Table 3-1 Summary of the Sieve Analysis

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percentage Passing</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Lightweight</td>
<td>Granite</td>
</tr>
<tr>
<td>1/2 in.</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>3/8 in.</td>
<td>100.0</td>
<td>95.4</td>
</tr>
<tr>
<td>1/4 in.</td>
<td>66.0</td>
<td>64.8</td>
</tr>
<tr>
<td>No. 4</td>
<td>23.7</td>
<td>40.1</td>
</tr>
<tr>
<td>No. 8</td>
<td>4.9</td>
<td>7.4</td>
</tr>
<tr>
<td>No. 16</td>
<td>3.5</td>
<td>2.4</td>
</tr>
<tr>
<td>No. 50</td>
<td>2.6</td>
<td>0.8</td>
</tr>
<tr>
<td>No. 100</td>
<td>2.2</td>
<td>0.7</td>
</tr>
<tr>
<td>No. 200</td>
<td>1.6</td>
<td>0.4</td>
</tr>
<tr>
<td>Median size, M</td>
<td>0.23 in. (5.70 mm)</td>
<td>0.21 in. (5.31 mm)</td>
</tr>
</tbody>
</table>
3.2.2 **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 sizes (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 the materials passing through all of the slots is divided by the total weight of the sample to give the percentage of flat particles by weight, or FI. 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 lightweight aggregate.
Table 3-2 Slot Sizes Required for Different Fractions of Aggregate Size

<table>
<thead>
<tr>
<th>Size of Aggregate</th>
<th>Slot Width, in.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Passing</strong></td>
<td><strong>Retaining</strong></td>
</tr>
<tr>
<td>1 in.</td>
<td>3/4 in.</td>
</tr>
<tr>
<td>3/4 in.</td>
<td>1/2 in.</td>
</tr>
<tr>
<td>1/2 in.</td>
<td>3/8 in.</td>
</tr>
<tr>
<td>3/8 in.</td>
<td>1/4 in.</td>
</tr>
<tr>
<td>1/4 in.</td>
<td>No. 4</td>
</tr>
</tbody>
</table>

Table 3-3 Flakiness Index of Aggregates

<table>
<thead>
<tr>
<th>Aggregate Type</th>
<th>Lightweight</th>
<th>Granite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flakiness Index (%)</td>
<td>8.28</td>
<td>24.01</td>
</tr>
</tbody>
</table>

Figure 3-2 Flakiness Index plate gauge
3.2.3 Average Least Dimension

The average least dimension, or ALD (H), is determined from the median particle size (M) and the FI. It is a reduction of the median particle size after accounting for flat particles. The ALD represents the expected AST thickness in the wheel paths where traffic forces the flat chips to lie on their flattest side. The ALD is calculated as follows:

\[ H = \frac{M}{1.139285 + (0.011506)(FI)} \times 100 \]  

(2)

where

\( H = \) average least dimension, in. or mm;

\( M = \) median particle size, in. or mm; and

\( FI = \) flakiness index, in percent.

Table 3-4 Average Least Dimension of Lightweight and Granite Aggregate

<table>
<thead>
<tr>
<th>Aggregate Type</th>
<th>Lightweight</th>
<th>Granite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Least Dimension</td>
<td>0.18 in. (4.57 mm)</td>
<td>0.15 in. (3.81 mm)</td>
</tr>
</tbody>
</table>

3.2.4 Bulk Specific Gravity

Bulk specific gravity (BSG) tests were performed on all aggregates. The aggregates were divided into three different sizes: the aggregate that 1) is retained on the No. 8 sieve; 2) passes the No. 8 sieve and is retained on the No. 200 sieve; and 3) passes the No. 200 sieve. These tests were conducted in accordance with standard test methods, and the results are summarized in Table 3-5. The BSG of the lightweight aggregate is 0.6 times that of the granite aggregate, as Figure 3-3 shows.
Table 3-5 Bulk Specific Gravity (BSG) of Aggregates

<table>
<thead>
<tr>
<th>Aggregate Type</th>
<th>Lightweight</th>
<th></th>
<th>Granite</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Standard</td>
<td>BSG</td>
<td>Standard</td>
<td>BSG</td>
</tr>
<tr>
<td>Retained on No. 8 Sieve</td>
<td>Tex-433A</td>
<td>1.62</td>
<td>ASTM C 127</td>
<td>2.65</td>
</tr>
<tr>
<td>Passing No. 8 and Retained on No. 200 Sieve</td>
<td>ASTM C 128</td>
<td>1.94</td>
<td>ASTM C 128</td>
<td>2.63</td>
</tr>
<tr>
<td>Passing No. 200 Sieve</td>
<td>ASTM D 854</td>
<td>2.55</td>
<td>ASTM D 854</td>
<td>2.52</td>
</tr>
<tr>
<td>Total BSG</td>
<td></td>
<td>1.66</td>
<td></td>
<td>2.64</td>
</tr>
</tbody>
</table>

Figure 3-3 Volume comparison of 0.22 lb (100 g) granite (left) and lightweight (right) aggregate
3.2.5 **Loose Unit Weight of 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 BSG 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 3-6:

\[
V = 1 - \frac{W}{62.4G}
\]

where

\( G = \) bulk specific gravity of the aggregate;

\( W = \) loose unit weight of the cover aggregate, ASTM C 29, lbs/ft\(^3\); and
\[ V = \text{voids in the loose aggregate, expressed as a decimal.} \]

Table 3-6 Loose Unit Weight Test Results

<table>
<thead>
<tr>
<th>Aggregate Type</th>
<th>Lightweight</th>
<th>Granite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Bulk Specific Gravity, (G)</td>
<td>1.66</td>
<td>2.64</td>
</tr>
<tr>
<td>Loose Unit Weight (lbs/ft(^3)), (W)</td>
<td>49.48</td>
<td>87.13</td>
</tr>
<tr>
<td>Voids in the Loose Aggregate, (V)</td>
<td>0.51</td>
<td>0.47</td>
</tr>
</tbody>
</table>

3.2.6 Aggregate Absorption

Because the lightweight aggregate is the rotary kiln expanded slate aggregate, it is expected to have higher surface voids and, thus, greater absorption. The research team checked its asphalt absorption values by using Rice’s vacuum saturation method, ASTM D 2041. PG 70-22 asphalt was mixed with the aggregate at 329°F using 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 lightweight aggregate has a higher asphalt absorption value than the granite, as shown in Table 3-7.
Table 3-7 Asphalt Absorption Test Results

<table>
<thead>
<tr>
<th>Aggregate Type</th>
<th>Lightweight</th>
<th>Granite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective Specific Gravity</td>
<td>1.64</td>
<td>2.67</td>
</tr>
<tr>
<td>Asphalt Absorption (%)</td>
<td>0.48</td>
<td>0.38</td>
</tr>
</tbody>
</table>

3.2.7 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 emulsion used in this project has 68-70% residual asphalt content, according to the test results provided by the NCDOT Materials and Tests Unit.
3.3 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 is 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. It is noted that the circular specimens were used in visual AST observation for determining the optimum AST application rates, and the rectangular specimens were used in the rest of research.

The wheel path under wandering MMLS3 loading is 7.1 in. wide, yielding a total area of 72.3 in\(^2\). Fabrication requires 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 (Figure 3-6(c)), 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 (d)). 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\(^2\) EAR on the felt disks and were then 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 emulsion 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 emulsion 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.

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

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 upright 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.

3.5 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 are attached to thin steel plates that are 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 is calculated by the following equation:
Aggregate Loss (%) = \frac{W_{\text{before}} - W_{\text{after}}}{W_{\text{before}}} \times 100 \tag{4}

where

\[ W_{\text{before}} = \text{weight of aggregate on AST specimen before any loading}; \text{ and} \]
\[ W_{\text{after}} = \text{weight of aggregate on AST specimen after MMLS3 loading}. \]

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

The complete MMLS3 test procedure involves the following steps:

1. preparing chip seal specimens using ASTM 7000D and curing them at 95°F for 24 hours in oven;
2. measuring the initial specimen weight;
3. attaching the specimens to thin steel plates that are fastened to a steel base plate;
4. setting up the MMLS3 over the specimens using a crane;
5. covering the MMLS3 and steel base with an environmental chamber using a crane;
6. connecting the control unit to a 220 VAC supply and connecting the three cables to the MMLS3;
7. connecting air ducts to the environmental chamber;
8. conditioning specimens at 77°F for 3 hours for the aggregate retention test;
9. conducting MMLS3 loading for 10 minutes by pushing the external start (green) and stop (red) buttons and then measuring the specimen weight;
10. conducting MMLS3 loading for 2 hours with periodic measurements of the specimen weight;

11. conducting a visual survey and other performance tests, such as the BPT, sand patch test, transverse profiling, etc.;

12. conditioning at 122°F for 3 hours for the bleeding test;

13. conducting MMLS3 loading for 4 hours at 122°F;

14. measuring the final specimen weight;

15. conducting a visual survey and obtaining other performance measures, including a bleeding measurement using digital image processing (DIP), sand patch test and BPT results, an embedment measurement using coating, and DIP results;

16. removing the MMLS3 from the steel base; and

17. maintaining and lubricating the MMLS3 according to the check-list in its manual.
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.
3.6 **Bleeding (or Flushing) Measurement**

Bleeding is caused by the spread of hot emulsion. In hot weather, when emulsion is soft, the emulsion will adhere to vehicle tires and be spread over the surrounding road surface. This soft state is called bleeding and is undesirable because the binder coats the microtexture of the aggregates, creating a lowered skid resistance. At the critical texture, the tires can sink down into the spaces between the aggregates and touch the surface of the emulsion. Although tracked bleeding emulsion may eventually wear down the aggregates, it also presents a hazard during the bleeding itself (Transit New Zealand 2005).

Flushing is caused by an excess of emulsion and creates a solid, smooth, black, slick surface. Flushing may occur as the natural end-of-life condition of a well-designed AST, or as an AST design or construction fault. In hot weather, a flushed surface may soften and bleed (Transit New Zealand 2005). Bleeding and flushing have different causes, but they exhibit the same behavior, which is reducing the skid resistance.

