EFFECT OF MIXTURE COMPONENT CHARACTERISTICS ON PROPERTY AND PERFORMANCE OF SUPERPAVE MIXTURES

By

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A DISSERTATION PRESENTED TO THE GRADUATE SCHOOL OF THE UNIVERSITY OF FLORIDA IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY

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To my beloved wife, Eunsong Lee and lovely son, Woobin Chun
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# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACKNOWLEDGMENTS</td>
<td>4</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>8</td>
</tr>
<tr>
<td>LIST OF FIGURES</td>
<td>9</td>
</tr>
<tr>
<td>LIST OF ABBREVIATIONS</td>
<td>12</td>
</tr>
<tr>
<td>ABSTRACT</td>
<td>14</td>
</tr>
<tr>
<td>CHAPTERS</td>
<td></td>
</tr>
<tr>
<td>1 INTRODUCTION</td>
<td>16</td>
</tr>
<tr>
<td>1.1 Background</td>
<td>16</td>
</tr>
<tr>
<td>1.2 Hypothesis</td>
<td>18</td>
</tr>
<tr>
<td>1.3 Objectives</td>
<td>19</td>
</tr>
<tr>
<td>1.4 Scope</td>
<td>19</td>
</tr>
<tr>
<td>1.5 Research Approach</td>
<td>20</td>
</tr>
<tr>
<td>2 CHARACTERIZATION OF MIXTURE GRADATION AND RESULTING VOLUMETRIC PROPERTIES (DOMINANT AGGREGATE SIZE RANGE - INTERSTITIAL COMPONENT (DASR-IC) MODEL)</td>
<td>22</td>
</tr>
<tr>
<td>2.1 Background</td>
<td>22</td>
</tr>
<tr>
<td>2.2 Dominant Aggregate Size Range (DASR)</td>
<td>22</td>
</tr>
<tr>
<td>2.3 DASR Porosity</td>
<td>23</td>
</tr>
<tr>
<td>2.4 Interstitial Component (IC) of Mixture Gradation</td>
<td>24</td>
</tr>
<tr>
<td>2.5 Disruption Factor (DF)</td>
<td>25</td>
</tr>
<tr>
<td>2.6 Effective Film Thickness (EFT)</td>
<td>26</td>
</tr>
<tr>
<td>2.7 Ratio between Coarse Portion and Fine Portion of Fine Aggregates</td>
<td>29</td>
</tr>
<tr>
<td>(CFA/FFA)</td>
<td>30</td>
</tr>
<tr>
<td>2.8 Summary</td>
<td></td>
</tr>
<tr>
<td>3 IMPLEMENTATION OF BINDER AND MIXTURE TESTS ON FIELD CORES FOR SUPERPAVE MIXTURES IN FLORIDA</td>
<td>31</td>
</tr>
<tr>
<td>3.1 Background</td>
<td>31</td>
</tr>
<tr>
<td>3.2 Binder Recovery and Binder Tests</td>
<td>31</td>
</tr>
<tr>
<td>3.2.1 Binder Recovery</td>
<td>32</td>
</tr>
<tr>
<td>3.2.2 Penetration Test</td>
<td>32</td>
</tr>
<tr>
<td>3.2.3 Viscosity Test</td>
<td>33</td>
</tr>
<tr>
<td>3.2.4 Dynamic Shear Rheometer Test (DSR)</td>
<td>35</td>
</tr>
<tr>
<td>3.2.5 Bending Beam Rheometer Test (BBR)</td>
<td>37</td>
</tr>
</tbody>
</table>
3.2.6 Multiple Stress Creep Recovery Test (MSCR) .................................................. 39
3.3 Mixture Tests ........................................................................................................... 42
  3.3.1 Test Specimen Preparation .................................................................................. 42
    3.3.1.1 Measuring, Cataloguing, and Inspecting ....................................................... 42
    3.3.1.2 Cutting ........................................................................................................... 43
    3.3.1.3 Gage Points Attachment ............................................................................. 44
  3.3.2 Test Procedure ..................................................................................................... 45
    3.3.2.1 Resilient Modulus Test .............................................................................. 45
    3.3.2.2 Creep Test .................................................................................................... 46
    3.3.2.3 Tensile Strength Test ................................................................................ 48
  3.3.3 Superpave IDT Test Results ................................................................................ 50
  3.3.4 Moisture-Damaged Projects ............................................................................. 58
3.4 Summary .................................................................................................................... 63

4 EVALUATION OF FIELD MIXTURE PERFORMANCE USING DASR-IC
MODEL PARAMETERS ...................................................................................................... 64
  4.1 Background .............................................................................................................. 64
  4.2 Implementation of Gradation Analysis for Superpave Mixtures ............................. 64
  4.3 Evaluation of Gradation Analysis Results for Superpave Projects ......................... 67
  4.4 Evaluation of Field Mixture Performance .............................................................. 69
    4.4.1 Field Performance: Rutting ............................................................................ 70
    4.4.2 Field Performance: Cracking .......................................................................... 74
  4.5 Summary .................................................................................................................. 78

5 IDENTIFICATION OF PREDICTIVE MIXTURE PROPERTY RELATIONSHIPS
AND MODEL DEVELOPMENT ......................................................................................... 80
  5.1 Background .............................................................................................................. 80
  5.2 Existing Material Property Models in the HMA-FM-E Model .................................. 81
    5.2.1 AC Stiffness Aging Sub-Model ..................................................................... 81
    5.2.2 Fracture Energy Limit Aging Sub-Model ....................................................... 83
  5.3 Key Elements for Material Property Relationships ................................................ 85
    5.3.1 DASR-IC Model Parameters ...................................................................... 86
    5.3.2 Initial Material Properties ............................................................................ 87
    5.3.3 Factors Related to Non-Healable Permanent Damage .................................... 88
  5.4 Development of Predictive Material Property Relationships .................................. 88
    5.4.1 Relationships for Initial Material Properties .................................................. 89
      5.4.1.1 Initial Fracture Energy Relationship ....................................................... 90
      5.4.1.2 Initial Creep Rate Relationship .............................................................. 92
    5.4.2 Models for Changes in Material Properties .................................................... 98
      5.4.2.1 AC Stiffness Model .............................................................................. 99
      5.4.2.2 Fracture Energy Limit Model ............................................................... 100
  5.5 Summary .................................................................................................................. 103
6 EVALUATION OF DASR-IC CRITERIA USING HMA-FM-E MODEL .......... 104

6.1 Background ........................................................................................................ 104
6.2 Enhanced HMA Fracture Mechanics Based Model (HMA-FM-E Model) ........ 104
6.3 Input Module ...................................................................................................... 105
6.4 Model Prediction Results ..................................................................................... 108
6.5 Relationships between DASR-IC Criteria and Model Prediction Results ...... 110
6.6 Summary ............................................................................................................ 113

7 CLOSURE .............................................................................................................. 114

7.1 Summary and Findings ....................................................................................... 114
7.2 Conclusions ........................................................................................................ 115
7.3 Recommendations and Future Works ................................................................. 116

APPENDIX: DETERMINISTIC PROCEDURE FOR ESTIMATION OF CRACK INITIATION TIME BASED ON CRACK RATING DATA .............................................. 118

LIST OF REFERENCES ............................................................................................ 121

BIOGRAPHICAL SKETCH ....................................................................................... 124
<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>Mixture information of Superpave projects evaluated</td>
<td>20</td>
</tr>
<tr>
<td>3-1</td>
<td>Asphalt binder used for Superpave projects evaluated</td>
<td>31</td>
</tr>
<tr>
<td>3-2</td>
<td>Mixture information for 11 Superpave projects</td>
<td>50</td>
</tr>
<tr>
<td>3-3</td>
<td>Project information for moisture-damaged sections</td>
<td>59</td>
</tr>
<tr>
<td>4-1</td>
<td>Mixture information of Superpave projects analyzed</td>
<td>65</td>
</tr>
<tr>
<td>4-2</td>
<td>DASR-IC parameters calculated for Superpave projects</td>
<td>65</td>
</tr>
<tr>
<td>4-3</td>
<td>Crack initiation time and cracking status determined for Superpave projects</td>
<td>75</td>
</tr>
<tr>
<td>6-1</td>
<td>Summary of input data characteristics for the HMA-FM-E model</td>
<td>106</td>
</tr>
<tr>
<td>6-2</td>
<td>Data used for model prediction</td>
<td>107</td>
</tr>
<tr>
<td>6-3</td>
<td>Predicted top-down cracking performance using HMA-FM-E model</td>
<td>109</td>
</tr>
</tbody>
</table>
### LIST OF FIGURES

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-1</td>
<td>DASR and IC for three different types of mixture (After Kim et al., 2006)</td>
<td>17</td>
</tr>
<tr>
<td>1-2</td>
<td>Overall research approach flowchart</td>
<td>21</td>
</tr>
<tr>
<td>2-1</td>
<td>Mixture components for calculation of DASR porosity (Kim et al., 2006)</td>
<td>24</td>
</tr>
<tr>
<td>2-2</td>
<td>Configuration of different DF values (Guarin, 2009)</td>
<td>26</td>
</tr>
<tr>
<td>2-3</td>
<td>Effective film thickness vs. theoretical film thickness</td>
<td>27</td>
</tr>
<tr>
<td>2-4</td>
<td>Conceptual drawing of film thickness effect</td>
<td>28</td>
</tr>
<tr>
<td>2-5</td>
<td>Determination of CFA/FFA</td>
<td>29</td>
</tr>
<tr>
<td>3-1</td>
<td>Penetration test results for Superpave projects</td>
<td>33</td>
</tr>
<tr>
<td>3-2</td>
<td>Viscosity test results for Superpave projects</td>
<td>34</td>
</tr>
<tr>
<td>3-3</td>
<td>Change in viscosity with aging for Superpave projects</td>
<td>35</td>
</tr>
<tr>
<td>3-4</td>
<td>$G'\cdot\sin\delta$, 10 rad/sec at 25 °C (77 °F) for Superpave projects</td>
<td>36</td>
</tr>
<tr>
<td>3-5</td>
<td>$S(t)$, 60 seconds loading time at -12 °C (10.4 °F) for Superpave projects</td>
<td>38</td>
</tr>
<tr>
<td>3-6</td>
<td>$m$-value, 60 seconds loading time at -12 °C (10.4 °F) for Superpave projects</td>
<td>38</td>
</tr>
<tr>
<td>3-7</td>
<td>MSCR average recovery at 64 °C (147.2 °F) for Superpave projects</td>
<td>40</td>
</tr>
<tr>
<td>3-8</td>
<td>MSCR nonrecoverable compliance at 64 °C (147.2 °F) for Superpave projects</td>
<td>41</td>
</tr>
<tr>
<td>3-9</td>
<td>Measuring, cataloguing, and inspecting work for field cores</td>
<td>43</td>
</tr>
<tr>
<td>3-10</td>
<td>Cut specimens for Superpave IDT tests</td>
<td>43</td>
</tr>
<tr>
<td>3-11</td>
<td>Cutting machine used in this study</td>
<td>44</td>
</tr>
<tr>
<td>3-12</td>
<td>Gage points attachment</td>
<td>44</td>
</tr>
<tr>
<td>3-13</td>
<td>Superpave IDT tests</td>
<td>45</td>
</tr>
<tr>
<td>3-14</td>
<td>Power model of creep compliance</td>
<td>47</td>
</tr>
<tr>
<td>3-15</td>
<td>Determination of fracture energy and dissipated creep strain energy to failure</td>
<td>49</td>
</tr>
<tr>
<td>3-16</td>
<td>Change in resilient modulus over time</td>
<td>52</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Description</td>
<td></td>
</tr>
<tr>
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<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
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<tr>
<td>AC</td>
<td>Asphalt Concrete</td>
<td></td>
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<tr>
<td>APT</td>
<td>Accelerated Pavement Testing</td>
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<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
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<tr>
<td>BBR</td>
<td>Bending Beam Rheometer</td>
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<tr>
<td>CFA/FFA</td>
<td>Ratio of Coarse Fine Aggregate to Fine Fine Aggregate</td>
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<tr>
<td>DASR</td>
<td>Dominant Aggregate Size Range</td>
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<td>DF</td>
<td>Disruption Factor</td>
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<tr>
<td>DSR</td>
<td>Dynamic Shear Rheometer</td>
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<tr>
<td>EAC</td>
<td>Effective Asphalt Content</td>
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<tr>
<td>EFT</td>
<td>Effective Film Thickness</td>
<td></td>
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<tr>
<td>ESAL</td>
<td>Equivalent Single Axle Load</td>
<td></td>
</tr>
<tr>
<td>FDOT</td>
<td>Florida Department of Transportation</td>
<td></td>
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<tr>
<td>FWD</td>
<td>Falling Weight Deflectometer</td>
<td></td>
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<tr>
<td>JMF</td>
<td>Job-Mix-Formula</td>
<td></td>
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<tr>
<td>HMA</td>
<td>Hot-Mix Asphalt</td>
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<tr>
<td>HMA-FM-E</td>
<td>Enhanced Hot-Mix Asphalt Fracture Mechanics Based Model</td>
<td></td>
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<tr>
<td>IA</td>
<td>Independent Assurance</td>
<td></td>
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<tr>
<td>IC</td>
<td>Interstitial Component</td>
<td></td>
</tr>
<tr>
<td>IDT</td>
<td>Indirect Tension Test</td>
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<tr>
<td>ITLT</td>
<td>Indirect Tension Test at Low Temperatures</td>
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</tr>
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<td>IV</td>
<td>Interstitial Volume</td>
<td></td>
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<tr>
<td>MEPDG</td>
<td>Mechanistic Empirical Pavement Design Guide</td>
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<td>MSCR</td>
<td>Multiple Stress Creep Recovery</td>
<td></td>
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<tr>
<td>Acronym</td>
<td>Description</td>
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<tr>
<td>NCHRP</td>
<td>National Cooperative Highway Research Program</td>
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<tr>
<td>PAV</td>
<td>Pressure Aging Vessel</td>
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<tr>
<td>PG</td>
<td>Performance Grade</td>
<td></td>
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<tr>
<td>QA</td>
<td>Quality Assurance</td>
<td></td>
</tr>
<tr>
<td>QC</td>
<td>Quality Control</td>
<td></td>
</tr>
<tr>
<td>TCE</td>
<td>Trichloroethylene</td>
<td></td>
</tr>
<tr>
<td>TF</td>
<td>Theoretical Film Thickness</td>
<td></td>
</tr>
<tr>
<td>UF</td>
<td>University of Florida</td>
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<tr>
<td>VMA</td>
<td>Voids in Mineral Aggregate</td>
<td></td>
</tr>
</tbody>
</table>
This study was conducted to evaluate the effect of mixture component characteristics (i.e. DASR and IC) on properties and performance of Superpave mixtures, specializing in the development of a set of implementable gradation and volumetric criteria, and Hot-Mix-Asphalt (HMA) mixture property predictive relationships based on mixture component characterization.

Four DASR-IC model parameters including DASR porosity, DF, EFT, and CFA/FFA have formed the DASR-IC criteria to effectively address the two primary components (i.e. DASR and IC) of asphalt mixtures that play a major role on properties and performance. Field performance evaluation of different Superpave mixtures was conducted to identify the relationships between the four DASR-IC parameters and field performance. Based on results analyzed, it was found that the introduction of DASR-IC criteria as performance-related design parameters to current mix design guidelines and specifications will lead to better and more consistent field rutting and cracking performance of Superpave mixtures.

In addition, the DASR-IC criteria will also provide a more rational method to consider the effect of DASR and IC on mixture behavior which strongly affects HMA
fracture properties. Therefore, it is expected that this criteria will have a potential to identify the effect of mixture gradation and volumetric characteristics on mixture fracture properties which is more reliably related to performance of asphalt mixtures. Relationships able to predict initial fracture energy and creep rate, which are the properties known to govern the change in material property over time and are also required for performance model predictions, were developed in this study.

Furthermore, conceptual relationships were identified to describe changes in these properties over time (aging) by including the effect of the non-healable permanent damage related to load and moisture. This can serve as the foundation for further development of improved models to predict mixture properties as a function of age in the field based on additional field data and laboratory studies using more advanced laboratory conditioning procedures. The verified relationships will also serve to provide reliable inputs for prediction of service life using pavement performance prediction models.
1.1 Background

It is now generally agreed that aggregate gradation is one of the most important factors that affects the properties and performance of asphalt mixtures. Having suitable gradation characteristics including appropriate aggregate particle size distribution and resulting volumetric properties is obviously important to ensure good field mixture performance. Therefore, aggregate related parameters have been studied to identify their effects on observed field performance of asphalt mixtures. Although different parameters, including effective film thickness and other volumetric parameters were found to affect mixture performance, consensus has not been reached regarding rational design guidelines and criteria, especially as related to the selection of the best aggregate blend to achieve optimal performance.

