Investigation into the Use of Carbonate Coarse Aggregates in Alabama Concrete Pavements: Final Report

by Nathan Klenke, E.I., Matthew Stone, Ph.D., Jay K. Lindly, Ph.D., Eric R. Giannini, Ph.D.

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Executive Summary

Carbonate coarse aggregates such as limestone and dolomite are not currently permitted in mainline concrete pavements in Alabama. However, up to 50% of the coarse aggregate in asphalt pavement surface courses may be carbonates. When siliceous aggregates must be imported for road construction, this situation often places concrete pavements at an economic disadvantage compared to asphalt pavements. A study sponsored by the Portland Cement Association – Southeast Region was conducted at The University of Alabama to investigate the possibility of reintroducing carbonate coarse aggregates for use in concrete pavements in Alabama.

This report addresses concerns about friction loss following diamond grinding of concrete pavements containing carbonate coarse aggregates, how to best characterize the polishing resistance of coarse aggregates, and how to design and construct new pavements to minimize the need for diamond grinding. On each topic, a review of current practices, specifications, and research is presented, specification improvements suggested, and research needs identified.

New approaches to materials characterization, design, and construction of pavements also offer the possibility of improved service life and reduced construction costs. A performance-based classification for coarse aggregates will allow ALDOT to classify them as polishing or non-polishing based on test results, rather than mineralogy; this ensures that softer materials are identified whether they are carbonate or siliceous. The adoption of Mechanistic-Empirical Pavement Design Guide (MEPDG) practices will enable pavement designers to account for the influence of joint spacing, coefficient of thermal expansion (CTE), and other variables that have a significant influence on pavement performance. A parametric study demonstrates how pavement designs can be optimized for CTE and that longer service life is possible without increasing thickness in some cases. The use of aggregate gradations optimized for paving applications and two-lift paving are presented as additional measures to reduce construction costs and improve the durability of pavement concrete. Demonstration projects are suggested to validate the performance of both optimized-graded pavement mixes and two-lift paving.

The restriction on carbonate aggregates in concrete pavements is primarily the result of post-diamond grinding friction loss of several pavement sections in the 1980s. However, the grinding was conducted using specifications now determined to be outdated. A review of state-of-the-art pavement rehabilitation methods suggests that new diamond grinding specifications and textures incorporating deeper grooves may result in improved friction retention, particularly when carbonate coarse aggregates are present. Controlled laboratory and field studies, including demonstration projects, are needed to validate this hypothesis. Some research is already underway in other states, and a framework is presented for future studies.

Finally, a transition to the use of the AASHTO M 286 (ASTM E524) smooth tire in pavement friction testing is proposed, pending a study to investigate the correlation to wet-
weather accident statistics. Research in other states suggests that smooth tire friction data will offer a better correlation to wet-weather accidents. If a similar conclusion can be made in Alabama, the adoption of smooth tire testing will allow ALDOT to better characterize safe, marginal, and unsafe friction levels for concrete pavements.

A summary of key findings in this report follow:

- A survey of SASHTO member DOTs and TxDOT found that nine of eleven agencies responding to the survey allow carbonate coarse aggregates in mainline rigid pavements, including all four states bordering Alabama.
- Aggregate performance characteristics, particularly with respect to polishing and soundness, may be better assessed using a combination of the Micro-Deval test and the unconfined freeze-thaw test. A new classification scheme in Tennessee may serve as a model for revising ALDOT’s aggregate specifications for rigid pavements.
- Locked-wheel friction test results exhibit a stronger correlation to wet weather accidents when a smooth, rather than ribbed tire, is used. A multi-year study is suggested for Alabama to assess the value of adopting this test on a system-wide scale.
- New diamond grinding techniques for rigid pavements that incorporate deeper grooves and wider land areas may provide sufficient macrotexture to offset polishing effects and maintain safe levels of pavement friction. Ongoing research in several states, combined with demonstration projects in Alabama, may provide verification of this hypothesis.
- The need for diamond grinding may be minimized by quality construction that meets and exceeds ALDOT smoothness requirements, and by the use of design and construction methods to resist cracking and faulting during the life of the pavement.
- The coefficient of thermal expansion (CTE) of concrete is dominated by the CTE of the aggregates, is rarely assessed, and is a major factor in pavement performance.
- Use of shorter joint spacing is a more cost-effective and efficient method to compensate for high CTE concrete than increasing the slab thickness.
- Adoption of MEPDG design procedures will allow pavement designers to better account for the influence of CTE and joint spacing.
- Two-lift paving allows for the use of more economical materials in the lower layer, potentially offsetting the inherent increase in construction costs incurred by the need for duplicate batching and paving equipment. However, research is still needed to determine the degree of variation in material properties between the two layers that can be tolerated.
- Use of optimized aggregate gradations specifically designed for slipform paving may lead to improvements in workability of the concrete, reduced drying shrinkage, improved durability, and reduced constructions costs. Recent research in Oklahoma may provide a model for new ALDOT gradation specifications for pavement concrete.
# Table of Contents

## TABLE OF CONTENTS

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>IV</td>
<td>TABLE OF CONTENTS</td>
<td></td>
</tr>
<tr>
<td>VI</td>
<td>TABLE OF TABLES</td>
<td></td>
</tr>
<tr>
<td>VIII</td>
<td>TABLE OF FIGURES</td>
<td></td>
</tr>
</tbody>
</table>

1. Introduction ...........................................................................................................1

2. Materials Characterization and Comparison .........................................................4
   2.1. Materials Characterization ...............................................................................4
       2.1.1. Soundness Tests ......................................................................................4
       2.1.2. Abrasion Tests .......................................................................................5
       2.1.3. The CTE Test .........................................................................................6
   2.2. Material Specification Comparison ....................................................................7
   2.3. Material Comparison .......................................................................................9
   2.4. Summary .........................................................................................................10

3. Impact of Modern Design and Construction Methods ............................................12
   3.1. Background .....................................................................................................12
   3.2. Design and Construction Considerations .........................................................13
       3.2.1. Two-Lift Paving .....................................................................................13
       3.2.2. Optimized Gradation .............................................................................13
       3.2.3. CTE and Joint Spacing ..........................................................................15
   3.3. Parametric Study in Pavement ME / MEPDG ..................................................17
   3.4. Summary .........................................................................................................21

4. Pavement Texture and Friction ..............................................................................23
   4.1. Pavement Texture and Measurement Techniques .............................................23
       4.1.1. Texture Measurement ............................................................................23
   4.2. Pavement Friction ............................................................................................25
       4.2.1. Friction Measurement .............................................................................25
       4.2.2. Locked Wheel Test: Smooth vs. Ribbed Tire ...........................................27
       4.2.3. International Friction Index (IFI) ............................................................28

iv
4.3. Texture Application in the Laboratory ........................................29
4.4. Accelerated Pavement Wear Options ........................................29

5. Diamond Grinding Techniques and Specifications .........................32
  5.1. Background ...........................................................................32
  5.2. State of the Art ......................................................................33
  5.3. Grinding New Pavements .......................................................38
  5.4. Recent Research ....................................................................39
    5.4.1. Conventional Grinding Studies ..........................................39
    5.4.2. Alternatives to Conventional Grinding ...............................41
  5.5. Research Needs ......................................................................42
  5.6. Summary ..............................................................................43

6. Summary and Recommendations ....................................................45
  6.1. Summary ..............................................................................45
  6.2. Recommended Specification and Test Method Improvements .......46
  6.3. Research Priorities ..................................................................49

ACKNOWLEDGEMENTS 52
REFERENCES 53
Table of Tables

Table 2.1: A summary of aggregate and concrete characterization tests ..........4

Table 2.2: Material testing standards for SASHTO states and Texas ............8

Table 2.3: States which use a polishing susceptibility test for asphalt coarse aggregates and the standards for each test. Information was taken from each states construction specifications. ........................................8

Table 2.4: Sources listed are in-state sources from Alabama’s approved aggregate list that are also listed on the approved aggregate lists of those states. ..................................................................................9

Table 2.5: Hardness results for selected aggregates. The test kit used had an interval of 0.5. Numbers listed are 0.5 less than the number of the material that scratched them. ........................................................................10

Table 3.1. Typical Values for Coefficient of Thermal Expansion of Concrete, per AASHTO T 336 procedure, adapted from (FHWA 2011). ...........................................................................................................16

Table 3.2. Design details for Phase I Pavement ME analyses. ..................18

Table 3.3. Phase I Pavement ME analysis results for a failure threshold of 10% slab cracking. .................................................................................................................................19

Table 3.4. Phase I Pavement ME analysis results for a failure threshold of 15% slab cracking. .................................................................................................................................19

Table 3.5. Results of Phase II Pavement ME analysis of Design 3 (12 in. thick slab with 15 ft. joint spacing). The governing failure mode for each CTE is in bold. .............................................................................................................20

Table 4.1: A summary of spot texture measurement methods. Modified from (Hall, et al. 2009). .................................................................................................................................24

Table 4.2: A summary of pavement profile measurement devices. Modified from (Jones and Omundson 2004, Koski 1994). .........................................................................................24

Table 4.3: A summary of friction measurement methods that can be used for lab and field measurement. Modified from (Henry 2000). .........................................................26

Table 4.4: A summary of friction measurement methods the can be used for field measurement. Modified from (Henry 2000). .........................................................26
Table 5.1: ACPA and IGGA recommended diamond grinding dimensions for hard and soft aggregate. Adapted from: (IGGA 2014a).......................34

Table 5.2: Diamond grinding specifications for each SASHTO state and Texas (NC, TN, and VA are excluded because their dimensions were not published in their Construction Specifications. Bold land widths calculated from number of grooves/blades per foot, assuming a 0.125 in. blade width. Groove/blade counts in italics are calculated based on the land area requirements, assuming a 0.125 in. blade width. ...............36

Table 5.3: Average service life of diamond ground rigid pavements by aggregate type, carbonate vs. siliceous, for SASHO member states and Texas. Bold states border Alabama.................................................................37

Table 5.4: Average service life of diamond ground rigid pavements by aggregate type, carbonate vs. siliceous, for SASHTO member states and Texas when limited by loss of friction after grinding. Bold states border Alabama.................................................................................................37
# Table of Figures

**Figure 1.1:** Use of carbonate coarse aggregate in mainline rigid pavements in SASHTO member states and Texas. ........................................3

**Figure 3.1.** Recommended optimized aggregate gradation range for slipform paving. The percentage retained on each sieve should fall between the upper and lower limits indicated by the dashed lines. From (Cook, et al. 2013).................................................................15

**Figure 3.2.** Phase I Pavement ME analysis results for a failure threshold of 10% slab cracking..........................................................19

**Figure 3.3.** Phase I Pavement ME analysis results for a failure threshold of 15% slab cracking..........................................................20

**Figure 3.4.** Phase II Pavement ME analysis results for Design 3. Slab cracking, joint faulting, and roughness were considered as failure modes ......................................................................................21

**Figure 4.1:** Visual of micro-texture and macro-texture. From (Hall, et al. 2009).....................................................................................23

**Figure 4.2:** Visual of mechanisms that create friction force in pavements. From (Hall, et al. 2009) .....................................................................................25

**Figure 4.3:** Laboratory grinding device at UT-Austin (Fentress n.d.)..............29

**Figure 4.4:** Schematic of the Model Mobile Load Simulator (Rado 2009) .........30

**Figure 5.1a (left):** Diamond cutting blades mounted on the cutting head. (Correa and Wong 2001) ..................................................................................32

**Figure 5.1b (right):** An example of diamond ground pavement including the fins. (Correa and Wong 2001) ........................................................................32

**Figure 5.2:** Profile of corduroy texture produced by diamond grinding. (Correa and Wong 2001) ........................................................................33

**Figure 5.3:** Map showing how state grinding specifications compare to the recommended dimensions. Bold outline area includes SASHTO states and Texas, the main states investigated in this study........................35

**Figure 5.4:** Surface finishing requirements of new construction rigid pavements in the United States by state..................................................39
1. Introduction

Alabama’s quarries produce mainly carbonate aggregates such as limestone and dolomite. Constructing concrete pavements with harder siliceous aggregates imported from other states increases the construction cost of concrete pavements in Alabama. In the 1980’s the Alabama Department of Transportation (ALDOT) performed diamond grinding on a number of existing concrete pavements in an effort to restore smoothness and improve ride quality. Several years after grinding, pavements with limestone coarse aggregate that had been diamond ground experienced polishing, and friction numbers fell below acceptable limits. Concrete pavements constructed with harder aggregates (e.g. river gravel or crushed granite) maintained higher friction values after grinding. As a result, ALDOT discontinued using limestone aggregate in mainline pavement.

The cement and concrete industry would like to explore reintroducing carbonate coarse aggregate as a material option when constructing mainline rigid pavements. Despite past issues with post-grinding friction loss, there are potential benefits of using carbonate coarse aggregates. Initial construction costs can be reduced if in-state aggregate sources are permitted. Not only are materials transport costs reduced, but many carbonate aggregates have lower coefficient of thermal expansion (CTE) than siliceous aggregates. The use of lower CTE materials results in pavements that are less likely to crack as a result of large temperature variations.

At a preliminary meeting between ALDOT, concrete industry representatives, and researchers at The University of Alabama (UA), two major questions were asked. First, is it possible to rehabilitate rigid pavements containing carbonate coarse aggregates and avoid a similar loss in friction number? Second, is it possible to improve the initial performance of rigid pavement containing carbonate coarse aggregate through means of joint spacing and CTE consideration?

This report addresses those questions in the following ways:

- A review and comparison of Alabama construction specifications and the construction specifications for Southeastern Association of State Highway and Transportation Officials (SASHTO) members and Texas. Areas of interest were 1) use of carbonate aggregates in mainline rigid pavement 2) justification for any restrictions of their use and 3) surface texture and friction requirements.
- A recommended list of laboratory tests to reliably characterize coarse aggregates and to predict their performance in concrete pavements.
- An investigation into friction measurement techniques and recommended field and laboratory tests to assess pavement friction in wet weather conditions.
- A comparison of approved carbonate aggregate sources in Alabama, SASHTO member states, and Texas.
A review of state-of-the-art pavement design, construction, and rehabilitation techniques that may prevent or minimize loss of surface texture and friction in rigid pavements. The review of design and construction considerations focused on the influence of joint spacing, CTE, and initial pavement texturing. The discussion of rehabilitation techniques focused mostly on diamond blade configuration and its effect on friction retention for pavements with different aggregate mineralogy.

A parametric study using AASHTO Pavement ME Design to evaluate the influence of joint spacing, pavement thickness, and CTE on predicted pavement performance in terms of slab cracking, joint faulting, and loss of smoothness.

To aid in this investigation, a short online survey was sent to state departments of transportation to provide additional information about construction practices and clarification of specifications. The survey was sent to a total of fifteen agencies representing SASHTO members and Texas. Of those fifteen agencies, eleven provided responses to the online survey (73% response rate), and ALDOT engineers have added substantial discussions regarding their specifications and practices with the authors. The survey questions and responses are discussed in appropriate sections of this report.

