EVALUATION OF ASPHALT STABILIZED LAYER EQUIVALENCY FACTORS FOR USE IN AIRFIELD FLEXIBLE PAVEMENT DESIGN

BY

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THESIS

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ABSTRACT

A California Bearing Ratio (CBR) design method has been used for Air Force airfield pavement design since the late 1920s. The procedures, calculations, and related nomographs have been updated to reflect developments in the pavement design, pavement analysis, and unique landing gear configurations and load levels introduced by new aircraft. Throughout these changes, the basic objective of CBR procedures is to design a pavement from the bottom up, through the consideration of the strength of a material (based on its CBR) and determining the amount of cover needed to protect that material.

In a 1970s era update to the CBR procedure, equivalency factors were developed to allow incorporation of stabilized layers into the airfield pavement structure. The equivalency factor is a ratio which identifies the required stabilized material layer thickness needed to replace a specified thickness of traditional, unbound pavement. These equivalency factors were based on full scale tests performed by the Army Corps of Engineers at the Waterways Experiment Station.

Current efforts to update and revise tri-service airfield pavement design guidance have focused on the established equivalency factors – with a desire to evaluate these factors to ensure their appropriateness, given new laboratory material testing methods and mechanistic modeling methods which have been developed in the last 40 years. This study outlines the process used to apply both linear and non-linear pavement evaluation techniques to determine the adequacy of established equivalency factors for asphalt stabilized base and subbase materials.

Using material properties obtained from literature, various pavement structures containing both stabilized and unbound materials were evaluated under a standardized aircraft gear load. Load induced pavement critical responses were obtained using BISAR 3.0 and GT Pave. Analyzing these critical response magnitudes, the adequacy of established asphalt stabilized material factors can be determined and recommendations for more appropriate, yet still conservative, equivalency factors can be made.
DISCLAIMER

The views expressed in this thesis are those of the author and do not reflect the official policy or position of the United States Air Force, Department of Defense, or the U.S. Government.
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CHAPTER 1
INTRODUCTION

1.1 Airfield Pavement Design Method Overview

Military airfield pavement design procedures are outlined in Unified Facilities Criteria (UFC) 3-260-02, *Airfield Pavement Design*. Two design procedures are outlined for flexible pavements, a California Bearing Ratio (CBR) design method and a linear elastic design method. Both methods require inputs describing airfield traffic levels and design aircraft; however, material inputs and design philosophies vary greatly between the two methods.

Regardless of the method chosen, the same descriptors of traffic and load are utilized. The first is airfield type. As seen in Table 1.1, the airfield type classification is based upon the type and frequency of aircraft operations. While Table 1.1 was generated based upon aircraft type and typical traffic levels at “selected” Air Force bases, airfields can be designed for alternate aircraft or pass levels depending on requirements (UFC 3-260-02).

Table 1.1. Design Gross Weights and Pass Levels based upon Airfield Type (UFC 3-260-02)

<table>
<thead>
<tr>
<th>Airfield Type</th>
<th>Design Aircraft</th>
<th>A Traffic Area</th>
<th>B Traffic Area</th>
<th>C Traffic Area</th>
<th>D Traffic Area</th>
<th>Overruns</th>
<th>Shoulder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light</td>
<td>F-15 C/D C-17</td>
<td>68,000</td>
<td>400,000</td>
<td>68,000</td>
<td>400,000</td>
<td>NA</td>
<td>4,000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>585,000</td>
<td>400</td>
<td>585,000</td>
<td>400</td>
<td>NA</td>
<td>4</td>
</tr>
<tr>
<td>Medium</td>
<td>F-15 E C-17 B-52</td>
<td>81,000</td>
<td>100,000</td>
<td>81,000</td>
<td>100,000</td>
<td>60,750</td>
<td>1,000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>585,000</td>
<td>400</td>
<td>585,000</td>
<td>400</td>
<td>438,750</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>490,000</td>
<td>120,000</td>
<td>490,000</td>
<td>120,000</td>
<td>300,000</td>
<td>400</td>
</tr>
<tr>
<td>Heavy</td>
<td>F-15 E C-17 B-52</td>
<td>81,000</td>
<td>100,000</td>
<td>81,000</td>
<td>100,000</td>
<td>60,750</td>
<td>1,000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>585,000</td>
<td>400</td>
<td>585,000</td>
<td>400</td>
<td>438,750</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>490,000</td>
<td>120,000</td>
<td>490,000</td>
<td>120,000</td>
<td>300,000</td>
<td>400</td>
</tr>
<tr>
<td>Modified</td>
<td>F-15 E C-17 B-1</td>
<td>81,000</td>
<td>100,000</td>
<td>81,000</td>
<td>100,000</td>
<td>60,750</td>
<td>1,000</td>
</tr>
<tr>
<td>Heavy</td>
<td></td>
<td>585,000</td>
<td>400</td>
<td>585,000</td>
<td>400</td>
<td>438,750</td>
<td>400</td>
</tr>
<tr>
<td></td>
<td></td>
<td>490,000</td>
<td>120,000</td>
<td>490,000</td>
<td>120,000</td>
<td>300,000</td>
<td>400</td>
</tr>
<tr>
<td>Assault</td>
<td>C-130</td>
<td>175,000</td>
<td>50,000</td>
<td>100,000</td>
<td>50,000</td>
<td>60,750</td>
<td>2,000</td>
</tr>
<tr>
<td>Landing Zone</td>
<td></td>
<td>502,000</td>
<td>100,000</td>
<td>502,000</td>
<td>100,000</td>
<td>502,000</td>
<td>100,000</td>
</tr>
</tbody>
</table>

The second descriptor of traffic and load is traffic area. The traffic area is selected based upon the location of the pavement structure within the airfield system. A general schematic illustrating traffic areas is shown in Figure 1.1. Type A areas are loaded with channelized, full weight traffic and are used on
primary taxiways and on the end portions of runways. Type B areas are loaded with non-channelized, full weight traffic and are used on aprons. Type C areas are loaded with channelized, partial weight traffic loads and are used on the center portions of runways and secondary taxiways. Due to lift generated by aircraft wings, the full aircraft weight is not imparted on the pavement in the center portions of runways. Landing aircraft which primarily use the secondary taxiways typically have a low fuel load, leading to partial weight traffic. Finally, very minimal to no aircraft loading is intended on Type D areas, which are used for shoulders. Type D areas are primarily used to reduce foreign object debris generation at the edges of the runway and are generally designed to endure loading by snow removal and pavement maintenance equipment.

![Traffic Area Schematic (UFC 3-260-02)](image)

**Figure 1.1.** Traffic Area Schematic (UFC 3-260-02)

CBR measurements have been used in military airfield construction dating as early as 1928. The design procedure took its current form in 1959, when nomographs were first produced to guide the process. The nomographs were updated throughout the subsequent decades, as pavement design theories
developed (Ahlvin 1991). The nomographs, which are specific to particular traffic levels and aircraft types, indicate the level of cover required to adequately protect each underlying material from load induced stresses and strains. Measured CBR values of the various pavement and in-situ materials at specific compaction levels are required inputs to the nomograph. The pavement is designed from the top down, ensuring that each subsequent layer has the required level of protection to withstand loading of the designated aircraft.

The linear elastic design method is a more recent development, first introduced in 1975 as an optional design procedure for those airfields intended to serve heavy aircraft (Ahlvin 1991). Subsequently, the method has been expanded to all aircraft types. This method utilizes the same traffic level and aircraft type inputs as the CBR method. As opposed to CBR, though, Young’s modulus (E) and Poisson’s ratio are used to characterize each pavement material. Burmister’s equation for multi-layered elastic continua is used to calculate strain at the bottom of the asphalt surface and at the top of the subgrade (UFC 3-260-02). To simplify and expedite the process, linear elastic design is typically performed using United States Army Corps of Engineers (USACE) Pavement-Transportation Computer Assisted Structural Engineering (PCASE) software due to the calculations required.