In this research, *bleeding* is the term used for both bleeding and flushing. Bleeding performances of AST specimens were quantified employing DIP, which essentially involves three steps: (a) digital image acquisition of the AST specimen surface; (b) image processing; and (c) data analysis for quantifying the amount of bleeding. The DIP is illustrated in Fig. 3-9. Before and after the MMLS3 bleeding test, the AST specimen surface is scanned by a HP Scanjet 4850 into an 8-bit grayscale digital image that consists of a single plane of pixels. Each pixel is encoded using a single number representing grayscale values from 0 to 225. To scan the specimen surface without any disturbance of the aggregate, the scanner is turned upside down and held over the specimen instead of turning over the specimen itself, which could potentially dislodge some of the aggregate. National Instruments Vision Assistant
(NIVA) 7.0 is then used for the analysis of the digital image to generate a histogram of grayscale intensities. This histogram is then converted into a data file. Because Windows BMP is the simplest image format, and is the native image format in the Microsoft Windows operating systems (Zhang 2003), 8-bit uncompressed BMP format images are considered in this research.

Resolution and contrast contribute to the quality of the overall image that a scanner produces during image acquisition. In broad terms, *resolution* is an expression of the amount of information transmitted or received. For a scanner, resolution is the maximum number of pixels per inch and is measured in dots per inch (dpi). The required resolution of a scanner is

Figure 3-9 DIP procedure of AST specimen: (a) actual specimen; (b) image acquisition using scanner (grayscale); (c) image processing; (d) data file
determined by the smallest feature to be inspected. For example, if the smallest aggregate particles with a size of 0.15 mm (sieve size #100), i.e., 0.006 in., must be recognized, the resolution of the scanner is at least \(1/0.006=170\) dpi. In this study, 300 dpi of resolution is chosen for digital image acquisition. Contrast defines the differences in intensity values between the object under inspection and the background. Contrast and resolution are closely related factors that contribute significantly to the quality of the image. Generally, a high-resolution scanner has enough contrast to distinguish objects from the background (Zhang 2003).

### 3.6.1 Bleeding Calculation

In order to quantify AST bleeding performance, two major DIP procedures were required along with the particle analysis and the histogram analysis. The gray intensity values (GIVs) acquired from a digital image do not necessarily allow a distinction to be made between the bleeding area and the unbleeding area because the image expresses the texture of the AST surface and aggregate color. A single GIV with a textured surface reflects the various levels of light strength to the charge-coupled device in the scanner and produces the various GIVs. The range of GIVs obtained from the shaded areas between the aggregates can overlap with that of the bleeding area. Another difficulty is that, due to its mineral composition, the aggregate within the AST specimen has a dark-gray color similar to that of the emulsion.

To overcome this problem in the digital image involving the texture of AST surface and the aggregate color, mapping the actual bleeding areas with selected AST specimens was conducted, and the particle analysis using DIP was employed for quantifying the bleeding
area. These actual bleeding areas were used to define the critical GIVs of bleeding in the GIV histogram. The defined critical bleeding GIVs were applied to the histograms from other AST specimens.

Two approaches were taken for mapping the bleeding with AST specimens: a) the bleeding area was mapped using the AST specimen that showed little bleeding b) the unbleeding area was mapped using the AST specimen that showed the most bleeding. In this study, both measurement approaches are introduced.

The bleeding area mapping was carried out on a transparency placed over the color AST specimen photo, as shown in Figure 3-10. Then, this transparency was scanned to obtain its digital image, and the particle analysis was conducted using NIVA. The main goal of particle analysis processing is to recognize the bleeding area, which involves isolating most of the bleeding area from the background. This technique is so-called *thresholding* in DIP. After isolating the bleeding area from the background, DIP can analyze the number of pixels representing the bleeding. To carry out thresholding, a threshold of GIVs should be set so that the bleeding area with a lower GIV than the threshold can be isolated. Thresholding consists of segmenting an image into two regions, a bleeding region and a background region. The bleeding areas are characterized by an intensity range and are composed of pixels with GIVs belonging to a given threshold interval. All other pixels are considered to be part of the background. Thresholding works by setting all pixels that belong to the threshold interval to 1, and setting all other pixels in the image to 0. Thus, a grayscale image with GIVs ranging from 0 to 255 is converted into a binary image (black and white), with a GIV of 0 or 1. This information is then output as an Excel file. Adding up the all pixels with a GIV of 0 or 1 produces the number of pixels in the bleeding area or unbleeding area.
To quantify the bleeding performance of other AST specimens without the mapping procedure, the critical bleeding GIV is determined using the pixel number of the bleeding area and the histogram obtained from the specimen used for mapping. At the equivalent pixel number, from 0 to higher GIVs in the histogram to the pixel number of the bleeding area determined by the mapping and the particle analysis, the highest GIV is defined as the critical bleeding GIV. Therefore, the percentage of bleeding using other GIV histograms can be calculated by the following equation:

\[
\text{Bleeding (\%)} = \frac{A_{\text{Bleeding}}}{A_{\text{Total}}} \times 100
\]  

(5)

where

\[A_{\text{Total}} = \text{area of AST specimen (total number of pixels)}; \text{ and}\]

\[A_{\text{Bleeding}} = \text{area of bleeding on AST specimen (sum of pixels that are smaller than the critical bleeding GIV)}.
\]
Figure 3-10 Photos of AST surface: (a) digital image of AST; (b) mapped bleeding area

Figure 3-11 Determination of critical bleeding GIV
4. EVALUATION OF AGGREGATE GRADATION ON AGGREGATE RETENTION PERFORMANCE

4.1 Selection of Optimum Aggregate and Emulsion Application Rates

The application rates for aggregate and emulsion are major factors that affect AST aggregate retention performance. Therefore, in order to evaluate the effects of aggregate gradation on aggregate retention, it is imperative to test AST specimens with various 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 (AAR) depends on the aggregate gradation, shape, and specific gravity. The emulsion application rate (EAR) 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 lightweight aggregate were designed using the McLeod design method, which resulted in the AAR of 15.03 and 11.15 lb/yd² and the EAR of 0.14-0.19 and 0.19-0.26 gal/yd² for granite and lightweight aggregates, respectively. Table 4-1 summarizes the 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 lightweight 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 lightweight aggregate.

Table 4-1 Design Input Parameters for McLeod and Modified Kearby Methods

<table>
<thead>
<tr>
<th>Type of Aggregate</th>
<th>Lightweight</th>
<th>Granite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Median Particle Size (in.)</td>
<td>0.23</td>
<td>0.21</td>
</tr>
<tr>
<td>Flakiness Index (%)</td>
<td>8.28</td>
<td>24.01</td>
</tr>
<tr>
<td>Average Least Dimension (in.)</td>
<td>0.18</td>
<td>0.15</td>
</tr>
<tr>
<td>Voids in the Loose Aggregate</td>
<td>0.51</td>
<td>0.47</td>
</tr>
<tr>
<td>Bulk specific Gravity</td>
<td>1.66</td>
<td>2.64</td>
</tr>
<tr>
<td>Asphalt Absorption (%)</td>
<td>0.48</td>
<td>0.38</td>
</tr>
<tr>
<td>Existing Pavement Texture</td>
<td>Smooth, nonporous</td>
<td></td>
</tr>
<tr>
<td>Percentage of Waste Allowed</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Residual Asphalt Content of Emulsion</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Construction Season for Field Adjustment</td>
<td>Summer</td>
<td></td>
</tr>
</tbody>
</table>

The modified Kearby method was used to design ASTs with lightweight aggregate. This design method was developed to cover both conventional and lightweight aggregates by using a laboratory board test method. The test is conducted by placing one layer of aggregate on a 1/2 yd² area board, as shown in Figure 4-1, and determining the AAR by converting the aggregate weight into a unit of lb/yd² of roadway. The EAR is determined from the dry loose unit weight and the dry bulk specific gravity of the aggregate, average embedment 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 lightweight aggregate results in approximately 30% more embedment than the one used for
conventional aggregate for the same mat thickness (Gransberg 2005). The AARs and EARs for lightweight aggregate used in this study are found to be 8.4 lb/yd² and 0.28-0.32 gal/yd², respectively.

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

<table>
<thead>
<tr>
<th>Type of Aggregate</th>
<th>No.</th>
<th>AAR (lb/yd²)</th>
<th>EAR (gal/yd²)</th>
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</thead>
<tbody>
<tr>
<td>Lightweight</td>
<td>1</td>
<td>0.26</td>
<td></td>
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<tr>
<td></td>
<td>2</td>
<td>0.35</td>
<td></td>
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<tr>
<td></td>
<td>3</td>
<td>0.40</td>
<td></td>
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<tr>
<td></td>
<td>4</td>
<td>0.26</td>
<td></td>
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<tr>
<td></td>
<td>5</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>0.40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>0.26</td>
<td></td>
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<tr>
<td></td>
<td>8</td>
<td>0.35</td>
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<tr>
<td></td>
<td>9</td>
<td>0.40</td>
<td></td>
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<tr>
<td></td>
<td>10</td>
<td>0.26</td>
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<tr>
<td></td>
<td>11</td>
<td>0.35</td>
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<td>12</td>
<td>0.40</td>
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<td></td>
<td>13</td>
<td>0.20</td>
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<td></td>
<td>14</td>
<td>0.25</td>
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<td>15</td>
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<td></td>
<td>16</td>
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<td>17</td>
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<td></td>
<td>18</td>
<td>0.35</td>
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<td></td>
<td>19</td>
<td>0.30</td>
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<td></td>
<td>20</td>
<td>0.20</td>
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<tr>
<td>Granite</td>
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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 lightweight and granite aggregates, based on their field experience. The optimum application rates were selected as 9 lb/yd²-0.26 gal/yd² for the lightweight aggregate and 14 lb/yd²-0.20 gal/yd² for the granite aggregate. These designs were chosen unanimously.