From the survey, nine agencies (AR, FL, GA, KY, NC, MS, TN, WV, and TX) confirmed that they allow the use of carbonate coarse aggregate in mainline rigid pavement construction, and two (SC and VA) do not allow their use (Figure 1.1). Further information about use of carbonate aggregates in several states is listed below:

- The nine states that allow carbonate coarse aggregate in rigid pavement construction all confirmed carbonate aggregate has been used in projects.
- Tennessee has updated specifications; when using a polishing aggregate it is required to be mixed with a polishing resisting aggregate.
- Further information regarding practices in SC and VA was not available.
- Though Alabama did not respond to the survey, its personnel confirmed that carbonate coarse aggregate is not permitted in mainline pavement, but it can be used in concrete shoulders.
Figure 1.1: Use of carbonate coarse aggregate in mainline rigid pavements in SASHTO member states and Texas.

This report is divided into six chapters, including this introduction chapter. Chapter 2 introduces aggregate characterization for concrete pavements, a comparison of material specifications of states of interest, and a short comparison of material data. Chapter 3 reviews state-of-the-art construction practices that aim to increase the performance life of rigid pavements, as well as results from a parametric study in MEPDG. Chapter 4 characterizes friction for rigid pavements. Chapter 5 is a review of state-of-the art diamond grinding practices and a comparison of diamond grinding specifications for all fifty states. Finally, Chapter 6 presents conclusions, recommended improvements to specifications, and prioritized research needs.
2. Materials Characterization and Comparison

2.1. Materials Characterization

Aggregate constitutes the majority of pavement by mass; therefore, it is important to be able to characterize the performance of aggregates to characterize the performance of pavement. Pavement texture and friction is initially influenced by fine aggregates in the top layer of mortar (Dahir and Henry 1978). Once the mortar layer has been removed, either through pavement wear or diamond grinding, coarse aggregate is the main influence of texture and friction (Dahir and Henry 1978). Pavement joint distresses and joint movement can be greatly influenced by expansion and contraction due to temperature change. In pavement, this behavior is heavily dominated by coefficient of thermal expansion of aggregates. Therefore, it is important for state agencies to test both (1) the resistance of aggregates to wearing and polishing and (2) the coefficient of thermal expansion. Table 2.1 gives a brief summary of popular tests to characterize several properties of aggregates.

Table 2.1: A summary of aggregate and concrete characterization tests

<table>
<thead>
<tr>
<th>Name</th>
<th>Standard</th>
<th>Property Tested</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sulfate Soundness</td>
<td>ASTM C 88, AASHTO T 104</td>
<td>Resistance to disintegration by weathering</td>
</tr>
<tr>
<td>LA Abrasion</td>
<td>ASTM C 131/131M, AASHTO T 96</td>
<td>Aggregate abrasion by impact</td>
</tr>
<tr>
<td>Micro-Deval</td>
<td>ASTM D 6928, AASHTO T 327</td>
<td>Coarse aggregate wet abrasion</td>
</tr>
<tr>
<td>Unconfined Freeze-Thaw</td>
<td>AASHTO T 103</td>
<td>Aggregate resistance disintegration by freeze-thaw</td>
</tr>
<tr>
<td>Coefficient of Thermal Expansion</td>
<td>AASHTO T 336</td>
<td>Concrete change in length per degree of temperature change</td>
</tr>
</tbody>
</table>

2.1.1. Soundness Tests

The sulfate soundness test (ASTM C 88 and AASHTO T 104) involves immersing an aggregate sample, separated by sieve size, of known mass in sodium or magnesium sulfate solution for five to ten cycles to simulate freezing and thawing resistance. After each cycle of immersion, the sample is dried at high temperatures. The pressures generated by the crystallization of sulfate salts in the pore spaces of the aggregates during the drying cycle serve as a proxy for ice crystallization pressures that occur during freezing cycles. Once the test is completed the sample is washed, dried, and each sieve size is hand sieved to discard undersized particles. Then, mass loss is calculated. Some research indicates that sulfate soundness test results correlate well with pavement performance; other research indicates the opposite (Wu, et
It is generally accepted the test presents poor repeatability, though magnesium sulfate has shown to be slightly more repeatable than sodium sulfate due to lower variability of solubility of magnesium sulfate (Wu, et al. 1998).

An alternative to sulfate soundness are unconfined freezing and thawing tests (AASHTO T 103 and CSA A23.2-24A), which tests for aggregate degradation when exposed to alternate cycles of freezing and thawing in water. The sulfate soundness test was introduced when refrigeration technology couldn’t efficiently freeze water; now that freezing water is easily achievable, it is perhaps appropriate to consider a switch to an unconfined freeze thaw test. The Ontario Ministry of Transportation has found the CSA A23.2-24A test to be one of the best predictors of pavement performance in freezing and thawing conditions, with a 6% maximum loss limit correlating well to acceptable field performance (Senior and Rogers 1991). The Virginia Department of Transportation (VDOT) has adopted AASHTO T 103 with a 5% loss limit (VDOT 2007).

However, the AASHTO T 103 standard has been criticized for not specifying the temperature range and thawing rate to be used in the test, allowing for multiple interpretations. This can result in subjecting the sample to unrealistically severe temperature changes and thawing rates that lead to failure of aggregate with established histories of good field performance (Fowler, et al. 2006). The CSA A23.2-24A test specifies the temperature range and thawing rate to be used; this should at least result in more consistent results, if not a better correlation to field performance. Given that Alabama’s climate is less severe than Virginia or Canada with respect to freezing and thawing, it is unclear what percentage loss limit in unconfined freeze thaw testing would be appropriate for ALDOT if it were to implement one of these test methods.

2.1.2. Abrasion Tests

The LA Abrasion test (ASTM C 131 and AASHTO T 96) is commonly mistaken to characterize abrasion or polishing susceptibility, but it is actually an impact resistance test. An aggregate sample is placed in a large drum along with an abrasion charge consisting of large steel spheres roughly 1.5-inches in diameter weighing approximately 400 grams apiece. The drum is rotated, and interior shelves lift the dry aggregate sample and the spheres and drop them. That action causes degradation of the aggregate pieces. However, the aggregate is more likely to lose mass from impact experienced in the fall or contact with the steel spheres, rather than as a result of true abrasion.

While impact resistance of aggregates is an important characteristic to determine if they will be susceptible to breakage during mixing and placement, the polishing susceptibility of coarse aggregate is crucial to estimating the friction performance of rigid pavement, especially once the top mortar layer has worn away and the coarse aggregate is exposed to traffic. The Micro-Deval (ASTM D 6928 and AASHTO T 327) test better characterizes the abrasion wear of coarse and fine aggregates. Micro-Deval rotates aggregates in a drum similar to LA Abrasion,
though the size of the drum and abrasion charges are smaller, and there are no shelves that pick up aggregate and drop it. The Micro-Deval also includes water, which creates a wet abrasion test.

Multiple studies have been done to verify the usefulness of the Micro-Deval test and to set a value of percent material loss below which the material can be classified as non-polishing for pavement applications. Research by the Ontario Ministry of Transportation suggests for rigid pavements that the combination of unconfined freeze-thaw and Micro-Deval gives the best indication of aggregate performance (Senior and Rogers 1991). After testing aggregates with known performance in rigid pavements in the Micro-Deval, the Ministry set an upper limit of twenty percent (20%) loss or less for aggregates to be classified as non-polishing. The International Center for Aggregate Research (ICAR) examined the success rate of combinations of aggregate tests and their ability to predict performance in concrete pavements. They concluded, similarly to the Ontario Ministry of Transportation, that the unconfined freeze-thaw and Micro-Deval tests give the best prediction of field performance, with an eighty-eight percent (88%) success rate. The combination of the magnesium sulfate soundness and Micro-Deval tests was only slightly less effective, with an eight-five percent (85%) success (Fowler, et al. 2006).

2.1.3. The CTE Test

Coefficient of thermal expansion (CTE) is a measure of change in length per degree of temperature change. Concrete expands and contracts with changes in temperature, which causes degradation. Knowledge of this rate of length change can be used to design a pavement with a longer service life because lower CTE translates to less degradation. Length change of a saturated cylindrical concrete specimen in a metal frame is measured as it is submerged in water and subjected to a temperature change of 10 to 50°C. Previously, CTE was measured using AASHTO TP 60 but the test showed low repeatability, both between laboratories and between equipment (Crawford, et al. 2010). This drawback has prompted several changes to the test method that reduced variance in measured values of CTE. As a result of an inter-laboratory conducted by the Federal Highway Administration (FHWA) Mobile Concrete Laboratory program, a new test standard, AASHTO T 336, was created with the following changes (Tanesi, et al. 2012):

- Use of a calibration specimen with a known CTE is now required to determine the correction factor for the equipment.
- A verifications specimen, other than the calibration specimen, with a known CTE should be used to verify that equipment is working properly.
- Linear variable differential transformers (LVDT) should be positioned so that their cores are at the midpoint and should be calibrated every six months.
- Water bath temperature should be verified every time a verification specimen is tested.
- Temperature sensors in the water bath were reduced from four to one.
• Water level in water bath during testing shall be the same as the water level during equipment calibration.
• End conditions of concrete specimens shall meet tolerances specified in AASHTO T 22, “Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimen”.
• Two specimens from each mixture should be tested.

The newer AASHTO T 336 test reports lower CTE values than the AASHTO TP 60 method, which is no longer in use. Except for Texas, which uses its own CTE test (Tex-428-A) all states have, at least in principle, adopted AASHTO T 336 (Tanesi, et al. 2012). However, that does not necessarily mean they are actually testing their concrete mixes in practice.

The CTE of coarse aggregate has a large effect on the CTE of concrete and therefore many methods of determining CTE of coarse aggregate have been developed. Several methods have been developed though the most recent research by Mukhopadhyay and Zollinger (2009) tests for CTE of coarse and fine aggregate as received in bulk. The test uses a dilatometer to test water volume changes; saturated aggregate samples are put into the dilatometer and subjected to cycles of temperature changes (10-50°C). With known CTE of the water and dilatometer material, the change in volume of the water is calculated and from there the CTE of the aggregate. Mukhopadhyay and Zollinger found the dilatometer method to obtain results very similar to other methods (strain gage) and those obtained from literature.

2.2. Material Specification Comparison

As mentioned above, rigid pavement aggregates are most commonly characterized by LA abrasion and sulfate soundness test results. A review of specifications for concrete coarse aggregates of SASHTO member states and Texas is presented in Table 2.2. The table shows that Alabama has standards for aggregates that are similar to other states’ standards. States with similar specifications to ALDOT have carbonate aggregates that meet those specifications and use those aggregates in mainline rigid pavement construction. Therefore, many Alabama carbonate aggregates would qualify for use in other states.

Neither the LA abrasion nor sulfate soundness tests are a good indicator of aggregate polishing susceptibility. To assess polishing susceptibility, alternative tests like the Micro-Deval or British Pendulum Test should be used. Of the states represented in Table 2.2, few required either test in their specifications, and those states only required these tests of aggregates to be used in flexible pavement applications. Table 2.3 gives a summary of southeastern states that use polishing susceptibility tests and the required standards. It should be noted that both North and South Carolina use the Micro-Deval test, and their limit on material loss is similar to limits set for asphalt pavement aggregates presented in the literature.
Table 2.2: Material testing standards for SASHTO states and Texas.

<table>
<thead>
<tr>
<th>State</th>
<th>Limestone in Mainline Pavement</th>
<th>Abrasion: Maximum Percent</th>
<th>Soundness: Maximum Percent Loss</th>
<th>Additional Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alabama</td>
<td>No</td>
<td>50</td>
<td>Sodium: 10</td>
<td></td>
</tr>
<tr>
<td>Arkansas</td>
<td>Yes</td>
<td>40</td>
<td>Sodium: 12</td>
<td></td>
</tr>
<tr>
<td>Florida</td>
<td>Yes</td>
<td>45</td>
<td>Sodium: 12</td>
<td></td>
</tr>
<tr>
<td>Georgia</td>
<td>Yes</td>
<td>40</td>
<td>Magnesium: 15</td>
<td></td>
</tr>
<tr>
<td>Kentucky</td>
<td>Yes</td>
<td>40</td>
<td>Sodium: 12</td>
<td>Freeze Thaw (KM-64-626/ASTM C 666): 0.06% Max. Expansion, 80% Min. Durability Factor</td>
</tr>
<tr>
<td>Louisiana</td>
<td>Yes</td>
<td>40</td>
<td>Magnesium: 15</td>
<td>Carbonate Aggregates must test innocuous by alkali reactivity (ASTM C 289) and X-Ray diffraction analysis</td>
</tr>
<tr>
<td>Mississippi</td>
<td>Yes</td>
<td>40</td>
<td>Magnesium: 16</td>
<td>Mortar Bar Expansion (ASTM C 227): 0.5% Max. at 6 Months; 1.0% Max at 1 year</td>
</tr>
<tr>
<td>North Carolina</td>
<td>Yes</td>
<td>55</td>
<td>Sodium: 15</td>
<td></td>
</tr>
<tr>
<td>South Carolina</td>
<td>No</td>
<td>60</td>
<td>Sodium: 25</td>
<td></td>
</tr>
<tr>
<td>Tennessee</td>
<td>Yes*</td>
<td>40</td>
<td>Sodium: 9</td>
<td></td>
</tr>
<tr>
<td>Virginia</td>
<td>Yes</td>
<td>40-50**</td>
<td>Magnesium: 15</td>
<td>Freeze Thaw (AASHTO T 103): 5% Max. Loss</td>
</tr>
<tr>
<td>West Virginia</td>
<td>Yes</td>
<td>40</td>
<td>Sodium: 15</td>
<td></td>
</tr>
<tr>
<td>Texas</td>
<td>Yes</td>
<td>35</td>
<td>Sodium or Magnesium: 15</td>
<td>Micro-Deval used by engineer to indicate further investigation</td>
</tr>
</tbody>
</table>

* Tennessee has changed their specification as of 2015 to allow blends of polishing and polishing-resistant material in rigid pavement mix designs. ** Range given for type of aggregate.

Table 2.3: States which use a polishing susceptibility test for asphalt coarse aggregates and the standards for each test. Information was taken from each states construction specifications.

<table>
<thead>
<tr>
<th>Test</th>
<th>Alabama</th>
<th>North Carolina</th>
<th>South Carolina</th>
<th>Tennessee</th>
<th>Texas</th>
</tr>
</thead>
<tbody>
<tr>
<td>BPN</td>
<td>Value determines the % of carbonate aggregate used in SMA/Superpave</td>
<td></td>
<td></td>
<td>By aggregate type: Type I: &gt;30 Type II: &gt;30 Type III: &gt;25 Type IV: &gt;22</td>
<td>Used as indicator for the need for further investigation</td>
</tr>
<tr>
<td>Micro-Deval</td>
<td>Max. Loss: 18%</td>
<td>Max. Loss: 15%</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
2.3. Material Comparison

The approved aggregate lists for each SASHTO member state, and Texas, were investigated to determine which states were using Alabama carbonate sources in their rigid pavement. Results from that search of twelve states (not including Alabama or Puerto Rico) found four states have aggregates from Alabama on their approved aggregate list; those states include Florida, Louisiana, Mississippi, and Tennessee. The shared limestone aggregate sources are listed in the Table 2.4.

Table 2.4: Sources listed are in-state sources from Alabama’s approved aggregate list that are also listed on the approved aggregate lists of those states.