1.2 Equivalency Factor Overview

While stabilized layers can be easily incorporated into the linear elastic design process by using the applicable modulus and Poisson’s ratio, accounting for these improved layers in a pavement structure designed using the CBR method is an empirical process.

The use of improved layers in pavement became popular in the highway community after the American Association of State Highway and Transportation Officials road test in the 1960s. USACE testing at that time indicated that equivalency factors in use by the highway community were not adequate for airfield use. In order to obtain acceptable airfield equivalency factors, USACE undertook full scale field testing at the Waterway Experiment Station in 1974. Based on these test results, equivalency factors for airfield pavement were introduced in 1977 (Ahlvin, 1991).

In the current CBR design procedures, the entire pavement structure design is completed using traditional, unbound materials. If the use of an improved layer is desired, an equivalency factor is used to correlate the needed thickness of the improved (or stabilized) pavement layer to the design thickness of the traditional material in the same pavement position (base or subbase) to obtain the same level of structural benefit. For example, when considering a traditional aggregate base and an asphalt stabilized base, the asphalt stabilized base might have an equivalency factor of 1.70. This means that one inch of asphalt stabilized base provides the same structural benefit to the pavement as 1.70 inches of traditional aggregate base. Thus, a 10 inch asphalt stabilized base can replace a 17 inch traditional aggregate base.
Current equivalency factors for asphalt stabilized materials are provided in Table 1.2. It can be seen that equivalency factors change based on stabilized material type and position in the pavement structure.

Table 1.2. Established Stabilized Asphalt Equivalency Factors (after UFC 3-260-02)

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Base EF</th>
<th>Subbase EF</th>
</tr>
</thead>
<tbody>
<tr>
<td>All-Bituminous Concrete</td>
<td>1.15</td>
<td>2.30</td>
</tr>
<tr>
<td>GW, GP, GM, GC</td>
<td>1.00</td>
<td>2.00</td>
</tr>
<tr>
<td>SW, SP, SM, SC</td>
<td>Not Used</td>
<td>1.50</td>
</tr>
</tbody>
</table>

It should be noted that the “all-bituminous” category refers to a hot mix asphalt concrete, similar to a leveling binder. The gravel and sand categories refer to the respective material cold mixed with an emulsion or cutback. Additionally, under current guidance, no base course layer thickness reduction is realized when using an asphalt stabilized aggregate, as the assigned equivalency factor is 1.00.

### 1.3 Asphalt-Stabilized Layer Design

In order for the equivalency factors shown in Table 1 to be applicable, the various asphalt stabilized layers should be designed using established methods.

#### 1.3.1 All-Bituminous Design

Hot mix asphalt – or the all bituminous layer in the stabilized layer vernacular – is designed in accordance with UFC 3-250-03, *Standard Practice Manual for Flexible Pavements*. Gradations for various nominal maximum size aggregate blends are first provided. After determining the aggregate blend, the design procedure is similar to the Marshall Mix design method, where Marshall Stability, flow, percent voids in the total mix, and percent voids filled with asphalt are used to determine the optimum asphalt content. Specifications are provided for compaction levels of both 50 and 75 blows of the Marshall Compaction hammer. Additionally, procedures for determining optimum asphalt content using a gyratory compactor are also provided. After optimum asphalt content has been determined from either of the above methods, moisture susceptibility is evaluated through determining the tensile strength ratio.

#### 1.3.2 Aggregate and Sand Design

While information on aggregate stabilization is briefly discussed in UFC 3-250-03, detailed information for both aggregate and sand is provided in UFC 3-250-11, *Soil Stabilization for Pavements*. Again, recommended gradations for both aggregate and sand are provided. Material gradation information is then used to estimate asphalt requirements, through the use of Table 1.3. It can be seen that percent
passing both the #10 and #200 sieves is used to determine emulsified asphalt content. If sieve data falls between the percentages indicated on the table, then interpolation is used to determine the proper emulsified asphalt content.

**TABLE 1.3. Estimated Asphalt Contents based on Sieve Analysis Data (UFC 3-250-11)**

<table>
<thead>
<tr>
<th>Percent Passing No. 200 Sieve</th>
<th>&lt;50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>6.0</td>
<td>6.3</td>
<td>6.5</td>
<td>6.7</td>
<td>7.0</td>
<td>7.2</td>
</tr>
<tr>
<td>2</td>
<td>6.3</td>
<td>6.6</td>
<td>6.7</td>
<td>7.0</td>
<td>7.2</td>
<td>7.5</td>
</tr>
<tr>
<td>4</td>
<td>6.5</td>
<td>6.7</td>
<td>7.0</td>
<td>7.2</td>
<td>7.5</td>
<td>7.7</td>
</tr>
<tr>
<td>6</td>
<td>6.7</td>
<td>7.0</td>
<td>7.2</td>
<td>7.5</td>
<td>7.7</td>
<td>7.9</td>
</tr>
<tr>
<td>8</td>
<td>7.0</td>
<td>7.2</td>
<td>7.5</td>
<td>7.7</td>
<td>7.9</td>
<td>8.2</td>
</tr>
<tr>
<td>10</td>
<td>7.2</td>
<td>7.5</td>
<td>7.7</td>
<td>7.9</td>
<td>8.2</td>
<td>8.4</td>
</tr>
<tr>
<td>12</td>
<td>7.5</td>
<td>7.7</td>
<td>7.9</td>
<td>8.2</td>
<td>8.4</td>
<td>8.6</td>
</tr>
<tr>
<td>14</td>
<td>7.2</td>
<td>7.5</td>
<td>7.7</td>
<td>7.9</td>
<td>8.2</td>
<td>8.4</td>
</tr>
<tr>
<td>16</td>
<td>7.0</td>
<td>7.2</td>
<td>7.5</td>
<td>7.7</td>
<td>7.9</td>
<td>8.2</td>
</tr>
<tr>
<td>18</td>
<td>6.7</td>
<td>7.0</td>
<td>7.2</td>
<td>7.5</td>
<td>7.7</td>
<td>7.9</td>
</tr>
<tr>
<td>20</td>
<td>6.5</td>
<td>6.7</td>
<td>7.0</td>
<td>7.2</td>
<td>7.5</td>
<td>7.6</td>
</tr>
<tr>
<td>22</td>
<td>6.3</td>
<td>6.5</td>
<td>6.7</td>
<td>7.0</td>
<td>7.2</td>
<td>7.5</td>
</tr>
<tr>
<td>24</td>
<td>6.0</td>
<td>6.3</td>
<td>6.5</td>
<td>6.7</td>
<td>7.0</td>
<td>7.2</td>
</tr>
<tr>
<td>25</td>
<td>6.2</td>
<td>6.4</td>
<td>6.6</td>
<td>6.9</td>
<td>7.1</td>
<td>7.3</td>
</tr>
</tbody>
</table>

While Table 1.3 provides the estimated quantity of emulsion needed, it does not provide specific detail on the residual asphalt content of the emulsion. The residual asphalt content is the percentage of asphalt binder in the emulsion (as opposed to water or emulsifying agent) and commonly is referred to in literature. UFC 3-250-11 does specify the types of emulsions to be used, therefore minimum residual asphalt percentages for these products can be determined from two applicable ASTM standard specifications: ASTM D977, *Standard Specification for Emulsified Asphalt* and ASTM D2397, *Standard Specification for Cationic Emulsified Asphalt*. Table 1.4 provides the minimum residual asphalt percentages specified in the ASTM specifications for the emulsion types outlined in UFC 3-250-11.