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

4.2 Effects 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 AARs and EARs on aggregate retention and bleeding. It is noted that a circular shaped specimen was used for the AST application rate study described in this chapter, and that the specimen was later modified to a rectangular shaped specimen, as described in Chapter 3. Figure 4-2 shows the FOT results for the two aggregate types at multiple rates. As expected, more aggregate is lost as the AAR increases and the EAR decreases.

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 lightweight and the granite aggregates. A similar trend is
observed from these figures to those from the FOT; that is, as the AAR increases and the EAR decreases, the percentage of aggregate loss increases. These results show only one side of the AST performance, however; that is, too low an AAR and/or too high an EAR will yield a lower aggregate loss percentage, but may cause bleeding. Therefore, 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 lightweight AST specimens after the bleeding test as the EAR changes. A visual examination of the surface textures, as presented in Figure 4-4, confirms that more bleeding occurs as the EAR 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 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 lightweight aggregate has better retention than the granite due to its uniform size and low FI. For the lightweight aggregate, most of the aggregate loss occurs during the initial trafficking, whereas for granite, continuous aggregate loss occurs after the initial trafficking.
Figure 4-2 Aggregate loss measured from the flip-over test: (a) lightweight aggregate; (b) granite aggregate
Figure 4-3 Aggregate loss measured from the MMLS3 test: (a) lightweight aggregate; (b) granite aggregate
Figure 4-4 Surface texture change after the bleeding test of AST specimens with 9 lb/yard² of lightweight aggregate and EARs of: (a) 0.26 gal/yard²; (b) 0.35 gal/yard²; (c) 0.4 gal/yard²
4.3 Effects of Fines Content

Because the cleanliness of the aggregate is a critical factor that influences the aggregate retention 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 lightweight aggregate and the granite aggregate 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 fines content 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 the fine particles attached to the 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.
Figure 4-5: Distribution of fines: (a) lightweight 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 EAR and AAR were kept the same for the different fines contents. In other words, the overall AAR was kept constant as the amount of fines was increased. Therefore, a higher fines content means a lower amount of coarse aggregate. This method was expected to yield a less significant effect of fines on aggregate retention because the effect of more fines and less coarse aggregate 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 AARs according to the appearance of the AST surface. Therefore, the critical fines content to be determined from this approach (i.e., a constant AAR with varying fines content) represents the maximum allowable amount of fines that does not affect the aggregate retention performance when no adjustments are made in the field.

Five fines 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 fines contents for both aggregate types. In general, the aggregate loss increases as the number of wheel loads increases and as the fines content increases. As can be seen in this figure, most of the lightweight aggregate loss occurs during the initial trafficking, whereas continuous aggregate loss is shown for the granite aggregate. In general, the lightweight aggregate has better retention than the granite aggregate. Moreover, the effect of additional fines on aggregate retention is much less significant with the lightweight than the granite aggregate.
Figure 4-7 presents the normalized aggregate loss, which is a ratio of aggregate loss at a specific fines content to the value at 0% fines content, as a function of fines content after the aggregate retention test. The rate of increase of the aggregate loss with increasing fines content becomes significantly greater between 0% and 2% fines content in both aggregate types, and after a 6% fines content in the granite aggregate. According to Figure 4-7, the specified fines content of 2% by Kandhal and Motter (1987) yields the normalized aggregate losses of 1.32 and 1.16 for granite and lightweight aggregates, respectively, which correspond to 12.6% and 6.1% of aggregate loss. The 1.5% fines content of the NCDOT specifications corresponds to 11.85% and 5.97% aggregate loss for the granite and lightweight aggregate, respectively. 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.
Figure 4-6 Effect of fines content and gradation on aggregate loss performance: (a) lightweight aggregate; (b) granite
Figure 4-7 Normalized aggregate loss as a function of fines content at 12,800 MMLS3 wheel loads.

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

The effects 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 aggregate and between 3/8 in. and No. 8 for the lightweight aggregate. The gradations for 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 lightweight aggregate, respectively. Due to the changed gradation, the AARs were redesigned using the board test, which is part of the modified Kearby method. The ratio of the AAR before the gradation modification to the rate after the modification was used to determine the EAR after the modification. AARs of 12.6 lb/yd² and 8.5 lb/yd² and EARs of 0.18 gal/yd² and 0.25 gal/yd² were determined for the granite and the lightweight aggregate, respectively.

The effect of aggregate gradation on aggregate retention 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 percentages for both aggregates, with a much more significant effect on the granite aggregate. The reason for this reduction in aggregate loss percentage is that the coarse aggregate particles sit atop the small aggregate particles in the graded aggregate and, therefore, do not become effectively coated by 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 lightweight AST. Therefore, it can be concluded that the lower aggregate loss percentage, shown in Figure 4-6 for the lightweight AST with the original gradation, has much more to do with the uniform gradation in the lightweight aggregate than
the fact that the aggregate is lightweight, has a higher FI, or that it has a higher absorption value than the granite aggregate.

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) that shows the granite AST specimen with the unmodified gradation. It can be seen that the effect of removing the passing No. 8 sieve aggregate is much greater in the granite AST (Figure 4-10) than in the lightweight AST (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) lightweight; (b) granite aggregate
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
Figure 4-11 Surface textures of lightweight AST specimens before and after the aggregate retention test: (a) original gradation with 2% fines; (b) retained on No. 8 sieve
4.5 Effects of Aggregate Type

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

The ratio of the bulk specific gravity of lightweight aggregate to granite aggregate was used for adjusting the application rate for the granite aggregate so that it would be compatible with the lightweight aggregate gradation. The same EAR determined by NCDOT engineers for the lightweight aggregate was used for the granite aggregate. The AAR and EAR for the granite aggregate with the lightweight aggregate gradation are found to be 14.31 lb/yd² and 0.26 gal/yd², 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 retention performance. In this comparison, the effects of gradation and fines content
differences are eliminated by matching the gradations of the two aggregates. Although these aggregates have different aggregate shapes, different levels of electrical interactions between aggregate and emulsion, and different asphalt absorption levels, the effects of these characteristics on the aggregate retention performance seems to be minor. Therefore, it is concluded that 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 is found that the lightweight aggregate has a more uniform texture than the granite aggregate and that the lightweight aggregate gradation is due to the more cubical shape of the lightweight aggregate (i.e., its high FI) as compared to the granite aggregate.
Figure 4-14 Surface textures of AST specimens before and after the aggregate retention test: (a) original gradation of lightweight aggregate with 2% fines; (b) granite with the lightweight aggregate gradation.
4.6 Analytical Evaluation of Effects of Aggregate Gradation on AST Performance

According to McLeod’s AST failure criteria, single application ASTs constructed with uniform-sized aggregates are assumed to be one-stone particle thick. The correct amount of emulsion should embed every aggregate particle in the emulsion to a certain percentage of the AST depth. In the case of graded aggregate, the particles that are embedded less than 50% in the emulsion are likely to be torn out by traffic (McLeod 1969). In other words, the graded aggregate creates aggregate loss or bleeding at the optimum AST rate because some of the aggregate particle sizes that are far from the median particle size (M) could be either fully coated or barely coated with emulsion. In the McLeod design, if traffic is moderate with 1000 to 2000 vehicles per day, the optimum EAR should fill about 70% of the voids between the AST aggregate particles in order to obtain good performance. The more graded aggregate has a higher percentage of aggregate particles that are smaller than 70% of the median particle size (M); this causes bleeding. Particles larger than twice 0.7×M cause aggregate loss, as shown in Figure 4-15.

![Figure 4-15 Scheme of McLeod’s AST failure criteria](image-url)
The typical characteristics of McLeod’s AST failure criteria are related to median size, uniformity of gradation, and the target embedment. The larger median size has a lower aggregate loss percentage and less bleeding, as shown in Figure 4-17. The more uniform gradation also has a lower aggregate loss percentage and less bleeding (Figure 4-18). The less deep the aggregate is embedded, the less bleeding occurs and the more aggregate loss occurs, and vice versa; that is, the greater the aggregate embedment depth, the more bleeding occurs and less aggregate loss occurs.

Figure 4-16 pertains to the calculated aggregate loss and bleeding based on the optimum embedment of 70% for the 78M specification. In this figure, the upper limit of 78M granite with a smaller median size shows more aggregate loss and more bleeding than the lower limit of 78M specification that has a larger median size. The optimum gradation showing the best AST performance in the 78M specification range can be used as a narrower guide gradation. To determine this gradation, the following procedure is utilized:

1. generate the most uniform gradations within the 78M specification range as shown in Figure 4-19. Herein, G1 generated follows the lower limit of 78M specification from No. 200 to No. 8 sieve size, then, it is modified to the most uniform gradation at the sieve size larger than No. 8 within the 78M specification range. G2 and G3 are generated using the same manner used for G1 gradation with exception of different sieve sizes that is a boundary between the lower limit of 78M specification and the modified gradation. These boundary sieve sizes were 3/8” and 1/2” for G1 and G2, respectively.

2. find M and calculate 0.7×M and 1.4×M for each generated gradation; and
3. find the percentage passing that corresponds to $0.7\times M$ and $1.4\times M$ for each generated gradation.

4. plot, as shown in.

Figure 4-20 shows the calculated bleedings and aggregate losses of the generated gradations as a function of median sizes. Since the G2 gradation shows the smallest percentages of aggregate loss and bleeding with 10.6% and 17.2%, respectively (as shown in Figure 4-20), this gradation can be the best AST gradation and also can be used as a guide gradation within the 78M specification range.

It is noted that, in determining a guide gradation, the standard sieve sizes used for aggregate classification (ASTM D448-03a) were used to generate the uniform gradation; this process excludes the 1/4” sieve size included in McLeod design method. The reason for this omission is that a guide gradation should be compatible with the standard sieve sizes used by AST aggregate industries.

The bleeding and the aggregate retention performance using McLeod’s AST failure criteria were calculated and are illustrated for lightweight aggregate and granite aggregate in Figure 4-21 and Figure 4-22, respectively, and summarized in Table 4-6. Because the median size of the lightweight aggregate is larger than that of the granite aggregate, and the gradation of the lightweight aggregate is more uniform than that of the granite aggregate, the calculated performance of the lightweight aggregate is better than that of the granite aggregate.