<table>
<thead>
<tr>
<th>Producer</th>
<th>Location</th>
<th>FL</th>
<th>LA</th>
<th>MS</th>
<th>TN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hoover, INC.</td>
<td>Allsboro</td>
<td></td>
<td></td>
<td></td>
<td>√</td>
</tr>
<tr>
<td>Martin Marietta Materials</td>
<td>Vance</td>
<td></td>
<td></td>
<td></td>
<td>√</td>
</tr>
<tr>
<td>Rogers Group, INC.</td>
<td>Lacey’s Spring</td>
<td>√</td>
<td></td>
<td></td>
<td>√</td>
</tr>
<tr>
<td>Vulcan Materials Company</td>
<td>Bessemer</td>
<td>√</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vulcan Materials Company</td>
<td>Calera</td>
<td></td>
<td>√</td>
<td>√</td>
<td>√</td>
</tr>
<tr>
<td>Vulcan Materials Company</td>
<td>Cherokee</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td></td>
</tr>
<tr>
<td>Vulcan Materials Company</td>
<td>Dolcito Quarry, Tarrant</td>
<td>√</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vulcan Materials Company</td>
<td>Russellville</td>
<td></td>
<td></td>
<td></td>
<td>√</td>
</tr>
<tr>
<td>Vulcan Materials Company</td>
<td>Trinity</td>
<td>√</td>
<td></td>
<td>√</td>
<td></td>
</tr>
<tr>
<td>Vulcan Materials Company</td>
<td>Tuscumbia</td>
<td>√</td>
<td>√</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vulcan Materials Company</td>
<td>Pride Quarry, Tuscumbia</td>
<td>√</td>
<td>√</td>
<td>√</td>
<td>√</td>
</tr>
</tbody>
</table>

It was determined through the previously mentioned survey that two states, Florida and Mississippi (two of the four states that border Alabama), confirmed to have used limestone from Alabama in their mainline rigid pavement. Florida specifically reported use of limestone from quarries near Calera in sections of I-10 west of Tallahassee in the 1980’s. It was noted in follow up discussions that material from these quarries is preferred because is more resistant to polishing than other limestones available for use in Florida (Bergin 2014). Several states (Texas, Arkansas, and West Virginia) reported in their survey responses that no Alabama carbonate sources currently are on their approved aggregate list, but they have no reason to prohibit their use, assuming all their material requirements were met. None of these states are neighboring states, and their distance from Alabama is the likely reason no Alabama sources are on their approved materials lists.

When several states approve the same carbonate aggregate source, it is difficult to find and compare the results of friction tests those states performed on the aggregate. First, states don’t always perform the same tests: most states gave both LA abrasion and sulfate soundness data; some states only reported abrasion data; and a few states did not give any data, even upon request. The second issue was that states use different versions of the same test; for example, the
LA abrasion test allows four different gradations to be tested, and data between the gradations cannot be compared. In the LA abrasion data made available, few states specified which gradation was used, so comparison of data cannot be done. The sulfate soundness test has similar issues. One issue is that magnesium sulfate data cannot be compared with sodium sulfate data.

Mineral hardness can also be used to gauge aggregate wear in service conditions. Mohs hardness is a test where an aggregate is scratched with minerals of known hardness on a scale of 1 to 10, starting with the least hard mineral. The test is stopped when a mineral scratches the aggregate, and the aggregate is said to have hardness less than the mineral it was scratched with. The University of Alabama research team used the Mohs scratch test in an attempt to see the difference in hardness of aggregate sources. Aggregate samples were taken from sources readily available in the UA concrete testing lab and were from several in-state and out-of-state sources representing both carbonate and siliceous aggregate. Their scratch hardness results are presented in Table 2.5. On the hardness scale; chert is typically an eight, quartz is seven, granites and gravels range from five to seven, and most carbonate stones are three or four.

Most of the tested sources performed differently than expected; limestone sources performed in the expected range of carbonate aggregates, and the siliceous sources performed in the expected range of siliceous aggregate. The mixed siliceous gravel from New Mexico, however, was of similar hardness as three of the four carbonate aggregates tested.

**Table 2.5:** Hardness results for selected aggregates. The test kit used had an interval of 0.5. Numbers listed are 0.5 less than the number of the material that scratched them.

<table>
<thead>
<tr>
<th>Source</th>
<th>Carbonates</th>
<th>Siliceous / Gravels</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Helena</td>
<td>Calera</td>
</tr>
<tr>
<td></td>
<td>AL</td>
<td>AL</td>
</tr>
<tr>
<td>Mineralogy</td>
<td>Dolomite</td>
<td>Dolomitic Limestone</td>
</tr>
<tr>
<td>Average Mohs Hardness Number</td>
<td>4.5</td>
<td>3.5</td>
</tr>
</tbody>
</table>

### 2.4. Summary

An investigation into material characterization and comparison yielded the following conclusions:
- Some siliceous aggregates may be as soft as or softer than some carbonate aggregates. A test method or combination of test methods is needed to characterize aggregate hardness and polishing resistance, and it should be applied to all aggregates, not just carbonates.
- If budgets permit:
States should include Micro-Deval testing in their aggregate approval process and investigate adopting similar limits suggested by the Ontario Ministry of Transportation (maximum of 20 percent loss).

States should investigate switching from the sulfate soundness test to an unconfined freeze thaw test to characterize freezing and thawing resistance, after first determining acceptable loss limits in the new test.

- CTE is an important aggregate characteristic because concrete pavement made with aggregates of lower CTE should experience less degradation than pavements of the same joint spacing made with higher CTE aggregates. In general, most carbonate aggregates have a lower CTE than most siliceous aggregates.
- ALDOT specifications for maximum allowable LA Abrasion loss and for maximum percent loss in sulfate soundness testing are similar to other SASHTO states.
- The following neighboring states confirmed that they have used Alabama carbonate stone in their mainline pavements: Florida and Mississippi.
3. Impact of Modern Design and Construction Methods

3.1. Background

One of ALDOT’s major concerns with using carbonate aggregates is that pavements may be more prone to polishing if they undergo diamond grinding rehabilitation. Diamond grinding and polishing concerns are discussed in detail in Chapter 5 of this report, but it should be noted that grinding is often triggered by a loss of smoothness resulting from transverse cracking and joint deterioration. However, if a new pavement can be constructed to avoid such deterioration, the need to restore smoothness by diamond grinding is reduced. This chapter briefly investigates how to design and construct long life concrete pavements, and how the use of carbonate coarse aggregates may actually have positive impacts on pavement performance.

In the survey of SASHTO member states and Texas conducted by The University of Alabama, eight of ten agencies responded that changes in “materials specifications, construction practices, or rehabilitation methods” have yielded improvements in rigid pavement service life. Several DOTs elaborated upon their response:

- Texas noted that introducing limits on coefficient of thermal expansion (CTE) has eliminated spalling issues with continuously reinforced concrete pavements (CRCP). The current construction specifications in Texas limit CTE to $6.0 \times 10^{-6}$ for CRCP (based on TxDOT's Tex-428-A procedure).
- South Carolina has abandoned both CRCP and jointed pavements designs with 18 to 25 ft. joint spacings. They now exclusively use doweled jointed plain concrete pavement (JPCP) and reported that a joint spacing of 15 ft. and increased dowel sizes provided the best performance. They did not state whether they had investigated joint spacings less than 15 ft.
- Florida notes that it is following FHWA recommendations for mix design, placement, finishing, and joints, and that this is leading to longer service life.
- Virginia is using wider lanes and more steel (neither value was quantified) in CRCP pavements and expects extended service life as a result.
- Kentucky reported they have added a freeze-thaw durability factor requirement (80% minimum) to mitigate future problems with aggregates susceptible to poor freeze-thaw performance. This is in addition to a prior requirement of 0.06% maximum expansion in freeze-thaw testing. The impact of this specification change has yet to be proven. (Freeze-thaw testing in Kentucky to determine expansion and durability factor of concrete follows Kentucky Method 64-626, which is based on ASTM C666 procedure B, Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing.)
- Tennessee reported they are considering further use of two-lift pavements.
In the same survey, Texas responded that they have a limit on CTE for concrete used in pavement applications. TxDOT’s limits (noted above) were put in place for CRCP only to avoid spalling issues that had affected some CRCPs containing high CTE aggregates. TxDOT does not limit the CTE for jointed pavements. The Texas limit of $6.0 \times 10^{-6}/\sqrt{ft}$ has also been reported as $5.5 \times 10^{-6}/\sqrt{ft}$ in the literature (Siddiqui and Fowler 2014). Follow-up discussions with TxDOT personnel provided confirmation that their limit has been reduced to $5.5 \times 10^{-6}/\sqrt{ft}$. It should be noted that TxDOT’s test method (Tex-428-A) results in somewhat higher values than the AASHTO T336 method used in other states (Siddiqui and Fowler 2014).

### 3.2. Design and Construction Considerations

A 2012 study sponsored by the Florida DOT included a review of current long-life concrete pavement design practices in use in several other states and in Europe, and performance history of existing pavements that have performed well over a long period of time. Illinois and Texas have 30+ year design standards for CRCP; unlike Texas, Illinois has not adopted a CTE limit. Washington and Minnesota have doweled JPCP design standards for 50-60 year service life that use 15 ft. joint spacings and combined “8-18” aggregate gradation (explained further in Section 3.2.2) in the concrete mix design. In some European countries, two-lift paving is widely used. (Tia, et al. 2012)

#### 3.2.1. Two-Lift Paving

Two-lift paving has yet to come to wide use in the US, but several demonstration projects have been conducted in recent years (Hu, et al. 2014). Following a 2006 tour of European pavements, FHWA began to advocate for two-lift paving in the US (Hall, et al. 2007). Potential benefits include being able to use polishing-susceptible aggregates in the lower layer, where they will have no chance of being exposed to traffic, and a greater use of recycled materials and leaner concrete mixes in the lower layer. The best quality materials can be placed in the top layer, which typically constitutes approximately one-third of the slab thickness (Hall, et al. 2007). There remain challenges to the wider use of this technique, including restrictive construction specifications (ALDOT requires single lift operations for slipform paving), contractor inexperience, and a perception that construction costs will be greater than single-lift paving. On some demonstration projects, however, two-lift paving costs were considerably less than traditional single-lift construction due to the lower cost of the materials in the lower lift (Hu, et al. 2014).

#### 3.2.2. Optimized Gradation

Combined, or optimized, aggregate gradations seek to maximize aggregate packing in a concrete mix by considering the overall gradation produced by the coarse and fine aggregate fractions together, rather than considering them separately. This can be used to reduce the paste
fraction of the concrete, yielding construction cost savings, reduced drying shrinkage of the concrete, and a reduced carbon footprint of the pavement (Cook, et al. 2013, Taylor, et al. 2007). Given that mixtures with crushed stone typically require a larger paste fraction to achieve a certain degree of workability than those containing river gravels (Siddiqui, et al. 2014), the benefits of optimized aggregate gradation are particularly relevant when considering reintroducing carbonate coarse aggregates for concrete paving in Alabama, because carbonates are almost exclusively crushed stone products.

Several generalized techniques have been developed to optimize aggregate packing, including the 0.45 power chart, the coarseness factor (CF), workability factor (WF), and “8-18” grading, in which percentage retained on most sieve sizes should fall between 8% and 18%. Shilstone (1990) developed and advocated use of CF and WF. ALDOT now mandates that contractors submit a combined gradation for paving mixes, along with a 0.45 power chart plot, CF, and WF data, but has not specified a particular combined gradation or mixture design (Alabama Department of Transportation 2012).

However, these tools may not yield concrete with the optimal workability for specific applications, such as slipform paving (Taylor, et al. 2007). Researchers at Oklahoma State University have worked to develop optimized gradation guidelines specific to a variety of applications, including slipform paving (Cook, et al. 2013). A recommended combined gradation specification for slipform paving is shown in Figure 3.1. This specification was developed along with a workability performance test called the “Box Test.” In a demonstration project funded in part by the FHWA Highways for Life program, 2.2 miles of CRCP on FM 1938 Fort Worth, TX were placed with a cementitious materials content of 4.75 sacks/yd³, or 94 lb./yd³ less than a non-optimized gradation concrete (Cook, et al. 2013). An estimated 10% savings on the material costs and a 25% reduction of the CO₂ emissions associated with the project were attributed to the application of optimized gradation principles (Cook, et al. 2013, Applying the Box Test in the Field). Drying shrinkage of similar optimized-graded concretes was considerably reduced compared to standard mixtures (Cook, et al. 2013); this would be expected to lead to a corresponding reduction in transverse slab cracking when applied in the field.
Figure 3.1. Recommended optimized aggregate gradation range for slipform paving. The percentage retained on each sieve should fall between the upper and lower limits indicated by the dashed lines. From (Cook, et al. 2013).

3.2.3. CTE and Joint Spacing

Two additional design and construction considerations of great significance are the CTE and joint spacing. The CTE of the concrete is strongly influenced by the CTE of the coarse aggregate. Table 3.1 shows typical CTEs (average and standard deviation) for concrete containing different aggregate types, with values based on the AASHTO T 336 procedure, as provided by LTPP Standard Data Release 25.0. The T 336 procedure corrects an error in the AASHTO TP 60 procedure that uses an incorrect value for the coefficient of thermal expansion of the reference bar, leading to an overestimation of the CTE. Corresponding TP 60 values for the materials in Table 3.1 would be approximately $0.7 \times 10^{-6} \, 1/\sqrt[3]{F}$ higher. Carbonate aggregates tend to have a lower CTE than siliceous aggregates, although dolomitic aggregates tend to have CTE values that are comparable to many granitic aggregates (Kim 2012, FHWA 2011, Taylor, et al. 2007, Siddiqui and Fowler 2014). Pavements containing carbonate coarse aggregate would therefore be expected to experience lower stresses as a result of thermal gradients and temperature changes than those with siliceous aggregates.

Joint spacing for JPCP designs, or more specifically, the spacing between transverse contraction joints, is particularly critical to mitigate and control cracking caused by drying shrinkage and thermal stresses. Currently, FHWA and the National Concrete Pavement Technology Center (CP Tech Center) recommend a maximum 15 ft. joint spacing for JPCP construction (Taylor, et al. 2007, FHWA 1990). The American Concrete Pavement Association (ACPA) recommends a maximum joint spacing of 24 times the slab depth for pavements with a
granular subbase, 21 times the slab depth for pavements with a stabilized base, and no more than 15 ft. regardless of the base/subbase conditions (ACPA 1991).

Table 3.1. Typical Values for Coefficient of Thermal Expansion of Concrete, per AASHTO T 336 procedure, adapted from (FHWA 2011).