**TABLE 1.4. Minimum Residual Asphalt**

<table>
<thead>
<tr>
<th>Emulsion Type</th>
<th>Minimum Residual Asphalt (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MS-2, CMS-2</td>
<td>65</td>
</tr>
<tr>
<td>SS-1, SS-1h, CSS-1, CSS-1h</td>
<td>57</td>
</tr>
</tbody>
</table>

Since the emulsion addition rates provided in Table 1.3 are approximate, the final design of the stabilized material is based on Marshall Stability test results. The stability test is to be performed in accordance with CRD-C 649-95, *Standard Test Method for Unit Weight, Marshall Stability, and Flow of Bituminous Mixtures*. The procedures outlined in CRD-C 649 perform the test on an unsoaked specimen.
UFC 3-250-11 indicates that a minimum stability of 500 pounds is required for sand mixtures. No minimum stability is provided for aggregate mixtures.

1.4 Need for Investigation

Details surrounding the 1977 field testing procedures (such as gear configuration, gear load, pavement structure, number of load repetitions, and testing cycles) and evaluation methods (such as gauge types, gauge locations, and data analysis procedures) are not readily available. Advances in computer modeling and increased understanding of mechanistic responses of pavement systems under loading can provide stress and strain information throughout the pavement structure, thus allowing for a more robust analysis of the affects of how asphalt stabilized layers affect the responses of the pavement structure. A robust mechanistic analysis of asphalt stabilized bases and subbases will either provide validity to the established equivalency factors or will provide a base for proposing new equivalency factors. Accurate equivalency factors will allow for conservative, yet economical new pavement design. Equivalency factors are also used in evaluation of existing airfield pavements, as described in UFC 3-260-03, *Airfield Pavement Evaluation*, thus accurate factors will facilitate safe aircraft operations on existing airfield pavements.

1.5 Organization

This thesis describes the evaluation of the existing asphalt stabilized layer equivalency factors and proposes updated equivalency factors based on both linear-elastic and non-linear pavement analyses.  

- Chapter 1 introduces the basic concepts of flexible military airfield pavement design, including design procedures and use of equivalency factors. Additionally, basic information on asphalt stabilized layer design is presented.
- Chapter 2 is an overview of available literature where asphalt stabilized base and subbase materials were tested to determine material properties. These mechanistic properties, combined with engineering judgment, were used to establish typical modulus ranges for asphalt stabilized materials.
- Chapter 3 develops the baseline pavement structure and standard gear load. The baseline pavement structure will be manipulated with asphalt stabilized layers of various thicknesses in subsequent chapters. Critical responses of the pavement (horizontal tensile strain at the bottom of the asphalt surface, vertical compressive stress at the top of the subgrade, and vertical compressive strain at the top of the subgrade) are evaluated under loading from the standard gear load on both the baseline and asphalt modified pavement system.
Chapter 4 presents the methodology used to perform the linear analysis, along with the results. Linear pavement analysis is used to evaluate existing equivalency factors for both base and subbase materials. The established equivalency factor analysis is then used as the cornerstone to propose new equivalency factors which better adhere to allowable pavement critical responses.

Chapter 5 details the process used to test the proposed equivalency factors developed in Chapter 4 using non-linear methods and the corresponding test results. Using the non-linear properties of various stabilized and unbound layers as determined from laboratory analyses found in the literature in conjunction with non-linear evaluation software, the effects of stress hardening and softening characteristics under loading can be better evaluated.

Chapter 6 provides final equivalency factor recommendations, summary, and conclusions.
CHAPTER 2
REVIEW AND ANALYSIS OF DATA IN LITERATURE

A literature review was conducted to determine characteristic mechanistic properties of asphalt stabilized materials. It was revealed that there are a wide range of mix design procedures and laboratory methods used. Additionally, as mentioned in Chapter 1, emulsion specifications are fairly general, only providing minimum residual asphalt contents and charge designation. The intent of this section is to not only review applicable literature, but to also compare it to asphalt stabilized mixture design guidance provided in UFC 3-250-11, as this document is expected to be referenced by Department of Defense users when an asphalt stabilized layer(s) is targeted for use in airfield pavement construction. Furthermore, UFC 3-250-11 does not provide any laboratory methods or standardized compliance testing to determine an acceptable mix design. This information is also discussed for reference and comparison between sources.

2.1 Stabilized Crushed Aggregate

2.1.1 Darter et al, 1978. This work studied how several different factors affect the structural responses of emulsion aggregate mixtures (EAM). The data presented in the report was later used to develop a mix design procedure for EAMs. Laboratory testing completed on specimens included modified Marshall Stability and resilient modulus (indirect tensile method) on both soaked and unsoaked specimens. Since stability and resilient modulus testing was completed on the same sample sets, it provides an opportunity to observe the measured material resilient modulus at the stability requirements prescribed by UFC 3-250-11.

A crushed limestone aggregate was used. The sieve analysis of the material, along with the limits outlined in UFC 3-250-11 for a three quarter inch maximum aggregate size, is shown in Figure 2.1. The crushed limestone does not meet specifications for the percent passing the #50 and #200 sieves. The aggregate fines had a plasticity index of 10.
The tests used a high float emulsion, with 70% asphalt. EAMs samples at 2.0%, 3.5%, and 5.0% residual asphalt contents were produced. Compaction occurred at 2.5% moisture content under 75 blows of a Marshall Compaction hammer. Both stability and resilient modulus test results are based on a 14 day dry curing period and dry testing at 72 degrees Fahrenheit.

Figure 2.2 shows both the stability and resilient modulus for the range of residual asphalt contents. The dry stability results are plotted to match the procedure outlined in CRD-C 649-95. It can be seen that resilient modulus and stability follow the same trend over the range of asphalt contents.

Using the sieve analysis, the asphalt content estimated by Table 1.2 can be determined in order to compare the UFC estimate to the range of residual asphalt contents used in this study. The percent passing the #10 sieve was interpolated. The residual asphalt content estimated by UFC 3-250-11 is shown in

**FIGURE 2.1.** Darter Crushed Limestone Gradation

**FIGURE 2.2.** Resilient Modulus and Stability Data for Darter Crushed Limestone
Table 2.1. Residual asphalt contents used in this study correspond relatively well to those estimated by Table 1.2.

Table 2.1. Residual Asphalt Content Estimate Based on UFC Guidance for Darter Crushed Limestone

<table>
<thead>
<tr>
<th>Percent Passing #200 Sieve</th>
<th>25%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent Passing #10 Sieve</td>
<td>44%</td>
</tr>
<tr>
<td>Percent Emulsion (by weight)</td>
<td>6.2%</td>
</tr>
<tr>
<td>Residual Asphalt Content (70% Asphalt Emulsion)</td>
<td>4.3%</td>
</tr>
</tbody>
</table>

2.1.2 Anderson and Thompson, 1995. The goal of Anderson and Thompson’s work was to perform testing on EAMs outside of the more common stability, tensile strength, and fatigue performance ranges reported by other researchers. Their work focused on shear strength, repeated load response, and dynamic cone penetrometer characteristics of EAMs. Materials used in this work were similar to what was currently being used in the State of Illinois for EAM road bases and included both laboratory and mixing plant prepared aggregate blends.

All EAM mixture designs were generated using the Illinois Department of Transportation (IDOT) procedures. Based on IDOT’s Physical Research Report No. 91, samples are compacted with 75 blows of the Marshall Compaction hammer prior to a 24 hour cure at 72 degrees Fahrenheit. After a four day soaking procedure, stability was measured. To be an acceptable design, the sample must have a minimum soaked stability of 500 pounds.

For laboratory testing specimens, the aggregate, emulsion, and any additional mixing water was mixed. The specimen remained in the uncompacted state until 50% of the moisture had evaporated. The specimen was then compacted and allowed to cure at 100 degrees Fahrenheit and 25% relative humidity, until the samples reached a constant weight. It is reported that curing time was “several months,” however exact measurements of curing time were not provided.