The Superpave field monitoring project recently completed at the University of Florida has determined that existing mix design criteria included in Superpave system such as Voids in Mineral Aggregate (VMA), gradation control points, and effective asphalt content do not capture all critical aspects of gradation and resulting volumetric properties found to be most strongly related to field mixture performance (Roque et al., 2011). Thereby, Superpave mixture performance varied significantly among mixtures that met all existing design and construction specification criteria. Therefore, there was a need to identify and verify additional criteria that can assure better and more consistent Superpave mixture performance. It was also found that differences in performance could not explained by differences in binder properties between mixtures.
It appeared that differences in performance were primarily controlled by differences in gradation and resulting volumetric properties between mixtures.

According to previous work conducted by University of Florida researchers, gradation characteristics of mixture can be expressed by separating the gradation into two major components: Dominant Aggregate Size Range (DASR) and Interstitial Component (IC) (After Kim et al., 2006). It has been shown that parameters describing the characteristics of these components, which were determined based on packing theory and particle size distributions, seemed to be well correlated to mixture performance. Kim et al. (2006) indicated in their research that porosity can be used as a criterion to ensure contact between DASR particles within the asphalt mixture to provide adequate interlocking and resistance to deformation and fracture. The work has clearly shown that DASR porosity can be used as an indicator which reflects the characteristics of coarse aggregate structure. The schematic of the DASR and IC concept for three different types of mixtures is illustrated in Figure 1-1.

![DASR and IC schematic](image)

(a) SMA Mixture     (b) Coarse Dense Mixture     (c) Fine Dense Mixture

Figure 1-1. DASR and IC for three different types of mixture (After Kim et al., 2006)

The work has also concluded that properties and characteristics of IC will strongly influence rutting and cracking resistance of asphalt mixtures. For the purpose of IC
characterization, a new parameter called Disruption Factor (DF) was conceived and developed by Guarin (2009) to evaluate the potential of IC particles to disrupt the DASR structure. It was found that DF appeared to be one good indicator to describe IC characteristics with respect to volumetric distribution of IC. However, DF only considers volumetric distribution of IC to determine the potential of the finer portion of the mixture’s gradation (i.e., IC) to disrupt the DASR structure. Therefore, there was a need to identify and develop additional parameters to more effectively characterize the IC of asphalt mixture with regard to stiffening effect of IC on mixture and structure of IC.

1.2 Hypothesis

Since there was a need to address more aspects of IC characteristics to better capture the effects of IC on key mixture properties and expected performance, two new parameters, effective film thickness (EFT) and ratio of coarse fine aggregate to fine fine aggregate (CFA/FFA), were added in addition to DF in this study for further IC characterization. It was expected that these two parameters will provide a more rational way to identify mixture behavior as mastic which is considered to strongly affect Hot-Mix Asphalt (HMA) fracture properties. Therefore, the addition of EFT and CFA/FFA will give a more potential to identify the effect of IC characteristics on properties which is more reliably related to performance of asphalt mixtures. The following two hypotheses were made in this study.

- Interstitial component (IC) characteristics related to the aggregate structure and binder distribution within the IC have an important effect on mixture fracture properties as well as on pavement cracking performance.

- Gradation and volumetric parameters that effectively characterize the DASR and IC can be used to predict mixture fracture properties.
1.3 Objectives

The primary objective of this study is to evaluate the effect of mixture component characteristics (i.e., DASR and IC) on properties and performance of Superpave mixtures. Detailed objectives are summarized as follows.

- Further develop the DASR-IC criteria to more effectively characterize both coarse (DASR) and fine (IC) portions of mixture gradation and resulting volumetric properties that play a major role on mixture performance.
- Evaluate and verify the DASR-IC criteria as an effective and implementable set of gradation and volumetric criteria for mixture design and construction specification that can help to assure better and more consistent field mixture performance.
- Identify key mixture component characteristics associated with mixture fracture properties and changes in these properties (i.e., fracture energy and creep rate) over time.
- Develop predictive relationships for mixture fracture properties, specifically fracture energy and creep rate, which have shown to strongly affect (or influence) pavement cracking performance in the field.
- Identify and further develop improved forms of the HMA fracture property aging model in an effort to more accurately predict pavement cracking performance.

1.4 Scope

Eleven Superpave monitoring project field sections were evaluated including different types of gradation, aggregate, and asphalt binder. It is noted that fairly wide ranges of Superpave mixture were evaluated in this study. All mixture data were obtained or determined from field cores including gradation, binder properties, volumetric properties, and mixture properties. The standard University of Florida (UF) Superpave Indirect Tension Tests (IDT) were performed at 10 °C and 20 °C, to obtain the HMA fracture properties of the different Superpave mixtures used for evaluation. Enhanced Hot-Mix Asphalt Fracture Mechanics Based Model (HMA-FM-E) was
employed to evaluate criteria developed as a performance prediction model. Table 1-1
summarizes the mixture information of Superpave projects evaluated in this study.

<table>
<thead>
<tr>
<th>Project (UF) ID</th>
<th>Aggregate Type</th>
<th>Binder Type</th>
<th>Mixture Type</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Top</td>
<td>Bottom</td>
<td>Top</td>
</tr>
<tr>
<td>1</td>
<td>Granite</td>
<td>PG 67-22</td>
<td>PG 67-22</td>
</tr>
<tr>
<td>2</td>
<td>Granite</td>
<td>PG 64-22</td>
<td>PG 64-22</td>
</tr>
<tr>
<td>3</td>
<td>Limestone</td>
<td>PG 67-22</td>
<td>PG 67-22</td>
</tr>
<tr>
<td>4</td>
<td>Limestone</td>
<td>PG 67-22</td>
<td>PG 67-22</td>
</tr>
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<td>6</td>
<td>Limestone</td>
<td>PG 64-22</td>
<td>N/A</td>
</tr>
<tr>
<td>7</td>
<td>Limestone</td>
<td>PG 64-22</td>
<td>PG 64-22</td>
</tr>
<tr>
<td>8</td>
<td>Limestone</td>
<td>PG 76-22</td>
<td>PG 76-22</td>
</tr>
<tr>
<td>9</td>
<td>Granite</td>
<td>ARB-5</td>
<td>PG 64-22</td>
</tr>
<tr>
<td>10</td>
<td>Granite</td>
<td>ARB-5</td>
<td>PG 64-22</td>
</tr>
<tr>
<td>11</td>
<td>Granite</td>
<td>PG 76-22</td>
<td>PG 64-22</td>
</tr>
<tr>
<td>12</td>
<td>Granite</td>
<td>ARB-5</td>
<td>PG 64-22</td>
</tr>
</tbody>
</table>

Note: Mixture Type: C = Coarse Mixtures, F = Fine Mixtures, N/A = Not Applicable

1.5 Research Approach

This research is mainly focused on evaluating the effect of mixture component
characteristics on properties and performance of Superpave mixtures in order to
develop more implementable performance-related criteria and predictive mixture
property relationships. The overall research approach to accomplish the objectives of
this study is shown in Figure 1-2. Details for each phase of this research are described
in the following sections.

- Development of DASR-IC criteria: (1) Identify key mixture component
  characteristics; (2) Identify and develop parameters to effectively characterize
  mixture component characteristics identified; (3) Evaluate relationships between
  parameters identified and field mixture performance; (4) Develop implementable
  performance-related criteria using parameters evaluated (i.e., DASR-IC criteria).

- Development of predictive mixture property relationships: (1) Identify key mixture
  properties to be evaluated; (2) Evaluate relationships between properties identified
  and mixture performance; (3) Evaluate relationships between parameters
  employed in DASR-IC criteria and mixture fracture properties; (4) Identify key
  elements associated with initial mixture properties and changes in these properties
over time (aging); (5) Develop predictive relationships for initial mixture properties; (6) Develop improved forms of mixture property aging model.

- Evaluation of criteria developed: (1) Evaluate criteria developed using performance prediction model (HMA-FM-E model); (2) Validate and refine criteria developed using additional field and laboratory data.

Figure 1-2. Overall research approach flowchart
CHAPTER 2
CHARACTERIZATION OF MIXTURE GRADATION AND RESULTING VOLUMETRIC PROPERTIES (DOMINANT AGGREGATE SIZE RANGE-INTERSTITIAL COMPONENT (DASR-IC) MODEL)

2.1 Background

Research recently conducted at the University of Florida has concluded that the gradation of mixtures can be characterized by separating the gradation into two major components: The Dominant Aggregate Size Range (DASR) and the Interstitial Components (IC) (Kim et al., 2006, Guarin, 2009, Roque et al., 2011). It was also shown that parameters describing the characteristics of two components, which were determined based on packing theory and particle size distributions, seemed to be well correlated to mixture performance. These parameters are DASR porosity, Disruption Factor (DF), Effective Film Thickness (EFT), and ratio of Coarse Fine Aggregate to Fine Aggregate (CFA/FFA), which are used to address the following aspects of gradation characteristics:

- DASR porosity: coarse aggregate interlocking
- Disruption Factor (DF): volumetric distribution of the IC
- Effective Film Thickness (EFT): stiffening effect of IC on mixture
- CFA/FFA: structure of the IC

Detailed descriptions with regard to the definition and calculation procedure of each parameter are included in the following sections.

2.2 Dominant Aggregate Size Range (DASR)

The concept and theoretical development of DASR, which was defined as the interactive range of particle sizes that forms the dominant structural network of aggregate, was introduce by Kim et al. (2006). According to the DASR approach, there is an interactive range of particle sizes that primarily contributes to aggregate interlocking in asphalt mixtures. Particle sizes interacting with each other will form the
primary structure to resist deformation and fracture. Particle sizes smaller than the DASR will serve to fill the voids between DASR particles, called the interstitial volume. The IC particles combined with binder form a secondary structure to help resist deformation and fracture, and it is the primary source of adhesion and resistance to tension. Particle sizes larger than the DASR will simply float in the DASR matrix and will not play a major role in the aggregate structure.

The DASR, which is determined by conducting particle interaction analysis based on packing theory, can be composed of one size or multiple sizes. It was concluded that the DASR should be composed of coarse enough particles, and that all contiguous particle sizes determined to be interactive can be considered as part of the DASR.

2.3 DASR Porosity

Porosity has been widely used in the field of soil mechanics as a dimensionless parameter that indicates the relative ratio of voids to total volume. It has been determined that the porosity of granular materials should be no greater than 50 % for particles to have contact with each other (i.e. to be interactive) (Lambe and Whitman, 1969). Research conducted by Kim et al. (2006) indicated that porosity can be used as a criterion to ensure contact between DASR particles within the asphalt mixture to provide adequate interlocking and resistance to deformation and fracture.

The basic principles related to the calculation of DASR porosity are as follows. The Voids in Mineral Aggregate (VMA) of asphalt mixtures, which indicates the volume of available space between aggregates in a compacted mixture, is comparable to volume of voids in soil. Porosity can be calculated for any DASR by assuming that a mixture has certain effective asphalt content and air voids (i.e. VMA) for a given gradation (Figure 2-1). Finally DASR porosity can be calculated using Equation 2-1.
2.4 Interstitial Component (IC) of Mixture Gradation

As illustrated in Figure 1-1, the interstitial component is the material including asphalt, fine aggregates, and air voids that exists within the interstices of the DASR, and volume of this material is considered as the interstitial volume (IV). Research conducted by Guarin (2009) concluded that properties and characteristics of the IC will strongly influence the rutting and cracking resistance of asphalt mixtures.

The IC should fill the voids within the aggregates larger than the IC without disrupting the DASR structure. As the DASR-IC model assumes that the particles bigger than the DASR are floating in the DASR structure, it would be reasonable to accept that

$$\eta_{DASR} = \frac{V_{V(DASR)}}{V_{T(DASR)}} = \frac{V_{ICAGG} + V_{MA}}{V_{TM} - V_{AGG>DASR}}$$

Where, $\eta_{DASR} = DASR$ porosity, $V_{V(DASR)} = \text{volume of voids within DASR}$, $V_{T(DASR)} = \text{total volume available for DASR particles}$, $V_{ICAGG} = \text{volume of IC aggregates}$, $V_{MA} = \text{voids in mineral aggregate}$, $V_{TM} = \text{total volume of mixture}$, $V_{AGG>DASR} = \text{volume of particles bigger than DASR}$. 

Figure 2-1. Mixture components for calculation of DASR porosity (Kim et al., 2006)
the effect of the DASR voids structure could be utilized to evaluate the total voids structure for the IC including the particles bigger than the DASR. Information on the IC characteristics is fundamental to understand and predict how the IC will fit into the IV and consequently to determine whether the DASR structure would be disrupted by the IC. The characteristics of the IC are expected to have a strong influence on key mixture properties including fracture energy and creep rate, as well as property changes due to aging. Therefore, it was expected that DASR-IC parameters would correlate well with the mixture performance, including rutting and cracking.

2.5 Disruption Factor (DF)

A new parameter called the Disruption Factor (DF) was conceived and developed by Guarin (2009) to determine the potential of the finer portion of the mixture’s gradation to disrupt the DASR structure. It was shown in laboratory studies that the DF can effectively evaluate the potential of IC particles to disrupt the DASR structure. DF can be calculated using the following equation.

\[
DF = \frac{Volume \ of \ potentially \ distruptive \ IC \ particles}{Volume \ of \ DASR \ voids}
\]  

(2-2)

Guarin also proffered an optimal DF range to attain better rutting and cracking performance of asphalt mixture. According to the DF approach, the IC aggregates would not be involved in transmitting load between the DASR aggregates if the DF is low. In this case, the DASR structure would get no additional support or benefit from the IC particles. In the case of high DF, mixture performance would be negatively affected because the DASR structure would be disrupted by the IC aggregates. Lastly, if the DF is in the optimal range, better mixture performance would be expected because the IC aggregates will be involved in resisting shear stresses with the DASR structure.
Therefore, it is expected that the DF will appear to be one good indicator to describe the IC characteristics with respect to the volumetric distribution of IC particles, and a link between the DF and material properties which are related to the performance of asphalt mixtures. Figure 2-2 is a pictorial representation of the different configurations of DF values: low, optimal, and high.

![Figure 2-2. Configuration of different DF values (Guarin, 2009)](image)

2.6 Effective Film Thickness (EFT)

The film thickness of asphalt mixtures has been used to help explain aging phenomena, and many researchers have attempted to evaluate the relationship between film thickness and mixture performance. Kandhal and Chakraborty (1996) have shown that this parameter can be utilized as an indicator to characterize the durability and fatigue resistance of asphalt mixtures. However, it is still controversial with regard to its application in the mix design of HMA. More importantly, Superpave system does not have any requirements or guidelines regarding film thickness.

Typically, apparent film thickness (or theoretical film thickness, TF), which is calculated by dividing the effective binder volume by the surface area of the aggregates, has been used for film thickness analysis. However, many researchers have questioned the relevance of this concept because it may not represent the distribution of binder in
Nukunya et al. (2001) introduced a new concept of effective film thickness (EFT), which can be calculated by using the effective volumetric properties of asphalt mixture. They concluded that the effective volumetric properties including the EFT seem to effectively evaluate aging effects and correlated well with mixture properties.

In this study, the EFT was selected to act as a surrogate to stiffening effect of interstitial component on mixture. Figure 2-3 is a pictorial representation of the difference between EFT and TF. The EFT can be calculated by using the following equation.

\[
EFT \ (\text{microns}) = \frac{V_{be}}{SA \cdot W_T \cdot PF_{AGG}} = \left[ \frac{P_b - (Abs/100) \cdot P_{AGG}}{SA \cdot PF_{AGG} \cdot G_b} \right] \times 1000 \tag{2-3}
\]

Where, \( V_{be} = \) effective volume of asphalt binder, \( SA = \) surface area of fine aggregate, \( W_T = \) total weight of mixture, \( PF_{AGG} = \) percent fine aggregate by mass of total mixture, \( P_b = \) asphalt content percent by mass of total mixture, \( Abs = \) absorption, \( P_{AGG} = \) percent aggregate by mass of total mixture, \( G_b = \) Specific gravity of asphalt binder.

![Diagram of EFT and TF](image-url)

Figure 2-3. Effective film thickness vs. theoretical film thickness
Adequate interstitial volume is important for mixtures to have sufficient strain tolerance, which can be controlled by having an acceptable range of effective film thickness (EFT). EFT of asphalt mixtures is related to the stiffening effect of IC on mixture, and the fineness of the IC aggregates is the primary factor to control the EFT. In this study, the parameter EFT was evaluated and used to establish a set of performance-related design criteria and predictive relationships for fracture properties of asphalt mixtures. It was expected that the EFT is associated with the time-dependent response and brittleness of asphalt mixtures. For example, higher EFT results in higher creep rate and higher fracture energy. Figure 2-4 shows a schematic that conceptually illustrates how EFT affects mixture properties for two cases which have same component materials.