<table>
<thead>
<tr>
<th>Type of Coarse Aggregate</th>
<th>Average Concrete CTE (x 10^-6 1/°F)</th>
<th>Standard Deviation (x 10^-6 1/°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chert</td>
<td>6.01</td>
<td>0.42</td>
</tr>
<tr>
<td>Sandstone</td>
<td>5.32</td>
<td>0.52</td>
</tr>
<tr>
<td>Quartzite</td>
<td>5.19</td>
<td>0.50</td>
</tr>
<tr>
<td>Dolomite</td>
<td>4.95</td>
<td>0.40</td>
</tr>
<tr>
<td>Gneiss</td>
<td>4.87</td>
<td>0.08</td>
</tr>
<tr>
<td>Granite</td>
<td>4.72</td>
<td>0.40</td>
</tr>
<tr>
<td>Limestone</td>
<td>4.34</td>
<td>0.52</td>
</tr>
<tr>
<td>Basalt</td>
<td>4.33</td>
<td>0.43</td>
</tr>
<tr>
<td>Rhyolite</td>
<td>3.84</td>
<td>0.82</td>
</tr>
</tbody>
</table>

Under the AASHTO 1993 pavement design guide, the CTE is not included as a design input (AASHTO 1993). With the introduction of Mechanistic-Empirical Design Guide (MEPDG) in 2008 and its associated design software (originally DARWin-ME, now Pavement ME Design) in 2011, many new design inputs can be used in pavement design, including the CTE and joint spacing. This is the first design approach to use CTE as an input parameter, and recent research indicates that CTE is one of the most significant inputs, leading to substantial interest in accurate CTE measurements (Tanesi, et al. 2010). It should be noted that the CTE values used in Pavement ME Design are based on the older AASHTO TP 60 test method at the present time; T 336 test results should not be used as inputs (Tanesi, et al. 2010; Tanesi, et al. 2012). In the design process, these inputs have considerable influence on the predicted development of transverse cracking and joint faulting modes of distress. Other variables such as slab thickness, anticipated traffic loading, and concrete mechanical properties (elastic modulus and modulus of rupture), strongly influence performance as well, but were previously included as inputs in the 1993 AASHTO design guide.

Recent studies in Florida (Tia, et al. 2012) and Georgia (Kim 2012) have also highlighted the significant influence of CTE and demonstrated the usefulness of MEPDG in accounting for this critical design variable for JPCP designs. Both studies demonstrated that lower CTE concrete pavements are predicted to have better performance than pavements with higher CTE concrete. CTE was found to have the greatest influence on transverse slab cracking, with lesser influence on pavement roughness and joint faulting.

The MEPDG analysis in the Georgia study noted that a CTE greater than 6.0 x 10^-6 1/°F (roughly equivalent to 5.3 x 10^-6 1/°F by the AASHTO T 336 procedure) was associated with a
large increase in transverse cracking and consequent reduction in predicted service life. A CTE of $5.5 \times 10^{-6} \, \text{in}^2/\text{F}$ (or $4.8 \times 10^{-6} \, \text{in}^2/\text{F}$ per AASHTO T 336) or less was determined to be associated with acceptable performance for the 10 in. JPCP with 15 ft. joint spacings. The Georgia study investigated the influence of joint spacing, but only for a single thickness and CTE (10 in. and $6.0 \times 10^{-6} \, \text{in}^2/\text{F}$). Predicted cracking performance was acceptable for 12 ft. and 15 ft. spacings, but decreased dramatically at 18 and 20 ft. spacings. Because pavement exposure conditions, traffic loads, and other design inputs vary widely, this should not be interpreted to suggest a firm limit on CTE. However, the findings of this study do illustrate the importance of considering both CTE and joint spacing as design parameters, and that shorter joint spacings are appropriate for pavements with high CTE aggregates.

Interestingly, two of the three aggregates used in the Florida study were from Alabama: limestone from Vulcan’s Calera quarry (CTE = $5.99 \times 10^{-6} \, \text{in}^2/\text{F}$) and a siliceous river gravel (CTE = $7.2 \times 10^{-6} \, \text{in}^2/\text{F}$). The CTE values were based on the AASHTO TP 60 procedure, and would correspond to approximately 5.29 and $6.5 \times 10^{-6} \, \text{in}^2/\text{F}$, for the limestone and gravel, respectively. Concrete made with the river gravel was predicted to perform markedly worse than either the Calera limestone or the Florida limestone. That study did not address the influence of joint spacing, however, using a fixed joint spacing of 15 ft. for all design cases.

### 3.3. Parametric Study in Pavement ME / MEPDG

To further investigate the influence of joint spacing and CTE, a parametric study was performed using Pavement ME Design (v.2.1.24), AASHTO’s MEPDG design software. The authors worked with Dr. Feng Mu, pavement engineer for CEMEX, on this effort. Parameters and pavement designs were selected jointly, and Dr. Mu performed the service life analyses.

The study involved two phases, with the first phase focused solely on cracking as the trigger for rehabilitation, and the second phase examining potential changes in failure mode that would trigger rehabilitation. All designs were for a doweled JPCP pavement in Huntsville, Alabama with 2500 average annual daily truck traffic (AADTT) in year 1 and 2% annual traffic growth. Ninety-five percent (95%) of truck traffic was placed in the outside (design) lane, which equates to approximately 22 million equivalent single axle loads (ESALs) at an age of 20 years. All CTE inputs are based on the AASHTO TP 60 method of measuring CTE, rather than the newer AASHTO T 336 test method, because Pavement ME national calibrations have yet to be updated to accept CTE values obtained from the AASHTO T 336 method. As a result, the CTE values shown in this study are approximately $0.7 \times 10^{-6} \, \text{in}^2/\text{F}$ greater than those that would be obtained by the AASHTO T 336 method.

For both phases, a 95% reliability level was used in the analyses. This means that for a particular design under the same traffic loading and climatic exposure, it would be expected to meet or exceed the predicted years to failure (rehabilitation threshold) 95 times out of 100.
Correspondingly, there is a 5% chance the design will reach the failure/rehabilitation threshold before that time.

In the first phase, only cracking was considered as a failure criterion triggering rehabilitation, while joint spacing, slab depth, and CTE were varied. Cracking limits of 10% of slabs and 15% of slabs were considered. These are common terminal distress values used in MEPDG analyses. Table 3.2 summarizes the designs investigated in Phase I and the variables investigated. The baseline design was designed to fail in a short time unless CTE was extremely low, while the other four designs illustrate attempts to optimize performance and pre-rehabilitation service life.

Table 3.2. Design details for Phase I Pavement ME analyses.

<table>
<thead>
<tr>
<th>Slab Depth</th>
<th>Baseline</th>
<th>Design 2</th>
<th>Design 3</th>
<th>Design 4</th>
<th>Design 5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10 in.</td>
<td>10 in.</td>
<td>12 in.</td>
<td>12 in.</td>
<td>10 in.</td>
</tr>
<tr>
<td>Lane Width</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>12 ft.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Joint Spacing</td>
<td>15 ft.</td>
<td>12 ft.</td>
<td>15 ft.</td>
<td>12 ft.</td>
<td>15 ft.</td>
</tr>
<tr>
<td>Base</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6 in. granular</td>
<td>None</td>
<td>Tied</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shoulder</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AADTT</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2500 (95% in design lane)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CTE Range</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4.5 to 6.5 x 10^{-6}°F</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Tables 3.3 and 3.4 show the predicted times to 10% and 15% slab cracking for the Phase I designs, respectively. These results are also illustrated in Figs. 3.2 and 3.3. The Pavement ME software will not predict a service life over 100 years; the results of several analyses for Design 4 are therefore reported as “100+” years for the time to failure. When the predicted time to failure exceeds the targeted service life for a pavement (i.e. 40 years), it can be considered overdesigned and more economical design details may be used to optimize the design.

The results of the Phase I analysis illustrate the tremendous influence of CTE and joint spacing on pavement service life when cracking is the governing failure mode. A lower CTE will lead to a longer service-life for a given design. Using low CTE materials may not be an option in some cases, but increasing slab thickness or changing the joint spacing is always an option.

Reducing the joint spacing also results in significant improvements in predicted service life; this is most clearly seen in comparisons of the Baseline design vs. Design 2, and Design 3 vs. Design 4. Simply reducing the joint spacing of the baseline design (Baseline → Design 2) increases predicted service life from 4 to 33 years (based on a 10% cracking threshold) for concrete with a CTE of 5.5 x 10^{-6}°F. Although this option does require 20% more joints to be constructed and maintained, no additional concrete is required for this option, and performance is predicted to be dramatically improved as a result of reduced movement at each joint.

Increasing the slab thickness from 10 to 12 in. (Baseline → Design 3) without changing the joint spacing also yields improvements in predicted service life, but this requires 20% more
concrete and yields only limited benefits at higher CTE values. A combination of reduced joint spacing and increased slab thickness yields a very robust design (Design 4) that is most likely over-designed for concrete with a low CTE. The use of a tied shoulder (Design 5) also yielded improvements in predicted time to failure, although these were not as dramatic as those obtained by increasing slab thickness or reducing the joint spacing.

**Table 3.3.** Phase I Pavement ME analysis results for a failure threshold of 10% slab cracking.

<table>
<thead>
<tr>
<th>CTE (x 10^-6/°F)</th>
<th>Time to cracking of 10% of slabs (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Baseline</td>
</tr>
<tr>
<td>4.5</td>
<td>24</td>
</tr>
<tr>
<td>5.0</td>
<td>13</td>
</tr>
<tr>
<td>5.5</td>
<td>4</td>
</tr>
<tr>
<td>6.0</td>
<td>1</td>
</tr>
<tr>
<td>6.5</td>
<td>0.5</td>
</tr>
</tbody>
</table>

**Table 3.4.** Phase I Pavement ME analysis results for a failure threshold of 15% slab cracking.

<table>
<thead>
<tr>
<th>CTE (x 10^-6/°F)</th>
<th>Time to cracking of 15% of slabs (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Baseline</td>
</tr>
<tr>
<td>4.5</td>
<td>36</td>
</tr>
<tr>
<td>5.0</td>
<td>18</td>
</tr>
<tr>
<td>5.5</td>
<td>6</td>
</tr>
<tr>
<td>6.0</td>
<td>3</td>
</tr>
<tr>
<td>6.5</td>
<td>2</td>
</tr>
</tbody>
</table>

**Figure 3.2.** Phase I Pavement ME analysis results for a failure threshold of 10% slab cracking.
Figure 3.3. Phase I Pavement ME analysis results for a failure threshold of 15% slab cracking.

Design 3 was selected for the second phase, in which faulting (0.12 in. at joints) and loss of smoothness (IRI > 172 in./mi.) were also considered as failure criteria. The intent was to discern whether cracking would be a dominant failure mechanism for these designs, or whether faulting and/or smoothness might trigger rehabilitation even earlier than cracking.

The results of the Phase II analysis are presented in Table 3.5 and Fig. 3.4. In Table 3.5, the governing failure mode at each CTE value is in bold text. The results clearly indicate that cracking is the governing failure mode at all but the lowest CTE value (4.5 x 10^{-6} °F) examined in this study, and only if the 15% cracking limit is used. The influence of CTE on predicted time to failure for faulting and IRI can also be seen. Predicted service life with respect to these failure modes is increased as CTE decreases; a nearly linear relationship can be seen in Fig. 3.4. Cracking, in contrast, tends to exhibit an exponential increase in service life as CTE decreases.

Table 3.5. Results of Phase II Pavement ME analysis of Design 3 (12 in. thick slab with 15 ft. joint spacing). The governing failure mode for each CTE is in bold.

| CTE (x 10^{-6} °F) | Years to failure, defined by: |  |  |  |
|-----------------|-------------------------------|-------------------------------|-------------------------------|-------------------------------|-------------------------------|-------------------------------|-------------------------------|-------------------------------|
|                 | 10% Cracking                  | 15% Cracking                  | Faulting > 0.12 in.          | IRI > 172 in/mi               | 10% Cracking                  | 15% Cracking                  | Faulting > 0.12 in.          | IRI > 172 in/mi               |
| 4.5             | 69                            | 94                            | 83                            | 74                            | 41                            | 56                            | 64                            | 67                            |
| 5.0             | 41                            | 56                            | 64                            | 67                            | 41                            | 56                            | 64                            | 67                            |
| 5.5             | 20                            | 30                            | 55                            | 52                            | 20                            | 30                            | 55                            | 52                            |
| 6.0             | 9                             | 16                            | 48                            | 38                            | 9                             | 16                            | 48                            | 38                            |
| 6.5             | 4                             | 7                             | 40                            | 26                            | 4                             | 7                             | 40                            | 26                            |
Figure 3.4. Phase II Pavement ME analysis results for Design 3. Slab cracking, joint faulting, and roughness were considered as failure modes.

3.4. Summary

Modern rigid pavement design, analysis, and construction methods can collectively contribute to longer service lives, and more efficient and sustainable use of materials. For carbonate coarse aggregate, longer life means that diamond grinding may be significantly postponed. The use of MEPDG in design enables pavement engineers to account for the impact of many variables that are not considered in the AASHTO 1993 design guide. The two most significant variables are coefficient of thermal expansion (CTE) of the concrete and joint spacing. A parametric study in Pavement ME Design demonstrated that:

- CTE is an influential factor in determining predicted service life and a significant influence on the development of slab cracking. A 0.5 x 10^-6 1/°F reduction in CTE can result in doubling the predicted service life of a particular pavement design.
- Lower CTE concretes are also predicted to experience less faulting and loss of smoothness (as indicated by IRI values).
- Reducing the joint spacing of JPCP from 15 ft. to 12 ft. had a much greater impact on service life than increasing the slab thickness from 10 in. to 12 in.
- Reducing the joint spacing enables the use of a more economical pavement design, thereby reducing initial construction costs.
- For high CTE concrete pavements, a combination of reduced joint spacing and increased slab thickness can be used to obtain a much longer service life than would be possible by either strategy on its own.
Because low CTE concrete is less susceptible to cracking and faulting distress, it should reduce or delay the need for diamond grinding rehabilitation to restore smoothness.

Several courses of action can be recommended based on the information presented in this chapter in order to improve the design and construction process for concrete pavements in Alabama:

- Adopt MEPDG and Pavement ME Design software. This will enable pavement design engineers to fully account for the influence of CTE, joint spacing, and other variables in pavement design.
- Develop a database of CTE values for concrete mixture designs used for pavements to ensure the proper inputs are used with the Pavement ME Design software. Because the primary influence on CTE will come from the aggregates, an otherwise standard mixture design using a constant paste volume can be used to compare the expected CTE of pavements with different aggregates.
- Require measurement of CTE of concrete actually used in pavement construction to ensure that it is equal to or less than was considered in the design. On the jobsite, this would only require casting several additional test cylinders at the same time concrete is sampled for other quality control tests.
- Consider utilizing shorter joint spacing as an option to improve pavement performance without increasing slab thickness.
- Adopt optimized gradation specifications targeted towards slipform paving concrete to be used in concert with a workability performance test, such as the Box Test developed at Oklahoma State University.
- Consider modifying specifications to allow two-lift paving, with fewer restrictions on materials in the lower layer, and seek to apply for FHWA support for a demonstration project in Alabama.
4. Pavement Texture and Friction

4.1. Pavement Texture and Measurement Techniques

Pavement texture is the deviations of the surface from a true planar surface. Portland Cement Concrete Pavements (PCCP) can be textured in the fresh (wet) or hardened state. Fresh concrete texturing techniques include longitudinal or transverse tining, and burlap or carpet drags. Hardened concrete is most commonly textured by diamond grinding.