A crushed dolomitic limestone was used in testing. Aggregate fines had a plasticity index of 4. The sieve analysis of the material is shown in Figure 2.3. The aggregate gradation used falls outside of UFC requirements slightly.
The aggregate was tested with two different types of emulsions, a high float emulsion with 67.5% residual asphalt and a CSS-1 emulsion with 62.3% residual asphalt. Based on IDOT mix design procedures, it was determined that a 4.0% residual asphalt content met the minimum stability requirements.

Resilient modulus was determined using the triaxial method. A range of bulk stresses between 25 pounds per square inch and 150 pounds per square inch were used. Resilient moduli measured for the two emulsions over the range of bulk stresses are depicted in Figure 2.4. It can be seen that emulsion type can have a significant impact on the stress dependent properties of an asphalt stabilized material.

**Figure 2.3. Anderson Dolomitic Crushed Limestone Gradation**

**Figure 2.4. Resilient Modulus Data Based on Emulsion Type for Anderson Dolomitic Crushed Limestone**
Using the sieve analysis, the asphalt content estimated by Table 2.2 can be determined in order to compare the UFC estimate to the range of residual asphalt contents used in this study. The percent passing the #10 sieve was interpolated. The residual asphalt content estimated by UFC 3-250-11 is shown in Table 2.2. Residual asphalt content used in this study is lower than the estimates provided by Table 1.2.

Table 2.2. Residual Asphalt Content Estimate Based on UFC Guidance for Anderson Dolomitic Crushed Limestone

<table>
<thead>
<tr>
<th>Percent Passing #200 Sieve</th>
<th>9%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent Passing #10 Sieve</td>
<td>18%</td>
</tr>
<tr>
<td>Percent Emulsion (by weight)</td>
<td>7.2%</td>
</tr>
<tr>
<td>Residual Asphalt Content</td>
<td></td>
</tr>
<tr>
<td>(67.5% Asphalt HF Emulsion)</td>
<td>4.9%</td>
</tr>
<tr>
<td>Residual Asphalt Content</td>
<td></td>
</tr>
<tr>
<td>(62.3% Asphalt CSS-1 Emulsion)</td>
<td>4.5%</td>
</tr>
</tbody>
</table>

2.1.3 Farrar and Ksaibati, 1996. This paper focused on finding the modulus properties of lean emulsion stabilized base materials and determining how the addition of emulsion affected the resilient modulus of the materials. The emulsion mixes were compacted to 100% modified proctor maximum density. The rationale behind the chosen emulsion contents was not provided.

Initially, a crushed Minnekahta limestone was tested. The aggregate had non-plastic fines. The sieve analysis of the material is shown in Figure 2.5. The aggregate gradation used falls outside of UFC requirements slightly.

Figure 2.5. Farrar Minnekahta Limestone Gradation
A CSS-1H emulsion was used in the testing. No additional emulsion information was provided, however it is likely that the emulsion met ASTM specifications since the testing was completed for the Wyoming Department of Transportation (WYDOT). WYDOT’s Standard Specifications for Road and Bridge Construction require use of emulsions meeting ASTM D977 and D2397 (2010). Emulsion was added at one, two, and three percent.

The limestone emulsion mixtures were allowed to cure for three weeks before testing. Information on curing conditions was not provided. Resilient modulus was determined using the triaxial method. A range of bulk stresses between 12 pounds per square inch and 80 pounds per square inch were used. Resilient moduli measured for the three different emulsion contents over the range of bulk stresses are depicted in Figure 2.6. The effects of emulsion content can be seen on the resilient modulus of the mixture.

![Figure 2.6. Resilient Modulus Data Based on Emulsion Content for Farrar Minnekahta Limestone](image)

Using the sieve analysis, the asphalt content estimated by Table 1.2 can be determined in order to compare the UFC estimate to the range of residual asphalt contents used in this study. The percent passing the #10 sieve was interpolated. The emulsion content estimated by UFC 3-250-11 is shown in Table 2.3. Emulsion content used in this study is significantly lower than the Table 1.2 estimate. This is expected, as the study investigated the properties of material stabilized with small amounts of emulsion.

<table>
<thead>
<tr>
<th>Percent Passing #200 Sieve</th>
<th>10%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent Passing #10 Sieve</td>
<td>28%</td>
</tr>
<tr>
<td><strong>Percent Emulsion (by weight)</strong></td>
<td><strong>7.2%</strong></td>
</tr>
</tbody>
</table>

Table 2.3. Emulsion Content Estimate Based on UFC Guidance for Farrar Minnekahta Limestone
2.2 Stabilized Partially Crushed Aggregate

2.2.1 Darter et al, 1978. Using the same methods discussed in Section 2.1.1, asphalt stabilized mixes made with pit run gravel were also evaluated. The sieve analysis of the material is shown in Figure 2.7. The aggregate gradation used meets UFC requirements.

![Figure 2.7. Darter Pit Run Gravel Gradation](image)

Again, stability and modulus data can be plotted together over the range of residual asphalt contents. These results can be seen in Figure 2.8. It can be seen that both modulus and stability fall as residual asphalt increases.

![Figure 2.8. Resilient Modulus and Stability Data for Darter Pit Run Gravel](image)

Using the sieve analysis, the asphalt content estimated by Table 1.2 can be determined in order to compare the UFC estimate to the range of residual asphalt contents used in this study. The percent passing
the #10 sieve was interpolated. The emulsion content estimated by UFC 3-250-11 is shown in Table 2.4. The Table 1.2 estimate falls within the range of residual asphalt contents used in this study.

Table 2.4. Residual Asphalt Content Estimate Based on UFC Guidance for Darter Pit Run Gravel

<table>
<thead>
<tr>
<th>Percent Passing #200 Sieve</th>
<th>7%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent Passing #10 Sieve</td>
<td>28%</td>
</tr>
<tr>
<td>Percent Emulsion (by weight)</td>
<td>6.9%</td>
</tr>
<tr>
<td><strong>Residual Asphalt Content</strong> (70% Asphalt Emulsion)</td>
<td><strong>4.8%</strong></td>
</tr>
</tbody>
</table>

2.2.2 Anderson and Thompson, 1995. Using the same methods discussed in Section 2.1.2, asphalt stabilized mixes made with river gravel were also evaluated. The sieve analysis of the material is shown in Figure 2.9. The aggregate gradation falls outside of UFC requirements.

![Figure 2.9. Anderson River Gravel Gradation](image)

Resilient modulus was determined using the triaxial method. A range of bulk stresses between 25 pounds per square inch and 150 pounds per square inch were used. The river gravel was only tested with the HFE emulsion. Figure 2.10 shows these results, along with the general stress hardening nature of the material.
Using the sieve analysis, the asphalt content estimated by Table 1.2 can be determined in order to compare the UFC estimate to the range of residual asphalt contents used in this study. The percent passing the #10 sieve was interpolated. The emulsion content estimated by UFC 3-250-11 is shown in Table 2.5. The Table 1.2 estimate is again higher that the 4.0% residual asphalt content used in this study.

Table 2.5. Residual Asphalt Content Estimate Based on UFC Guidance for Anderson River Gravel

<table>
<thead>
<tr>
<th>Percent Passing #200 Sieve</th>
<th>4%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent Passing #10 Sieve</td>
<td>34%</td>
</tr>
<tr>
<td>Percent Emulsion (by weight)</td>
<td>6.5%</td>
</tr>
<tr>
<td><strong>Residual Asphalt Content</strong></td>
<td><strong>4.4%</strong></td>
</tr>
</tbody>
</table>

(67.5% Asphalt HF Emulsion)

In addition to the river gravel, a lab mixed partially crushed gravel was also investigated. The sieve analysis of the material is shown in Figure 2.11. The aggregate gradation falls slightly outside of UFC requirements. The aggregate had non-plastic fines.
Resilient modulus was determined using the triaxial method. A range of bulk stresses between 25 pounds per square inch and 150 pounds per square inch were used. Resilient moduli measured for the two emulsions over the range of bulk stresses are depicted in Figure 2.12. Again, the stress hardening behavior for both emulsions is seen, along with differing behavior based on the emulsion used.