Note: Then $\varepsilon_A > \varepsilon_B$, therefore, $E_A < E_B$

Figure 2-4. Conceptual drawing of film thickness effect

The white color portion of Figure 2-4 shows the asphalt binder part, while the gray color portion represents the aggregates. In the case of thicker EFT represented by case 1, material will tolerate higher strain (i.e. less brittle) than the thinner EFT (case 2) and it
will generally be broken by micro-damage development with high strain at low stress level. However, in the case of thinner EFT described by case 2, material will exhibit less strain tolerance and failed in a brittle manner with low strain at high stress (local stress) level. Therefore, mixtures should have an acceptable range of EFT for adequate strain tolerance and EFT can be controlled by limiting fineness of fine aggregate portion (i.e. IC) of mixture’s gradation.

2.7 Ratio between Coarse Portion and Fine Portion of Fine Aggregates (CFA/FFA)

Preliminary analyses indicated that the fineness of the fine aggregate portion of the interstitial components was strongly related to effective film thickness. However, EFT does not reflect the effect of particle interaction within the IC, which could be one important factor for IC characterization. Therefore, a new parameter CFA/FFA, which is the ratio between the coarse portion and fine portion of the IC particles, was introduced to characterize the structure of the IC of mixture’s gradation.

![Figure 2-5. Determination of CFA/FFA](image)
In this study, CFA/FFA was used as an indicator to represent the fineness and aggregate structure of the IC. It was hypothesized that CFA/FFA was related to the creep response or time-dependent response of asphalt mixture. Figure 2-5 describes the basic principle of determining the CFA/FFA.

2.8 Summary

For DASR-IC characterization, two existing parameters including DASR porosity and DF, and two more parameters including EFT and CFA/FFA were newly added, especially for further IC characterization. Finally, four parameters identified including DASR porosity, DF, EFT, and CFA/FFA have formed the DASR-IC criteria to effectively address the two primary components of asphalt mixtures (i.e. both coarse (DASR) and fine (IC) portions of mixture gradation and resulting volumetric properties) that play a major role on properties and performance. These parameters (i.e. four DASR-IC parameters) were used for evaluation conducted in this study.
CHAPTER 3
IMPLEMENTATION OF BINDER AND MIXTURE TESTS ON FIELD CORES FOR SUPERPAVE MIXTURES IN FLORIDA

3.1 Background

Binder and mixture tests on field cores were conducted to determine binder and mixture properties for different Superpave projects in Florida. The information obtained was used to establish reasonable and effective mixture design guidelines and criteria, performance-related laboratory properties, and parameters, and predictive mixture property relationships. All binder tests were performed according to FDOT test methods, and HMA fracture mechanics model was used to analyze mixture test results.

3.2 Binder Recovery and Binder Tests

Asphalt binder recoveries and binder tests were conducted for cores obtained from different Superpave projects. Binder tests, including penetration test at 25 °C, viscosity test at 60 °C, dynamic shear rheometer (DSR) test, bending beam rheometer (BBR) test, and multiple stress creep recovery (MSCR) test, were performed in this study. The binder testing plan is summarized below, and Table 3-1 represents the asphalt binder used on the Superpave projects evaluated.

- Penetration test at 25 °C (77 °F)
- Viscosity test at 60 °C (140 °F)
- Dynamic shear rheometer (DSR) test at 25 °C (77 °F)
- Bending beam rheometer (BBR) test at -12 °C (10.4 °F)
- Multiple stress creep recovery (MSCR) test at 64 °C (147.2 °F)

Table 3-1. Asphalt binder used for Superpave projects evaluated

<table>
<thead>
<tr>
<th>Project</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer A</td>
<td>PG</td>
<td>PG</td>
<td>PG</td>
<td>PG</td>
<td>PG</td>
<td>PG</td>
<td>PG</td>
<td>ARB</td>
<td>ARB</td>
<td>PG</td>
<td>ARB</td>
<td></td>
</tr>
<tr>
<td>Layer B</td>
<td>PG</td>
<td>PG</td>
<td>PG</td>
<td>PG</td>
<td>N/A</td>
<td>PG</td>
<td>PG</td>
<td>PG</td>
<td>PG</td>
<td>PG</td>
<td>PG</td>
<td>PG</td>
</tr>
</tbody>
</table>

Note: N/A = Not Applicable
3.2.1 Binder Recovery

Asphalt recovery was performed by using the solvent extraction method for cut cores obtained from the different Superpave projects, including Superpave top and bottom layers which were denoted as layer A and B, respectively. Trichloroethylene (TCE) was used as a solvent for binder recovery and the test procedure was carefully followed to minimize any additional aging of the binder during the binder recovery operation according to FDOT test methods.

3.2.2 Penetration Test

The penetration test is one of the oldest and simplest empirical tests used to measure the consistency of asphalt binder. In general, penetration test is performed at 25 °C which is considered approximately representative value of average service temperature for asphalt pavement. The depth of penetration is measured in units of 0.1 mm and reported in penetration units. For example, if the penetration depth of the needle is 8 mm, the penetration number of asphalt binder is 80. The description and practice of standard penetration test method is designated and reported in AASHTO T 49 and ASTM D 5. Penetration tests were conducted at 25 °C. Figure 3-1 represents penetration test results from binder recovered for the Superpave projects evaluated.

Results show that penetration measured for binder extracted from top layer denoted as layer A generally has lower value than for binder obtained from bottom layer denoted as layer B. This was expected because the effect of oxidative aging for top layer is generally more severe than bottom layer. Binders obtained from top layer of Project 9 and 10, which were rubber modified binder (ARB-5) exhibited especially lower penetration.
Note: (C) = Cracked, (U) = Uncracked
Figure 3-1. Penetration test results for Superpave projects

3.2.3 Viscosity Test

Viscosity represents the resistance to flow of a fluid and it can be simply defined as the ratio of shear stress to shear rate. As opposed to other empirical tests including penetration test, viscosity is a fundamental property. However, viscosity is generally measured at only one temperature, so it does not cover the full range of construction and service conditions.

Viscosity test is usually performed at 60 °C which is approximately considered to be representative of the maximum in-service surface temperature of asphalt pavement. The description and practice of standard absolute viscosity test method is described in AASHTO T 202 and ASTM D 2171. Figure 3-2 exhibits current viscosity measured from extracted binder and Figure 3-3 shows the change in viscosity over time for the Superpave projects evaluated.
Viscosity Test Results for Superpave Projects

Note: (C) = Cracked, (U) = Uncracked

Figure 3-2. Viscosity test results for Superpave projects

Change in Viscosity with Aging (Layer A)

(a) Layer A
Due to more severe effect of oxidative aging caused by higher surface temperature, the top layer showed higher viscosity as well as higher rate of increase in viscosity than the bottom layer. Specifically, top layer (Layer A) of Project 8 through 12 which included polymer modified (PG 76-22) and rubber modified binder (ARB-5) sections indicated higher viscosity with around six to nine years of aging in the field. Also, as indicated in Figure 3-3 (a), these sections showed higher rate of increase in viscosity with aging.

3.2.4 Dynamic Shear Rheometer Test (DSR)

The dynamic shear rheometer (DSR) test is used in the Superpave system to characterize the viscous and elastic behavior of asphalt binder at intermediate and high service temperatures. The DSR measures the complex shear modulus $G'$ and phase angle $\delta$ of asphalt binder to determine the characteristics of elastic and viscous components at pavement service temperatures. Specifically, $G'$ and $\delta$ measured are
utilized as the indicators to predict two HMA distresses: rutting and fatigue cracking.

The description and practice of standard DSR test method is designated and reported in AASHTO TP 5. In the Superpave asphalt binder specification, two parameters have been chosen (G’/sinδ, and G’·sinδ) for evaluation of rutting and fatigue cracking, respectively.

Since the Superpave projects investigated have six to eleven years of service period from the construction, all recovered binders obtained were considered as PAV aged binders. As the DSR test for PAV aged binder, samples were tested by using 8mm spindle at intermediate temperature determined based on the PG grade of original binder used. Figure 3-4 represents the parameter G’·sinδ for all Superpave projects evaluated.

![DSR G*Sinδ 10rad/s at 25°C (77°F)](image)

Note: (C) = Cracked, (U) = Uncracked

Figure 3-4. G’·sinδ, 10 rad/sec at 25 °C (77 °F) for Superpave projects

Figure 3-4 shows that all binders met the Superpave specification requirement for a maximum G’·sinδ of 5000 kPa except for the top layer of Project 9 (ARB-5) and the
top and bottom layer of Project 10 (Top: ARB-5. Bottom: PG 64-22). \( G' \cdot \sin \delta \) is typically considered as an indicator of resistance to fatigue cracking because it indicates an amount of energy dissipated meaning that higher \( G' \cdot \sin \delta \) is related to higher energy loss. However, based on the results shown in Figure 3-4, and considering the cracking performance, it appeared questionable whether the parameter \( G' \cdot \sin \delta \) was consistently correlated with cracking performance of mixtures.

3.2.5 Bending Beam Rheometer Test (BBR)

The bending beam rheometer (BBR) test is used in the Superpave system to determine the propensity of asphalt binders to thermal cracking at low temperatures. The BBR calculates the creep stiffness of asphalt binder \( (S(t)) \) and the rate of change of the stiffness \( (m\text{-value}) \). The creep stiffness \( (S(t)) \) is related to the thermal stresses developed in the HMA pavement as a result of thermal contraction, while the slope of the stiffness curve, \( m\text{-value} \), is associated with the ability of HMA pavement to relieve thermal stresses. In other words, \( m\text{-value} \) is an indicator of the binder’s ability to relax stresses by asphalt binder flow. The Superpave binder specification requires a maximum limit of creep stiffness and the minimum limit of \( m\text{-value} \). The description and practice of standard BBR test method is designated and reported in AASHTO TP 1.

The BBR tests for PAV aged binder samples were tested at PG grade temperature according to their original specification. Figures 3-5 and 3-6 represent the parameters \( S(t) \) and \( m\text{-value} \) as a result of the BBR testing for all Superpave project sections, respectively. Figure 3-5 shows that all binders met the Superpave specification requirement for a maximum \( S(t) \) of 300 MPa. Figure 3-6 indicates that all binders also met the Superpave specification requirement for a minimum \( m\text{-value} \) of 0.3 except for the top layers of Project 9 (ARB-5) and Project 10 (ARB-5).
The BBR test results including $S(t)$ and $m$-value are typically evaluated to determine the propensity of binder for thermal cracking. However, based on the results...
shown by Figure 3-5 and 3-6, it appeared also questionable whether the parameters $S(t)$ and m-value were consistently correlated with cracking performance of mixtures.

### 3.2.6 Multiple Stress Creep Recovery Test (MSCR)

The multiple stress creep recovery (MSCR) test is used to identify the presence of elastic response in the asphalt binder and the change of elastic response under shear creep and recovery using two different stress levels at a specified temperature. In general, the percent recovery of asphalt binders in the MSCR test is affected by the type and amount of polymer used in the polymer modified asphalt binder. Thus, it can be used as an indicator for determining whether polymer was utilized. In addition, non-recoverable creep compliance has been used as an indicator of the asphalt binder’s resistance to permanent deformation under repeated load. D’Angelo et al. (2009, 2010) found that rutting is typically reduced by half as the non-recoverable creep compliance is reduced by half.

The description and practice of standard MSCR test method is designated and reported in AASHTO TP 70-07 and ASTM D 7405. The MSCR test was conducted by using an 8 mm spindle at the environmental grade temperature (64 °C) for the State of Florida. Figure 3-7 and 3-8 represent the MSCR test results including average recovery and non-recoverable compliance for all Superpave project sections, respectively. Figure 3-7 clearly shows that MSCR average percent recovery can distinguish the presence of polymers in asphalt binders. In general, percent recovery of polymer modified binders was greater than base binders including PG 64-22 and PG 67-22 for both stress levels. Rubber modified binders also showed relatively high percent recovery than base binders. Since the percent recovery indicates the elastic response of asphalt binder,
polymer modified binders (PG 76-22) appear to exhibit higher elastic response and less sensitivity to change of stress level.

Figure 3-7. MSCR average recovery at 64 °C (147.2 °F) for Superpave projects
Figure 3-8. MSCR nonrecoverable compliance at 64 °C (147.2 °F) for Superpave projects

Based on Figure 3-8, polymer and rubber modified binders normally showed lower non-recoverable compliance than base binders for both stress levels. According to
D’Angelo et al. (2009), nonrecoverable compliance can be used for evaluating the rutting resistance of asphalt binder. However, on the basis of the results analyzed, it seemed questionable whether it is consistently correlated with rutting performance of mixtures in the field.

**3.3 Mixture Tests**

Superpave IDT tests were performed on field cores obtained from the Superpave projects evaluated to determine mixture properties including modulus, creep compliance, strength, failure strain, and fracture energy and to identify the change in key mixture properties as a function of age in the field. Tests were performed at 10 °C and 20 °C.

**3.3.1 Test Specimen Preparation**

Specimens were prepared for laboratory testing using field cores obtained from Superpave projects evaluated. Specific gravity (\( G_{mb} \)) test was conducted on each cut cores and air voids were calculated using the \( G_{mb} \) and original (first time of coring) maximum specific gravity (\( G_{mm} \)). It should be noted that \( G_{mm} \) could change with time, especially for moisture-damaged projects. For moisture-damaged projects, air voids determined using original \( G_{mm} \) are probably conservatively low (i.e. true air voids of moisture-damaged projects are likely higher than air voids calculated using original \( G_{mm} \)). Cores of similar air voids were grouped for standard Superpave IDT tests.

**3.3.1.1 Measuring, Cataloguing, and Inspecting**

Each core obtained was cleaned and the layer of each different asphalt mixture was properly identified, measured, and catalogued with appropriate markings to prevent any confusion. For quality control purposes, cores were inspected and compared to construction information to verify the presence of different mixtures and thicknesses. Figure 3-9 shows the measuring, cataloguing, and inspecting work for field cores.
3.3.1.2 Cutting

Once the data was properly logged and verified, the core was sliced to obtain test specimens for Superpave top and bottom layers for testing purposes. A cutting device, which has a diamond cutting saw and a special attachment to hold the cores, was used to slice the cores into specimens of desired thickness. Because the saw uses water to keep the blade wet, the cut specimens were placed in the dehumidifier for at least two days (i.e. 48 hours) to negate the moisture effects in testing. Figure 3-10 represents the cut specimens prepared for Superpave IDT tests and Figure 3-11 shows the cutting machine used in this study.
3.3.1.3 Gage Points Attachment

Gage points were attached to the specimens using a steel template, a vacuum pump setup, and a strong adhesive. Four gage points (5/16 inch diameter by 1/8 inch thick) were placed with epoxy on each side of the specimens at distance of 19 mm (0.75 in.) from the center, along the vertical and horizontal axes. Figure 3-12 shows the gage point attachment procedure.
During this process, the loading axis previously marked on the specimens was checked and clarified. This procedure helped for the placement of specimen in the testing chamber and assured proper loading of the specimen.

3.3.2 Test Procedure

One set of Superpave IDT tests including resilient modulus, creep compliance, and strength test were performed on each specimen for the Superpave projects evaluated to determine modulus, creep compliance, strength, failure strain, and fracture energy at 10 °C and 20 °C. These test results provide the properties to identify changes in key mixture properties over time with aging environment in the field. In addition, as it mentioned previously, this information was also critical to identify material properties and prediction model evaluation, and to calibrate and validate the pavement performance prediction model. The material testing system (MTS) used for this study, and test configuration of Superpave IDT test set-up are shown in Figure 3-13.

![Superpave IDT tests](image)

Figure 3-13. Superpave IDT tests

3.3.2.1 Resilient Modulus Test

The resilient modulus is defined as the ratio of the applied stress to the recoverable strain when repeated loads are applied. The test was conducted according
to the system developed by Roque et al. (1997) to determine the resilient modulus and the Poisson’s ratio. The resilient modulus test was performed in a load controlled mode by applying a repeated haversine waveform load to the specimen for a 0.1 second followed by a rest period of 0.9 seconds. The load was selected to keep the horizontal resilient deformations within the linear viscoelastic range, where horizontal deformations are typically between 100 to 180 micro-inches during the test.

The resilient modulus and Poisson’s ratio can be calculated by the following equations, which were developed based on three dimensional finite element analysis by Roque and Buttlar (1992). The equation is incorporated in the Superpave Indirect Tension Test at Low Temperatures (ITLT) computer program, which was developed by Roque et al. (1997).

\[ M_R = \frac{P \times GL}{\Delta H \times t \times D \times C_{compl}} \]  

\[ \nu = -0.1 + 1.480 \times (X/Y)^2 - 0.778 \times (t/D)^2 \times (X/Y)^2 \]  

Where, \( M_R \) = Resilient modulus, \( P \) = Maximum load, \( GL \) = Gage length, \( \Delta H \) = Horizontal deformation, \( t \) = Thickness, \( D \) = Diameter, \( C_{compl} = 0.6354 \times (X/Y)^{-1} - 0.332 \), \( \nu \) = Poisson’s ratio, and \( (X/Y) \) = Ratio of horizontal to vertical deformation.