As defined by the Permanent International Association of Road Congress (PIARC), texture has three levels: micro-texture, macro-texture, and mega-texture. Micro texture has a wavelength of less than 0.5 mm and amplitude of 1 to 500 μm. It is most greatly influenced by fine aggregate and is important in maintaining friction in dry conditions and wet conditions with speeds under 45 mph. Macro-texture has a wavelength greater than 0.5 mm but less than 50 mm and amplitude between 0.1 and 20 mm. Figure 4.1 gives visual definition of micro-texture and macro-texture levels. Macro-texture is determined by the type of surface finish and is essential to maintain friction in high speed, wet conditions. It also prevents hydroplaning. Mega-texture relates to pavement ride quality (smoothness or roughness); it is undesirable and affects texture and friction measurement by causing roughness measuring devices to bounce on the pavement.

![Figure 4.1: Visual of micro-texture and macro-texture. From (Hall, et al. 2009).](image)

4.1.1. Texture Measurement

Pavement texture can be measured either in spot tests or continuous tests. Each method has a unique index for texture and has its own advantages and disadvantages. A summary of commonly used measurement techniques, their method of measurement, and their advantages and disadvantages can be found in the Tables 4.1 and 4.2. Spot measurement techniques, shown in Table 4.1, are primarily sensitive to macro-texture; continuous measurement techniques, shown in Table 4.2, are more sensitive to mega-texture and are used to characterize pavement roughness.
### Table 4.1: A summary of spot texture measurement methods. Modified from (Hall, et al. 2009).

<table>
<thead>
<tr>
<th>Test Method</th>
<th>Standard</th>
<th>Description</th>
<th>Measurement</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand Patch Method (SPM)</td>
<td>ASTM E 965</td>
<td>This volumetric-based spot test method provides the mean depth of pavement macro-texture. The operator spreads a known volume of glass beads in a circle onto a clean surface and determines the diameter and the subsequent mean texture depth (MTD).</td>
<td>Mean texture depth (MTD) of macro-texture is computed as: $MTD = \frac{4V}{\pi D^2}$ Where $V = \text{Sample volume, in}^3$, $(\text{mm})^3$ and $D = \text{average material diameter, in.} (\text{mm})$</td>
<td>Simple method and inexpensive equipment. When combined with other data can provide friction information. Widely used method</td>
<td>Method is slow and requires lane closure. Only represents a small area. Only macro-texture is evaluated. Sensitive to operator variability. Labor intensive activity.</td>
</tr>
<tr>
<td>Circular Texture Meter (CTM)</td>
<td>ASTM E 2157</td>
<td>This non-contact laser device measures the surface texture in an 11.65 in (295 mm) diameter circular profile of the pavement surface at intervals of 0.034 in. (0.868 mm), matching the measurement path of the Dynamic Friction Tester (DFT). It rotates at 20 ft/min (6 m/min) and provides profile traces and mean profile depth for the pavement surface.</td>
<td>Indices provided by the CTM include the mean profile depth (MPD) and the root mean square (RMS) macro-texture. MPD can be converted to an estimated texture depth (ETD) to be used in calculating International Friction Number (IFN). $ETD = 0.008 + 0.8 * MPD$</td>
<td>Measures the same diameter as DFT, allowing texture-friction comparisons. Repeatable, reproducible, and independent of operators. Correlates well with MTD. Measures positive and negative profile. Is small and portable. Set up time is short (less than 1 minute).</td>
<td>Method is slow (about 45 seconds to complete) and requires lane closures. Represents a small surface area.</td>
</tr>
</tbody>
</table>

### Table 4.2: A summary of pavement profile measurement devices. Modified from (Jones and Omundson 2004, Koski 1994).

<table>
<thead>
<tr>
<th>Test Method</th>
<th>Standard</th>
<th>Description</th>
<th>Measurement</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Profilograph</td>
<td>ASTM E 1274</td>
<td>A metal frame trailer (profilograph) is driven down a section of pavement. The vertical movement between a center mounted measuring wheel and the reference frame of the vehicle are recorded on a continuous graph.</td>
<td>The Profile Index (PI) is calculated by summing the total number of vertical deviations over a 0.1 mile pavement section. Profiles can be filtered with a blanking band that ignores deviations less than a predetermined height. Blankin bands are usually 0.1 or 0.2 inches.</td>
<td>Frame can be dismantled for easy transportation. Low cost and easy to use. Repeatable results. Profiles can be fed through computers which avoids time consuming hand reduction and calculation of vertical deviations (scallops).</td>
<td>Low Speed. No correlation between the types of profilographs.</td>
</tr>
<tr>
<td>Inertial Profiler</td>
<td>AASHTO R 57 ASTM E 950</td>
<td>Height between pavement surface and accelerometer in host vehicle is measured by computer program that converts vertical acceleration into distance. Height is measured with non contact sensor, commonly a laser transducer. From this a continuous profile of the pavement segment is obtained.</td>
<td>Profile data is run through a computer system, filtered twice (once with a 9.85 in. moving average, and again with quarter-car simulation). Divide profile by segment length to get International Roughness Index (IRI). Units of in/mi.</td>
<td>Profiles can be taken at normal highway speeds. Quicker setup, test time, and data upload than profilograph. Collects a true profile allowing for other ride quality indexes to be calculated.</td>
<td>Lack of accuracy and repeatability. Lack of knowledge of equipment and measurement. Can be unable to compensate for longitudinal texturing (e.g. diamond grinding, tining).</td>
</tr>
</tbody>
</table>
4.2. Pavement Friction

Friction is the force that resists relative motion between the tire and the pavement. Friction is a key factor in keeping vehicles on the pavement and gives drivers the ability to control and maneuver the vehicle in a safe manner in the longitudinal and lateral directions. The longitudinal component of the friction force is generated by the forward motion of the wheel rolling over the pavement. The lateral component of the friction force accounts for vehicles changing direction. The higher the friction between the tire and the pavement, the better control the driver has.

Friction is the result of a combination of two mechanisms: adhesion and hysteresis. Adhesion is small scale bonding of the tire rubber and the pavement surface, and hysteresis is the enveloping of the tire around the texture (Figure 4.2). Both components are dependent on the tire-pavement contact, the material properties of the tire, and the texture of the pavement. Adhesion is responsive to micro-texture and governs the overall friction on smooth textures and dry pavements. Hysteresis is responsive to macro-texture and responsible for friction on coarse textures and wet pavements. Other components (tire rubber shear) play a role in the creation of frictional forces but are insignificant when compared to adhesion and hysteresis and are therefore ignored. (Hall, et al. 2009)

![Figure 4.2: Visual of mechanisms that create friction force in pavements. From (Hall, et al. 2009).](image)

4.2.1. Friction Measurement

Several tests have been developed to evaluate the friction of paved surfaces. Like pavement texture, friction can be measured over a continuous pavement segment or in spot tests. Tables 4.3 and 4.4 summarize each method, their corresponding indices of friction, and their advantages and disadvantages.
Table 4.3: A summary of friction measurement methods that can be used for lab and field measurement. Modified from (Henry 2000).

<table>
<thead>
<tr>
<th>Test Method</th>
<th>Standard</th>
<th>Description</th>
<th>Measurement</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
</table>
| British Pendulum Tester (BPT)   | AASHTO T 278      | Produces a low-speed sliding contact between a standard rubber slider and the pavement surface. The elevation to which the arm swings after contact provides an indicator of the frictional properties. Data from five readings are typically collected and recorded by hand. | The British Pendulum Number (BPN) based on the pendulum height of a calibrated BPT. | • It is used worldwide as a measure of friction and texture.  
  • It is suitable for both laboratory and field evaluation.  
  • The BPT can be used to measure both longitudinal and lateral pavement-tire friction. | • BPN variability is large and can be affected by operator procedures and wind effects.  
  • For both tests traffic control is required. They do not always simulate pavement-tire characteristics. Both devices collect only spot measurements and cannot be used for network evaluation. To quantify a given section of pavement, several measurements must be made inside the section. |
| Dynamic Friction Tester (DFT)   | ASTM E 1911       | Measures the torque necessary to rotate three small, spring loaded, rubber pads in a circular path over the pavement surface at speeds from 3 to 55 mph (5 to 89 km/hr). Water is applied at 0.95 gal/min (3.6 L/min) during testing. Rotational speed, rotational torque, and downward load are measured and recorded electronically. | Produces DTF numbers or friction coefficients and a graph of the friction coefficients for different rotational speeds. This device also reports the peak friction, associated peak slip speed, and the international friction index (IFI), designated by F(60) and S_p. | • Provides good repeatability and reproducibility and is unaffected by operators or wind.  
  • It also provides friction coefficients that are representative of high speed values.  
  • It can be used to produce the IFI statistics  
  • Correlates well with BPN. |                                                                                  |

Table 4.4: A summary of friction measurement methods that can be used for field measurement. Modified from (Henry 2000).

<table>
<thead>
<tr>
<th>Test Method</th>
<th>Standard</th>
<th>Description</th>
<th>Measurement</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
</table>
| Locked-Wheel      | AASHTO T 242      | This Device is installed on a trailer which is towed behind the measuring vehicle at a typical speed of 40 mph (64 km/hr). Water (0.2 in [0.5mm] thick) is applied in front of the test tire, the test tire is lowered as necessary, and a braking system is forced to lock the tire. Then the resistive drag force is measured and averaged for 1 to 3 seconds after the test wheel is fully locked. Measurements can be repeated after the wheel reaches a free rolling state again. | The measured resistive drag force (F) and the wheel load (W) applied to the pavement are used to compute the coefficient of friction, μ. Friction is reported as friction number FN or skid number SN.  
  
  \[ FN = \frac{F}{W} \times 100 \]  | • Well developed and widely used in the U.S.  
  • More than 40 states use the locked-wheel devices.  
  • Systems are user friendly, relatively simple, and not time consuming. | • Can only be used on straight segments (no curves, T-sections or roundabouts).  
  • Can miss low friction spots because measurements are intermittent. |
| Side-Force        | AASHTO T 266      | Side-force friction measuring devices measure the pavement side friction or cornering force perpendicular to the direction of travel of one or two skewed tires. The Mu-Meter trailer is commonly used to obtain this measurement. Water is placed on the pavement surface and one or two, free rotating wheels are pulled over the surface (typically at 40 mph [64 km/h]). Side force, tire load, distance, and vehicle speed are recorded. Data is typically collected every 1 to 5 inches and averaged over 3 foot intervals. | The side force perpendicular to the plane of rotation is measured and averaged to compute the Mu Number (MuN), or the sideforces way coefficient, SFC.  
  • Relatively well controlled skid condition similar to fixed-slip device results.  
  • Measurements are continuous through test section.  
  • Method is commonly used in Europe. | • Very sensitive to road irregularities (potholes, cracks, etc.) which can destroy tires quickly.  
  • Mu-Meter device is primarily only used for airports in the U.S. |                                                                                           |
There are other methods in measuring continuous friction along a paved surface. These include the fixed slip device and the variable slip device; neither is commonly used in the United States (Henry 2000). The fixed slip device measures rotational resistance at a constant slip speed which may not always coincide with the critical slip values and requires skillful data reduction. It also does not have an ASTM or AASHTO standard test method for measurement at this time. The variable slip device measures friction as a function of slip from 0 to 100 percent slip. The equipment is large, complex, and expensive, and the data processing is complex. For these reasons they were not included in the previous table. (Hall, et al. 2009)

4.2.2. Locked Wheel Test: Smooth vs. Ribbed Tire

When measuring friction using the locked-wheel test, the option is given to use a ribbed tire (ASTM E501) or a smooth tire (ASTM E524). Which tire states use to measure pavement friction has been of much interest to researchers. The National Cooperative Highway Research Program (NCHRP) has twice published research on states pavement friction measurement practices, once in 2000 (Henry 2000) and again in 2009 (Hall, et al. 2009); both of which sent a survey to states asking which tire, ribbed or smooth, was used in pavement friction measurement. In 2000, of the 39 agencies responding, 27 reported they exclusively used the ribbed tire, five exclusively used the smooth tire, and seven used both tires. In 2009, 41 agencies responded, of which 23 reported they exclusively used the ribbed tire, six agencies used the smooth tire exclusively, and 12 used both tires.

These surveys show an increase in the number of states including the smooth tire in their pavement friction testing program, but this increase is primarily a result of states testing friction with both tires, rather than replacing the ribbed tire with the smooth tire. Between 2000 and 2009, five states (Connecticut, Florida, Kansas, Kentucky, and Michigan) switched from testing with only the ribbed tire to testing with both ribbed and smooth tires. Of these five states, Kansas and Florida use the smooth tire only for research purposes, and not routine friction testing. Oklahoma switched from testing with only ribbed tires to testing with only smooth tires. North Carolina switched from using both tires to only testing with the ribbed tire, and did not provide an explanation.

A study by Henry (1980) investigated the differences in friction numbers obtained from both the ribbed and smooth tires. He found that the grooves in ribbed tires provide channels for water to flow out of the tire-pavement interface, limiting the influence of water flow on the ribbed tire test results. Measurements with ribbed tires are therefore less sensitive to macro-texture and more sensitive to micro-texture. Smooth tires provide the extreme case of zero drainage and are sensitive to both micro-texture and macro-texture. Ideally, both tests would be performed and the results compared to estimate the levels of micro-texture and macro-texture to assess the cause of poor skid resistance. In the case where only one test can be performed, Henry suggested the smooth tire should be used because of its sensitivity to macro-texture which is known to be a significant factor in the creation of friction in wet conditions (Henry 1980).
Several studies have been done to find a correlation between friction numbers measured with smooth or ribbed tires and accident data. Rizenbergs, et al. (1976) attempted such a correlation on rural two-lane roads in Kentucky. 230 pavement sections, of which 13 were concrete, were tested using the now-discontinued ASTM E249 standard tire. The ASTM E249 ribbed tire would be expected to result in 40 mph skid numbers approximately 4% lower than the currently used ASTM E501 tire (Weiner, et al., 1976). The study by Rizenbergs and others found that (1) the ratio of wet-to-dry pavement accidents correlated poorly to skid number, and (2) wet pavement accident frequency correlated poorly with skid number. In both cases, the $R^2$ values were less than 0.43. The authors did not distinguish between test data for asphalt vs. concrete pavements in their discussion. A study on Connecticut roads found high correlation between wet weather accidents and smooth tire friction numbers, and a low correlation between wet weather accidents and ribbed tire friction numbers. It was observed on one stretch of roadway that ninety-five percent (95%) of wet weather accidents occurred when smooth tire friction numbers were less than 15. Over every roadway investigated in the study, no accidents caused by hydroplaning occurred with smooth tire friction numbers above 25 (Ganung and Kos 1979). A study of Florida roads also found decreased wet weather accidents when smooth tire friction numbers were above 25 (Henry 2000).

In a later pavement friction study by Henry (2000), he commented that state Departments of Transportation may be unwilling to switch from a ribbed tire test to a smooth tire test because the smooth tire test produces lower friction numbers than the ribbed tire test. The inability to compare new smooth tire friction numbers to historical friction performance of pavements would also contribute to a reluctance to change specifications.

4.2.3. **International Friction Index (IFI)**

With a wide variety of methods to measure friction, PIARC found it necessary to find a way to harmonize the results of the methods. A PIARC study by Boulet, et al. (1995) evaluated the relationships of texture and friction measurements and developed a unit for converting results from different devices into a single scale. The result was the International Friction Index (IFI). IFI consists of two numbers, one related to macro-texture and one related to friction measurement. Therefore, it is necessary to measure both in order to calculate the IFI.