Using the sieve analysis, the asphalt content estimated by Table 1.2 can be determined in order to compare the UFC estimate to the range of residual asphalt contents used in this study. The percent passing the #10 sieve was interpolated. The emulsion content estimated by UFC 3-250-11 is shown in Table 2.6. The Table 1.2 estimate is again higher than the 4.0% residual asphalt content used in this study.
Table 2.6. Residual Asphalt Content Estimate Based on UFC Guidance for Anderson Partially Crushed Gravel

<table>
<thead>
<tr>
<th>Percent Passing #200 Sieve</th>
<th>9%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent Passing #10 Sieve</td>
<td>27%</td>
</tr>
<tr>
<td>Percent Emulsion (by weight)</td>
<td>7.1%</td>
</tr>
<tr>
<td><strong>Residual Asphalt Content</strong> (67.5% Asphalt HF Emulsion)</td>
<td>4.8%</td>
</tr>
<tr>
<td><strong>Residual Asphalt Content</strong> (62.3% Asphalt CSS-1 Emulsion)</td>
<td>4.4%</td>
</tr>
</tbody>
</table>

2.2.3 Farrar and Ksaibati, 1996. Using the same methods discussed in Section 2.1.3, alluvial terrace gravel was also tested. The sieve analysis of the material is shown in Figure 2.13. The aggregate gradation nearly meets UFC requirements. Additionally, this aggregate had non-plastic fines.

Figure 2.13. Farrar Alluvial Terrace Gravel Gradation

A CSS-1H emulsion was again used for producing the asphalt stabilized material. However, emulsion was only added at one and two percent. The gravel emulsion was allowed to cure for two weeks. However, the two week resilient modulus data for the two percent emulsion addition rate was not presented in the text. Resilient modulus was determined using the triaxial method. A range of bulk stresses between 12 pounds per square inch and 80 pounds per square inch were used. Resilient moduli measured for the one percent emulsion content cured for two weeks and the two percent emulsion content cured for one week are shown in Figure 2.14.
Using the sieve analysis, the asphalt content estimated by Table 1.2 can be determined in order to compare the UFC estimate to the range of residual asphalt contents used in this study. The percent passing the #10 sieve was interpolated. The emulsion content estimated by UFC 3-250-11 is shown in Table 2.7. Emulsion content used in this study is significantly lower than the Table 1.2 estimate.

**Table 2.7. Emulsion Content Estimate Based on UFC Guidance for Farrar Minnekahta Limestone**

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent Passing #200 Sieve</td>
<td>6%</td>
</tr>
<tr>
<td>Percent Passing #10 Sieve</td>
<td>38%</td>
</tr>
<tr>
<td>Percent Emulsion (by weight)</td>
<td><strong>6.6%</strong></td>
</tr>
</tbody>
</table>

2.2.4 Finberg, Quire, and Thomas, 2008. Data presented in this section was collected in order to design an asphalt stabilized base for a road in Las Vegas. The base was to be constructed in place - with the asphalt surface being removed and the in situ base material being processed with emulsion and a new asphalt surface being installed. Thus, samples collected from the project site composed the aggregate used for the study.

The aggregate was poorly graded sand with silty clay and gravel. The fines had a plasticity index of seven. The sieve analysis of this material plotted with the UFC 3-250-11 requirements is presented in Figure 2.15. The material does not meet gradation requirements.
A cationic emulsion with 65% residual asphalt was used in the mixes. Samples were produced with emulsion contents of 4, 5, 6, and 7 percent. After mixing, the samples were compacted in a Superpave gyratory compactor for 30 gyrations under an 87 pound per square inch load. After compaction, the samples were cured for 72 hours at a temperature of 104 degrees Fahrenheit. The samples were allowed to cool to 77 degrees Fahrenheit before testing. Resilient modulus testing was accomplished using the indirect tensile method. The resilient modulus measured at each emulsion content is provided in Figure 2.16. Resilient modulus decreases as emulsion content increases.

Using the sieve analysis, the asphalt content estimated by Table 1.2 can be determined in order to compare the UFC estimate to the range of residual asphalt contents used in this study. The percent passing
the #10 sieve was interpolated. The emulsion content estimated by UFC 3-250-11 is shown in Table 2.8. Emulsion content used in this study is higher than the Table 1.2 estimate.

<table>
<thead>
<tr>
<th>Percent Passing #200 Sieve</th>
<th>11%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent Passing #10 Sieve</td>
<td>40%</td>
</tr>
<tr>
<td>Percent Emulsion (by weight)</td>
<td>7.3%</td>
</tr>
</tbody>
</table>

Table 2.8. Emulsion Content Estimate Based on UFC Guidance for Finberg Base

2.3 Stabilized Sand

Research which investigated asphalt stabilized sand was limited. However, Darter et al, did include a sand mixture in the previously discussed work. The methods and techniques outlined in Sections 2.1.1 and 2.2.1 also apply to the sand mixture. The sieve analysis of the material is shown in Figure 2.17. The aggregate gradation used meets UFC requirements.

![Figure 2.17. Darter Sand Gradation](image)

Again, stability and modulus data can be plotted together over the range of residual asphalt contents. These results can be seen in Figure 2.18. The resilient modulus peaks at the 3.5% residual asphalt content, while the stability continues to increase over the range of residual asphalt contents. It can also be seen that the stability at all asphalt contents is below the UFC 3-250-11 requirement of 500 pounds.
2.4 Summary

A summary of all the literature reviewed is provided in Table 2.8. Several points can be concluded based on the literature review:

1. The aggregate gradation specifications provided in UFC 3-250-11 may be overly stringent. A well graded aggregate produces an asphalt stabilized material with better engineering properties than a poorly graded aggregate, as the asphalt emulsion coats the aggregate as opposed to filling voids, as with lime and cement. However, the desire for a very well graded aggregate needs to be balanced with the fact that marginal aggregates are often chosen for stabilization.

2. Methods for performing mix design, laboratory specimen production, and testing are widely varied. These variations can cause considerable differences in test results.

3. Emulsion type used can have a considerable effect on the engineering properties of the asphalt stabilized material. The range of material properties is evident in the literature reviewed, even though most (one study did not report on exact emulsion properties) emulsions met applicable ASTM standards.

4. UFC 3-250-11 Table 1.2 generally returns high emulsion contents compared to the mix designs performed in the literature reviewed.

5. The established asphalt stabilized material equivalency factors from UFC 3-260-02 only address all bituminous, aggregate, and sand layers. However, based on the literature review, there is a discernible difference in modulus between crushed and uncrushed aggregate.