The granite aggregate, popularly used in North Carolina, shows a poor AST performance with a 23.94% aggregate loss – 10% is considered reasonable for high speed roadways (DOT and Public Facilities 2001) – and 28.77% bleeding (the specification for
bleeding is not available). The AST performance was calculated based on a one-stone particle thick layer of aggregate particles. However, observations indicate that the finer material is not distributed as one-stone thick throughout an AST constructed with graded aggregates. The finer sizes of granite tend to stack vertically, two or more particles in depth, as shown in Figure 4-23. Not having a one-stone thick layer with graded aggregate is contrary to McLeod’s assumption. This aggregate stacking problem regarding the graded aggregate should be taken into consideration in the analysis and prediction of the AST performance with the McLeod’s AST failure criteria.

Figure 4-16 Performance prediction of 78M specification
Figure 4-17 AST performance in terms of median sizes

Figure 4-18 AST performance in terms of gradations
Figure 4-19 The most uniform gradations within 78M specification range

Figure 4-20 Determination of median size showing the best AST performance
Table 4-3 Generated Optimum Gradation in 78M Specification Range

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2 in.</td>
<td>100</td>
</tr>
<tr>
<td>3/8 in.</td>
<td>100</td>
</tr>
<tr>
<td>No. 4</td>
<td>20</td>
</tr>
<tr>
<td>No. 8</td>
<td>0</td>
</tr>
<tr>
<td>No. 16</td>
<td>0</td>
</tr>
<tr>
<td>No. 50</td>
<td>0</td>
</tr>
<tr>
<td>No. 100</td>
<td>0</td>
</tr>
<tr>
<td>No. 200</td>
<td>0</td>
</tr>
</tbody>
</table>
Figure 4-21 Performance prediction of lightweight aggregate gradation

Figure 4-22 Performance prediction of granite aggregate gradation
Figure 4-23 Aggregate particle setting comparison in ASTs: (a) lightweight aggregate with 9 lb/yd$^2$ and 0.25 gal/yd$^2$; (b) granite aggregate with 14 lb/yd$^2$ and 0.2 gal/yd$^2$

4.7 Performance-Based Uniformity Coefficient

The uniformity coefficient (UC) is the ratio of the particle size that is 60% finer ($D_{60}$) by weight to the particle size that is 10% finer ($D_{10}$) by weight on the grain size distribution curve (Das 1993). The UC 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 AST performance-based uniformity coefficient (PUC) using UC concept and McLeod’s AST failure criteria can be effective as a performance indicator of aggregate gradation. The PUC is the ratio of the percentage passing at a given embedment depth to the percentage passing at twice the embedment depth in a sieve analysis curve.

$$PUC = \frac{P_{EM}}{P_{2EM}}$$  \hspace{1cm} (6)

where
\[ PUC = \text{performance-based uniformity coefficient}; \]

\[ P_{EM} = \text{percentage passing at a given embedment depth (} E) \text{ of median particle size (} M) \]
in sieve analysis curve; and

\[ P_{2EM} = \text{percentage passing at twice the embedment depth of median particle size in} \]
sieve analysis curve.

The closer the PUC is to zero, the more uniformly the aggregate is graded. In other words, the \( P_{EM} \) that is closer to 0% and the \( P_{2EM} \) that is closer to 100% indicate a more uniform gradation that corresponds to a better AST performance, less aggregate loss and less bleeding.

Examples of gradations are provided in Figure 4-24 to show the difference between the UC and the PUC. The three different gradations were created within the range of the 78M specification. Information about these three gradations is provided in Table 4-4. The PUCs are 0.19, 0.35, and 0.55 for G1, G2, and G3 gradations, respectively, which indicates that the G1 gradation is the best gradation among the three gradations for AST performance. However, their UCs show the opposite trend in terms of gradation types. The gradation closest to a UC of 1 is the G3 gradation, which indicates that the G3 gradation is the most uniform gradation of the UCs.
Figure 4-24 Examples of gradation uniformity

Table 4-4 Summary of Example Gradations

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
</tr>
</thead>
<tbody>
<tr>
<td>D60 (mm)</td>
<td>6.9</td>
<td>7.2</td>
<td>7.6</td>
</tr>
<tr>
<td>D10 (mm)</td>
<td>1.6</td>
<td>2.6</td>
<td>2.9</td>
</tr>
<tr>
<td>UC</td>
<td>4.34</td>
<td>2.77</td>
<td>2.62</td>
</tr>
<tr>
<td>$P_{EM}$ (%)</td>
<td>17.2</td>
<td>27.9</td>
<td>38.6</td>
</tr>
<tr>
<td>$P_{2EM}$ (%)</td>
<td>89.4</td>
<td>79.5</td>
<td>69.7</td>
</tr>
<tr>
<td>PUC</td>
<td>0.19</td>
<td>0.35</td>
<td>0.55</td>
</tr>
</tbody>
</table>
To confirm the PUC’s effectiveness, the MMLS3 aggregate retention tests using were carried out using the three gradations with different PUCs. The three gradations were created between the No. 8 sieve and the 3/4” sieve within the range of the 78M specification. The aggregates passing the No. 8 sieve were removed to eliminate the effects of the small particles and fines, as shown in Figure 4-25. The same AST application rates were applied for the three different gradations because the aggregate gradations have the same median size. The AST application rate was determined using the McLeod design method with a 70% embedment depth. Information about the three gradations and their performance results under the MMLS3 test are shown in Table 4-5.

The percentage of aggregate loss from the MMLS3 test is comparable to the calculated aggregate loss (100-\(P_{2EM}\)) from the gradation and McLeod failure criteria. The observation made from these data is that the aggregate loss percentage in the MMLS3 test is smaller than that in the 100-\(P_{2EM}\). The calculated loss and the measured aggregate loss show the same rankings for the three gradations. The G1 gradation and G3 gradation show the best and the poorest performance, respectively. The ratio increases of both the calculated and measured loss in terms of aggregate gradation are similar to each other. It is expected that a bleeding analysis of these three PUCs, in addition to the results regarding aggregate loss, could yield a more complete evaluation that can support the PUC as an AST performance indicator.

The PUC using the calculated AST performance data is based on the one-stone thick layer AST and the thickness of AST layer depends on the AST application rate and the aggregate gradation. The optimum AAR or a less than optimum AAR is more likely to achieve a one-stone thick layer than an AAR that is higher than the optimum rate. With the
graded aggregate, a poor AST performance can be expected due to the high PUC with the one-stone thick layer. However, it should be noted that in reality an optimum AAR using graded aggregate should not be used with a one-stone thick layer due to its poor performance, nor can the thickness even be a one-stone thick layer naturally, as shown in Figure 4-12. Furthermore, McLeod’s AST failure criterion for the graded aggregate can be one of several failure modes, including 1) the lack of emulsion coating on the aggregates that are on top of smaller aggregate particles, which causes aggregate loss; 2) the crushing of the aggregate that is on top of smaller aggregate due to traffic loads; and 3) inadequate aggregate embedment depth; etc. Conclusively, further research evaluating and clarifying the AST failure criteria in terms of the aggregate gradation is required to use the PUC as a better AST performance indicator.

Table 4-6 provides the PUCs, including: the gradations of the upper limit and lower limit of the 78M specification; the gradation of the best AST performance in the 78M specification range; the gradations of lightweight aggregate and the lightweight aggregate retained on the No. 8 sieve; and the gradations of granite aggregate and the granite aggregate retained on the No. 8 sieve size. It is noted that all performance calculations in Table 4-6 are based on the standard sieve sizes used to classify aggregate (ASTM D448-03a). Among the eight gradations shown in this table, the generated best gradation in 78M specification range and the lightweight aggregate retained on No. 8 sieve size show the lowest PUC. Those two gradations are fairly similar each other in the median size and the gradation shape. One of the findings from the MMLS3 test results is the aggregate loss reductions that are due to the modification of the original gradations to the more uniform gradation using the aggregates retained on the No. 8 sieve size for both the lightweight aggregate and the granite aggregate.
This finding is confirmed by the 100-$P_{2EM}$ and PUC reductions from the same modification of aggregate gradations as the MMLS3 test, as shown in Figure 4-6 and Table 4-6. However, the aggregate losses due to the modified gradations for the granite and lightweight aggregate are about the same in the MMLS3 test, but not in the 100-$P_{2EM}$ for those two aggregates. This difference can be explained by two reasons. The first reason is the error of McLeod’s assumption from the layer difference between the one-stone thick layer in his assumption and the multiple-stone thick layer in reality. The other reason is the different levels of embedment depth for each aggregate type in the actual AST specimen used for the MMLS3 test. Based on the same 100-$P_{2EM}$ for both the aggregate types and using the embedment depth of the lightweight aggregate as a reference, the percentages of embedment depths are 70% and 78% for the lightweight and the granite aggregate retained on the No. 8 sieve, respectively.
Figure 4-25 Three created gradations with the same median for performance comparison

Table 4-5 Data Summary of Three Different Gradations

<table>
<thead>
<tr>
<th></th>
<th>G1</th>
<th>G2</th>
<th>G3</th>
</tr>
</thead>
<tbody>
<tr>
<td>P(_{EM}) (%)</td>
<td>17.2</td>
<td>27.9</td>
<td>38.6</td>
</tr>
<tr>
<td>P(_{2EM}) (%)</td>
<td>89.4</td>
<td>79.5</td>
<td>69.7</td>
</tr>
<tr>
<td>100-P(_{2EM}) (%)</td>
<td>10.6</td>
<td>20.5</td>
<td>30.3</td>
</tr>
<tr>
<td>Ratio of 100-P(_{2EM})</td>
<td>1.00</td>
<td>1.93</td>
<td>2.86</td>
</tr>
<tr>
<td>PUC</td>
<td>0.2</td>
<td>0.4</td>
<td>0.6</td>
</tr>
<tr>
<td>Ratio of PUC</td>
<td>1.00</td>
<td>1.80</td>
<td>2.90</td>
</tr>
<tr>
<td>AAR (lb/yd(^2))</td>
<td></td>
<td>18.01</td>
<td></td>
</tr>
<tr>
<td>EAR (gal/yd(^2))</td>
<td></td>
<td>0.17</td>
<td></td>
</tr>
<tr>
<td>Aggregate Loss in MMLS3 Test (%)</td>
<td>4.0</td>
<td>7.10</td>
<td>9.30</td>
</tr>
<tr>
<td>Ratio of Aggregate Loss in MMLS3 Test</td>
<td>1.00</td>
<td>1.78</td>
<td>2.32</td>
</tr>
</tbody>
</table>
Table 4-6 Summary of AST Performance Expectations Using Gradations