The macro-texture term in the IFI is the speed constant of wet pavement friction ($S_p$). While preferably given as mean profile depth (MPD), measured continuously along the test section by an inertial profiler, it can be measured by sand patch test or circular texture meter (CTM) (ASTM E 1960 2011). The friction term used to calculate IFI is the Friction Number at 60 km/hr (F60). Friction can be obtained through the locked-wheel or side-force methods, as well as the Dynamic Friction Tester (DFT). The PIARC study found the IFI to have small error when measuring texture as MPD and measuring friction with any of friction methods.
4.3. Texture Application in the Laboratory

Standard concrete pavement textures (such as tining, burlap or carpet drag, and broom finish) are easy to apply to specimens in the lab and achieve similar textures as found in field application. Difficulty arises in diamond grinding lab specimens. Diamond grinding lab specimens with field equipment is not realistic, and creating textures in the lab similar to textures in the field can require custom made equipment. A laboratory grinding device developed by the International Grooving and Grinding Association (IGGA) and The University of Texas at Austin (UT-Austin) is an example of one such device (Figure 4.3).

Figure 4.3: Laboratory grinding device at UT-Austin (Fentress n.d.).

4.4. Accelerated Pavement Wear Options

There are several options available to simulate traffic wear of pavement specimens. While most options were originally designed to simulate wear on asphalt concrete pavement specimens, with minor modifications they could be applicable to PCC pavement specimens.

The Circular Track Polishing Machine (CTPM) is capable of simulating wear on twelve specimens at a time constructed of either asphalt or concrete. Concrete specimens are usually trapezoidal sections (7.5 x 28 x 2 in) cut from larger slabs, so desired texture is easily applied. When used to wear concrete specimens, different wheel types are used to properly simulate wear of the top layer and expose the coarse aggregate in a manner similar to field conditions. First, standard pneumatic tires are used to polish the as-built surface, followed by steel tires to expose the coarse aggregate, and finally rubber tires to polish the coarse aggregate. Wheels are loaded to 72 pounds of vertical force, and one test cycle is typically eight hours of exposure at 30 rpm.
After polishing with the CTPM, friction can be measured using the BPT at various points on the specimen.

The Three Wheel Polishing Device (TWPD) has been developed by the National Center for Asphalt Technology (NCAT). It is similar to the Circular Track Polishing Machine, but is built to the same dimensions as the CTM and DFT, allowing texture and friction of specimens to be evaluated easily (Rado 2009).

A potential issue with circular polishing methods is that real-world pavements are textured parallel or perpendicular to the travel direction of traffic. However, due to their circular path, devices such as the TWPD only simulate the alignment of traffic and texture at two points along that path.

Another accelerated wear device is the Model Mobile Load Simulator (MMLS-3), a one-third-scale accelerated wear testing device, which has the ability to simulate wear in a continuous longitudinal direction over concrete or asphalt slabs. The device consists of four pneumatic tires (one third the size of standard truck tires) that are each loaded to 607 pounds and run over a four foot long section of pavement (Figure 4.4). The device was originally developed to simulate rutting in asphalt concrete pavements but has successfully been applied to concrete pavements by the Pennsylvania Department of Transportation as well as the Virginia Department of Transportation. (Rado 2009)

![Figure 4.4: Schematic of the Model Mobile Load Simulator (Rado 2009).](image)

4.5. Summary

Friction and surface texture are important components in pavement serviceability. Both micro- and macro-texture contribute to the development of friction and skid resistance between vehicle tires and the pavement in dry conditions, while macro-texture dominates the friction characteristics of a pavement in wet weather. Given ALDOT’s concerns about the loss of friction
in diamond-ground pavements containing limestone coarse aggregate, a review of texture and friction characterization methods was conducted.

Continuous texture measurement conducted on pavements in the field aim to detect undesirable mega-texture, or roughness, characterized as either the profile index (PI) or IRI. Beneficial macro- and micro-texture are characterized via spot measurements such as the sand patch test or CTM; both tests can be performed on laboratory specimens. Friction tests can also be used to indirectly characterize macro- and micro-texture. Some tests such as the locked wheel and side force friction tests are conducted at speeds of 40 mph or higher and can only be performed in the field or on test tracks. The British Pendulum and Dynamic Friction tests can be performed both on laboratory samples and in the field.

ALDOT’s friction and texture measurement methodologies are generally in line with the practices of most state transportation agencies, using profilograph and inertial profiler to measure smoothness and roughness, and relying on a locked-wheel skid trailer with a ribbed tire to characterize friction, macro-texture, and micro-texture. However, friction of wet pavements, which is the greater concern for traffic safety, may be better characterized by using smooth tires in the locked wheel test. Smooth tires are more likely to hydroplane in the presence of sufficient water and insufficient macro-texture, better representing the performance of vehicles with minimal remaining tread on their tires. A smooth tire FN of 25 or higher was cited in the literature as sufficient for reducing wet-weather accidents.

At a meeting in December 2014, ALDOT engineers expressed some concern about interpreting smooth tire friction data if the ribbed tire version of the test were to be abandoned in favor of the smooth tire. Therefore, the recommendation of this research team is for ALDOT to begin collecting smooth tire friction data in addition to ribbed tire data over a limited portion of ALDOT’s interstate highway network for a period of three years. This will allow ALDOT to determine (1) whether smooth tire friction numbers provide a better correlation to wet-weather accident frequencies than ribbed tire friction numbers, (2) whether there is a reasonable correlation between smooth and ribbed tire friction numbers, (3) whether a smooth tire FN of 25 is appropriate as a lower limit for concrete pavements, and (4) whether the ribbed tire test should be replaced completely by smooth tire friction testing.

In order to predict the texture and friction retention of pavements with carbonate coarse aggregate, both before and after diamond grinding/grooving, it will be desirable to correlate the results of lab tests to expected field performance. Several options were presented for texturing, accelerated wearing, and characterization of laboratory specimens. While there is no perfect solution, laboratory testing will allow evaluation of a wider range of textures (including different diamond grinding and grooving configurations) before selecting the best candidates for testing in the field or on test tracks. The potential for research in this area will be discussed in greater detail in the following chapter, after first providing a review of diamond grinding and grooving technologies.

31
5. Diamond Grinding Techniques and Specifications

5.1. Background

Diamond grinding is a Concrete Pavement Preservation (CPP) technique that is used in both new construction and rehabilitation to improve ride quality and extend service life. In new construction, it may be used to ensure that newly-placed pavements meet initial smoothness requirements. In older pavements, it is commonly used to fix joint faulting and pavement roughness, to improve pavement friction, and to reduce noise caused by tire-pavement contact. Structural and material deficiencies (e.g. D-cracking, longitudinal cracking, corner break, and reactive aggregate) cannot be fixed with diamond grinding and should be addressed before any diamond grinding activity is performed. Diamond grinding is often used in combination with other CPP techniques, usually those meant to restore structural capacity. (Correa and Wong 2001)

Diamond grinding equipment consists of diamond saw blades mounted on a cutting head that is typically 36-48 inches wide and includes 50 to 60 blades per foot. The blades are arranged on the cutting head in an alternating pattern of blades and spacers sized to get the desired texture for the aggregate hardness. Figure 5.1a shows an example of diamond blades on a cutting head. The machine works similarly to a wood plane; the front wheels pass over a fault, the cutting head shaves it down, and the back wheels ride over the smooth surface. This process leaves behind a new corduroy texture running parallel to the direction of travel; this is shown in Figure 5.1b.

![Diamond cutting blades mounted on the cutting head.](image1)

![An example of diamond ground pavement including the fins.](image2)

The corduroy texture is defined by three components: groove width, groove height, and land area. Groove width is the width of the cutting blade, and groove height is the distance from the bottom of the groove to the top of the land area. Land area is the distance between blades and has the greatest contribution to pavement texture and friction. A profile view of the texture can
be seen in Figure 5.2. The cutting processes can leave material above the intended land area surface. These protrusions are called *fins* and are intended to break off under traffic loading, leaving behind the intended groove height and land area. Combining a hard aggregate and wide spacing (e.g. 50 blades per foot) can cause fins to persist longer than anticipated, while combining a soft aggregate and narrower spacing (e.g. 60 blades per foot) may create weak fins that break off flush with the groove, leaving behind a nearly flat surface prone to rapid polishing. Therefore, it is generally recommended that blade spacing, and therefore land area, be larger for pavement containing soft coarse aggregates than for pavements containing hard coarse aggregates. (ACPA 2000).

![Diagram](image.png)

**Figure 5.2:** Profile of corduroy texture produced by diamond grinding. (Correa and Wong 2001)

Diamond concentration, diamond size, and bond hardness must also be considered when diamond grinding. Concentration is how many diamonds are on the blade; typically it is the overriding factor in grinding efficiency, as more diamonds mean a harder blade and more efficient grinding. Size and bond hardness are important factors in blade life and cutting speed. Diamond size acts similarly to tooth size for cutting wood and metal; larger diamonds are used for softer aggregates and smaller diamonds for harder aggregates.

### 5.2. State of the Art

The ACPA and IGGA have released recommended dimensions for diamond grinding (ACPA 2000). Table 5.1 gives their recommended dimensions for groove width, land area, groove height, and number of grooves per foot. These recommended dimensions are intended to help avoid excessive fin production and improve grinding efficiency for pavements containing hard or soft aggregates. They may improve texture and friction for pavements with soft aggregates, but there have been no conclusive studies to date. Newer recommendations from IGGA are focusing more on blade counts (grooves per foot) rather than land and groove dimensions (Scofield 2014). However, FHWA continues to recommend the dimensions and blade counts presented in Table 5.1.
The ACPA and IGGA have recommended separate blade spacing for hard (such as granite) and soft (such as limestone) aggregate for over twenty years. Several research projects have been initiated to confirm whether optimizing land width dimensions for hard and soft aggregates can increase and prolong skid resistance. These projects are not completed, but it is known that optimal groove spacing helps curb fin production, which has a large influence on the smoothness of diamond ground pavement. Few states have amended their construction specifications to match these recommendations, despite the potential for increased and prolonged friction numbers and improved/retained smoothness of ground pavements.

A review of the standard construction specification of the 50 states, shown in Figure 5.3, indicates that thirty states have published their diamond grinding specifications. Of those thirty, twenty-one have no reference to aggregate hardness, two states use the recommended land width for hard aggregates, and two states use the recommended land width for soft aggregate. Five states reference aggregate hardness as a design factor in diamond grinding equipment, but only three states (NY, WV, and IA) provide detailed specifications.

It can be seen that the Southeast has the biggest variety in diamond grinding specifications. Four states (TN, VA, MS, and NC) have not published their grinding specifications, and four states (TX, AR, KY, and SC) do not reference aggregate hardness in their grinding specifications. Alabama and Florida use only the recommended hard aggregate land width dimension, though their specifications do not reference them as hard aggregate dimensions. Georgia and Louisiana provide a range in land width but do not specify which is for hard or soft aggregate. Only West Virginia provides specifications of land area for hard and soft aggregates that mirrors the dimensions published by ACPA and IGGA. Table 5.2 provides the grinding specifications for SASHTO states and Texas.

There is also discontinuity between the recommended dimension for groove height and the specifications for several states. The ACPA and IGGA give a recommended groove height of 0.06 in. While the majority of states conform to this specification, some states, including AL, AR, and FL, use 0.03 in. which is half of the recommended depth. It is unclear whether this has any significance for pavement performance and friction, given that only blade width and spacing can actually be controlled during diamond grinding.

<table>
<thead>
<tr>
<th></th>
<th>Range of Values</th>
<th>Hard Coarse Aggregate</th>
<th>Soft Coarse Aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Groove Width (in.)</td>
<td>0.08 – 0.16</td>
<td>0.10-0.16</td>
<td>0.10 – 0.16</td>
</tr>
<tr>
<td>Land Area (in.)</td>
<td>0.06-0.14</td>
<td>0.08</td>
<td>0.10</td>
</tr>
<tr>
<td>Groove Height (in.)</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
</tr>
<tr>
<td>Blades/Grooves per Foot</td>
<td>50-60</td>
<td>55-60</td>
<td>50-54</td>
</tr>
</tbody>
</table>
From the investigation of diamond grinding specifications it was clear that a better understanding was needed of each state’s experiences with diamond grinding. The survey sent to SASHTO members and Texas by the UA team helped to fill gaps concerning usage of diamond grinding as a rigid pavement rehabilitation method, and the average post-rehabilitation service life of rigid pavements with carbonate and siliceous coarse aggregate. Every responding state answered that they use diamond grinding to restore smoothness to rigid pavements. No state reported alternative rehabilitations methods to restore pavement smoothness.
Questions concerning post-rehabilitation service life of diamond ground pavements were asked in two parts:

1) What is the average service life of diamond ground pavements for carbonate and siliceous coarse aggregate?

2) What is the average service life of diamond ground pavements for carbonate and siliceous coarse aggregates when service life was limited by loss of friction?

The second part was intended to determine if states noted cases where loss of friction reduced the post-rehabilitation service life. Results of average service life of diamond ground pavements are summarized in Table 5.3, and the results of average service life of diamond ground pavements when limited by friction loss are summarized in Table 5.4. Several responses were reported as “Undetermined” in these tables, and encompass responses that indicated the survey respondent did not have that information available or that most diamond ground segments had not reached a point where a second rehabilitation or reconstruction was required.

Most states provided the same answer to both questions, but Virginia noted that some pavements containing siliceous aggregates had their post-grinding service life reduced by up to ten years because of friction loss. A wide range of values were reported among the eight states that provided numerical answers. According to this survey, diamond grinding provides between five and twenty years of additional service life for siliceous aggregate pavements, and between three and twenty years for carbonate aggregate pavements. Only Tennessee reported markedly worse performance for pavements with carbonate coarse aggregates.
Table 5.3: Average service life of diamond ground rigid pavements by aggregate type, carbonate vs. siliceous, for SASHO member states and Texas. Bold states border Alabama.

<table>
<thead>
<tr>
<th></th>
<th>Carbonate</th>
<th>Siliceous</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arkansas</td>
<td>Up to 20 years</td>
<td>Undetermined</td>
</tr>
<tr>
<td>Florida</td>
<td>No response</td>
<td>No response</td>
</tr>
<tr>
<td>Georgia</td>
<td>10-15 years</td>
<td>10-15 years</td>
</tr>
<tr>
<td>Kentucky</td>
<td>Undetermined</td>
<td>Undetermined</td>
</tr>
<tr>
<td>Mississippi</td>
<td>10+ years</td>
<td>10+ years</td>
</tr>
<tr>
<td>North Carolina</td>
<td>No response</td>
<td>10-15 years</td>
</tr>
<tr>
<td>South Carolina</td>
<td>No response</td>
<td>Undetermined</td>
</tr>
<tr>
<td>Tennessee</td>
<td>3-4 years</td>
<td>10-12 years</td>
</tr>
<tr>
<td>Virginia</td>
<td>No response</td>
<td>15-20 years</td>
</tr>
<tr>
<td>West Virginia</td>
<td>5+ years</td>
<td>5+ years</td>
</tr>
<tr>
<td>Texas</td>
<td>Undetermined</td>
<td>Undetermined</td>
</tr>
</tbody>
</table>

Table 5.4: Average service life of diamond ground rigid pavements by aggregate type, carbonate vs. siliceous, for SASHTO member states and Texas when limited by loss of friction after grinding. Bold states border Alabama.