Focusing on the resilient modulus ranges from Table 2.9, Table 2.10 summarizes the typical ranges for resilient moduli values based on material type.
### Table 2.9. Summary of Values from Literature Review

<table>
<thead>
<tr>
<th>Source</th>
<th>Type</th>
<th>Gradation</th>
<th>Meet Spec</th>
<th>Emulsion</th>
<th>Stability</th>
<th>Asphalt Content of Mix</th>
<th>Resilient Modulus</th>
<th>Test Method</th>
<th>Bulk Stress Range, Measured Value, ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Darter</td>
<td>Crushed Limestone</td>
<td>No</td>
<td>HFE</td>
<td>Yes</td>
<td>1483-3173</td>
<td>2.0% - 5.0%</td>
<td>14 @ 72F</td>
<td>Indirect Tensile</td>
<td>137-377 (945-2599)</td>
</tr>
<tr>
<td>Anderson</td>
<td>Crushed Limestone</td>
<td>No</td>
<td>HFE-300</td>
<td>Yes</td>
<td>&gt;500</td>
<td>4.0%</td>
<td>30+ @100F³</td>
<td>Triaxial</td>
<td>25-150 (172-1034) 43-99 (296-683)</td>
</tr>
<tr>
<td>Anderson</td>
<td>Crushed Limestone</td>
<td>No</td>
<td>CSS-1</td>
<td>Yes</td>
<td>&gt;500</td>
<td>4.0%</td>
<td>30+ @100F³</td>
<td>Triaxial</td>
<td>25-150 (172-1034) 87-213 (600-1469)</td>
</tr>
<tr>
<td>Farrar</td>
<td>Crushed Limestone</td>
<td>No</td>
<td>CSS-1H</td>
<td>N/T</td>
<td>n/a</td>
<td>1.0%</td>
<td>21⁴</td>
<td>Triaxial</td>
<td>12-80 (83-552) 40-163 (276-1124)</td>
</tr>
<tr>
<td>Farrar</td>
<td>Crushed Limestone</td>
<td>No</td>
<td>CSS-1H</td>
<td>N/T</td>
<td>n/a</td>
<td>2.0%</td>
<td>21⁴</td>
<td>Triaxial</td>
<td>12-80 (83-552) 89-210 (614-1448)</td>
</tr>
<tr>
<td>Farrar</td>
<td>Crushed Limestone</td>
<td>No</td>
<td>CSS-1H</td>
<td>N/T</td>
<td>n/a</td>
<td>3.0%</td>
<td>21⁴</td>
<td>Triaxial</td>
<td>12-80 (83-552) 96-226 (662-1558)</td>
</tr>
<tr>
<td>Darter</td>
<td>Pit Run Gravel</td>
<td>Yes</td>
<td>HFE</td>
<td>Yes</td>
<td>394-792</td>
<td>2.0% - 5.0%</td>
<td>14 @ 72F</td>
<td>Indirect Tensile</td>
<td>46-100 (317-689)</td>
</tr>
<tr>
<td>Anderson</td>
<td>Partially Crushed Gravel</td>
<td>No</td>
<td>HFE-300</td>
<td>Yes</td>
<td>&gt;500</td>
<td>4.0%</td>
<td>30+ @100F³</td>
<td>Triaxial</td>
<td>25-150 (172-1034) 83-145 (572-1000)</td>
</tr>
<tr>
<td>Anderson</td>
<td>Partially Crushed Gravel</td>
<td>No</td>
<td>CSS-1</td>
<td>Yes</td>
<td>&gt;500</td>
<td>4.0%</td>
<td>30+ @100F³</td>
<td>Triaxial</td>
<td>25-150 (172-1034) 71-176 (490-1213)</td>
</tr>
<tr>
<td>Anderson</td>
<td>Plant Mix Gravel</td>
<td>No</td>
<td>HFE-300</td>
<td>Yes</td>
<td>&gt;500</td>
<td>3.4%</td>
<td>30+ @100F³</td>
<td>Triaxial</td>
<td>25-150 (172-1034) 50-95 (345-655)</td>
</tr>
<tr>
<td>Farrar</td>
<td>Alluvial Gravel</td>
<td>No</td>
<td>CSS-1H</td>
<td>N/T</td>
<td>n/a</td>
<td>1.0%</td>
<td>14⁴</td>
<td>Triaxial</td>
<td>12-80 (83-552) 37-82 (255-565)</td>
</tr>
<tr>
<td>Farrar</td>
<td>Alluvial Gravel</td>
<td>No</td>
<td>CSS-1H</td>
<td>N/T</td>
<td>n/a</td>
<td>2.0%</td>
<td>7⁴</td>
<td>Triaxial</td>
<td>12-80 (83-552) 145-374 (1000-2579)</td>
</tr>
<tr>
<td>Vegas</td>
<td>Silty Gravel Base Course</td>
<td>No</td>
<td>CSS</td>
<td>Yes</td>
<td>N/T</td>
<td>4.0% - 7.0%</td>
<td>3 @ 304F</td>
<td>Indirect Tensile</td>
<td>181-441 (1247-3041)</td>
</tr>
<tr>
<td>Darter</td>
<td>Sand</td>
<td>Yes</td>
<td>HFE</td>
<td>Yes</td>
<td>363-449</td>
<td>2.0% - 5.0%</td>
<td>14 @ 72F</td>
<td>Indirect Tensile</td>
<td>42-60 (290-414)</td>
</tr>
</tbody>
</table>

N/T = not tested  
n/a = not applicable  
¹ Refers to Specifications Outlined by UFC 3-250-11  
² Source did not provide emulsion asphalt content  
³ Samples were allowed to cure until a constant weight was achieved, exact number of days was not recorded  
⁴ No curing temperature provided in literature

### Table 2.10. Resilient Modulus Ranges for Asphalt Stabilized Materials

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Resilient Modulus Range, ksi (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed Aggregate</td>
<td>87 – 213 (600 – 1469)</td>
</tr>
<tr>
<td>Uncrushed Aggregate</td>
<td>50 – 176 (345 – 1213)</td>
</tr>
<tr>
<td>Sand</td>
<td>42 – 60 (290 – 414)</td>
</tr>
</tbody>
</table>
CHAPTER 3
BASELINE PAVEMENT STRUCTURE AND STANDARD GEAR LOAD

Stabilized material equivalency factors are utilized as part of the CBR based flexible pavement design in UFC 3-260-02. However, since the effects of utilizing a stabilized layer within a pavement structure will be evaluated utilizing linear elastic theory in this study, it is logical to design the baseline pavement using the linear elastic design (LED) for flexible pavements. Utilizing LED will allow for a more direct correlation between the baseline pavement design and the modeling completed using BISAR 3.0 (Shell Bitumen, 1998), as modulus and Poisson’s ratio are the required layer property inputs for both processes.

Both the CBR and LED-based flexible design methods define compaction requirements based on the layer’s depth from the surface. Generally, the deeper a layer is within a pavement system, the less stringent the compaction requirements. For this analysis, compaction differences at various depths will not be considered.

3.1 Layer Properties

The goal of this study is to determine how stabilized layers effect pavement structures. Therefore, typical, unstabilized layer properties will be utilized in the baseline pavement structure. The typical layer properties used in the baseline design are outlined in Table 3.1. Modulus values generally correspond to PCASE default values and Poisson’s ratios are based on recommended values from UFC 3-260-02.

<table>
<thead>
<tr>
<th>Elastic Modulus, ksi (MPa)</th>
<th>Poisson’s Ratio</th>
<th>Typical Material Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subgrade</td>
<td>6.0 (41.4)</td>
<td>0.4</td>
</tr>
<tr>
<td>Subbase</td>
<td>24.0 (165.5)</td>
<td>0.3</td>
</tr>
<tr>
<td>Base</td>
<td>61.0 (421.6)</td>
<td>0.3</td>
</tr>
<tr>
<td>Asphalt Surface</td>
<td>350.0 (2413.2)</td>
<td>0.5</td>
</tr>
</tbody>
</table>

A one season pavement design will be completed in PCASE, using an asphalt surface modulus which corresponds to the critical, low modulus hot temperature conditions. Thus, only one modulus value is required to describe the asphalt surface. Likewise, a weak subgrade will be utilized, which corresponds to the critical conditions in order to elicit the most extreme pavement structure, allowing the effects of stabilized layers to be conservatively analyzed.
3.2 Pavement Design

Based on UFC 3-260-02, asphalt pavement is generally used in Type C traffic areas. Type C traffic corresponds to areas which experience low traffic volumes or areas where the operating aircraft is applying less weight to the pavement than its design weight. Type C pavements are generally found in the center portions of runways and on secondary taxiways, as shown in Figure 1.1.

PCASE was used to perform an LED pavement design for a Type C traffic area for the standard Air Force heavy traffic pattern. Heavy aircraft impart significant loading on the base, subbase, and subgrade layers of the pavement system compared to lighter aircraft.