3.3.2.2 Creep Test

Creep compliance is a function of time-dependent strain over stress. The creep compliance curve was originally developed to predict thermally induced stress in asphalt pavement. However, it can also be used to evaluate the rate of damage accumulation of asphalt mixture. As shown in Figure 3-14, \( D_0 \), \( D_1 \), and m-value are mixture parameters obtained from creep compliance tests. Although \( D_1 \) and m-value are related to each
other, $D_1$ is more related to the initial portion of the creep compliance curve, while $m$-value is more related to the longer-term portion of the creep compliance curve.

![Diagram of creep compliance with equations and graphs]

where:

$D(t) = \text{Creep compliance at time, } t$

$D_0, D_1, m = \text{Power model constants}$

Figure 3-14. Power model of creep compliance

The creep test was performed in the load controlled mode by applying a static load in the form of a step function to the specimen and then holding it for 1000 seconds. The magnitude of load applied was selected to maintain the accumulated horizontal deformations in the linear viscoelastic range, which is below the total horizontal
deformation of 750 micro-inches. Although the range of horizontal deformation at 100 seconds can vary depending upon test temperature, specimen type, and the level of aging, a horizontal deformation of 100 to 130 micro-inches at 100 seconds is generally considered to be acceptable.

The Superpave Indirect Tension Test at Low Temperatures (ITLT) computer program was used to determine creep properties of the mixtures by analyzing the load and deformation data. Creep compliance and Poisson’s ratio are computed by the following equations.

\[ D(t) = \frac{\Delta H \times t \times D \times C_{\text{compl}}}{P \times GL} \]  
\[ \nu = -0.1 + 1.480 \times (X/Y)^2 - 0.778 \times (t/D)^2 \times (X/Y)^2 \]

Where, \( D(t) \) = Creep compliance at time t (1/psi), \( \Delta H \), t, D, \( C_{\text{compl}} \), GL, \( \nu \), P, and \( (X/Y) \) are same as described above.

**3.3.2.3 Tensile Strength Test**

Failure limits including tensile strength, failure strain, and fracture energy were determined from strength test. These properties can be used for estimating the cracking resistance of the asphalt mixtures. The strength test was conducted in a displacement controlled mode by applying a constant rate of displacement of 50 mm/min until the specimen failed. The maximum tensile strength is calculated as the following equation.

\[ S_t = \frac{2 \times P \times C_{sx}}{\pi \times b \times D} \]  

Where, \( S_t = \) Maximum indirect tensile strength, \( P = \) Failure load at first crack, \( C_{sx} = 0.948 - 0.01114 \times (b/D) - 0.2693 \times \nu +1.436(b/D) \times \nu \), \( b = \) Thickness, \( D = \) Diameter, and \( \nu = \) Poisson’s ratio.
Fracture energy and dissipated creep strain energy can be determined from the strength test and the resilient modulus test. Fracture energy is the total energy necessary to induce fracture. Dissipated creep strain energy (DCSE) is the absorbed energy that damages the specimen, and dissipated creep strain energy to failure is the absorbed energy to fracture (DCSE_f). As shown in the Figure 3-15, fracture energy and DCSE_f can be determined as described below. The ITLT program also calculates fracture energy automatically.

\[
M_R = \frac{S_t}{\varepsilon_f - \varepsilon_0} \Rightarrow \varepsilon_0 = \frac{M_R \varepsilon_f - S_t}{M_R} \tag{3-6}
\]

\[
\text{Elastic Energy (EE)} = \frac{1}{2} S_t (\varepsilon_f - \varepsilon_0) \tag{3-7}
\]

\[
\text{Fracture Energy (FE)} = \int_{0}^{\varepsilon_f} S(\varepsilon) d\varepsilon \tag{3-8}
\]

\[
\text{Dissipated Creep Strain Energy (DCSE}_f) = \text{FE} - \text{EE} \tag{3-9}
\]

Where, \( S_t \) = Tensile strength, and \( \varepsilon_f \) = Failure strain.

![Figure 3-15. Determination of fracture energy and dissipated creep strain energy to failure](image)

Fracture Energy = DCSE_f + E.E.
In addition, a parameter, Energy Ratio (ER), which represents the asphalt mixture’s potential for top-down cracking was recently developed by Roque et al. (2004). This parameter allows the evaluation of cracking performance on different pavement structures by incorporating the effects of mixture properties and pavement structural characteristics. The energy ratio is expressed in the equation shown below. The ITLT program calculates energy ratio automatically.

\[ ER = \frac{DCSE_f}{DCSE_{min}} = \frac{a \times DCSE_f}{m^{2.98} \times D_1} \]  

(3-10)

Where, \( DCSE_f \) = Dissipated creep strain energy (in KJ/m\(^3\)), \( DCSE_{min} \) = Minimum dissipated creep strain energy for adequate cracking performance (in KJ/m\(^3\)), \( D_1 \) and \( m \) = Creep parameters, \( a = 0.0299\sigma^{-3.1} (6.36 - S_t) + 2.46 \times 10^{-8} \) in which, \( \sigma \) = Tensile stress of asphalt layer (in psi), and \( S_t \) = Tensile strength (in MPa).

### 3.3.3 Superpave IDT Test Results

Table 3-2 summarizes the mixture information for the Superpave projects evaluated.

<table>
<thead>
<tr>
<th>Project (UF) ID</th>
<th>Year Aged</th>
<th>Binder Type Top</th>
<th>Binder Type Bottom</th>
<th>Mixture Type Top</th>
<th>Mixture Type Bottom</th>
<th>Traffic Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11</td>
<td>PG 67-22</td>
<td>PG 67-22</td>
<td>9.5C</td>
<td>19.0C</td>
<td>D/5</td>
</tr>
<tr>
<td>2</td>
<td>11</td>
<td>PG 64-22</td>
<td>PG 64-22</td>
<td>12.5C</td>
<td>19.0C</td>
<td>D/5</td>
</tr>
<tr>
<td>3</td>
<td>11</td>
<td>PG 67-22</td>
<td>PG 67-22</td>
<td>12.5C</td>
<td>19.0C</td>
<td>D/5</td>
</tr>
<tr>
<td>4</td>
<td>11</td>
<td>PG 67-22</td>
<td>PG 67-22</td>
<td>9.5C</td>
<td>19.0C</td>
<td>E/6</td>
</tr>
<tr>
<td>6</td>
<td>12</td>
<td>PG 64-22</td>
<td>N/A</td>
<td>12.5F</td>
<td>N/A</td>
<td>C/4</td>
</tr>
<tr>
<td>7</td>
<td>12</td>
<td>PG 64-22</td>
<td>PG 64-22</td>
<td>12.5F</td>
<td>12.5F</td>
<td>C/4</td>
</tr>
<tr>
<td>8</td>
<td>9</td>
<td>PG 76-22</td>
<td>PG 76-22</td>
<td>12.5C</td>
<td>12.5C</td>
<td>D/5</td>
</tr>
<tr>
<td>9</td>
<td>7</td>
<td>ARB-5</td>
<td>PG 64-22</td>
<td>FC-6</td>
<td>12.5F</td>
<td>C/4</td>
</tr>
<tr>
<td>10</td>
<td>7</td>
<td>ARB-5</td>
<td>PG 64-22</td>
<td>FC-6</td>
<td>12.5F</td>
<td>B/4</td>
</tr>
<tr>
<td>11</td>
<td>6</td>
<td>PG 76-22</td>
<td>PG 64-22</td>
<td>12.5C</td>
<td>12.5C</td>
<td>E/6</td>
</tr>
<tr>
<td>12</td>
<td>6</td>
<td>ARB-5</td>
<td>PG 64-22</td>
<td>FC-6</td>
<td>12.5F</td>
<td>C/4</td>
</tr>
</tbody>
</table>
The test results obtained from the Superpave IDT were analyzed using the ITLT computer program developed at the University of Florida. A comprehensive analysis of test results was conducted to identify the trend of changes in key mixture properties including fracture energy, creep rate, resilient modulus, creep compliance, tensile strength, and failure strain as a function of age and environment for different Superpave mixtures.

Results of resilient modulus ($M_R$), which is a measure of elastic stiffness, are presented in Figure 3-16. These include initial and current values of resilient modulus obtained from field cores indicating the trend in resilient modulus over time for the Superpave projects. For most cases, resilient modulus decreased over time, which clearly indicates the presence of permanent damage and the existence of incomplete healing beyond after some level of aging. The top layer (Layer A) generally exhibited higher rates of reduction in resilient modulus than the bottom layer (Layer B). This reflects that the effect of permanent damage induced by traffic load is more severe for top layer than bottom layer.
Figure 3-16. Change in resilient modulus over time

Creep compliance results are shown in Figure 3-17. Creep compliance is related to the ability of a mixture to relax stresses. In general, higher creep compliance indicates that mixtures can relax stresses faster than mixtures with lower creep compliance, which is critical for evaluating thermal stresses. However, higher creep compliance may also be an indication of permanent damage, and the reduction in creep compliance is expected if there is no permanent damage effect.
Figure 3-17. Change in creep compliance over time

Creep rate, or the rate of change of creep compliance, is related to rate of damage. Figure 3-18 shows the creep rate results. For mixtures not affected by moisture damage (Project 1 through 8), creep rate of the top and bottom layers generally decreased over time, which indicates that oxidative aging had a predominant effect on change in creep rate.
Figure 3-18. Change in creep rate over time

However, for mixtures affected by moisture damage (Project 9 through 12), three cases (Layer B of Project 9, 10, and 12) showed clear increase as well as two cases (Layer A of Project 9 and 11) exhibited slight increase of creep rate over time as opposed to the effect of oxidative aging, which indicates the effect of non-healable permanent damage induced by moisture. The other three cases exhibited clear decrease of creep rate over time.

Tensile strength indicates the maximum tensile stress that the mixture can sustain before fracture. Figure 3-19 shows the tensile strength results, which exhibit a similar trend as the results of resilient modulus. It was also determined that tensile strength decreased over time, indicating the presence of permanent damage and the existence of incomplete healing after a certain level of aging.

The top and bottom layers of Project 8 exhibited a lower rate of reduction in tensile strength which appears to be related to the effect of polymer modification. However, the top layer of Project 11, which also used a polymer modified binder, exhibited an
unusually high rate of reduction in tensile strength over time, which seemed to be associated with the effect of moisture damage.

Figure 3-19. Change in tensile strength over time

Failure strain characterizes the brittleness of a mixture. This value is related to the severity of aging condition and the mixture susceptibility to aging, especially oxidative
aging. Figure 3-20 shows initial (less than six months after construction) and current failure strain for the Superpave projects evaluated. As expected, the rate of reduction in failure strain for top layer was generally greater than for the bottom layer. The top and bottom layers of Project 2 and the top layer of Project 6 exhibited the highest rate of reduction in failure strain of sections 1 to 8.
Projects 9 through 12, which showed evidence of moisture damage, also exhibited a high reduction in failure strain. High initial and current air voids as well as the increase in air voids over time caused by the moisture damage for these sections may have accelerated the effect of oxidative aging, so the mixture embrittled within a relatively short period of time.

![Change in Fracture Energy with Aging - Layer A](image)

(a) Layer A

![Change in Fracture Energy with Aging - Layer B](image)

(b) Layer B

Figure 3-21. Change in fracture energy over time
Fracture energy reflects the mixture’s resistance to damage without fracturing. It has been identified as a good indicator of cracking performance of asphalt pavements having similar pavement structure, traffic and environmental condition. Fracture energy results for the eleven Superpave projects are presented in Figure 3-21. As expected, fracture energy has decreased over time. This observation was the basis for the fracture energy aging model introduced in the NCHRP Project 01-42A.

Based on the results shown in Figure 3-21, it seemed clear that higher initial FE results in higher rate of reduction in FE with aging. Also, the top layers exhibited relatively higher rates of reduction in FE than the bottom layers for most projects. However, the rate of reduction in FE for projects showing evidence of moisture damage (Project 9, 10, 11 and 12) exhibited unusually high rates of reduction in FE regardless of the initial FE magnitude and layer depth.

3.3.4 Moisture-Damaged Projects

Moisture damage of asphalt mixture is a major distress mode that can result in significant costs for repair and rehabilitation. The effect of moisture on asphalt mixture involves various factors acting simultaneously including the effect of moisture susceptibility of asphalt mixture, stresses induced by traffic load, environmental condition, and moisture. Many researchers have tried to identify relationships between asphalt mixture properties and moisture (Schmidt et al. 1972, Fwa et al. 1995, and Lottman 1986). However, the mechanism and effect of moisture damage have not yet been fully identified or verified.

During the inspection process of the cores obtained, evidence of moisture damage was visually identified in the form of stripping for Project 9, 10, 11, and 12. Stripping was particularly prominent at the interface between top and bottom Superpave layers. These
projects are relatively new pavements (six to seven years of age). Specifically, as shown in Table 3-3, use of granite aggregates was common to all moisture damaged projects. No moisture damage was observed for projects produced with limestone aggregate. Three of the four projects (Projects 9, 10, and 12) had fine-graded mixtures with rubber modified binder (ARB-5) in the top layer and PG 64-22 binder in the bottom layer. The fourth project had a coarse-graded mixture with SBS-modified binder in the top layer and PG 64-22 binder in the bottom layer. Table 3-3 summarizes project information and the moisture damaged sections are highlighted.

Table 3-3. Project information for moisture-damaged sections

<table>
<thead>
<tr>
<th>Project (UF) ID</th>
<th>Year</th>
<th>Aged</th>
<th>Aggregate Type</th>
<th>Binder Type Top</th>
<th>Binder Type Bottom</th>
<th>Mixture Type Top</th>
<th>Mixture Type Bottom</th>
<th>Traffic Level</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11</td>
<td>11</td>
<td>Granite</td>
<td>PG 67-22</td>
<td>PG 67-22</td>
<td>9.5C</td>
<td>19.0C</td>
<td>D/5</td>
</tr>
<tr>
<td>2</td>
<td>11</td>
<td>11</td>
<td>Granite</td>
<td>PG 64-22</td>
<td>PG 64-22</td>
<td>12.5C</td>
<td>19.0C</td>
<td>D/5</td>
</tr>
<tr>
<td>3</td>
<td>11</td>
<td>11</td>
<td>Limestone</td>
<td>PG 67-22</td>
<td>PG 67-22</td>
<td>12.5C</td>
<td>19.0C</td>
<td>D/5</td>
</tr>
<tr>
<td>4</td>
<td>11</td>
<td>11</td>
<td>Limestone</td>
<td>PG 67-22</td>
<td>PG 67-22</td>
<td>9.5C</td>
<td>19.0C</td>
<td>E/6</td>
</tr>
<tr>
<td>6</td>
<td>12</td>
<td>12</td>
<td>Limestone</td>
<td>PG 64-22</td>
<td>N/A</td>
<td>12.5F</td>
<td>N/A</td>
<td>C/4</td>
</tr>
<tr>
<td>7</td>
<td>12</td>
<td>12</td>
<td>Limestone</td>
<td>PG 64-22</td>
<td>PG 64-22</td>
<td>12.5F</td>
<td>12.5F</td>
<td>C/4</td>
</tr>
<tr>
<td>8</td>
<td>9</td>
<td>9</td>
<td>Limestone</td>
<td>PG 76-22</td>
<td>PG 76-22</td>
<td>12.5C</td>
<td>12.5C</td>
<td>D/5</td>
</tr>
<tr>
<td>9</td>
<td>7</td>
<td>7</td>
<td>Granite</td>
<td>ARB-5</td>
<td>PG 64-22</td>
<td>FC-6</td>
<td>12.5F</td>
<td>C/4</td>
</tr>
<tr>
<td>10</td>
<td>7</td>
<td>7</td>
<td>Granite</td>
<td>ARB-5</td>
<td>PG 64-22</td>
<td>FC-6</td>
<td>12.5F</td>
<td>B/4</td>
</tr>
<tr>
<td>11</td>
<td>6</td>
<td>6</td>
<td>Granite</td>
<td>PG 76-22</td>
<td>PG 64-22</td>
<td>12.5C</td>
<td>12.5C</td>
<td>E/6</td>
</tr>
</tbody>
</table>

Note: N/A = Not Applicable

Several unique trends were identified for moisture damaged sections with regard to the change in fracture energy and air voids over time. Relatively high initial and/or current air voids were measured on field cores obtained from layer A for Project 9 and 10. In some cases (Layer A of Project 9 and 11 and layer B of Project 9, 11, and 12), the air voids increased over time, which appeared to be related to the displacement of material caused by moisture damage. Figure 3-22 shows the change in air voids for Superpave projects evaluated including layers A and B, respectively.
Note: “WP” denotes the “Wheel Path” Figure 3-22. Change in air voids over time

In addition, as shown in Figure 3-23, much greater rate of reduction in normalized fracture energy was obtained for moisture damaged sections. Rate of reduction of normalized fracture energy over time was calculated to account for the difference in age between moisture damaged sections and other sections. Figure 3-23 shows the initial
rate of reduction in normalized fracture energy for Superpave projects evaluated including layer A and B, respectively.