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>M (in.)</td>
<td>0.20</td>
<td>0.31</td>
<td>0.26</td>
<td>0.25</td>
<td>0.26</td>
<td>0.22</td>
<td>0.23</td>
</tr>
<tr>
<td>Calculated Bleeding, $P_{EM}$ (%)</td>
<td>30.90</td>
<td>24.5</td>
<td>18.50</td>
<td>21.50</td>
<td>18.37</td>
<td>28.77</td>
<td>26.34</td>
</tr>
<tr>
<td>$P_{2EM}$ (%)</td>
<td>74.00</td>
<td>81.00</td>
<td>94.00</td>
<td>91.05</td>
<td>94.16</td>
<td>76.06</td>
<td>79.82</td>
</tr>
<tr>
<td>Calculated Agg. Loss, $100 - P_{2EM}$ (%)</td>
<td>26.00</td>
<td>19.00</td>
<td>6.00</td>
<td>8.95</td>
<td>5.84</td>
<td>23.94</td>
<td>20.18</td>
</tr>
<tr>
<td>$P_{UC}$</td>
<td>0.42</td>
<td>0.30</td>
<td>0.20</td>
<td>0.24</td>
<td>0.20</td>
<td>0.38</td>
<td>0.33</td>
</tr>
<tr>
<td>Agg. Loss (%)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>6.4</td>
<td>3.8</td>
<td>10.5</td>
<td>5.1</td>
</tr>
</tbody>
</table>
5. DEVELOPMENT OF ASPHALT SURFACE TREATMENT DESIGN METHOD

5.1 Comparison of AST Design Rates Using Various Design Methods

The differences in AST application rates among the different design methods can be explored to determine an optimum design rate. The six different design methods – 2004 New Zealand, McLeod (1969), modified Kearby (1974), Hanson (1935), California (1949), and Lovering (1954) methods – are used for the AST design. Additionally, the optimum AST design rates selected by NCDOT engineers, as shown in Chapter 4.1, and the rates recommended by ASTM D 1369-84 and NCDOT specifications (NCDOT Standard Specifications for Roads and Structures 2002) are summarized and presented in Table 5-1, Figure 5-1, and Figure 5-2.

It is found that the smallest and largest designed AARs for the lightweight and the granite aggregate are in the 2004 New Zealand (approximately 40% of AAR design reference voids) and the Hanson method (20% of AAR design reference voids), respectively. The 2004 New Zealand method has approximately twice larger then the EAR of Hanson method for the granite aggregate. These different AST design rates with different reference voids show the importance of the reference voids as a design factor.

The largest AARs are 33% and 75% larger than the smallest AARs for the lightweight and the granite aggregate, respectively. The smallest EARs for both aggregate types are obtained from the Lovering method, and the largest EARs for both aggregate types are designed using the modified Kearby method and the NCDOT specifications. The largest EARs are approximately twice and three times larger than the smallest EARs for the lightweight and the granite aggregates, respectively. Such different AST design rates
illustrate the need for evaluating the current design methods and developing a new design method that can show the best AST performance.

Table 5-1 Summary of Designed AST Rates

<table>
<thead>
<tr>
<th>Design Methods or Specifications</th>
<th>AAR (lb/yd²)</th>
<th>EAR (gal/yd²)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Lightweight</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2004 New Zealand</td>
<td>8.39</td>
<td>0.25</td>
</tr>
<tr>
<td>McLeod (1969)</td>
<td>11.15</td>
<td>0.26-0.19</td>
</tr>
<tr>
<td>Modified Kearby (1974)</td>
<td>8.40</td>
<td>0.32-0.28</td>
</tr>
<tr>
<td>Hanson (1935)</td>
<td>12.33</td>
<td>0.2</td>
</tr>
<tr>
<td>California (1949)</td>
<td>8.66</td>
<td>0.26</td>
</tr>
<tr>
<td>Lovering (1954)</td>
<td>10.01</td>
<td>0.13</td>
</tr>
<tr>
<td>Engineers’ Selection</td>
<td>9.00</td>
<td>0.26</td>
</tr>
<tr>
<td><strong>Granite</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2004 New Zealand</td>
<td>11.12</td>
<td>0.21</td>
</tr>
<tr>
<td>McLeod</td>
<td>15.03</td>
<td>0.19-0.14</td>
</tr>
<tr>
<td>Modified Kearby</td>
<td>13.68</td>
<td>0.19-0.16</td>
</tr>
<tr>
<td>Hanson</td>
<td>19.94</td>
<td>0.12</td>
</tr>
<tr>
<td>California</td>
<td>17.28</td>
<td>0.24</td>
</tr>
<tr>
<td>Lovering</td>
<td>19.45</td>
<td>0.13</td>
</tr>
<tr>
<td>Engineers’ Selection</td>
<td>14.00</td>
<td>0.20</td>
</tr>
<tr>
<td><strong>No. 8</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASTM D 1369-84</td>
<td>16.32</td>
<td>0.19</td>
</tr>
<tr>
<td><strong>No. 78M</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NCDOT Specifications</td>
<td>17-22</td>
<td>0.35-0.4</td>
</tr>
</tbody>
</table>

Note: 1. The recommended AST application rates in the ASTM D 1369-84 and NCDOT specifications are based on the aggregates having the typical specific gravity of around 2.6.
Figure 5-1 AST application rates using different methods: lightweight aggregate

Figure 5-2 AST application rates using different methods: granite aggregate
5.2 Development of Performance-Based Asphalt Surface Treatment Design Method

Designing ASTs involves determining the optimum amount of AST material using the AST material properties. Several factors affect the AST design, as shown in Table 2-4. The major factor among them is the reference voids, and some of the other factors also relate to the reference voids. For example, the reference voids change according to aggregate gradation shape, particle shape, median size, traffic volume, existing pavement conditions, etc.

The performance-based AST design equation can be developed using the fundamental voids concept developed by Hanson (1935) as well as the AST performance test using the MMLS3. The concept of this AST design development is shown in Figure 5-3.

To develop the performance-based AST design equation, the laboratory MMLS3 test using several AST rate combinations is investigated. This investigation provides the AST performance characteristics that, in turn, help develop the voids reduction model in terms of design factors from the laboratory as well as help develop the optimum application determination. Based on the factors at the optimum AST rates, the design equation under MMLS3 test conditions is provided.

The voids reduction model, the AST design equation, and other findings developed from the laboratory testing must be verified and calibrated against the field AST. The field ASTs will be constructed using the AST design equation developed by the MMLS3 test, and the AST samples obtained periodically from the field AST will be measured to determine the volume of voids and embedment depth. The laboratory results and the field results will be compared to calibrate the laboratory-developed design equation.
This research covers the AST design equation as a function of aggregate gradation under MMLS3 test conditions as part of a comprehensive methodology of AST design equation development. The procedure for developing a performance-based AST design under MMLS3 test conditions is presented in Figure 5-3 (in bold).

### 5.2.1 Experimental Program

Knowing the actual AST performances using the various AST application rates can be valuable for providing fundamental information needed for AST performance predictions and the optimum application determination. AST performance, including aggregate loss, bleeding performance, and aggregate embedment depth, was evaluated using the MMLS3 in 32 and 28 application rate combinations for the lightweight aggregate and the granite aggregate, respectively, as shown in Table 5-2. This experimental program for MMLS3 testing was developed based on the findings from previous MMLS3 testing and flip-over tests (FOTs), design results from the McLeod and modified Kearby methods, and input from NCDOT engineers.
Design Factors

- Aggregate gradation
- Aggregate types
- Aggregate median size
- Emulsion types
- etc.

AST performance tests using MMLS3
- Aggregate retention
- Bleeding (DIP application)

Development of voids reduction models under MMLS3 trafficking

Determination of optimum AST application rates based on performance criteria

Determination of reference voids and embedment depth for optimum AAR and EAR

Development of an AST design equation under MMLS3 conditions

Correlation between laboratory tests and field tests

AST performance tests in field
- Visual observation
- Samples for laboratory measurements

Development of voids reduction models under field conditions

Development of an AST design method under field conditions

Figure 5-3 Procedure for developing performance-based AST design
Table 5-2 Experimental Program

<table>
<thead>
<tr>
<th>Aggregate Application Rate (lb/yd²)</th>
<th>Lightweight</th>
<th>Emulsion Application Rate (gal/yd²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 0.2</td>
<td>0.25</td>
<td>0.3</td>
</tr>
<tr>
<td>8 0.2</td>
<td>0.25</td>
<td>0.3</td>
</tr>
<tr>
<td>9 0.2</td>
<td>0.25</td>
<td>0.3</td>
</tr>
<tr>
<td>10 0.2</td>
<td>0.25</td>
<td>0.3</td>
</tr>
<tr>
<td>11 0.2</td>
<td>0.25</td>
<td>0.3</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>0.3</td>
</tr>
<tr>
<td>13 0.2</td>
<td>0.25</td>
<td>0.3</td>
</tr>
<tr>
<td>Granite</td>
<td>11 0.15</td>
<td>0.2</td>
</tr>
<tr>
<td>12 0.15</td>
<td>0.2</td>
<td>0.25</td>
</tr>
<tr>
<td>13 0.15</td>
<td>0.2</td>
<td>0.25</td>
</tr>
<tr>
<td>14 0.15</td>
<td>0.2</td>
<td>0.25</td>
</tr>
<tr>
<td>15 0.15</td>
<td>0.2</td>
<td>0.25</td>
</tr>
<tr>
<td>16 0.15</td>
<td>0.2</td>
<td>0.25</td>
</tr>
<tr>
<td>17 0.15</td>
<td>0.2</td>
<td>0.25</td>
</tr>
</tbody>
</table>
5.2.2 Aggregate Loss Performance

The aggregate loss performance using the MMLS3 test in terms of various AARs and EARs under the experimental program is presented in Figure 5-4. In general, more aggregate is lost as the AAR increases and the EAR decreases. The rate of increase of aggregate loss with an increasing AAR becomes greater as the EAR decreases, for the lightweight aggregate, but the rate remains fairly constant with each EAR decrease for the granite aggregate.