A four layer pavement system was designed utilizing material properties identified in Table 3.1. Allowing PCASE to compute the asphalt, base, and subbase thicknesses; the design in Table 3.2 was generated.

<table>
<thead>
<tr>
<th>Pavement Layer</th>
<th>Calculated Thickness, in (m)</th>
<th>Thickness Used for Analysis, in (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subbase</td>
<td>36.62 (0.9301)</td>
<td>26.5 (0.6731)</td>
</tr>
<tr>
<td>Base Course</td>
<td>8.00 (0.2032)</td>
<td>20.0 (0.5080)</td>
</tr>
<tr>
<td>Asphalt Surface</td>
<td>9.52 (0.2418)</td>
<td>6.5 (0.1651)</td>
</tr>
</tbody>
</table>

It can be seen in Table 3.2 that PCASE dissipates the heavy loading by utilizing the minimum base course thickness and a significantly thicker subbase course. An 8 inch base course is not conducive to investigating the impact of a stabilized base course, as UFC 3-260-02 does not allow stabilized layer to be any thinner than the prescribed minimum layer thicknesses. Therefore, the base course thickness was manually entered into PCASE in order to produce a pavement design which has similar base and subbase layer thicknesses.

The final baseline pavement design is shown in Table 3.3. Layer thicknesses used for the analysis are also shown. It is common with Air Force pavement design to “conservatively round” (0.20 or greater rounds to 0.5 and 0.70 or greater rounds to the next even inch) computed layer thicknesses to the nearest one-half inch.

<table>
<thead>
<tr>
<th>Pavement Layer</th>
<th>Calculated Thickness, in (m)</th>
<th>Thickness Used for Analysis, in (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subbase</td>
<td>26.29 (0.6678)</td>
<td>26.5 (0.6731)</td>
</tr>
<tr>
<td>Base Course</td>
<td>20.00 (0.5080)</td>
<td>20.0 (0.5080)</td>
</tr>
<tr>
<td>Asphalt Surface</td>
<td>6.36 (0.1615)</td>
<td>6.5 (0.1651)</td>
</tr>
</tbody>
</table>
3.3 Standard Gear Load

Commensurate with the heavy traffic pattern used to develop the baseline pavement structure, it is logical to select a gear configuration and wheel load corresponding to an aircraft which would operate from a similarly designed airfield pavement. UFC 3-260-03, *Airfield Pavement Evaluation*, categorizes Air Force aircraft into various groups from 1 to 14. Aircraft groups are generally comprised of aircraft of similar size and with similar landing gear configurations. A larger group number corresponds to aircraft which impart greater loads on the pavement structure. For example, Group 2 contains common fighter aircraft, while Group 8 contains the Boeing 767 and KC-135. Within each group, the aircraft considered the most damaging to the pavement are designated as the “controlling aircraft.”

The controlling aircraft of Group 14, the Boeing B-52 Stratofortress, will be used as the standard gear load for this investigation. A picture of the aircraft, including the landing gear is shown in Figure 3.1, with a schematic of the single gear shown in Figure 3.2(a). Figure 3.2(b) shows the coordinate axis used in this analysis, in addition to the coordinates for the center of each wheel load.

![Figure 3.1. Picture of B-52 Landing Gear (Boeing)](image)

![Figure 3.2(a). B-52 Landing Gear Assembly Schematic (after UFC 3-260-02)](image)
As shown in Figure 3.3, the B-52 has two gear assemblies, with both sharing the geometry depicted in Figure 3.2. In addition to the two main assemblies, small single wheel landing gears can be seen towards the end of each wing. Loading from the wing gears is considered to be negligible, as their main purpose is to simply keep the wing from contacting the ground due to flexure during ground operations.

Based on the significant distance between the front and rear assemblies, no load interaction between the gears will be considered.

UFC 3-260-02 indicates that the design gear load for the B-52 is 250,000 pounds with a tire pressure of 240 pounds per square inch. Guidance indicates that loads on a Type C pavement are 75% of the design gear load, whereas full design gear load is considered for Type A and B pavements. Since the
baseline pavement design was for a Type C area, a design gear load of 187,500 pounds will be used. It will be assumed that the total gear load is evenly distributed across all wheels. This produces an individual wheel load of 46,875 pounds.
Pavement critical response magnitudes under the gear load will be calculated for both the unbound baseline pavement structure (with no stabilized layers) and the baseline pavement structure containing stabilized layers. Critical responses include tensile strain at the bottom of the asphalt surface, compressive stress at the top of the subgrade, and compressive strain at the top of subgrade. These critical response locations, in addition to the baseline unbound pavement layers is presented in Figure 4.1. It is hypothesized that measured critical response magnitudes will be significantly lower in the baseline pavement structure with stabilized layers.

![Figure 4.1. Critical Response Locations and Unbound Baseline Pavement Structure](image)

Critical response magnitudes calculated in the unbound baseline pavement structure (developed using PCASE) are assumed to be within levels found to be acceptable in airfield pavements. Therefore, the stabilized layer thicknesses will be reduced until the calculated critical response magnitudes in the stabilized pavement structure are similar or less than those of the unbound baseline pavement structure. The resulting stabilized layer thicknesses will be recorded to the nearest half inch. The ratio formed between the unbound baseline pavement structure layer thickness and the stabilized layer thickness measured will be used to calculate the proposed equivalency factors for each material.

### 4.1 Unbound Baseline Pavement Critical Responses

BISAR 3.0 was used to analyze the critical responses induced in the unbound baseline pavement structure by the design gear load outlined in Chapter 3. UFC 3-260-03 recommends checking critical responses at the locations designated with a black dot in Figure 3.2(a). The coordinates for these
locations, based on the established coordinate system, are presented in Table 4.1. Note that these coordinates are for the unbound baseline pavement structure. Differing coordinates in the z direction are expected when evaluating various stabilized materials in this structure.

<table>
<thead>
<tr>
<th>Measured Value</th>
<th>P1</th>
<th>P2</th>
<th>P3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal Strain, Bottom of AC</td>
<td>(18.5, 0.0, 6.5)</td>
<td>(37.0, 0.0, 6.5)</td>
<td>(68.0, 0.0, 6.5)</td>
</tr>
<tr>
<td>Compressive Strain, Top of Subgrade</td>
<td>(18.5, 0.0, 53.0)</td>
<td>(37.0, 0.0, 53.0)</td>
<td>(68.0, 0.0, 53.0)</td>
</tr>
<tr>
<td>Compressive Stress, Top of Subgrade</td>
<td>(18.5, 0.0, 53.0)</td>
<td>(37.0, 0.0, 53.0)</td>
<td>(68.0, 0.0, 53.0)</td>
</tr>
</tbody>
</table>

Table 4.2 provides the calculated critical responses calculated at each designated point. It can be seen that the responses under Point P2 have the largest magnitude. The Point P2 responses will be used as the maximum allowable critical response magnitudes for the subsequent analysis of pavement structures with stabilized layers.

4.2 Stabilized Material Properties

Before BISAR can be used to evaluate stabilized layers, however, modulus and Poisson’s ratios for each stabilized material must be established. Table 4.3 shows the values used for this analysis. Engineering judgment was applied to the literature review results in order to obtain conservative material properties for each material type.

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Modulus, ksi (MPa)</th>
<th>Poisson’s Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>All-Bituminous Concrete</td>
<td>750.0 (5171.1)</td>
<td>0.30</td>
</tr>
<tr>
<td>Crushed Aggregate</td>
<td>150.0 (1034.2)</td>
<td>0.50</td>
</tr>
<tr>
<td>Uncrushed Aggregate</td>
<td>100.0 (689.5)</td>
<td>0.50</td>
</tr>
<tr>
<td>Sand</td>
<td>50.0 (344.7)</td>
<td>0.50</td>
</tr>
</tbody>
</table>
It should be noted that an additional material type is included in Table 4.3, which is not included in the established equivalency factors (Table 1.1). Based on the literature review, there is an appreciable modulus difference between stabilized materials produced with crushed and uncrushed aggregates. The creation of separate crushed and uncrushed aggregate equivalency factors will better account for this modulus difference. Additionally, a visual inspection can determine whether an aggregate is crushed or uncrushed, therefore; the addition will not hinder the expedient application of the CBR design process.