(a) Layer A

(b) Layer B

Note: $dFEn(t)/dt$ at $t=0$ denotes the initial rate of reduction in normalized fracture energy over time.

Figure 3-23. Initial rate of reduction in normalized fracture energy over time.
Birgisson et al. (2004) indicated that the Energy Ratio (ER) can be used to evaluate the effect of moisture damage on changes in fracture resistance of asphalt mixtures.

Figure 3-24. Initial rate of reduction in normalized energy ratio over time

Note: \( \frac{dER_n}{dt} \) at t=0 denotes the initial rate of reduction in normalized energy ratio.
Figure 3-24 clearly shows that a much greater reduction in normalized ER was observed in moisture damaged sections. In other words, the effect of moisture dramatically reduced the fracture resistance of asphalt mixtures, and the ER was capable of detecting the effect of moisture damage. Figure 3-24 clearly indicates that the ER is very sensitive to, and therefore able to capture the effects of moisture damage. As expected, high rate of reduction in normalized ER with aging was identified for moisture damaged sections.

3.4 Summary

Binder and mixture tests were performed on field cores to determine key binder and mixture properties to identify the change in these properties as a function of age and environment in the field for different Superpave projects in Florida. Test results were carefully analyzed and further used to develop implementable mixture design criteria (i.e. DASR-IC criteria), predictive mixture property relationships, and material property prediction model evaluation.
CHAPTER 4
EVALUATION OF FIELD MIXTURE PERFORMANCE USING DASR-IC MODEL PARAMETERS

4.1 Background

Four DASR-IC model parameters including DASR porosity, DF, EFT, and CFA/FFA have formed the DASR-IC criteria to effectively address the two primary components (i.e. DASR and IC) of asphalt mixtures that play a major role on properties and performance. Field performance evaluation of different Superpave mixtures was conducted to identify the relationships between the four DASR-IC parameters and field performance.

4.2 Implementation of Gradation Analysis for Superpave Mixtures

Gradation analysis was conducted for different Superpave mixtures evaluated as part of Superpave monitoring projects sponsored by FDOT. Table 4-1 summarizes mixture information of Superpave projects analyzed. The actual reliability level for all binder’s true grades is 98% for the seven-day average high temperature and the one-day low temperature.

In this study, extensive sampling was made by taking field cores from different Superpave mixtures constructed throughout the state of Florida. It is noted that in-place gradations were determined from field cores. These gradations are not simply job-mix-formula (JMF) gradations that may or may not be representative of the final results in the field. In addition, asphalt binder recovery was performed by using the solvent extraction method, and all mixture volumetric properties required were obtained from cut cores.
Table 4-1. Mixture information of Superpave projects analyzed

<table>
<thead>
<tr>
<th>Project (UF) ID</th>
<th>US Route</th>
<th>County</th>
<th>Aggregate Type</th>
<th>Binder Type</th>
<th>Mixture Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>I-10 WB</td>
<td>Madison</td>
<td>Granite</td>
<td>PG 67-22</td>
<td>9.5C</td>
</tr>
<tr>
<td>2</td>
<td>I-75 SB</td>
<td>Hamilton</td>
<td>Granite</td>
<td>PG 64-22</td>
<td>12.5C</td>
</tr>
<tr>
<td>3</td>
<td>I-75 SB</td>
<td>Hamilton</td>
<td>Limestone</td>
<td>PG 67-22</td>
<td>12.5C</td>
</tr>
<tr>
<td>4</td>
<td>I-10 EB</td>
<td>Duval</td>
<td>Limestone</td>
<td>PG 67-22</td>
<td>9.5C</td>
</tr>
<tr>
<td>6</td>
<td>US-301 SB</td>
<td>Marion</td>
<td>Limestone</td>
<td>PG 64-22</td>
<td>12.5F</td>
</tr>
<tr>
<td>7</td>
<td>FL Turnpike NB</td>
<td>Palm Beach</td>
<td>Limestone</td>
<td>PG 64-22</td>
<td>12.5F</td>
</tr>
<tr>
<td>8</td>
<td>I-10 WB</td>
<td>Leon</td>
<td>Limestone</td>
<td>PG 76-22</td>
<td>12.5C</td>
</tr>
<tr>
<td>9</td>
<td>SR-121 SB</td>
<td>Alachua</td>
<td>Granite</td>
<td>ARB-5</td>
<td>FC-6</td>
</tr>
<tr>
<td>10</td>
<td>SR-16 EB</td>
<td>Bradford</td>
<td>Granite</td>
<td>ARB-5</td>
<td>FC-6</td>
</tr>
<tr>
<td>11</td>
<td>I-295 SB</td>
<td>Duval</td>
<td>Granite</td>
<td>PG 76-22</td>
<td>12.5C</td>
</tr>
<tr>
<td>12</td>
<td>SR-73 SB</td>
<td>Calhoun</td>
<td>Granite</td>
<td>ARB-5</td>
<td>FC-6</td>
</tr>
</tbody>
</table>

Note: 1. Mixture Type: C = Coarse Mixtures, F = Fine Mixtures, N/A = Not Applicable
2. WB = West Bound, SB = South Bound, EB = East Bound, and NB = North Bound

All DASR-IC parameters including DASR porosity, DF, EFT, and CFA/FFA were determined using the in-place gradations and mixture volumetric properties obtained from field cores for each project. Table 4-2 summarizes all parameters calculated.

Figure 4-1 shows the initial values of fracture energy and creep rate, which are key mixture properties to control performance of asphalt pavement (Zhang et al. 2001), measured from field cores for Superpave projects evaluated.

Table 4-2. DASR-IC parameters calculated for Superpave projects

<table>
<thead>
<tr>
<th>Project (UF) ID</th>
<th>DASR (mm)</th>
<th>DASR Porosity (%)</th>
<th>DF</th>
<th>EFT (Microns)</th>
<th>CFA/FFA</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.75 – 1.18</td>
<td>48.0</td>
<td>0.64</td>
<td>19.5</td>
<td>0.31</td>
</tr>
<tr>
<td>2</td>
<td>4.75</td>
<td>60.2</td>
<td>1.02</td>
<td>37.1</td>
<td>0.60</td>
</tr>
<tr>
<td>3</td>
<td>4.75 – 2.36</td>
<td>43.8</td>
<td>0.52</td>
<td>23.5</td>
<td>0.35</td>
</tr>
<tr>
<td>4</td>
<td>4.75 – 2.36</td>
<td>47.5</td>
<td>0.56</td>
<td>32.5</td>
<td>0.46</td>
</tr>
<tr>
<td>6</td>
<td>4.75 – 1.18</td>
<td>56.2</td>
<td>0.92</td>
<td>13.7</td>
<td>0.29</td>
</tr>
<tr>
<td>7</td>
<td>9.5 – 1.18</td>
<td>50.2</td>
<td>0.86</td>
<td>12.7</td>
<td>0.30</td>
</tr>
<tr>
<td>8</td>
<td>4.75 – 2.36</td>
<td>48.8</td>
<td>0.60</td>
<td>28.3</td>
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</tr>
<tr>
<td>9</td>
<td>4.75 – 1.18</td>
<td>51.0</td>
<td>0.69</td>
<td>15.2</td>
<td>0.29</td>
</tr>
<tr>
<td>10</td>
<td>9.5 – 1.18</td>
<td>50.3</td>
<td>0.71</td>
<td>14.4</td>
<td>0.31</td>
</tr>
<tr>
<td>11</td>
<td>4.75 – 1.18</td>
<td>40.6</td>
<td>0.56</td>
<td>24.8</td>
<td>0.39</td>
</tr>
<tr>
<td>12</td>
<td>4.75 – 1.18</td>
<td>61.3</td>
<td>0.76</td>
<td>30.4</td>
<td>0.24</td>
</tr>
</tbody>
</table>
Figure 4-1. Initial fracture energy and creep rate for Superpave projects evaluated

Detailed descriptions of gradation analysis results for each project are included in the following section, including a brief introduction of material composition used, gradation characteristics, evaluation of DASR-IC parameters calculated and mixture property characteristics.
4.3 Evaluation of Gradation Analysis Results for Superpave Projects

Project 1 was constructed with 9.5 mm coarse-graded Superpave mixture using granite aggregate. PG 67-22 binder was used for this project. Gradation analysis results indicated that Project 1 exhibited a continuous gradation pattern with good interaction within the DASR structure. All gradation parameters were within the acceptable range. Mixture test results indicated that initial fracture energy and creep rate were in a range associated with good-performing mixtures.

Project 2 was constructed using 12.5 mm coarse-graded Superpave mixture. Granite aggregate was used with PG 64-22 binder. Project 2 exhibited high values of DASR porosity and DF with poor interaction within the DASR structure. In addition, uncommonly high EFT and CFA/FFA were identified. It appeared that the gradation characteristics mentioned above resulted in the unusually high initial fracture energy and creep rate.

Project 3 was constructed with 12.5 mm coarse-graded Superpave mixture. Limestone aggregate was used along with PG 67-22 binder. As with Project 1, Project 3 exhibited a continuous gradation pattern with good interaction within the DASR structure. However, DF was relatively low which led to high creep rate and is probably related to the poor cracking performance observed for Project 3.

Project 4 was constructed with 9.5 mm coarse-graded Superpave mixture. Limestone aggregate was used with PG 67-22 binder. Project 4 exhibited an acceptable DASR porosity, but also had relatively high EFT and CFA/FFA with low DF, indicating that the IC aggregates would not be involved in transmitting load between the DASR aggregates. Mixture test results indicated relatively high initial creep rate, which
appeared to be associated with the high EFT and CFA/FFA along with the low DF of Project 4.

Project 6 was constructed with 12.5 mm fine-graded Superpave mixture. Limestone aggregate was used along with PG 64-22 asphalt binder. Based on the gradation analysis results, Project 6 had high DASR porosity and relatively high DF, indicating potentially poor mixture performance. In fact, relatively poor field performance for both rutting and cracking was identified based on the PCS data and field investigation.

Project 7 was constructed with 12.5 mm fine-graded Superpave mixture. Limestone aggregate was used with PG 64-22 binder. Project 7 exhibited a continuous gradation pattern with good interaction within the DASR structure. However, the DASR porosity was in the marginal range with DF within the acceptable range. Acceptable values of initial fracture energy and creep rate were determined for mixture tests.

Project 8 was constructed with 12.5 mm coarse-graded Superpave mixture. Limestone aggregate was used along with an SBS modified binder. Project 8 exhibited a continuous gradation pattern with good interaction within the DASR structure. However, the DASR porosity was in the marginal range with DF within the acceptable range. Project 8 mixture exhibited relatively high EFT and CFA/FFA which may be associated with high damage rate. However, mixture test results indicated that initial creep rate was within the range considered to be acceptable. This was probably due to the beneficial effect of polymer modification. More details regarding material property relationships will be introduced in Chapter 5.
Projects 9 through 12 were categorized as unusual projects from the standpoint of gradation effects and material property relationships because evidence of moisture damage was observed. Results of gradation analysis for these sections are included in the Table 4-2. Also, results of initial fracture energy and creep rate are included along with normal projects in Figure 4-1. However, moisture damaged sections have to be dealt with differently from other sections for the performance evaluation. Therefore, moisture damaged sections, Project 9, 10, 11, and 12, were excluded from performance evaluation in this study.

4.4 Evaluation of Field Mixture Performance

Performance evaluation of different Superpave mixtures was conducted to identify effects of gradation characteristics on field performance using the DASR-IC model parameters. Field rutting and cracking performance data were collected as part of Superpave monitoring projects conducted at the University of Florida. Structural deficiencies were not found for Superpave projects evaluated in this study. Based on moduli determined by backcalculation of falling weight deflectometer test (FWD) results obtained at multiple times during the service life, changes in base and subgrade moduli were not significant over time. Actual backcalculated base moduli varied from 0.30 to 0.52 Gpa (44100 to 74900 psi), which indicated competent base moduli for asphalt pavement. Gradation analysis results were used as gradation parameters for the evaluation. Based on the results introduced in prior research (Kim et al. 2006, Guarin 2009, Roque et al. 2011) and analyses performed in this study, acceptable ranges of each parameter were identified:

- DASR porosity (%): 38 – 52 (48 – 52: Marginal DASR porosity)
- DF: 0.60 – 0.90
• EFT (microns): 12.5 – 25.0
• CFA/FFA: 0.28 – 0.36

It is noted that binder properties obtained from field cores still met the Superpave specification requirement regarding six to eleven year-old pavement for Superpave projects evaluated (Roque et al. 2011). In other words, parameters from binder tests did not appear to be consistently correlated with field performance of Superpave mixtures.

Furthermore, based on the Quality Control (QC), Quality Assurance (QA), and Independent Assurance (IA) data, all projects met relevant construction specification criteria including job-mix-formula (JMF) for gradation, air voids, binder content and in-place density.

4.4.1 Field Performance: Rutting

Comprehensive monitoring of field rutting performance was conducted for Superpave mixtures as part of Superpave monitoring projects sponsored by FDOT. Field rut depth measured from construction throughout the pavement’s life using transverse profilograph was used for the evaluation. The results are presented in terms of rut depth per ESALs (inch/ESALs×10^6) to normalize the effect of traffic volume between the different projects. In addition, the rut depth/ESALs at two years values were used for analysis to account for the fact that the rate of rutting generally decreases with time. Careful analysis of rut profiles with time has clearly shown that this phenomenon is associated with the fact that the rut basin continues to widen with continued loading, while rut depth does not increase very much. Figure 4-2 shows the rut depth/ESALs at two years for the Superpave projects evaluated. Projects 1, 2 and 6 exhibited relatively high rut depth/ESALs, which indicates poor field rutting performance, compared to other projects.
Figure 4-2. Field rutting performance for Superpave projects evaluated

Figure 4-3 shows the results of rutting performance evaluation using DASR-IC parameters for Superpave projects. Figure 4-3 (a) clearly indicates that mixtures with good DASR porosity exhibited much lower rut depth/ESALs, which reflects good field rutting performance, than mixtures with high DASR porosity (i.e. outside the acceptable range). For mixtures with marginal DASR porosity, it was noted that mixtures could exhibit either good or bad rutting performance.

Based on Figure 4-3 (b), the range of DF appears to be correlated with DASR porosity. Mixtures with DF within the acceptable range exhibited lower rut depth/ESALs, except for Project 1. Project 1 had marginal DASR porosity which appeared to result in relatively bad performance. Mixtures with high DF clearly exhibited high rut depth/ESALs. However, Projects 3 and 4 exhibited good rutting performance, even though the DF was relatively low. Gradation analysis results indicated that these two sections had good DASR porosity with good interaction within the DASR structure. Similar trends were also identified for relations between rut depth/ESALs and EFT and
CFA/FFA (see Figures 4-3 (c) and (d)). It is noted that Project 8 exhibited good rutting performance with marginal DASR porosity which was probably related to the beneficial effect of polymer modification.
In summary, the characteristics of the coarse aggregate structure as reflected by the DASR porosity, is the most important parameter to control field rutting performance. It was observed that good IC characteristics including DF, EFT, and CFA/FFA could not...
overcome the problems associated with a mixture with DASR porosity outside the acceptable range. Therefore, the DASR porosity criteria introduced appeared to provide an effective tool that can accurately distinguish the field rutting performance of Superpave mixtures.

4.4.2 Field Performance: Cracking

Field cracking performance was also monitored for the Superpave projects as part of Superpave monitoring projects sponsored by FDOT. As the indicator of cracking performance evaluation, crack initiation time for each project was estimated using the approach based on the information obtained from comprehensive field investigation conducted by the University of Florida research team and the crack rating history data from the pavement condition survey (PCS) performed by the FDOT (Roque et al. 2011). Figure 4-4 indicates the deterministic procedure used to estimate crack initiation time based on PCS data for Projects 1 and 2. Appendix A includes the deterministic procedure to estimate crack initiation time using crack rating data for all Superpave projects evaluated.

![Figure 4-4. Determination of observed crack initiation time for Project 1 and 2](image-url)
Table 4-3 shows the crack initiation time and cracking status determined for Superpave projects evaluated and Figure 4-5 shows the crack initiation time estimated for the Superpave projects evaluated. Projects 1 and 7 exhibited relatively longer crack initiation times, which reflect better field cracking performance than other projects.