As well as the AAR, the residual state after aggregate loss is important because the ASTs are without excessive aggregate during most of their life. From the point of view of AST structure, the higher the residual aggregate rate (RAR), the more densely packed is the aggregate structure in the AST, which may result in a better structural roll for the pavement due to the resistance to heavier and a greater volume of traffic. If all applied aggregates remain on the specimens without losing aggregate particles, their RAR and AAR are the same; these rates can be expressed as a straight diagonal line, called the line-of-equality (LOE) in the AARs vs. RARs chart, as shown in Figure 5-6. The inequality represents the aggregate loss. Figure 5-6 presents RAR changes in various lightweight and granite AST application rates in the MMLS3 tests. In general, the higher the AAR or EAR, the greater the RAR for both aggregate types, but the RAR remains constant after certain AARs with each EAR in the lightweight aggregate. The granite aggregate shows a continuous RAR increase with an AAR increase. This RAR behavior difference between the lightweight and the granite aggregate can be explained by the gradation effect. In this research, the lightweight aggregate has a relatively uniform particle size gradation. The characteristic of uniform gradation in terms of AAR is shown in Figure 5-5 (a). The voids between the aggregate particles decrease as the AAR increases from state A to state C. After the voids reach the
smallest with the one-stone thick layer (state C), the additional aggregate particles that sit on the top of the bottom layer and that are not coated or barely coated by emulsion (state D) are easily removed by traffic; this factor yields a constant RAR. However, the granite aggregate, which has relatively well-graded aggregate particles, can retain the additional aggregate particles in the voids and can form a multi-stone thick layer, as shown in state e in Figure 5-5 (b); this factor yields a continuous RAR increase in the ranges of the AARs and EARs tested in this research.

Generally speaking, the 10% allowable aggregate loss has been applied to the AST application rate design. The thresholds between the slopes and the planes, shown in Figure 5-6 (a), show approximate agreement as seen by the 10% allowable aggregate loss line. However, this observation is not applicable to the granite aggregate due to the absence of the thresholds. This finding confirms that the 10% allowable aggregate loss is reasonable at a certain level of uniform gradation.

![Figure 5-4](image)

Figure 5-4 Aggregate loss performances with various AAR and EAR combinations: (a) lightweight aggregate; and (b) granite aggregate
Figure 5-5 Section views of ASTs: (a) uniform-sized aggregate; and (b) graded aggregate

Figure 5-6 Residual and applied rates on AST specimens in MMLS3 test: (a) lightweight aggregate; and (b) granite aggregate
5.2.3 Bleeding Performance of Asphalt Surface Treatments

As well as aggregate loss, bleeding is a critical performance factor of ASTs. To quantify the bleeding performance, the 2-D digital image processing (DIP) procedure described in Chapter 3 was carried out after the MMLS3 bleeding test at 50°C.

Mapping the bleeding areas is conducted using four selected AST specimens for each type of aggregate. Their bleeding areas are quantified by the particle analysis in the DIP. The averaged critical bleeding GIVs for the lightweight and the granite aggregates are determined based on matching the bleeding area to the GIV histograms, as shown in Table 5-3. The average critical bleeding GIVs are applied to all the histograms obtained from the AST application rate combinations, as shown in Figure 5-7. The percentage of bleeding is defined as the percentage of the number of pixels having smaller GIVs than the critical bleeding GIV in the total number of pixels in the histogram.

Table 5-3 Summary of Threshold Intensities for Bleeding Measurement

<table>
<thead>
<tr>
<th>AST Rates (lb/yd² – gal/yd²)</th>
<th>AST State</th>
<th>Critical Bleeding GIV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lightweight</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7-0.4</td>
<td>After bleeding</td>
<td>104</td>
</tr>
<tr>
<td>10-0.4</td>
<td>Before bleeding</td>
<td>84</td>
</tr>
<tr>
<td>10-0.2</td>
<td>After bleeding</td>
<td>49</td>
</tr>
<tr>
<td>13-0.2</td>
<td>Before bleeding</td>
<td>49</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>71.5</td>
</tr>
<tr>
<td>Granite</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11-0.3</td>
<td>After bleeding</td>
<td>67</td>
</tr>
<tr>
<td>14-0.15</td>
<td>Before bleeding</td>
<td>38</td>
</tr>
<tr>
<td>14-0.3</td>
<td>After bleeding</td>
<td>63</td>
</tr>
<tr>
<td>17-0.15</td>
<td>Before bleeding</td>
<td>69</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>59</td>
</tr>
</tbody>
</table>
Figure 5-7 Critical GIVs in GIV histograms: (a) lightweight aggregate; (b) granite aggregate
The major cause of bleeding is excessive emulsion. In a hot temperature (50°C) test, the emulsion is soft between the aggregate particles, which means that the aggregate particles in the emulsion are more easily pitched and rolled by MMLS3 trafficking. This movement of the aggregate can squeeze emulsion upward and between the aggregate particles; the raised emulsion is then tracked over the AST specimen by the MMLS3 tires.

More residual aggregate or more applied aggregate has a greater aggregate surface to be coated by the emulsion and, at a constant EAR, the more residual aggregate or more applied aggregate leads to less bleeding. Based on this relationship, the bleeding performance in terms of EAR and AAR can be expressed as seen in Figure 5-8 and confirmed with the bleeding performances of the lightweight and the granite aggregate, as shown in Figure 5-9. The bleeding performances are separated into two groups above and below about 55% for the lightweight aggregate and 60% for the granite aggregate. These 55% and 60% values for each aggregate type are used for the critical percentage of bleeding as failure criteria.

The residual emulsion rates (RERs) and RARs based on a square yard pavement area are calculated to confirm the relationship between the bleeding performance and AST application rates. The RER is the emulsion rate after full curing (water evaporation) of the AST. In this research, the percentage of residue for the emulsion is about 70%, as shown in Figure 3-7. Table 5-4 summarizes the bleeding performance and the volume ratio of RER to RAR in the AAR and EAR combinations. In this table, the bleeding failures are expressed with the highlighted cells. The critical volume ratios representing the bleeding failure are approximately 0.31 for the lightweight aggregate and approximately 0.26 for the granite aggregate. In other words, emulsion volumes greater than 31% and 26%, based on aggregate volumes of each type, cause bleeding failure. Based on the critical RER volume ratios for
each aggregate, it is confirmed that the uniform particle gradation of the lightweight aggregate has a better bleeding performance at the same volume of applied aggregate. This performance is explained by the fact that the more well-graded aggregate has a wider aggregate particle size distribution range, allows small particles to be embedded more deeply, and exposes more emulsion on the surface of the AST due to aggregate loss with large aggregate, as shown for states C and c in Figure 5-5.

![Figure 5-8 Scheme of AST bleeding performance](image-url)

Figure 5-8 Scheme of AST bleeding performance
Figure 5-9 Bleeding performances after MMLS3 bleeding test: (a) lightweight aggregate with AARs; (b) lightweight aggregate with RARs; (c) granite aggregate with AARs; (d) granite aggregate with RARs
## Table 5-4 Volume Ratios of RER to RAR in AAR and EAR Combinations

<table>
<thead>
<tr>
<th>Aggregate Application Rate (lb/yd²)</th>
<th>Emulsion Application Rate (gal/yd²)</th>
<th>0.15</th>
<th>0.2</th>
<th>0.25</th>
<th>0.3</th>
<th>0.35</th>
<th>0.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lightweight</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>0.28</td>
<td>0.35</td>
<td>0.42</td>
<td>0.49</td>
<td>0.55</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>0.25</td>
<td>0.31</td>
<td>0.37</td>
<td>0.43</td>
<td>0.49</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>0.23</td>
<td>0.28</td>
<td>0.34</td>
<td>0.39</td>
<td>0.44</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>0.21</td>
<td>0.27</td>
<td>0.31</td>
<td>0.36</td>
<td>0.41</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>0.21</td>
<td>0.26</td>
<td>0.30</td>
<td>0.33</td>
<td>0.37</td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.29</td>
</tr>
<tr>
<td>13</td>
<td>0.21</td>
<td>0.26</td>
<td>0.29</td>
<td>0.32</td>
<td>0.34</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Granite</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>0.22</td>
<td>0.30</td>
<td>0.36</td>
<td>0.43</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>0.21</td>
<td>0.27</td>
<td>0.33</td>
<td>0.39</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>0.20</td>
<td>0.26</td>
<td>0.32</td>
<td>0.37</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>0.20</td>
<td>0.24</td>
<td>0.30</td>
<td>0.35</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>0.19</td>
<td>0.24</td>
<td>0.28</td>
<td>0.34</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>0.18</td>
<td>0.23</td>
<td>0.27</td>
<td>0.32</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>0.18</td>
<td>0.22</td>
<td>0.26</td>
<td>0.31</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
5.2.4 Design Methods Comparison for AST Performance

Several AST design methods are currently available, as shown in Chapter 5.1. Their aggregate loss and bleeding performances have been evaluated using the MMLS3 test for lightweight and granite aggregate.