<table>
<thead>
<tr>
<th></th>
<th>Carbonate</th>
<th>Siliceous</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arkansas</td>
<td>No response</td>
<td>Undetermined</td>
</tr>
<tr>
<td>Florida</td>
<td>Undetermined</td>
<td>No response</td>
</tr>
<tr>
<td>Georgia</td>
<td>10-15 years</td>
<td>10-15 years</td>
</tr>
<tr>
<td>Kentucky</td>
<td>Undetermined</td>
<td>Undetermined</td>
</tr>
<tr>
<td>Mississippi</td>
<td>No response</td>
<td>No response</td>
</tr>
<tr>
<td>North Carolina</td>
<td>No response</td>
<td>No response</td>
</tr>
<tr>
<td>South Carolina</td>
<td>No response</td>
<td>Undetermined</td>
</tr>
<tr>
<td>Tennessee</td>
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</tr>
<tr>
<td>Virginia</td>
<td>No response</td>
<td>10-12 years</td>
</tr>
<tr>
<td>West Virginia</td>
<td>5+ years</td>
<td>5+ years</td>
</tr>
<tr>
<td>Texas</td>
<td>Undetermined</td>
<td>Undetermined</td>
</tr>
</tbody>
</table>

Several states provided further explanation of their answers and practices. South Carolina has only recently begun diamond grinding and has minimal experience; they have yet to reach the end of service life for diamond ground sections. Florida does not use friction retention to determine pavement service life. Florida also provided information on the performance of sections of I-10 constructed with coarse aggregate from Calera, Alabama. The average time to the first rehabilitation for that district was sixteen years; common first rehabilitation methods were slab replacement and diamond grinding (Dietrich 2015, Cunagin and Dietrich 2013). The average time to the second rehabilitation was eight years, which was usually an asphalt overlay.
5.3. Grinding New Pavements

ALDOT has expressed concerns that grinding may be required on new concrete pavements to meet profile index (PI) acceptance criteria. If grinding were required, that could lead to unacceptably low friction values within a few years and could be a drawback to concrete pavements using limestone. ALDOT’s current PI requirements for new pavement are as follows, with the profile index given in in./mi. with zero blanking bands:

- >50: Must correct
- 20 to <50: Payment penalty on a scale
- 10 to <20: 100% pay
- <10: 100% + bonus pay

ALDOT has completed two, recent, major projects using concrete pavements: a 10-inch un-bonded overlay on I-59 and a pavement reconstruction on I-65. ALDOT shared comments from the project manager of these projects:

- I-59: Grinding was only allowed 25 feet on each side of an end-of-day paving joint. No grinding was required.
- I-65: Grinding was allowed if PI was greater than 50 to bring within acceptable smoothness. Only one pass of grinding was allowed. If, after grinding, smoothness levels were not met, pavement was to be removed and replaced. The contractor did need to grind several sections to meet smoothness requirements.

This recent, limited experience suggests two conclusions: required grinding for PI acceptance is a possibility for new concrete pavements, and it is possible to construct new concrete pavements that meet smoothness requirements without grinding.

A review of other state construction specifications shows that diamond grinding newly-constructed concrete pavements is not a common requirement for surface texture. Figure 5.4 shows the surface finish requirements by state. Only three states (FL, SC, and NJ) require all concrete pavements to be diamond ground at construction. The majority of states require longitudinal or transverse tined texture, but allow contractors to diamond grind pavements in order to meet smoothness requirements.

A more detailed review of pavement smoothness correction practices in the southeastern US shows that ALDOT’s standards are the most restrictive. Alabama allows for 25 feet of grinding on either side of construction joints and a maximum additional 25 feet of grinding throughout a 528-foot test section. Grinding is only permitted to reduce the PI below 50 in./mi., not to improve the PI into the 100% pay range. No other state in the southeast sets a fixed maximum on the length of diamond grinding allowed in each test section. The most common
The specification language is “grind to meet smoothness requirements.” Georgia also requires that if there is more than 50 feet of grinding needed, the entire 528-foot test section is ground.

**Figure 5.4:** Surface finishing requirements of new construction rigid pavements in the United States by state.

### 5.4. Recent Research

This section summarizes the results of recent research on both conventional diamond grinding and newly-developed alternatives incorporating a combination of grinding and grooving.

#### 5.4.1. Conventional Grinding Studies

Diamond grinding is a concrete pavement preservation (CPP) technique that improves pavement smoothness. In some cases, improvements in friction have been reported, but that is not the primary goal of diamond grinding. It does not increase pavement structural capacity, and full or partial depth repairs or load transfer restoration should be made before diamond grinding. The state of Texas has experienced excellent results in improving IRI and retaining those
improvements when combining rehabilitation methods, most notably diamond grinding and dowel bar retrofit (Chen and Hong 2014). A section of US-287 in TxDOT’s Childress District displayed heavy faulting, and the district combined dowel bar retrofit and diamond grinding. Before rehabilitation, the IRI was 150 in./mi., and following rehabilitation it decreased just over fifty percent (50%) to 68 in./mi, or below the intervention level for a JPCP with moderate traffic levels (Correa and Wong 2001). The pavement continued to be monitored over eight years and increased to 94 in./mi. which is still a thirty-six percent (36%) decrease from the pre-rehabilitation value. The overall conclusion was that the combination of diamond grinding and dowel bar retrofit created a lasting, smooth, riding surface.

Chen and Hong (2014) also made several conclusions based on statewide data obtained from eleven diamond grinding projects. The type aggregates used in these pavements were not specified by the authors. When analyzing friction of several pavements, they found that on average, the skid number increased by 5.6 after grinding, and then decreased by approximately 2.0 per year. Furthermore, the net change in friction number over the eight year monitoring period is approximately 10 or 11 based on the values reported by the authors. From a friction standpoint, this could not be considered a successful project, and it serves to highlight the challenges with post-grinding retention of friction. For one of these eleven projects, accident statistics showed a sixty-two percent (62%) decrease in fatalities and a forty-six percent (46%) decrease in serious injuries after diamond grinding. Unfortunately the skid test data for this individual site was not presented, so it is not possible to conclude whether accidents and injuries decreased despite a continued loss of friction, or whether this happened to be a site that performed better than average with respect to retaining friction.

Other studies have shown dramatic improvements in friction realized immediately after grinding the concrete pavement. A study by Mosher (1985) examined five diamond grinding projects in the United States using a Saab Friction Tester and found that the average increase in the Saab Friction Value (SFV) from pre-grind friction levels was 38, a ninety percent (90%) improvement. However, the SFV is not obtained from a locked-wheel test, and it is unclear how much time passed after grinding before the SFV was measured. SFVs are obtained from a constant slip test, and they have the potential to produce friction values as much as twice those obtained from locked-wheel tests. Research on the performance of a CPP project in the City of London in Ontario, Canada using a skid trailer in accordance with ASTM Standard E274 (tire type was not reported) found that the average friction number obtained shortly after diamond grinding a section of Highbury Avenue was 48 compared to the pre-grinding average friction number of 25 in the driving lane (Bradbury and Kazmierowski 1997). In less than one year after diamond grinding, the FN had dropped to 33, and eventually stabilized in the upper 20s during the remaining six years of the study. Similar to the study by Chen and Hong (2014), this study reinforces the concept that friction improvements obtained by conventional diamond grinding can be very short-lived. Neither the study by Mosher nor the study by Bradbury and Kazmierowski reported the diamond grinding specifications used on these projects.
In general, the survival of the macrotexture and microtexture of a diamond ground pavement through time as the pavement undergoes traffic loading and other wear inducing activities such as winter maintenance has shown that the decrease in texture is much faster in the first few years after grinding and eventually stabilizes through time (Grady and Chamberlin 1981, Tyner 1981). A research project conducted by the University of Texas at Austin monitored the skid resistance of diamond ground CRCP sections of I-35W in the Fort Worth, Texas area containing limestone coarse aggregate for a period of 15 months. The grinding specification for the project called for a land area of 0.110 to 0.120 in., or between 49 to 51 blades per foot, and a groove depth of 0.047 in. (1.2 mm). Skid numbers were measured using a locked wheel skid trailer with smooth tires at 50 mph. The average skid number of the pavements increased from 21 before grinding to 34 immediately after, and held approximately steady for at least four months after grinding. After 15 months, however, the skid number had dropped to 27. Further monitoring of the pavement may show whether this trend continues or if the skid number will stabilize (Buddhavarapu, et al. 2014).

The IGGA is working on several diamond grinding projects in the southeastern US. An ongoing project in North Carolina is investigating the effect of blade spacing on friction levels of concrete pavement (Scofield 2013a). The Tennessee Department of Transportation is concerned that diamond grinding does not create sufficient long-term friction numbers and is working with the IGGA to test several pavements across the state, several of which were constructed with limestone aggregate (Scofield 2013b). The study is looking at both the long term friction effects of bump grinding new construction and the use of diamond grinding as a rehabilitation technique. Neither project has been completed; no data or conclusions have been published at this time. Other ongoing projects which investigate long-term friction retention include a 10 year old site in Ohio and an undisclosed location in the Midwest. Both of these projects include pavements constructed with polishing susceptible coarse aggregate. Their results have not been published (Scofield 2014).

5.4.2. Alternatives to Conventional Grinding

The next generation concrete surface (NGCS), developed in a joint study by Purdue University, the ACPA, and IGGA, is a diamond grinding configuration which is designed to provide a quieter driving experience. NGCS combines conventional diamond grinding (CDG) with deeper grooves. Unlike the positive, or upward, corduroy texture created by conventional diamond grinding after the fins break off, the grooves in a pavement with a NGCS texture are intentionally cut into the pavement (negative texture). The grooves are deeper (1/8 in. or more) and more widely spaced (1/2 to 5/8 in.); this creates a macro-texture that is intended to be more durable than that created by conventional grinding. While grinding is primarily an activity to improve smoothness, grooving is intended to improve friction and macrotexture (Martinez 1977). The deeper grooves also provide drainage routes for water during rain events (Wulf, et al. 2008).
Next Generation Concrete Surface was developed as a solution to loud pavement textures such as transverse tining. Several sections of NGCS (MnROADS, Kansas I-70, and Duluth I-35) have been on-board sound intensity (OBSI) tested and when compared to other pavement textures, NGCS is the quietest. Sections of the MnROADS test track that have been ground with the NGCS configuration were monitored for friction using both the smooth- and ribbed-tire locked wheel tests over a four year period between 2007 and 2011. When friction was measured with a ribbed tire, NGCS performed worse than CDG; when tested with a smooth tire, NGCS performed slightly better than CDG (Wilde 2013). Given that smooth tire tests are shown to correlate better to accident rates and wet weather friction, NGCS may be superior to CDG from a friction/safety perspective.

The IGGA has suggested a couple of potential improvements to the method of grinding pavements with limestone coarse aggregate. One suggestion was to diamond groove pavements with soft coarse aggregates if a loss of friction were to occur in the years after diamond grinding. A second suggestion was to use a renewable texture such as Optimized Texture for City Streets (OTCS, formerly known as NGCS lite). OTCS is a finer texture than conventional diamond grinding using 70 to 96 blades per foot, and is intended to provide a quieter riding experience for roads with a speed limit of 45 mph or less (IGGA 2014b). However, it is unclear how OTCS would improve friction for pavements with soft aggregates, given that in conventional grinding of such pavements, lower blade counts are recommended.

5.5. Research Needs

Research on the factors that influence the skid resistance of a diamond ground pavement over time has uncovered some interesting results. However, some areas warrant more investigation. There remains little reported research on the effect of blade spacing on the long-term friction/texture retention of pavements incorporating a range of aggregate hardness. Although wider blade spacings have been shown to have little impact on friction for hard aggregates, no data have been reported to show whether this is also the case for softer aggregates. The effectiveness of longitudinal grooving and/or NGCS textures on hard vs. soft aggregates is another area in need of study.

One method to validate the recommended spacings can be accomplished by testing diamond ground concrete pavement specimens containing soft or hard aggregates with the IGGA recommended blade spacing in the laboratory. To accomplish this task, pavement specimens containing either soft or hard aggregate would be ground using a range of blade spacings. For example, a specimen containing soft aggregate would be ground with a blade spacing of 60 blades per foot while another specimen with soft aggregate would be ground with a blade spacing of 50 blades per foot. These specimens will then be subjected to simulated traffic wear using a device such as the MMLS-3. Texture and friction data will be obtained at designated times during the wear simulation to analyze the effect that blade spacing for a given aggregate type has on the pavement surface through time. After all of the simulations are complete, an
analysis of the data will determine the effect that aggregate type and blade spacing has on the short-term and long-term texture and friction properties of the concrete pavement surface.

Field studies are tremendously valuable and are needed to validate laboratory studies but require large amounts of material, space, and funds. Although it can be less resource intensive to conduct field trials on existing pavements, obtaining and utilizing friction data requires extraordinary cooperation from the owner agency (e.g. state DOT). A purpose-built test track has the advantage that there should be no limitations on the collection and reporting of friction data. Chapter 4 of this report assessed the state of the art in friction and texture characterization in both the lab and field, in order to evaluate the best techniques for any planned experimental research activities.

Models generated from combined laboratory and field research could help predict the effect of diamond grinding on both short-term and long-term pavement friction. Where possible, data from other studies, such as the ongoing IGGA-coordinated studies, should be used to help build a predictive friction model. As an example, a model could be produced to predict the improvement in friction number immediately following grinding based on a quantitative measure of aggregate hardness, grinding configuration, and other measurable pavement characteristics at pre-grind levels. A long-term model could predict time of wear to unacceptable levels based on pavement and grinding factors. This type of model would prove valuable to state highway agencies when considering rehabilitation measures, while conducting life cycle cost analyses (LCCA), or when integrated into a pavement management system (PMS).

5.6. Summary

Paving contractors should be capable of meeting smoothness requirements for new pavements without diamond grinding, based on two recent interstate paving projects in Alabama. Therefore, concerns regarding the potential for rapid loss of friction if newly-constructed pavements containing carbonate coarse aggregates are diamond ground may be overstated. However, the use of diamond grinding remains a likely rehabilitation activity to restore smoothness later in the life of the pavement. It is critical that future grinding and/or grooving activities should be engineered not only to restore smoothness, but also to ensure that they do not result in a similar loss of friction as experienced on several projects in Alabama in the 1980s where carbonate coarse aggregates were used.

There are several opportunities to improve ALDOT’s grinding specifications, based on the literature reviewed for this study.

- Blade spacing: ALDOT specifies 60 blades per foot for all pavements without consideration for the hardness or softness of the coarse aggregate. Adjusting the required blade spacing based on the aggregate hardness is likely to both improve the efficiency of the grinding process and result in a wider land area for pavements with softer aggregates. A soft aggregate specification in the range of 50-54 blades per foot is recommended.
• Groove height: ALDOT currently specifies a groove height of 0.03 in., which is half of the ACPA and IGGA-recommended dimension and half of what the majority of DOTs across the US require. An increased groove height is likely to improve retention of the diamond-ground texture that is essential to maintaining pavement friction, particularly for pavements with softer aggregates that may be more prone to wearing to a uniformly smooth surface. Because groove height is in part dependent on blade spacing in conventional diamond grinding, this parameter is difficult to control directly, but it may be indirectly improved for pavements containing softer aggregates by increasing the blade spacing as recommended above.