Table 4-3. Crack initiation time and cracking status determined for Superpave projects

<table>
<thead>
<tr>
<th>Project (UF ID)</th>
<th>Age (year)</th>
<th>PCS-based Status</th>
<th>Observed Status</th>
<th>Decision Status</th>
<th>t₁ (year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11</td>
<td>U</td>
<td>U</td>
<td>U</td>
<td>16 (P)</td>
</tr>
<tr>
<td>2</td>
<td>11</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>10</td>
</tr>
<tr>
<td>3</td>
<td>11</td>
<td>C*</td>
<td>U</td>
<td>C</td>
<td>9</td>
</tr>
<tr>
<td>4</td>
<td>11</td>
<td>C</td>
<td>U</td>
<td>C</td>
<td>&lt; 11</td>
</tr>
<tr>
<td>6</td>
<td>11</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>11</td>
</tr>
<tr>
<td>7</td>
<td>11</td>
<td>U</td>
<td>U</td>
<td>U</td>
<td>18 (P)</td>
</tr>
<tr>
<td>8</td>
<td>9</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>9</td>
</tr>
</tbody>
</table>

Note: 1. U denotes “Uncracked” and C denotes “Cracked”  
2. P denotes the value determined based on extrapolation  
3. * denotes the final decision when an inconsistency occurred between our observation at coring time and the PCS data

Figure 4-5. Field cracking performance for Superpave projects evaluated

Figure 4-6 represents the results of cracking performance evaluation using DASR-IC parameters for Superpave projects. For cracking performance, IC characteristics
including DF, EFT and CFA/FFA criteria are important. According to Figure 4-6 (a), Projects 1 and 7 exhibited relatively good cracking performance with marginal DASR porosity, while Project 8 showed early crack initiation time with marginal DASR porosity.
Figure 4-6. Cracking performance evaluation using DASR-IC parameters

However, all three Projects (Project 1, 7 and 8) indicated good ranges of DF as shown in Figure 4-6 (b). In this case, the EFT and CFA/FFA, which characterize binder
distribution, fineness and aggregate structure of the IC, seemed to play an important role on cracking performance. Figures 4-6 (c) and (d) show that Projects 1 and 7 exhibited good ranges of both EFT and CFA/FFA. Project 8 exhibited higher ranges of EFT and CFA/FFA which potentially resulted in poor field cracking performance even though it had a polymer-modified binder. It is noted that mixtures should have acceptable ranges of EFT and CFA/FFA for adequate strain tolerance. The EFT and CFA/FFA can be controlled by limiting fineness of the fine aggregate portion.

In addition, Figures 4-6 (c) and (d) indicate that mixtures with high EFT and CFA/FFA clearly exhibited shorter crack initiation time, which reflects bad field cracking performance, than mixtures with good EFT and CFA/FFA. However, Project 3 and 6 exhibited bad cracking performance, even though the EFT and CFA/FFA were within the acceptable ranges. Gradation analysis results indicated that these two sections had either low (Project 3) or high (Project 6) DF (see Figure 4-6 (b)). The effect of bad DF is probably related to poor cracking performance for Projects 3 and 6. On the basis of field cracking performance evaluation, it appeared that the DF criteria should be considered in conjunction with EFT and CFA/FFA criteria to effectively distinguish the field cracking performance.

4.5 Summary

A comprehensive field performance evaluation for both rutting and cracking was conducted using the DASR-IC model parameters to identify and verify performance-related criteria. Based on results analyzed, it was expected that the introduction of DASR-IC criteria as performance-related design parameters to current mix design guidelines and specifications will lead to better and more consistent field rutting and cracking performance of Superpave mixtures. In addition, the DASR-IC criteria will also
provide a more rational method to consider the effect of IC on mixture behavior which strongly affects HMA fracture properties. Therefore, it is expected that this criteria will have a potential to identify the effect of mixture gradation and volumetric characteristics on properties which is more reliably related to performance of asphalt mixtures.
CHAPTER 5
IDENTIFICATION OF PREDICTIVE MIXTURE PROPERTY RELATIONSHIPS AND
MODEL DEVELOPMENT

5.1 Background

The current lack of material property models that can accurately describe the
changes in material properties over time under field conditions is probably the greatest
deficiency in our ability to accurately predict pavement performance. Therefore, there is
a need to evaluate existing material property models and develop improved models for
use in the prediction of pavement performance.

Previous research has shown that fracture energy, which is associated with
mixture’s tolerance to damage, and creep rate, which is related to the rate of damage
accumulation in the mixture, were key material properties that affect cracking
performance of asphalt pavements (Sedwick 1998, and Zhang et al. 2001). Thusly, two
models, part of the Enhanced Hot-Mix Asphalt Fracture Mechanics based Model (HMA-
FM-E) developed during the completion of NCHRP Project 01-42A (Roque et al. 2010)
and directly associated with these two material properties were selected for evaluation
in this study. These are the AC stiffness (creep compliance) aging sub-model and the
fracture energy limit aging sub-model.

Superpave monitoring projects, recently conducted at the University of Florida
(UF), provided an unique opportunity to have material property data for several
pavements throughout their early pavement life cycles, and to evaluate the selected
material property models using this historical data. In addition, four Dominant Aggregate
Size Range-Interstitial Component (DASR-IC) model parameters were identified from
the Superpave monitoring projects to be strongly related to cracking performance of
asphalt mixtures: DASR porosity, disruption factor (DF), effective film thickness (EFT),
and the ratio between the coarse and the fine portions of the fine aggregate (CFA/FFA) (Kim et al. 2006, Guarin 2009, and Roque et al. 2011).

In this study, these DASR-IC parameters, describing the characteristics of gradation and resulting volumetric properties, were used along with the historical material property data to develop improved predictive relationships for HMA fracture properties. These predictive relationships will help reduce the need for sophisticated laboratory mixture tests, thereby increasing the potential for implementation of more advanced pavement design systems such as the interim mechanistic-empirical pavement design guide (MEPDG) recently adopted by American Association of State Highway and Transportation Officials (AASHTO).

5.2 Existing Material Property Models in the HMA-FM-E Model

The two previously mentioned material property models (i.e., the AC stiffness aging model and the fracture energy limit aging model) were re-examined using this newly acquired data. Each model was briefly described, and then used to predict the respective changes in the HMA material properties versus time in service. These predictions were then compared to the historical data obtained from the field sections as part of the Superpave monitoring projects. These comparisons were used to improve the existing models, by proposing modifications which increase the accuracy and reduce the error between predicted and observed values.

5.2.1 AC Stiffness Aging Sub-Model

The AC stiffness aging sub-model was developed based on the global binder aging model (Mirza and Witczak, 1995) and the dynamic modulus model (Witczak and Fonseca, 1996) using a loading time of 0.1 seconds. In this model, the following empirical equation was identified to consider the effect of aging on mixture stiffness (S),
\[ S = \left| E^* \right|_0 \frac{\log \eta_t}{\log \eta_0} \] (5-1)

In this equation, \( |E^*|_0 \) represents the unaged mixture stiffness, and \( \eta_t \) and \( \eta_0 \) correspond to the aged and unaged binder viscosities at 10 °C, respectively. The general trend for the predicted change in mixture stiffness at surface of the pavement as a function of time or age is shown in Figure 5-1. The stiffness \( S(t) \) increases with age, and its rate of change decreases with time, where \( S_0 \) is the initial value and \( S_{\text{max}} \) is the maximum value after being aged for a sufficiently long time.

![Stiffness vs. time](image)

Figure 5-1. Schematic plot for AC stiffness at surface vs. age

Using the AC stiffness aging model, creep compliance values were calculated at 1, 2, 5, 10, 20, 50, 100, 200, 500, and 1000 seconds for three temperatures (i.e. 0, 10, and 20 °C) by taking the inverse of the AC stiffness values at the corresponding time for each of the temperatures. This results in three 1000-second creep compliance curves. These isothermal creep compliance curves were then used to generate a master curve and the creep rate can be obtained from this master curve, as done by Buttlar et al., in 1998. The general trend for the predicted creep rate aging curve in Figure 5-2 shows that the creep rate \( CR(t) \) decreases as age increases at a decreasing rate.
The predicted trend in Figure 5-1 is generally consistent with observations from the resilient modulus data during the early stages of pavement life. However, it is different from the data obtained at later stages of pavement life, during which it was found that the modulus actually decreases with age (Roque et al., 2011). It appears that oxidative aging alone is not sufficient to account for the change in AC stiffness over time. The effects of other factors on AC stiffness, such as load-induced damage, moisture-related damage, and healing potential, need to be considered for more accurate prediction. When the existing AC stiffness aging model is modified to include these key factors, the creep rate aging curve will be affected due to the dependence of creep rate on AC stiffness.

5.2.2 Fracture Energy Limit Aging Sub-Model

The fracture energy limit aging sub-model, also developed as part of the HMA-FM-E model, is expressed in the following form.

\[ FE_f(t) = FE_i - \left( FE_i - FE_{\text{min}} \right) \left( S_n(t) \right)^{1/k} \]  

(5-2)

Where, \( FE_i \) is the initial fracture energy of the HMA and \( FE_{\text{min}} \) is the minimum value of the FE after a sufficiently long aging period \( t_{\text{inf}} \). \( FE_{\text{min}} \) was estimated based on...
experimental data obtained from field cores to be approximately 0.2 kJ/m$^3$, and $t_{\text{inf}}$ is fixed at 50 years. The exponent $k_1$ is an aging parameter determined through field calibration (Roque et al., 2010), and $S_n(t)$ is the normalized change in stiffness (with respect to its initial value) at the surface of the AC layer, and is expressed as,

$$S_n(t) = \frac{S(t) - S_0}{S_{\text{max}} - S_0} \quad (5-3)$$

Where, $S(t)$, $S_0$, and $S_{\text{max}}$ were defined when describing Figure 5-1. Figure 5-3 shows that $S_n(t)$ has the same form as Figure 5-1, and varies between zero and one.

![Normalized change in stiffness vs. time](image)

Figure 5-3. Schematic plot for normalized change in AC stiffness vs. age

The predicted trend for the change in FE limit at the pavement surface as a function of pavement age is presented in Figure 5-4. As can be seen, the FE limit decreases with pavement age at a decreasing rate. This prediction generally agrees with field observations from measured FE limit data. However, the effects of load-induced damage, moisture-related damage, and healing potential on FE limit, which are not considered in the existing model, could affect the change in FE at later stages of pavement life (Roque et al., 2011). Therefore, it may be necessary to take these factors into account to improve the accuracy of the model. It is also noted that initial fracture
energy (FEi) for the existing model was predicted based on FE measured from field cores. Therefore, whereas the existing model requires at least one FE measurement, development of a model to predict initial FE would be of great value.

\[
\frac{FE_i - FE_f(t)}{FE_i - FE_{\text{min}}} = [S_n(t)]^k
\]

Figure 5-4. Schematic plot for FE limit vs. age

5.3 Key Elements for Material Property Relationships

The identification of appropriate material property relationships of key mixture properties is important to accurately predict pavement performance. As mentioned previously, the current or existing material property models for AC stiffness, creep rate, and fracture energy, as included as part of the HMA-FM-E model, are capable of predicting changes in these mixture properties due only to oxidative aging. These current models have the following deficiencies:

- Load-induced damage, moisture-related damage, and healing that could also affect these material properties, especially in the later stages of pavement life were not considered.

- Determination of creep rate, as a function of pavement age, was based on the current AC stiffness aging model using a simple inverse relationship. Consequently, the creep rate predictions were relatively inaccurate. However, the overall trend of the creep rate aging curve was generally correct. Therefore, measured creep rates obtained from field cores were needed to improve the accuracy of the model’s prediction.

- At least one FE measurement is required to predict initial FE for use in the existing fracture energy aging model.
Therefore, these existing mixture property models can be improved by incorporating these physical and environmental factors. Also, there is a need to develop models to predict initial fracture energy and creep rate, so that accurate model predictions can be achieved without relying on physical measurements from field cores.

As illustrated in Figure 5-5, two of material property relationships were identified and targeted for improvement and/or for additional development to meet these needs, and these are described below.

- **Material property relationship I:** This relationship associates or ties mixture gradation characteristics and volumetric properties to initial mixture properties, more specifically, the initial fracture energy and creep rate.

- **Material property relationship II:** This relationship improves the models by changing the mixture properties over time, which takes into account the effects of load-induced damage, moisture-related damage, and mixture healing, in addition to standard oxidative aging.

![Diagram showing the two material property relationships]

**Figure 5-5. Two material property relationships**

**5.3.1 DASR-IC Model Parameters**

As part of Superpave monitoring project recently completed at the University of Florida, field performance evaluation of different Superpave mixtures was conducted to
identify effects of gradation characteristics on field performance using DASR-IC model. Field rutting and cracking performance data and the DASR-IC model parameters determined based on the gradation analysis of these projects were used for the evaluation. Based on this evaluation, an acceptable range of each DASR-IC model parameter was recommended for optimal mixture performance (Roque et al., 2011):

- DASR porosity (%): 38 – 52 (48 – 52: Marginal DASR porosity)
- DF: 0.50 – 0.95
- EFT (microns): 12.5 – 25.0
- CFA/FFA: 0.28 – 0.36

It should be noted that acceptable range of DF was extended based on analyses conducted in this study for the purpose of development of predictive material property relationships. Further analyses were undertaken to identify whether any distinctive relationship or pattern existed between key mixture properties (i.e., fracture energy limit and creep rate) and the DASR-IC parameters within the recommended range.

5.3.2 Initial Material Properties

It was clear from the analysis that the initial value of material properties is one of the key elements that control their changes with aging (Roque et al., 2010). Figure 5-6 shows FE limit curves at three different values of initial fracture energy $F_E^i$, for a constant $k_1$ value, and differentiate the effect of the initial FE magnitude on the FE limit curve as aging continues. As shown, the overall FE limit curves move upward as $F_E^i$ increases. However, the initial reduction rate of the curves also increases. Therefore, models for initial material properties are key elements needed to further develop models for changes in these material properties (see Figure 5-5).
Figure 5-6. FE limit aging curve at different initial FE (k₁=3 (Roque et al. 2010))

5.3.3 Factors Related to Non-Healable Permanent Damage

Existing material property models for changes in AC stiffness, fracture energy, and creep rate, included in HMA-FM-E model, considered oxidative aging as the only factor. However, the trends based on these existing models did not correlate well with the results of previous laboratory and field research (Roque et al., 2007, 2011). In this study, new concepts and modified models were developed to appropriately describe the known trends with respect to the changes in fracture energy and creep rate with time, by including the effect of non-healable permanent damage induced by loading and moisture, in addition to the effect of oxidative aging.

5.4 Development of Predictive Material Property Relationships

Identification of mixture parameters that control mixture performance, including mixture characteristics, component properties/characteristics, and volumetric properties, led to the development of preliminary relationships that predict fundamental mixture properties (i.e. fracture energy and creep rate). DASR-IC model parameters, which were introduced to describe the characteristics of gradation and resulting volumetric
properties found to be the most strongly related to rutting and cracking performance of asphalt mixtures, were used to develop initial property prediction models. In addition, conceptual relationships were identified to describe changes in these properties over time (aging). This can serve as the foundation for further development of improved models to predict mixture properties as a function of age in the field. The verified relationships will also serve to provide reliable inputs for prediction of service life using pavement performance prediction models. Figure 5-7 shows the flowchart for development of predictive material property relationships.

![Flowchart for development of predictive material property relationships](image)

**Figure 5-7. Flowchart for development of predictive material property relationships**

**5.4.1 Relationships for Initial Material Properties**

As mentioned before, initial values of material properties are key elements governing the changes in material properties over time. Therefore, models for initial material properties are important for accurate prediction of their changes over time and for accurate prediction of overall pavement performance. Two relationships for initial
material properties which are presented in the sub-sections that follow were developed in this study.

**5.4.1.1 Initial Fracture Energy Relationship**

The initial value of fracture energy is one key parameter that governs the trend of FE limit aging curve. Based on the analyses of mixture characteristics and component properties and the results of mixture testing, the relationship between initial fracture energy and DASR-IC parameters were identified. Figures 5-8 through 5-10 present the relationships identified between initial fracture energy and DASR porosity, DF, and EFT, respectively. As shown, the initial fracture energy generally decreases with increasing DASR porosity and disruption factor. Also, it can be seen that the initial fracture energy increases with increasing effective film thickness. As a result, it was identified that there are unique relationships between initial fracture energy and each individual DASR-IC parameters, which control characteristics of the mixtures, particularly characteristics of the interstitial component (IC) of mixture. It should also be noted that this trend appears to hold only when DASR porosity, DF, and EFT are within the acceptable ranges.