The AST design results, calculated by the seven different methods for the lightweight aggregate and by the nine different methods for the granite aggregate, are provided in Table 5-1. Based on these AST rates, the percentage of aggregate loss and percentage of bleeding are obtained using Figure 5-4 and Figure 5-9 and are plotted in Figure 5-10. Because some of the designed AST rates exceed the experimental program range, limited AST design rates are evaluated, including the NCDOT engineers’ rate described in Chapter 4.1, and those from the 2004 New Zealand, modified Kearby, McLeod, California, and Hanson methods for the lightweight aggregate, and the NCDOT engineers’ rate, 2004 New Zealand, modified Kearby, McLeod designs and the ASTM recommendations for the granite aggregate. For the lightweight aggregate, the NCDOT engineers’ rate, 2004 New Zealand, and the California design methods show relatively good AST performance, but the modified Kearby method shows bleeding failure (more than 60% bleeding); the McLeod and Hanson methods show aggregate loss failure (more than 10%). For the granite aggregate, the NCDOT engineers’ selection shows the best AST performance among the five AST design methods.

Thirty-seven percent of agencies in the United States still use the quantities of emulsion and aggregate as determined by experience and/or precedent (Gransberg 2005), and the findings herein support the effectiveness of the AST rate determined by experience. However, the modified Kearby and McLeod methods, the most popular design methods in North America (Gransberg 2005), show poor performance in the MMLS3 test, which illustrates the need for a better AST design method.
Figure 5-10 Aggregate loss and bleeding performances with various design methods: (a) lightweight aggregate; and (b) granite aggregate
5.2.5 Determination of Optimum AST Application Rate

The determination of an optimum AST application rate based on aggregate retention and bleeding performances is made using the MMLS3 test results. The optimum rate combinations using AARs and EARs are determined using two criteria in the AST performance: 1) the AAR showing the aggregate loss close to but not exceeding 10%; and 2) a higher EAR that does not produce bleeding. A higher AAR yields more densely-packed aggregate in the AST and performs an improved structural role providing a surface course and protecting the existing pavement. A higher EAR shows the better performance in retaining the cover aggregate and preventing water infiltration into the existing pavement.

Table 5-5 provides the aggregate loss and bleeding performances under MMLS3 testing using the AST rate combinations. In this table, the highlighted cells represent the bleeding failure and the aggregate losses, in bold, represent the aggregate loss greater than 10%. For the lightweight aggregate, the application rate combinations that have an aggregate loss less than 10% but no bleeding failure are candidates for the optimum AST application rate. For the lightweight aggregate, all AARs with 0.2 gal/yard² and 0.25 gal/yard² of EARs, except the rate combinations that show aggregate loss and bleeding failures, are candidates. For the granite aggregate, AARs - EARs with 12 lb/yard² -0.15 gal/yard², 13 lb/yard² -0.2 gal/yard², and 14 lb/yard² -0.2 gal/yard² are the candidates. Based on the criteria for determining the optimum AST application rates, the optimum rates are determined to be 10 lb/yard² and 0.25 gal/yard² for the lightweight aggregate and 14 lb/yard² and 0.2 gal/yard² for the granite aggregate.
Table 5-5 AST Aggregate Loss and Bleeding Failure with AST Rate Combinations

<table>
<thead>
<tr>
<th>Aggregate Application Rate (lb/yd²)</th>
<th>Emulsion Application Rate (gal/yd²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lightweight</td>
<td>0.15</td>
</tr>
<tr>
<td>7</td>
<td>1.02</td>
</tr>
<tr>
<td>8</td>
<td>2.88</td>
</tr>
<tr>
<td>9</td>
<td>6.48</td>
</tr>
<tr>
<td>10</td>
<td>9.60</td>
</tr>
<tr>
<td>11</td>
<td>17.76</td>
</tr>
<tr>
<td>12</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td></td>
</tr>
<tr>
<td>Granite</td>
<td>11</td>
</tr>
<tr>
<td>12</td>
<td>10.25</td>
</tr>
<tr>
<td>13</td>
<td>11.63</td>
</tr>
<tr>
<td>14</td>
<td>16.53</td>
</tr>
<tr>
<td>15</td>
<td>17.44</td>
</tr>
<tr>
<td>16</td>
<td>20.93</td>
</tr>
<tr>
<td>17</td>
<td>25.78</td>
</tr>
</tbody>
</table>
5.2.6 Development of AST Design Equation under MMLS3 Testing

The AST design methods shown in Table 5-1 have different reference voids. For example, the reference voids for the AST designs are 20% voids for the Hanson and McLeod methods, 45-50% for the modified Kearby, and approximately 40% for the 2004 New Zealand. The voids concept developed by Hanson (1935) has been popularly used for the AST design methods. The percentage of reference voids in the AST optimum application rates for the lightweight aggregate and the granite aggregate, determined by the MMLS3 test, can be calculated using the fundamental voids concept developed by Hanson (1935).

\[ RAR = 46.8(1 - V_R)HG \]  
\[ EAR = \frac{(2.244)V_RHT}{R} \]

where

\( AAR \) = aggregate application rate, lb/yd\(^2\);
\( RAR \) = residual aggregate rate, lb/yd\(^2\);
\( V_R \) = percentage of reference voids, expressed as decimal;
\( H \) = average least dimension, in.;
\( G \) = bulk specific gravity of aggregate;
\( EAR \) = emulsion application rate, gal/yd\(^2\);
\( T \) = percentage of embedment depth of aggregate in emulsion, expressed as decimal;

and

\( R \) = percentage of residue of emulsion, expressed as decimal.
The voids in ASTs are reduced from the high voids volume at the loose aggregate state to the small voids volume at the compacted aggregate state where the compacted aggregate particles are oriented to lay on their flattest side. Because the above equation uses the ALD as the thickness of the AST, using the RAR is more reasonable than using the AAR for the application of the voids concept. The other reason is that the optimum AAR is normally larger than the optimum RAR because of the aggregate loss. If the above equation with the optimum AAR is used for calculating the percentage of voids, then the AAR produces smaller voids than the RAR does, as shown in Table 5-6. This also indicates that the voids volume at the initial loose AST state is lower than that at the compacted AST state, based on the volume of the unit area with the ALD thickness. However, the initial AST thickness is larger than the ALD. Therefore, the percentage of voids calculated by the AAR is not the actual physical voids volume, but the percentage of voids calculated by the RAR can be the physical voids volume due to the compatibility of the AST mat thickness and the ALD. The calculated reference voids at the compacted aggregate state using the RAR for the lightweight aggregate and the granite aggregate are 35.18% and 31.99%, respectively.

In order to reproduce the optimum AAR, which is the actual target application to reach the expected RAR state after ultimate compaction by MMLS3 trafficking, the additional factor of allowable aggregate loss (10%) should be applied. This additional factor is normally called the wastage factor (E). Using equation (8), the optimum EARs, and voids obtained from the RARs, the optimum embedment depths in emulsion under MMLS3 test conditions are calculated as 61.57% and 65.01% for the lightweight and the granite aggregate, respectively.
Another important observation regarding the voids in ASTs is the change in voids in terms of traffic compaction and aggregate type. The aggregate gradation derived from uniform particle sizes and well distributed particle sizes yields voids ranging in volume from large to small. It is known that the percentage of voids of ASTs before roller compaction is similar to that in the loose aggregate state (Hanson 1935 and McLeod 1969). The same voids ratio of lightweight to granite in the loose aggregate and in the compacted aggregate are found as 1.09, as shown in Table 5-7 and Figure 5-11. Furthermore, a similar voids reduction ratio of loose voids to compacted voids between the lightweight aggregate and the granite aggregate is found, as shown in Table 5-7. Based on these observations, it is concluded that the reference voids (the voids in compacted aggregate) can vary in terms of the aggregate gradation type.

To formulate the reference voids in terms of the loose voids, the correlation between loose voids and reference voids was carried out, and this result is shown in Figure 5-12. This correlation is applied to the reference voids in the Hanson voids concept as the following equation, which serves as the AST design equation under the MMLS3 test conditions and the type of aggregate used in the research.

\[ AAR = 46.8(1-V_R)HGE \]  

\[ E = \text{wastage factor.} \]

\[ LAAR = 46.8(1-(0.75V_l-0.0325))HGE \]  

\[ L2.244(0.75V_l-0.0325)HTEAR = R \]
Table 5-6 Summary of Optimum AST Rates and Design Factors

<table>
<thead>
<tr>
<th></th>
<th>Lightweight</th>
<th>Granite</th>
</tr>
</thead>
<tbody>
<tr>
<td>Optimum AAR (lb/yd(^2))</td>
<td>10.00</td>
<td>14.00</td>
</tr>
<tr>
<td>Voids of AAR (%)</td>
<td>28.49</td>
<td>24.46</td>
</tr>
<tr>
<td>Optimum RAR (lb/yd(^2))</td>
<td>9.06</td>
<td>12.60</td>
</tr>
<tr>
<td>Voids of RAR (%)</td>
<td>35.18</td>
<td>31.99</td>
</tr>
<tr>
<td>Optimum EAR (gal/yd(^2))</td>
<td>0.25</td>
<td>0.20</td>
</tr>
<tr>
<td>Embedment Depth (%)</td>
<td>61.57</td>
<td>65.01</td>
</tr>
</tbody>
</table>

Table 5-7 Voids Comparison between Loose Aggregate State and Compacted Aggregate State by MMLS3

<table>
<thead>
<tr>
<th></th>
<th>Reference Voids (Voids after MMLS3 Test)</th>
<th>Voids in Loose Aggregate</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lightweight</td>
<td>0.35</td>
<td>0.51</td>
<td>0.69</td>
</tr>
<tr>
<td>Granite</td>
<td>0.32</td>
<td>0.47</td>
<td>0.68</td>
</tr>
<tr>
<td>Ratio</td>
<td>1.09</td>
<td>1.09</td>
<td></td>
</tr>
</tbody>
</table>
Figure 5-11 Voids reduction of lightweight and granite aggregate by MMLS3

\[ y = 0.75x - 0.0325 \]

Figure 5-12 Voids relationship between loose aggregate and compacted aggregate by MMLS3
6. EVALUATION OF SKID RESISTANCE TEST PERFORMANCE OF ASPHALT SURFACE TREATMENTS

One of the performance characteristics that is important in well-performing ASTs is skid resistance. As 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 GripTester (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 NCDOT specifications is the LWST. Because 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 allows the use of the BPT in the laboratory to convert the BPN to the SN in order to check against the specifications.