• Intentional grooving (negative texture) appears to significantly improve friction retention compared to conventional diamond grinding. When a pavement containing softer aggregates is diamond ground to improve smoothness, the additional use of longitudinal grooves, similar to those used in the NGCS textures, should be considered.

• NGCS textures should be considered, at least as an option, for future concrete pavement rehabilitation/preservation. Over a four year monitoring period, NGCS textures from the MnROAD demonstrated superior friction retention over conventional diamond grinding textures when friction is measured using a smooth tire. However, longer term (10 year or more) data is needed to determine whether this trend is sustained.

It should be noted that the ACPA and IGGA recommended grinding parameters have not been proven through research to improve long-term pavement friction; they do reduce finish production which creates a smoother and quieter ride.

Research is greatly needed to develop laboratory test methods that can simulate pavement wear and correlate these test results to field performance of pavements. Similarly, the use of test tracks such as the MnROAD site is of high value in the research and development of improved pavement surfaces and rehabilitation methods. On a test track, friction numbers need not be kept confidential, and they can be used to verify laboratory test methods and long-held industry assumptions regarding recommended diamond grinding procedures.

Because Alabama has not constructed pavements containing carbonate coarse aggregates for over 25 years, this situation presents an interesting opportunity for ALDOT. Under newly introduced life-cycle cost assumptions, significant rehabilitation of concrete pavements is not expected until an age of 30 years. Even allowing for large variations in performance, if carbonate coarse aggregates are immediately reintroduced as an approved material for concrete pavement construction in Alabama, these pavements will not undergo rehabilitation until 2035 or 2040. Therefore, there is sufficient time to conduct long-term studies to determine what grinding and/or grooving texture will provide the best combination of ride quality and safety.
6. Summary and Recommendations

6.1. Summary

The UA research team evaluated the concept of reintroducing carbonate coarse aggregates as approved materials for use in mainline concrete (rigid) pavements in Alabama. The researchers sought to determine whether the use of carbonate coarse aggregates can be beneficial to the performance of newly constructed pavements and if such pavements can be safely rehabilitated without leading to an unacceptable loss of friction. The team also identified existing knowledge gaps and research needs.

The investigation included reviews of existing literature, ongoing research on pavement construction and rehabilitation, and current specifications for highway construction in a number of states, with a focus on SASHTO member states and Texas. A survey of these states’ departments of transportation supplemented the literature and specification review. Finally, a parametric study evaluated the influence of joint spacing, pavement thickness, and coefficient of thermal expansion (CTE) on predicted pavement performance using AASHTO Pavement ME Design. The researchers also sought to determine which materials characterization tests would provide the best correlation to field performance in pavement applications.

The most significant findings of this investigation follow:

- Nine of eleven state DOTs responding to the survey allow carbonate coarse aggregates in mainline rigid pavements, including all four states bordering Alabama. Florida has used limestone from quarries near Calera, located south of Birmingham, and noted in their survey response their experience has shown this particular limestone to be more resistant to polishing than material from other (unnamed) geologic formations.
- Polishing resistance is rarely measured directly, if at all, and the Micro-Deval test may be able to best simulate the type of wear that aggregates at the pavement surface will experience in service.
- Unconfined freezing and thawing test methods directly evaluate an aggregate’s resistance to freezing and thawing, and are an alternative to sulfate soundness tests.
- The coefficient of thermal expansion (CTE) of concrete is dominated by the CTE of the aggregates, is rarely assessed, and is a major factor in pavement performance. Mid-slab cracking development is strongly correlated to CTE, with higher CTE concrete experiencing more rapid cracking. Low CTE concrete pavements are more resistant to cracking, joint faulting, and loss of smoothness. Limestone aggregates typically have lower CTE than most siliceous aggregates.
- Use of shorter joint spacings is a more cost-effective and efficient method to compensate for high CTE concrete than increasing the slab thickness.
• Adoption of MEPDG design procedures will allow pavement designers to better account for the influence of CTE and joint spacing.

• Use of optimized aggregate gradations specifically designed for slipform paving may lead to improvements in workability of the concrete, reduced drying shrinkage, improved durability, and reduced constructions costs. Recent research at Oklahoma State University may provide a viable model for implementation at ALDOT.

• Locked-wheel friction test results exhibit a stronger correlation to wet weather accidents when a smooth, rather than ribbed tire, is used. A 40 mph smooth tire friction number of 25 or higher was found in several studies to be sufficient to significantly reduce wet-weather accident rates.

• Diamond grinding of pavements exposes the coarse aggregate to traffic; if this is a carbonate aggregate, it may tend to wear at the same rate as the cement paste, leading to a loss of microtexture (polishing). Over time, this can reduce pavement friction to unacceptable levels. New grinding techniques that incorporate deeper grooves and wider land areas may provide sufficient macrotexture to offset the loss of microtexture in the land area between the grooves. Ongoing research in several states being coordinated by IGGA may help verify this hypothesis.

• Completed research examining the performance of different diamond grinding configurations on pavements with aggregates of varying hardness is very limited. Ongoing research studies in several states may provide more insight, but long-term performance will take years to evaluate.

• The need for diamond grinding may be minimized by quality construction that meets and exceeds ALDOT smoothness requirements, and by the use of design and construction methods to resist cracking and faulting during the life of the pavement.

6.2. Recommended Specification and Test Method Improvements

Based on the findings of this study, the researchers recommend that several improvements be included in the ALDOT Standard Specifications. These are listed below by specification section numbers:

Section 450.02 (b) 2. Portland Cement Concrete Pavement - Coarse Aggregate:

The researchers recommend that all coarse aggregates to be used at the riding surface of mainline and ramp concrete pavements be classified by performance in a test or combination of tests that evaluate polishing resistance, such as Micro-Deval or the British Pendulum Test. Coarse aggregates would then be classified as either polishing or non-polishing based on the test results. Polishing aggregates would be permitted only if blended with non-polishing aggregate in a ratio sufficient that the overall coarse aggregate blend meets the requirements for non-polishing
aggregate. For the lower lift of two-lift concrete pavements and concrete used in shoulder pavement, the researcher recommend no restriction on the use of coarse aggregates classified as polishing.

Because the researchers found that some siliceous aggregates may be as soft as or softer than some carbonate aggregates, it is further advised that the performance-based classification system apply to coarse aggregates of all mineralogies, not just carbonates.

A performance-based approach is already in place in Sections 423 and 424 of the ALDOT construction specifications (Stone Matrix Asphalt and Superpave Bituminous Concrete), with the percentage of carbonate stone allowed linked to the BPN9 value measured in the British Pendulum Test. The 2015 specifications from the Tennessee Department of Transportation (TDOT) use a similar approach that may serve as a model. Table 903.24-1 from the TDOT specifications is given below for reference:

![Table 903.24-1: Quality Requirements for Type I, II, III, and IV Aggregate](image)

In the TDOT specifications, the threshold for non-polishing aggregates is less stringent for lower-volume roads. Some further study is likely needed to determine appropriate threshold values for classification, but it may be appropriate to allow a percentage of carbonate coarse aggregate based on the tables in Sections 423 and 424 in the ALDOT specifications immediately.
450.02 (e) Concrete Mix Design

Because of the significant influence of CTE of pavement performance, particularly with respect to crack development, the researchers recommend that testing of CTE per AASHTO T 336 be required as part of the mix design approval process. The average CTE of samples made using the approved mix design may not exceed the equivalent AASHTO T 336 value assumed during the pavement design process if the pavement is designed following MEPDG procedures. The equivalent AASHTO T 336 value will be determined by subtracting $0.7 \times 10^{-6} \ 1/\sqrt{T}$ from TP 60 values used in the Pavement ME Design software. Using current practices in Texas a model, researchers recommend that ALDOT require the contractor to submit two 4 x 8 in. concrete cylinder samples cast from a trial batch to an ALDOT materials laboratory for CTE testing.

450.03 (a) Placement of Concrete:

Currently, this specification requires that concrete pavements be placed in one lift and that separate lifts of concrete will not be allowed. It is recommended that ALDOT consider revising this specification to allow the use of two-lift pavements during slipform paving operations.

455.03 (d) Grinding Concrete Pavement Surfaces - Final Surface Finish:

A revision of this specification should include a range of blade spacing values that should be used based upon the aggregate type that is in the concrete surface on the project. An increase in the groove height from $1/32$ of an inch to $1/16$ of an inch is also recommended. These changes will result in wider land areas and more groove height for pavements with soft aggregate compared to the current specifications. It is recommended that the specification be changed to read as follows:

“Said line type texture shall consist of parallel longitudinal corrugations of approximately **50 to 60 evenly spaced grooves per foot** {300 mm} with the **ridges approximately 1/16 of an inch** {1.6 mm} higher than the bottom of the grooves. **Select the number of grooves per foot (meter) based on coarse aggregate type that is in the concrete surface on the project from the table below.**”

<table>
<thead>
<tr>
<th></th>
<th>Soft Aggregate</th>
<th>Hard Aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grooves per foot (per meter)</td>
<td>50 to 54 {164 to 177}</td>
<td>55 to 60 {180 to 197}</td>
</tr>
<tr>
<td>Groove height</td>
<td>1/16 in. (1.6 mm)</td>
<td></td>
</tr>
</tbody>
</table>
453.05 (a) 5. Rideability Acceptance

This section also contains language regarding diamond grinding of concrete pavements in the second-to-last paragraph. It is recommended that the same language and table given above for section 455.03 (d) also be inserted here.

Lastly, the researchers recommend ALDOT add a reference to AASHTO M 286-96 (Standard Specification for Smooth-Tread Standard Tire for Special-Purpose Pavement Frictional-Property Tests) to the ALDOT-401-04 test procedure (Pavement Friction Testing Procedure), and require that the test be conducted using both the AASHTO M 286 (smooth) and M 261 (ribbed) tires on limited basis. A proposed course of action to assess the value of test data generated with each tire is discussed in the next section.

6.3. Research Priorities

Throughout this report, the researchers have identified several opportunities for research to advance the state of the art in pavement design, construction, and rehabilitation. This section will present the top five research priorities.

- Performance-based Classification of Coarse Aggregates

Rather than qualifying or disqualifying aggregates for use in concrete pavements by mineralogy or rock type (e.g. gravel, limestone, granite), the researchers propose that ALDOT support the development and implementation of a new classification scheme based on performance in several tests. In addition to preventing the exclusion of harder carbonate aggregates from use in pavement construction, this system would ensure that softer siliceous materials are identified and classified appropriately.

The Micro-Deval test (AASHTO T 327) is proposed as a performance test to classify aggregates as polishing or non-polishing, based on the percentage loss during the test. Although this test method is not listed in the standard specifications at present, ALDOT has been collecting Micro-Deval data on its approved aggregate sources for some time, and the data will be very beneficial to this process.

Tennessee has implemented a tiered system for aggregates used in the riding surface of concrete pavements that is dependent on traffic volume that could be a model for a new classification system to be used by ALDOT. Other test data such as BPN9 (AASHTO T 279), and acid-insoluble residue (ASTM D 3042) would be considered as well, and some combination of tests is likely to be recommended, rather than a single test method.
**Validation of IGGA Recommended Blade Spacing**

While research on diamond grinding concrete pavements has uncovered some interesting results, there are some areas that need further investigation. One such area is the validation of the recommended blade spacing for concrete pavements with soft or hard aggregates. These recommendations were provided to optimize pavement texture by reducing the number of fins left after the grinding operations. However, the effect on short-term and long-term skid resistance using the recommended spacings has not been validated with research, particularly with respect to softer aggregates like limestone and dolomite. The research approach provided in Chapter 5 provides a method that could validate the recommended spacing through laboratory testing.

There is also a great need to correlate laboratory test results of diamond ground pavements with simulated pavement wear to field performance of pavements. Correlation of the laboratory test results with pavements subjected to real-world traffic conditions, either by the use of test sections on the highway system or by test tracks such as MnROAD, will provide validation of the laboratory testing methods. Such tests could allow for greater correlation of data gathered by small-scale testing methods with the data gathered from full-scale testing methods such as the locked wheel skid trailer.

**Grooving**

In addition to research needs on diamond grinding, it is recommended that research on diamond grooved concrete pavements be conducted in parallel. Similar to the laboratory and field testing of the IGGA recommended blade spacing, laboratory and field research can be conducted on the how well diamond grooved pavements perform over short-term and long-term periods in terms of texture and friction retention. Other performance characteristics of the grooved surface, such as the effect of the grooves on the ride quality of small vehicles and motorcycles, can be evaluated on the field test sections.

**Optimized Aggregate Gradation for Pavements**

The combined aggregate gradation methods currently in use by ALDOT for concrete mixture design may not be optimized for pavement concrete placed by slipforming, which has particular workability requirements. It is recommended that ALDOT consider adopting the recent advances made at Oklahoma State University, including a modified version of 8-18 grading, and the use of the Box Test to validate the workability of the resulting mixtures.

A small research project including a field demonstration project would serve to validate these principles, and modifications to the construction specifications and pavement mixture design procedures would be the final product of such a project. This will enable a reduction in
construction costs, cementitious materials content, and drying shrinkage of future concrete pavements.

- **Smooth vs. Ribbed Tire Correlation**

  Locked-wheel pavement friction testing using smooth tires provides greater correlation to wet weather accidents, and it is recommended that ALDOT consider incorporating the AASHTO M 286 smooth tire into their pavement friction testing procedures (ALDOT-401-04). As stated in the summary of Chapter 4, friction numbers obtained from locked-wheel friction testing using ribbed and smooth tires should be evaluated to examine the relationship between the two tests and to determine if the smooth tire friction numbers provide better correlation to wet-weather accidents.

  A three-year evaluation period is recommended, during which smooth and ribbed tire testing would be conducted annually on a randomly-selected fraction of the interstate highway network in Alabama. At the end of the evaluation period, an acceptable lower limit for the FN obtained using a smooth tire can be determined and should provide ALDOT with sufficient data to make the decision to maintain the current pavement friction testing procedures or update the procedure to include the use of the smooth tire during locked-wheel friction testing.

- **Two-Lift Pavements**

  FHWA is strongly supporting more widespread use of two-lift paving in the US. Therefore ALDOT could apply for funding for a demonstration project. Such a project would be most valuable if the pavement can be instrumented and monitored to help determine the impact of using materials with different physical, mechanical, and thermal properties in the upper and lower lifts.

  While it is desirable from a construction cost perspective to use lower cost materials such as recycled concrete aggregate in the lower layer, such materials will result in concrete with different drying shrinkage, flexural strength, stiffness, and thermal expansion characteristics than the upper layer. It is important to assess how much variation in material properties between the two layers can be tolerated and still meet the goals of longer service life and sustainability. A more detailed review is needed of the existing literature and ongoing research in order to identify the most significant knowledge gaps related to the materials and mechanistic performance of two-lift pavements.
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