![Figure 5-8. Relationship between initial fracture energy and DASR porosity](image-url)

Figure 5-8. Relationship between initial fracture energy and DASR porosity
The relationships between initial fracture energy and the three individual parameters are either proportional or inversely proportional. Since there is a proportional relationship between the initial fracture energy and EFT, and inversely proportional relationships between initial fracture energy and DASR porosity and DF, respectively, an empirical relationship was further developed to relate initial fracture energy to all three parameters, which resulted in the following equation.
Where, $FE_i =$ Initial fracture energy, $DASR = DASR$ porosity, $DF =$ Disruption factor, $EFT =$ Effective film thickness, $a = 0.251$, $b = -0.034$, $c = -0.039$, and $d = 0.706$. In this equation, $a$, $b$, $c$, and $d$ are fitting parameters determined through linear regression. Figure 5-11 shows the initial fracture energy as calculated using the predictive equation compared to the measured values from the field cores. As shown, all data points in the figure are close to the line of equality, which indicates that the predictions compare well with the measured values.

\[
FE_i = a \cdot \frac{EFT^b}{DASR^c \cdot DF^d}
\]  

5.4.1.2 Initial Creep Rate Relationship

Creep rate, also known as the rate of creep compliance, is related to the rate of damage, which is considered to be a good indicator for evaluating the cracking performance of asphalt pavement. Attempts were made to identify the relationship between the initial creep rate and each of the DASR-IC parameters: DASR porosity, DF,
and EFT. However, no distinct relationship or pattern was identified from the analysis. In other words, there was no relationship between these three parameters and the initial creep rate (see Figures 5-12 through 5-14). In these figures, “Solid Diamond (♦)” denotes unmodified mixtures for which all DASR-IC parameters are within the ranges considered to be acceptable, “Cross (×)” denotes unmodified mixtures for which at least one of the DASR-IC parameters is not within the ranges considered to be acceptable, and “Square (□)” denotes polymer-modified mixtures for which at least one of the DASR-IC parameters is not within the ranges considered to be acceptable.

However, prior research conducted at the University of Florida has shown that creep rate can be strongly influenced by parameters obtained using the DASR-IC model, including properties and characteristics of the IC (Kim et al., 2006, Guarin, 2009, and Roque et al., 2011). Therefore, it was expected that parameters other than those discussed above could be identified to uniquely define the initial creep rate relationship.

Figure 5-12. Relationship between initial creep rate and DASR porosity
Preliminary analyses showed that the ratio between coarse and fine portions of fine aggregate (CFA/FFA), which is associated with effective film thickness, was strongly related to initial creep rate of mixture. Therefore, this new parameter representing the composition of interstitial component was used to identify the relationship between initial creep rate and CFA/FFA presented in Figure 5-15. It
appears that CFA/FFA is better correlated to initial creep rate than effective film thickness. This may be explained by the fact that CFA/FFA reflects the effect of particle interaction within the IC, whereas EFT does not.

Figure 5-15. Relationship between initial creep rate and CFA/FFA

It also can be seen from Figure 5-15 that the initial creep rate generally increases with increasing CFA/FFA. Similar to the initial fracture energy relationships, it is noted that this relationship appears to hold only when CFA/FFA are within the acceptable range.

In addition, it was found that the initial creep rate relationship can be applied to mixtures with polymer modified binder. As shown in Figure 5-16, the continuous line represents the initial creep rate relationship for mixtures with unmodified binder, while the dashed line represents the relationship for mixtures with polymer modified binder. It appears that polymer modification resulted in lower initial creep rate, even when not all DASR-IC criteria were met. This observation is consistent with previous findings from
research that polymer modification generally helps to reduce the damage rate of asphalt mixtures (Kim et al., 2003).

![Relationship between Initial Creep Rate and CFA/FFA](image)

**Figure 5-16.** Effect of polymer modification on relationship between initial creep rate and CFA/FFA

Additional analyses were conducted to identify whether any clear pattern emerges between the initial creep rate and any binder property parameters, specifically viscosity, effective asphalt content, and $G^* \sin\delta$. Figures 5-17 through 5-19 present the relationships between initial creep rate and each of the parameters evaluated, respectively.

According to analysis results, it appeared that these binder parameters were not clearly correlated with initial creep rate. However, it was found that unmodified mixtures that met all DASR-IC criteria exhibited relatively low initial creep rate as highlighted in Figures 5-17 through 5-19. Unmodified mixtures that did not meet all DASR-IC criteria exhibited a broad range of creep rate, indicating that inadequate gradation can result in high damage rate even when binder properties are satisfactory.
Also, modified mixtures exhibited low initial creep rate even though not all DASR-IC criteria were met. This clearly indicates that if all DASR-IC parameters, including DASR porosity, DF, EFT, and CFA/FFA, are within the acceptable ranges, the unmodified mixtures will have relatively low initial creep rates which could result in better cracking performance.

Figure 5-17. Relationship between initial creep rate and viscosity

Figure 5-18. Relationship between initial creep rate and effective asphalt content
Figure 5-19. Relationship between initial creep rate and $G^*\cdot\sin\delta$

However, it does not necessarily imply that binder properties are not important. Binder properties do play a major role on creep rate and fracture resistance when gradation deficiencies are not present. Also, the results indicated that polymer modification is likely related to the initial creep rate magnitude. Therefore, both binder properties and gradation are important factors to control the initial creep rate.

5.4.2 Models for Changes in Material Properties

The identification of appropriate trends with respect to the changes in key material properties (e.g. fracture energy limit and creep rate) over time is important for accurate prediction of cracking performance of asphalt mixtures. As discussed earlier, the current lack of material property models of this type is probably the greatest shortcoming in the ability to accurately predict pavement performance. Possible goals to develop improved material property models are presented as follows.

- Adjust the changes in material property based on the changes in IC characteristics.
- Adjust the changes in material property by including the effects of non-healable permanent damage related to traffic load and moisture (environment).
However, due to the limited data available for this research, this part of the study was limited to the recommendation of new concepts for modification of the two candidate models under evaluation, i.e. AC stiffness (creep compliance) aging model and fracture energy limit aging model.

5.4.2.1 AC Stiffness Model

As mentioned before, the existing model in Figure 5-1 showed that the AC stiffness is continuously increasing with time. However, this trend does not coincide with the results of prior laboratory, field, and accelerated pavement testing (APT) research, which indicated that the stiffness generally reduces with time after a certain age (Roque et al. 2007, 2011). Therefore, a new concept was proposed and modification of the existing model was designed to appropriately describe the known trend of this property. Figure 5-20 describes the proposed modification including the effect of non-healable permanent damage related to load and moisture on the change in AC stiffness with time.

As shown in Figure 5-20, there is a critical time denoted as $t_d$ which separates pavement life into two stages. In the early stage, Stage I, there is no effect of non-healable permanent damage (related to load and moisture) on changes in AC stiffness. In Stage I, the trend of change in AC stiffness is mainly controlled by oxidative aging. This indicates that the healing process is a dominant factor as compared to the damage process in governing the trend of change in material property in this stage.

Also shown in Figure 5-20, the non-healable permanent damage process takes control in Stage II. The non-healable permanent damage includes both load-induced and moisture-related damage, which tends to reduce the AC stiffness after the critical time. Clearly, determination of $t_d$ is the first task in finalizing the proposed AC stiffness
model. The next challenge is how to quantify the effect of load-induced and moisture-related damage on AC stiffness after the critical time.

![Diagram of AC stiffness model](image)

**Figure 5-20. Proposed AC stiffness model**

### 5.4.2.2 Fracture Energy Limit Model

The fracture energy limit aging model developed as part of the NCHRP Project 01-42A showed that the FE limit generally decreases with aging, and eventually reaches some minimum value after a sufficiently long time (see Figure 5-4). However, the results of prior laboratory, field, and APT research did not coincide well with the trend introduced in the existing model (Roque et al., 2007, 2011). Therefore, modification of the existing model was designed to reasonably describe the observed trend of this property.

Figure 5-21 presents the proposed modifications including the effect of the non-healable permanent damage related to load and moisture on the change in fracture energy limit with time. Since the fracture energy limit in the existing model was associated with the AC stiffness in a normalized form (see Equation 5-2), the proposed modifications for the existing fracture energy limit model (see Figure 5-21 (b)) are related to the modified AC stiffness model (see Figure 5-21 (a)).
As shown in Figure 5-21, pavement life was separated into two stages by the critical time $t_d$: In Stage I when mixture healing potential is high, it was assumed that there is no permanent damage induced by load and moisture. Therefore, the existing relationship for surface AC stiffness $S(t)$, which accounts for the change in AC stiffness due to only oxidative aging, was used for determination of the normalized change in stiffness $S_n(t)$. As a result, the existing relationship for fracture energy limit can be used for this part of the model.

However, during Stage II when the time is greater than $t_d$, a modified relationship for surface AC stiffness, termed $S_d(t)$, was required to consider the permanent damage effect on the change in AC stiffness with time (see Figure 5-21 (a)). As a result, the normalized change in the modified AC stiffness can be expressed using the following equation.

$$S_{dn}(t) = \frac{S_{d0} - S_d(t)}{S_{d0} - S_{dmin}}$$ (5-5)

Where, $S_{dn}(t)$ is the normalized change in stiffness $S_d(t)$ defined for Stage II, and $S_{d0}$ and $S_{dmin}$ are stiffness values when $S_d(t)$ is at $t = t_d$ and $t = 50$ years, respectively.

Then, the modified FE limit aging function was introduced for Stage II by relating the normalized change in FE limit to the normalized change in stiffness $S_d(t)$ by a power of $k_2$, which is expressed in the following equation.

$$\frac{FE_{d0} - FE_d(t)}{FE_{d0} - FE_{min}} = \left[ S_{dn}(t) \right]^{k_2}$$ (5-6)

Where, $FE_d(t)$ is the fracture energy limit function defined for Stage II, $FE_{d0}$ is the value when $FE_d(t)$ is at $t = t_d$, $k_2$ is aging parameter for Stage II, $FE_{min}$ denotes minimum FE, and $S_{dn}(t)$ was defined in Equation 5-5.
It can also be seen from Figure 5-21, due to the incorporation of the permanent damage effect, the modified fracture energy limit function (Stage II) has a higher rate of reduction than the existing function, which is closer to the trend of the change in FE over time actually measured from field cores.

1) For $t < t_d$ (Stage I)
\[
S_n(t) = \frac{S(t) - S_0}{S_{max} - S_0}
\]

2) For $t > t_d$ (Stage II)
\[
S_{dn}(t) = \frac{S_{d0} - S_d(t)}{S_{d0} - S_{dmin}}
\]

Figure 5-21. Proposed FE limit model

In the case of the proposed fracture energy function, it is important to determine the critical time $t_d$, which indicates the point when permanent damage starts affecting the change in fracture energy with time. Also, the effect of non-healable permanent
damage due to load and moisture must be quantified in addition to the effect of oxidative aging after the critical time.

5.5 Summary

Material property relationships were identified using material properties measured from field cores over time and assigned DASR-IC model parameters calculated. Unique relationships were identified between initial mixture properties (i.e. fracture energy and creep) and DASR-IC parameters. It was found that the relationships for initial fracture energy and creep rate appeared to work best when all DASR-IC parameters were within the ranges considered to be acceptable. In addition, based on evaluation of existing material property aging models, a new concept and modifications were proposed to improve these models for more accurate prediction of changes in material properties over time. Due to the limited data available for this study, a full model could not be developed. Nevertheless, procedures to continue and complete the development of improved models were recommended for future works.
CHAPTER 6
EVALUATION OF DASR-IC CRITERIA USING HMA-FM-E MODEL

6.1 Background

The primary purpose of this chapter is the evaluation of DASR-IC criteria with regard to their relationships on field mixture performance using performance prediction model recently developed at the University of Florida, Enhanced HMA Fracture Mechanics Based Model (HMA-FM-E Model). Performance prediction effort using HMA-FM-E Model is presented in Section 6.2 through 6.4, including a brief introduction to this tool, its input module, and prediction results for Superpave projects evaluated. It is noted that Project 8 has a Portland cement concrete base. Therefore, this section was not included in this part of study.

6.2 Enhanced HMA Fracture Mechanics Based Model (HMA-FM-E Model)

The HMA-FM-E model was developed by the UF research team as part of the NCHRP Project 01-42A to predict top-down cracking performance in HMA layers. As indicated by the name, the model is an enhanced version of the existing HMA-FM model, which is the result of the continuous efforts of the UF team over the past years (Zhang et al. 2001, Roque et al. 2002, 2004, Sangpetngam et al. 2003, 2004, Kim et al. 2008). During the course of the NCHRP project, the enhanced performance model was formed by developing and incorporating into the existing model appropriate sub-models that account for effects of aging, healing, and transverse thermal stresses on top-down cracking performance. Furthermore, the enhanced model was calibrated and validated using data from Florida field sections (Roque et al. 2010).

The enhanced top-down cracking performance model has four major components, as shown in the general model framework presented in Figure 6-1, including:
- The load response and load-associated damage sub-models that are used to predict step-wise load induced damage.
- The thermal response and thermal-associated damage sub-models that are used to predict thermally induced damage.
- The damage recovery and accumulation process that is used to accumulate damage after taking into account healing effect. Once the accumulated damage reaches the threshold, a crack will initiate or propagate.
- The mixture properties sub-models that were devised to account for changes in mixture damage, fracture, and healing properties due to aging.

Details for each component are described elsewhere (Roque et al., 2010, Zou and Roque, 2011).

![Diagram of HMA-FM-E model framework]

Figure 6-1. General framework of the HMA-FM-E model

The enhanced model was used to predict top-down cracking performance in HMA layers for Superpave projects evaluated. The remaining parts of this section present the input module for the HMA-FM-E model, followed by model prediction results.

6.3 Input Module

The inputs for the HMA-FM-E model are divided into five categories, including traffic, climate, structure, mixture damage and fracture properties, and mixture healing.
properties. Table 6-1 summarizes the input data characteristics for the HMA-FM-E model.

Table 6-1. Summary of input data characteristics for the HMA-FM-E model

<table>
<thead>
<tr>
<th>Inputs</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic</td>
<td>Multi-year data:</td>
</tr>
<tr>
<td></td>
<td>- Based on current ESALs/Year as measured (No growth is counted)</td>
</tr>
<tr>
<td>Climate</td>
<td>Multi-year data:</td>
</tr>
<tr>
<td></td>
<td>Based on typical one-year data in Melrose, FL</td>
</tr>
<tr>
<td>Structure</td>
<td>- Three-layer</td>
</tr>
<tr>
<td></td>
<td>- AC modulus (multi-year data) predicted from initial data:</td>
</tr>
<tr>
<td></td>
<td>Gradation, binder type, volumetric information (for AC stiffness model)</td>
</tr>
<tr>
<td></td>
<td>- GB, SG moduli (current data) obtained from FWD data (No moisture effect)</td>
</tr>
<tr>
<td>AC damage and fracture</td>
<td>Multi-year data:</td>
</tr>
<tr>
<td>property</td>
<td>- Predicted using mixture property aging sub-models</td>
</tr>
<tr>
<td>AC healing property</td>
<td>Multi-year data:</td>
</tr>
<tr>
<td></td>
<td>- Predicted using the maximum healing potential aging sub-model</td>
</tr>
<tr>
<td>Analysis Type</td>
<td>- Deterministic analysis</td>
</tr>
</tbody>
</table>

Further descriptions of data characteristics for each input category are described as follows:

- Traffic: The traffic volume (in terms of million ESALs per year) for the year of field evaluation for each project was taken as the base value and applied to the corresponding pavement section for the entire simulation period, without considering annual traffic growth (see Table 6-2 (a)).

- Climate: Hourly temperature variation at different depths in the asphalt concrete (AC) layer for one typical year in Melrose, FL was used for all projects for the entire simulation period. The year-to-year change in temperature was not considered.

- Structure: A three-layer pavement structure was selected for the simulation (see also Table 6-2 (a)). Thickness for AC and Base was obtained from design values. Modulus for Base and Subgrade were determined based on backcalculation of falling weight deflectometer (FWD) testing data obtained at the time of field evaluation. The change in Base and Subgrade moduli due to moisture variation were not considered because the former can be ignored in Florida and of the latter has small effect on top-down cracking. AC layer modulus was predicted using the AC stiffness aging sub-model which requires gradation, binder type, and
volumetric properties measured at the unaged condition (see Table 6-2 (b)). In other words, the change in AC modulus due to aging was taken into account for the entire simulation period.

- **AC damage and fracture property**: AC damage and fracture properties including creep rate, fracture energy limit (FEf), were predicted using mixture property aging sub-models, taking into account the measured properties from field cores obtained at the time of field evaluation (see Table 6-2 (c)).

- **AC healing property**: AC healing potential was predicted using the maximum healing potential aging sub-model.