The three different tests, 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 6-1. They were constructed in 2003, and the skid resistance was measured between November 2004 and January 2005. The materials used on the selected roads are granite aggregate, lightweight aggregate, and their screenings (SCN). The tests were performed in both traffic directions.

6.1 Skid Resistance Tests and Results

The BPTs, as shown in Figure 6-1, 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 in Table 6-1 within a range from 71
to 90. Figure 6-2 shows the histogram of BPNs, which range from 53 to 100. The mean of the BPNs for all four sections 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 6-3) was performed on each of 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 in each traffic direction. The results from the LWST are reported as SNs and summarized in Table 6-1.

Figure 6-4 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 the test sections 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 SN 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 6-5, 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 a 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 both the GT and LWST, the GT towing vehicle moves from the center of the lane to the wheel path area. 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 6-6, the range of values is from 0.25 to 0.99, and the standard deviation of all data is 0.14. The average GN 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 Figure 2-7. Most of the measured values from the field are greater than 0.43, the minimum required for 40 mph testing, except for one section. Good skid resistance performance is evident from the GNs because 75% of all the data is greater than 0.8.

It can be concluded that, based on the above results from the three different test methods, the selected ASTs in the four counties have good friction values.
Table 6-1 Summary of Information for Skid Resistance Test Sections

<table>
<thead>
<tr>
<th>County</th>
<th>SR</th>
<th>Average</th>
<th>Aggregate Type For Top Layer</th>
<th>AST Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>SN</td>
<td>BPN</td>
<td>GN</td>
</tr>
<tr>
<td>Granville</td>
<td>1150</td>
<td>60.4</td>
<td>89</td>
<td>0.85</td>
</tr>
<tr>
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Figure 6-1 British Pendulum Test (BPT) on AST

Figure 6-2 Histogram of British Pendulum Numbers (BPNs)
Figure 6-3 Locked Wheel Skid Ttest (LWST)

Figure 6-4 Histogram of Skid Numbers (SNs)
Figure 6-5 Towing vehicle and Grip Tester (GT)

Figure 6-6 Histogram of Grip Numbers (GNs)
6.2 Effect of Aggregate Types on Skid Resistance

The type of AST cover aggregate is one of the factors that affects skid resistance performance. The skid resistance usually depends on the top layer of aggregate, but in the case of screening aggregate as the top layer, skid resistance depends on the top and second layers of aggregate. As shown in Table 6-2, the top layer of ASTs tested were composed of lightweight 5/16 in. aggregate, granite 78M aggregate, and their screening aggregates. In Figure 6-7, the test results from the three test methods are compared in terms of different aggregate types. It is noted that the GNs are presented on the right side of the y-axis, and the BPNs and SNs are presented on the left side of the y-axis.

The first observation to be made from Figure 6-7 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 draw 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.
Table 6-2 Average Skid Numbers for Test Sections

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<th>AST Type</th>
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132
6.3 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 6-1, are 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 6-8. A linear regression of the BPNs versus SNs shows 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 6-9 and Figure 6-10, respectively. Because 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 GNs. Compared to the SNs and BPNs, the GNs are relatively insensitive from one section to another, as can be seen in Figure 6-9 in the flat slope between the GN and SN relationship. Another reason for this poor correlation is that some GTs were conducted during different seasons than the LWSTs.

The BPNs and GNs are also used for performing the regression analysis. From Figure 6-10, 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 6-8 Correlation between average BPN and average SN](image_url)
Figure 6-9 Correlation between average SN and average GN

Figure 6-10 Correlation between average BPN and average GN
6.4 Skid Resistance Performance Evaluation using MMLS3

The fairly good linear regression results of the BPNs versus SNs that were obtained from the field tests were applied also to the MMLS3 AST performance evaluation procedure in the laboratory. The BPNs were obtained after the aggregate retention test (before the bleeding test) and after the bleeding test at 50°C for the lightweight and the granite aggregate.

Although there are several causes of bleeding and there are also different types of bleeding in ASTs, in whatever form, bleeding reduces the skid resistance. The BPN reductions of 13.8% and 10.4% for the lightweight and the granite aggregate, respectively, were observed after the MMLS3 bleeding test, as shown in Figure 6-11. The other observation made from this figure is that the lightweight aggregate has a higher BPN than the granite aggregate before and after the bleeding test; the same trend is found in the split seals, as shown in Figure 6-7. These observations show the ability of the MMLS3 test to simulate the texture of ASTs in the field. However, the BPT is the only skid resistance test method that can be used with the MMLS3 test in the laboratory. Using the correlation developed for the selected roads, the BPNs from the MMLS3 tests were converted to SNs and are presented in Figure 6-11. Based on the recommended SN of 40 in the NCDOT specifications, the lightweight and granite aggregates satisfy this condition for AST skid resistance performance under MMLS3 loading conditions.
Figure 6-11 BPN and SN of the granite and the lightweight aggregate on the straight seal
7. CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER RESEARCH

7.1 Conclusions

This research presents an asphalt surface treatment (AST) performance study of fines content and gradation effects, development of a performance-based AST design method under MMLS3 test conditions, and an evaluation of AST skid resistance. A new AST performance test method is herein developed using the MMLS3. It is found that this test method is an excellent means of evaluating aggregate loss and bleeding due to various mixture factors.

7.1.1 Effects of Fines Content and Gradation

Based on the fines content effect and aggregate gradation effect on AST aggregate retention performance tests and the FOT, the following conclusions are drawn:

1. The results from the MMLS3 aggregate retention tests and FOT confirm that the amount of aggregate loss decreases as the fines content decreases and the gradation becomes more uniform.

2. The amount of fines has much less of an effect on aggregate loss in the lightweight AST with a more uniform gradation than in the granite AST with a less uniform gradation.

3. The aggregate gradation plays a critical role in the aggregate retention performance regardless of the type of aggregate. The most critical factor in minimizing the aggregate loss in an AST is the uniformity of the gradation.
4. The PUC can be an indicator for AST performance in conjunction with the AST aggregate gradation.

7.1.2 Development of Performance-Based Asphalt Surface Treatment Design

Based on the characteristics of AST performance using the MMLS 3 test with 32 and 28 AST application rate combinations for the lightweight aggregate and the granite aggregate, their optimum application rates, critical factors, and the performance-based AST design equation under MMLS3 test conditions were found, and the following conclusions are drawn:

1. The results from the MMLS3 test confirm that the amount of aggregate loss decreases as the AAR decreases and the EAR increases.

2. The increase of the RAR with the AAR varies in terms of the aggregate gradation. The lightweight aggregate with the uniform gradation has a constant RAR after a certain AAR. Therefore, over a certain AAR using the uniform gradation aggregate, a significant aggregate loss is produced.

3. Taking into account aggregate wastage, the failure criterion of 10% allowable aggregate loss is reasonable for the aggregate gradation that is the most uniform.

4. The percentage of bleeding increases as the volume ratio of the residual emulsion to the residual aggregate increases, and the lightweight aggregate with a more uniform gradation can have a higher volume of emulsion without bleeding failure than the granite aggregate with a less uniform gradation.
5. The research found that the reference voids (the compacted voids by MMLS3 trafficking) vary in terms of the voids in loose aggregate that are affected by the aggregate gradation type.

7.1.3 Evaluation of Skid Resistance

Skid resistance was evaluated on 14 selected ASTs using three different tests: the BPT, LWST, and GT. 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 measured in the laboratory can be utilized for predicting the SN, which cannot be measured in the laboratory.

3. Lightweight aggregate has a higher BPN than granite aggregate before and after the bleeding test; the same trend is found in the results for the split seals. This observation confirms the ability of the MMLS3 test in simulating the texture (state) of ASTs in the field.

7.2 Recommendations for Further Research

7.2.1 Evaluation of Performance-Based Uniformity Coefficient (PUC)

Further research is recommended to evaluate the PUC developed during the course of this research. Even though the current research evaluates the PUC in terms of the aggregate retention performance, evaluating McLeod’s AST failure criteria used in the PUC with the aggregate gradation and embedment depth would help support the PUC application as an AST performance indicator.
To evaluate the PUC, an accurate AST performance measurement method is required for observing the behavior of the aggregate loss or the bleeding versus the embedment depth or the aggregate gradation. One candidate for this measurement method is the cross-cutting of an epoxy-reinforced AST combined using DIP.

The DIP technique is applied to the cross-section of an epoxy-reinforced AST specimen to measure the percentage of voids of the AST and the percentage of aggregate embedment. This procedure consists of three steps: (a) sample preparation; (b) image acquisition and the DIP; and (c) percentage of volume and embedment calculations. The digital image of the specimen’s cut surface is obtained using the same scanning method developed for the bleeding measurement. The digital image of the cross-section is used to determine the percentage of voids and embedment depth. The percentage of voids and the percentage of embedment depth are calculated based on the calculated ALD (Figure 7-1) and the measured voids using the following equations:

\[
\text{Voids } (\%) = \frac{A_{ALD} - A_{agg}}{A_{ALD}} \times 100
\]

\[
\text{Embedment Depth } (\%) = \frac{A_{Emul}}{A_{ALD} - A_{agg}} \times 100
\]

where

\(A_{ALD}\) = area of AST in the ALD thickness (i.e., ALD thickness multiplied by the sample width);

\(A_{agg}\) = area of aggregate in the ALD thickness; and

\(A_{Emul}\) = area of emulsion in the ALD thickness.
7.2.2 Development of Asphalt Surface Treatment Design Method

Further research, as described in Chapter 6, is required to extend the important findings of this study and transform the development of an AST design method that has limited factors and restricted conditions into a performance-based AST design method that considers more comprehensive design factors and types of ASTs (e.g., multilayered ASTs), as well as field verification and calibration.
CITED REFERENCES


BSI. *Test method for measurement of surface skid resistance using the GripTester braked wheel fixed slip device*. British Standards Institution, 7941-2.


Hanson, F.M. Bituminous Surface Treatment of Rural Highways, Proceedings, New Zealand Society of Civil Engineers, Vol. 21, 1934/35, pp. 89-179.