### Table 6-2. Data used for model prediction

<table>
<thead>
<tr>
<th>Project (UF) ID</th>
<th>Layer Thickness (in)</th>
<th>Layer Modulus (ksi)</th>
<th>Yearly Traffic (10^6 ESALs)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AC</td>
<td>Base/Subbase</td>
<td>Subbase</td>
</tr>
<tr>
<td>1</td>
<td>7.43</td>
<td>21.65</td>
<td>44.1</td>
</tr>
<tr>
<td>2</td>
<td>7.40</td>
<td>22.32</td>
<td>74.9</td>
</tr>
<tr>
<td>3</td>
<td>9.64</td>
<td>22.32</td>
<td>66.2</td>
</tr>
<tr>
<td>4</td>
<td>7.44</td>
<td>22.24</td>
<td>50.7</td>
</tr>
<tr>
<td>6</td>
<td>6.40</td>
<td>22.80</td>
<td>60.3</td>
</tr>
<tr>
<td>7</td>
<td>6.74</td>
<td>22.00</td>
<td>67.7</td>
</tr>
<tr>
<td>9</td>
<td>5.50</td>
<td>20.50</td>
<td>51.0</td>
</tr>
<tr>
<td>10</td>
<td>7.75</td>
<td>18.00</td>
<td>29.3</td>
</tr>
<tr>
<td>11</td>
<td>7.75</td>
<td>22.00</td>
<td>65.9</td>
</tr>
<tr>
<td>12</td>
<td>6.49</td>
<td>18.40</td>
<td>47.8</td>
</tr>
</tbody>
</table>

(a) Pavement structural and material property and traffic volume

<table>
<thead>
<tr>
<th>Project (UF) ID</th>
<th>Percent Passing by Weight</th>
<th>Binder Type</th>
<th>Veff (%)</th>
<th>Va (%)</th>
<th>MAAT (°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>100.0</td>
<td>99.0</td>
<td>64.0</td>
<td>45.0</td>
<td>69</td>
</tr>
<tr>
<td>2</td>
<td>100.0</td>
<td>89.0</td>
<td>45.0</td>
<td>4.9</td>
<td>68</td>
</tr>
<tr>
<td>3</td>
<td>100.0</td>
<td>95.0</td>
<td>67.0</td>
<td>4.4</td>
<td>68</td>
</tr>
<tr>
<td>4</td>
<td>100.0</td>
<td>88.0</td>
<td>70.0</td>
<td>4.2</td>
<td>71</td>
</tr>
<tr>
<td>5</td>
<td>100.0</td>
<td>88.0</td>
<td>70.0</td>
<td>4.5</td>
<td>75</td>
</tr>
<tr>
<td>6</td>
<td>100.0</td>
<td>90.0</td>
<td>73.0</td>
<td>5.5</td>
<td>69</td>
</tr>
<tr>
<td>7</td>
<td>100.0</td>
<td>88.0</td>
<td>70.0</td>
<td>5.4</td>
<td>69</td>
</tr>
<tr>
<td>8</td>
<td>100.0</td>
<td>80.0</td>
<td>70.0</td>
<td>5.4</td>
<td>68</td>
</tr>
<tr>
<td>9</td>
<td>100.0</td>
<td>90.0</td>
<td>59.0</td>
<td>4.7</td>
<td>5.3</td>
</tr>
<tr>
<td>10</td>
<td>100.0</td>
<td>88.0</td>
<td>70.0</td>
<td>5.4</td>
<td>5.8</td>
</tr>
<tr>
<td>11</td>
<td>100.0</td>
<td>90.0</td>
<td>59.0</td>
<td>3.5</td>
<td>5.6</td>
</tr>
<tr>
<td>12</td>
<td>100.0</td>
<td>90.0</td>
<td>59.0</td>
<td>3.5</td>
<td>5.7</td>
</tr>
</tbody>
</table>

(b) Mixture gradation, binder type, and volumetric property
### 6.4 Model Prediction Results

The predicted relative crack depth ($C_{Dr}$), crack amount (CA), and crack status for top-down cracking for each Superpave projects evaluated are presented in Table 6-3. It shows that at the time of field evaluation, two out of the ten sections (Project 3 and 4) had reached maximum crack amount to failure ($CA_{max}$), two (Project 6 and 7) just started to crack, three (Project 2, 9, and 11) did not but were about to crack, and the rest (Project 1, 10, and 12) were far from crack initiation.

It can also be seen from Table 6-3 that project sections 3 and 4 had worse cracking performance in terms of shorter crack initiation time ($t_i$) and higher average crack growth rate ($G_i$) than the other sections, while project section 1 had the best performance. Further discussion of the prediction results, including the increment of top-down crack amount over time and the relationships between the DASR-IC criteria and top-down cracking performance for individual projects, will be presented in the section 6.5.
Table 6-3. Predicted top-down cracking performance using HMA-FM-E model

<table>
<thead>
<tr>
<th>Project (UF) ID</th>
<th>Time of Evaluation (Year)</th>
<th>CD_r (%)</th>
<th>HMA-FM-E Top-Down Cracking Predictions</th>
<th>CA_max (ft/100ft)</th>
<th>Predicted Status</th>
<th>CA_i (ft/100ft)</th>
<th>t_i (Year)</th>
<th>G_t (ft/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>11</td>
<td>2.2</td>
<td>U</td>
<td>14.5</td>
<td>22.2</td>
<td>16.9</td>
<td>63.7</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>11</td>
<td>2.9</td>
<td>U</td>
<td>19.2</td>
<td>22.3</td>
<td>12.8</td>
<td>100.0</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>11</td>
<td>50.0</td>
<td>C</td>
<td>330.0</td>
<td>17.1</td>
<td>6.9</td>
<td>153.0</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>11</td>
<td>50.0</td>
<td>C</td>
<td>330.0</td>
<td>22.2</td>
<td>5.9</td>
<td>198.8</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>11</td>
<td>10.7</td>
<td>C</td>
<td>70.5</td>
<td>25.8</td>
<td>9.5</td>
<td>101.3</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>11</td>
<td>8.2</td>
<td>C</td>
<td>54.3</td>
<td>24.5</td>
<td>10.0</td>
<td>108.5</td>
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</tr>
<tr>
<td>9</td>
<td>7</td>
<td>4.1</td>
<td>U</td>
<td>27.4</td>
<td>30.0</td>
<td>7.7</td>
<td>97.7</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>7</td>
<td>1.5</td>
<td>U</td>
<td>10.1</td>
<td>21.3</td>
<td>14.7</td>
<td>66.1</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>6</td>
<td>2.8</td>
<td>U</td>
<td>18.8</td>
<td>21.3</td>
<td>6.8</td>
<td>133.8</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>6</td>
<td>1.5</td>
<td>U</td>
<td>9.7</td>
<td>25.4</td>
<td>15.8</td>
<td>63.6</td>
<td></td>
</tr>
</tbody>
</table>

Note: 1. In HMA-FM-E, CD_r was defined as crack depth over AC layer thickness (in %), and CA_max was determined to be 330ft/100ft when CD_r is equal to 50%.
2. CA_i is crack amount at crack initiation.

Figure 6-2 represents the predictions development of top-down crack amount over time. As shown, Projects 3, 4, and 11 exhibited relatively bad cracking performance with shorter crack initiation time (t_i) and higher average crack growth rate (G_t) than rest of the projects, while Projects 1, 10, and 12 showed good field cracking performance.
6.5 Relationships between DASR-IC Criteria and Model Prediction Results

Attempts were made to assess preliminary DASR-IC criteria established based on the comprehensive field performance evaluation of Superpave projects, including DASR...
porosity, DF, EFT, and CFA/FFA, for consistently enhanced cracking performance. The predicted top-down cracking performance in terms of crack initiation time \( t_i \) and average crack growth rate \( G_t \) were used along with DASR-IC parameters determined for each Superpave projects. For evaluation, all projects were grouped based on ranges of crack initiation time \( t_i \) and average crack growth rate \( G_t \) predicted by HMA-FM-E model. All project sections were also categorized considering ranges of DASR-IC parameters preliminarily determined to identify and evaluate the relationship between DASR-IC criteria and performance model prediction results.

For crack initiation time \( (t_i, \text{year}) \):

- Bad performance sections: \( t_i < 7 \)
- Intermediate performance sections: \( 7 \leq t_i < 13 \)
- Good performance sections: \( t_i \geq 13 \)

For average crack growth rate \( (G_t, \text{ft/year}) \):

- Bad performance sections: \( G_t \geq 130 \)
- Intermediate performance sections: \( 70 \leq G_t < 130 \)
- Good performance sections: \( G_t < 70 \)

For DASR-IC criteria:

- Mixtures that met all DASR-IC criteria
- Mixtures that not all DASR-IC criteria are met

Figures 6-3 (a) and (b) show the relationships between performance model prediction results, including crack initiation time \( t_i \) and average crack growth rate \( G_t \), and the DASR-IC criteria, respectively. According to result shown in Figure 6-3 (a), it was found that projects that met all DASR-IC criteria exhibited relatively long crack initiation time \( t_i \) (i.e. good or intermediate ranges of crack initiation time), while projects
that not all DASR-IC criteria are met showed a broad range of crack initiation time ($t_i$) indicating that inadequate DASR-IC criteria can result in bad field cracking performance.

Figure 6-3. Relationships between DASR-IC criteria and field cracking performance
In addition, result in Figure 6-3 (b) shows that projects that met all DASR-IC criteria exhibited relatively low average crack growth rate ($G_t$) (i.e. good or intermediate ranges of average crack growth rate), while projects that not all DASR-IC criteria are met showed a wide range of average crack growth rate ($G_t$) including range considered as bad performance. These results clearly indicate that if all DASR-IC parameters, including DASR porosity, DF, EFT, and CFA/FFA, are within the acceptable ranges, mixtures will have relatively good cracking performance in the field.

6.6 Summary

Preliminary DASR-IC criteria established in this study was evaluated to identify and verify their relationships on field cracking performance using model prediction results. Results indicated that acceptable ranges of DASR-IC parameters will result in consistently enhanced field cracking performance. Therefore, it appeared that the introduction of DASR-IC criteria as performance-related design parameters to current mix design guidelines and specifications will lead to better and more consistent field performance of Superpave mixtures. However, current criteria preliminarily established need to be further validated and refined using additional field and laboratory data.
CHAPTER 7
CLOSURE

7.1 Summary and Findings

This study was conducted to evaluate the effect of mixture component characteristics (i.e., DASR and IC) on properties and performance of Superpave mixtures, specializing in the development of a set of implementable gradation and volumetric criteria, and Hot-Mix Asphalt (HMA) mixture property predictive relationships based on mixture component characterization. Field performance evaluation for both rutting and cracking was conducted to identify and verify performance-related criteria using parameters from DASR-IC mixture gradation model. In addition, an evaluation of existing material property models was conducted to identify and develop improved material property relationships for more accurate prediction of cracking performance of asphalt mixtures. A summary of findings associated with these tasks are presented as follows:

- Mixtures having DASR porosity within the acceptable range clearly exhibited better field rutting performance than mixtures with high DASR porosity. Mixtures with marginal DASR porosity exhibited either good or bad rutting performance in the field.
- Mixtures with DF considered to be acceptable generally exhibited better cracking performance in the field than mixtures with the either low or high DF.
- DASR porosity, which reflects the characteristics of coarse aggregate structure, is the most dominant parameter to control rutting performance. IC characteristics including DF, EFT, and CFA/FFA could not overcome the problems associated with a mixture with DASR porosity outside the acceptable range.
- IC characteristics are more important than DASR porosity criteria to clearly differentiate field cracking performance. It appears that the DF criteria should be considered in conjunction with EFT and CFA/FFA criteria to effectively distinguish the field cracking performance.
- Unique relationships were identified between initial fracture energy and three DASR-IC parameters: DASR porosity, DF, and EFT. Initial fracture energy
generally decreases with increasing DASR porosity and DF and increases with increasing EFT.

- A new parameter, namely the ratio of the coarse portion and fine portion of fine aggregates (CFA/FFA), representing the composition or aggregate structure of the interstitial component of mixture’s gradation, was introduced to develop the relationship to predict initial creep rate.

- It was identified that the initial creep rate generally increased with increasing CFA/FFA. The effect of polymer modification helps to reduce the initial creep rate, which is the damage rate of an asphalt mixture.

- It was found that the relationships for initial fracture energy and creep rate appeared to work best when all DASR-IC parameters were within the ranges considered to be acceptable.

- Existing material property models for AC stiffness, fracture energy, and creep rate considered oxidative aging as the only factor responsible for changes in these properties with time. However, it was found that the trends based on existing models did not correlate well with the results of prior laboratory and field research.

**7.2 Conclusions**

The following key conclusions were drawn based on the findings and results of this study:

- Mixture gradation characteristics and resulting volumetric properties that effectively characterize the DASR and IC have an important effect on mixture fracture properties as well as on pavement performance in the field. DASR-IC parameters identified were able to appropriately describe the critical aspects of mixture gradation and volumetric properties that play a major role on mixture performance.

- Mixtures with acceptable range of DASR-IC parameters clearly exhibited better mixture performance in the field than those with DASR-IC parameters outside ranges identified as acceptable. Thus, the introduction of DASR-IC criteria as performance-related design parameters to current mix design guidelines and specifications will lead to better and more consistent field rutting and cracking performance of Superpave mixtures.

- Adequate gradation characteristics will result in more consistent initial mixture properties and provide predictive relationships for mixture fracture properties that will enhance our ability to accurately predict pavement performance.

- Inadequate gradation characteristics can result in improper mixture fracture properties even when binder properties are satisfactory. Binder properties do play
a major role on creep rate and fracture resistance when gradation deficiencies are not present. In other words, binder properties alone cannot overcome deficiencies in gradation.

- Predictive mixture property relationships will help reduce the need for sophisticated laboratory mixture tests, thereby increasing the potential for implementation of these more advanced pavement design systems such as the interim MEPDG recently adopted by AASHTO. The verified relationships will also serve to provide reliable inputs for more accurate prediction of service life using pavement performance prediction models.

- Appropriate trends with respect to the changes in fracture energy and creep rate over time were proposed by including the effects of the non-healable permanent damage. The challenges are to quantify these effects in addition to the effect of oxidative aging after the critical time, and to determine the critical time, which indicates the point when permanent damage begins.

### 7.3 Recommendations and Future Works

Based on extensive evaluations performed in this study, recommendations for further investigations regarding the effect of mixture component characteristics on property and performance of Superpave mixtures are summarized below:

- Since the preliminary criteria established in this study for consistently enhanced field mixture performance, including DASR porosity, disruption factor (DF), effective film thickness (EFT), and ratio between coarse and fine portion of the fine aggregate in the mixture (CFA/FFA), were only based on limited data from Superpave monitoring project, there is a need to validate and refine the criteria developed with a thorough laboratory study for incorporation into asphalt mix design.

- An effective and clear implementable set of gradation and volumetric criteria need to be further established for an enhancement of mix design procedure, which should result in longer lasting asphalt pavements, with the benefit being cost savings and less disruption to the public due to less frequent construction cycles.

- Conceptual relationships identified to describe changes in mixture properties over time will serve as the foundation for further development of improved models based on additional field data and laboratory studies using more advanced laboratory conditioning procedures.

- Since healing has been determined to be one of the most critical factors that affect cracking performance of asphalt mixtures, further investigations are needed to assess the relationships between DASR-IC criteria developed and healing
characteristics of asphalt mixtures using the healing test recently developed in other FDOT research.
APPENDIX
DETERMINISTIC PROCEDURE FOR ESTIMATION OF CRACK INITIATION TIME BASED ON CRACK RATING DATA

Figure A-1. Determination of observed crack initiation time for Project 1 and 2

Figure A-2. Determination of observed crack initiation time for Project 3 and 4
Figure A-3. Determination of observed crack initiation time for Project 5 and 6

Figure A-4. Determination of observed crack initiation time for Project 7 and 8
Figure A-5. Determination of observed crack initiation time for Project 9 and 10

Figure A-6. Determination of observed crack initiation time for Project 11 and 12
LIST OF REFERENCES


BIOGRAPHICAL SKETCH

Sanghyun Chun was born in Seoul, South Korea in 1977. He attended Kyunghee University and received a Bachelor of Engineering degree in civil engineering in 2003. In the middle of his undergraduate studies, he served as a sergeant in the Republic of Korean Army from 1998 to 2000. In February 2003, Sanghyun started a Master of Engineering program in civil engineering at the Kyunghee University. After finishing his master’s degree, in February 2005, he joined the Chungsuk Engineering Co. Ltd. in Seoul and he worked as a chief engineer in Road and Airport Division for two years. His academic pursuit led him to attend the Ph.D. program of pavement and materials group at the University of Florida in 2008 and worked as a graduate research assistant with doctoral advisor, Dr. Reynaldo Roque. After completing his Ph.D., he plans to work as academia, government, agencies, or private companies in civil engineering.