4.3 Testing Matrix

Design guidance indicates that utilizing a stabilized base layer with an unstabilized subbase layer is an acceptable practice. Therefore, the first set of analysis runs will be used to identify acceptable equivalency factors for base courses used over a typical, unbound subbase material, as shown in Figure 4.4.

<table>
<thead>
<tr>
<th>Run</th>
<th>Base Material</th>
<th>Base Thickness Based on</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>All-Bituminous</td>
<td>Established All Bituminous EF</td>
</tr>
<tr>
<td>2</td>
<td>All-Bituminous</td>
<td>Thinnest</td>
</tr>
<tr>
<td>3</td>
<td>Stabilized Crushed Aggregate</td>
<td>Established Aggregate EF</td>
</tr>
<tr>
<td>4</td>
<td>Stabilized Crushed Aggregate</td>
<td>Thinnest</td>
</tr>
<tr>
<td>5</td>
<td>Stabilized Uncrushed Aggregate</td>
<td>Established Aggregate EF</td>
</tr>
<tr>
<td>6</td>
<td>Stabilized Uncrushed Aggregate</td>
<td>Thinnest</td>
</tr>
</tbody>
</table>

* “Established EF” refers to the equivalency factors identified in Table 1.1. “Thinnest” refers to the thinnest base thickness for each base material which produces a pavement structure that does not exceed critical responses identified in Table 4.1 when loaded.

Using the proposed equivalency factors determined through the analysis runs outlined in Table 4.5, the subbase course equivalency factors can now be evaluated. The use of a stabilized subbase with an unbound base is not recommended, as the likelihood of having a saturated base course is high due to the relatively impermeable stabilized subbase layer. Therefore, it can be assumed that a stabilized base will always be used in conjunction with a stabilized subbase – making it unnecessary to analyze stabilized subbase courses with unbound base courses.
It can be noted in Table 4.5 that each subbase material is evaluated only using base materials with superior engineering properties. This follows the rule of traditional pavement design, where a superior quality upper pavement layer provides protection for the weaker underlying layer. Following this logic, it is not necessary to analyze an all bituminous subbase with a stabilized uncrushed aggregate base course.

Each subbase material will only have one equivalency factor, regardless of the base material used in conjunction with the subbase material. It is expected that the critical responses with the greatest magnitude will be seen with only one particular combination of base and subbase - on which the proposed equivalency factor will be based. This eliminates the ambiguity of having multiple subbase equivalency factors.

4.4 Results

4.4.1 Stabilized Base Course

The base course analysis results are summarized in Table 4.6. The results show that the established equivalency factors (Runs 1, 3, and 5) are very conservative when compared to the critical responses generated from the unbound baseline pavement layer, as seen in column P2 of Table 4.2. The controlling critical response for the pavement structures in search of the thinnest allowable stabilized base
layer (Runs 2, 4, and 6) are denoted with red, italicized text. The calculated stress on the top of the subgrade was the controlling response for all analysis cases.

Table 4.6. Linear Analysis Base Course Results

<table>
<thead>
<tr>
<th>Run</th>
<th>Base Material</th>
<th>Thickness, in (m)</th>
<th>Equivalency Factor</th>
<th>Tensile Strain, Bottom of AC, microstrain</th>
<th>Compressive Strain, Top of Subg, microstrain</th>
<th>Compressive Stress, Top of Subg, psi (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>All-Bituminous</td>
<td>17.5 (0.4445)</td>
<td>1.15</td>
<td>1.866e2</td>
<td>5.758e2</td>
<td>3.99 (27.51)</td>
</tr>
<tr>
<td>2</td>
<td>All-Bituminous</td>
<td>11.5 (0.2921)</td>
<td>1.74</td>
<td>2.649e2</td>
<td>7.507e2</td>
<td>5.00 (34.47)</td>
</tr>
<tr>
<td>3</td>
<td>Stabilized Crushed Aggregate</td>
<td>20.0 (0.5080)</td>
<td>1.00</td>
<td>2.970e2</td>
<td>7.434e2</td>
<td>4.71 (32.47)</td>
</tr>
<tr>
<td>4</td>
<td>Stabilized Crushed Aggregate</td>
<td>16.5 (0.4191)</td>
<td>1.21</td>
<td>2.991e2</td>
<td>8.732e2</td>
<td>5.32 (36.68)</td>
</tr>
<tr>
<td>5</td>
<td>Stabilized Uncrushed Aggregate</td>
<td>20.0 (0.5080)</td>
<td>1.00</td>
<td>4.144e2</td>
<td>8.110e2</td>
<td>5.06 (34.89)</td>
</tr>
<tr>
<td>6</td>
<td>Stabilized Uncrushed Aggregate</td>
<td>18.0 (0.4572)</td>
<td>1.11</td>
<td>4.183e2</td>
<td>8.618e2</td>
<td>5.39 (37.16)</td>
</tr>
</tbody>
</table>

Based on the results of Table 4.6, equivalency factors can be proposed for all three stabilized base types. All proposed base course equivalency factors are less conservative than the established equivalency factors. The proposed base course equivalency factors based in the linear analysis are shown in Table 4.7.

Table 4.7. Calculated Base Course Equivalency Factors

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Calculated EF</th>
<th>Established EF</th>
</tr>
</thead>
<tbody>
<tr>
<td>All-Bituminous Concrete</td>
<td>1.74</td>
<td>1.15</td>
</tr>
<tr>
<td>Stabilized Crushed Aggregate</td>
<td>1.21</td>
<td>1.00</td>
</tr>
<tr>
<td>Stabilized Uncrushed Aggregate</td>
<td>1.11</td>
<td>1.00</td>
</tr>
</tbody>
</table>

4.4.2 Stabilized Subbase Course

The subbase course results are summarized in Table 4.8. Unlike the base course results, it was found that some of the established subbase equivalency factors were overly conservative, while others were unconservative. All bituminous subbase and crushed aggregate subbase were found to have conservative established equivalency factors. Uncrushed aggregate subbase and sand subbase were found to have unconservative established equivalency factors. The critical response magnitude results when using the all bituminous subbase and the crushed aggregate base were very conservative compared to critical responses observed in the unbound baseline structure. However, it was found that the established equivalency factors for uncrushed aggregate were somewhat unconservative when comparing the critical responses. Based on these findings, the proposed equivalency factors for all bituminous subbase and crushed aggregate subbase are greater than the established equivalency factors, while the proposed equivalency factors for uncrushed aggregate subbase and sand subbase are less. Again, the red, italicized text denotes the controlling critical response. For this data, it can be seen that the controlling critical response was obtained with an all bituminous base was used with each subbase material.
<table>
<thead>
<tr>
<th>Run</th>
<th>Base Material</th>
<th>Base Thickness, in (m)</th>
<th>Subbase Material</th>
<th>Subbase Thickness, in (m)</th>
<th>Subbase Equivalency Factor</th>
<th>Tensile Strain, Bottom of AC, microstrain</th>
<th>Compressive Strain, Top of Subg, microstrain</th>
<th>Compressive Stress, Top of Subg, psi (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>2.30</td>
<td>1.529e2</td>
<td>4.562e2</td>
<td>3.88 (26.75)</td>
</tr>
<tr>
<td>8</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>All Bituminous</td>
<td>6.0 (0.1524)</td>
<td>4.42</td>
<td>2.087e2</td>
<td>6.266e2</td>
<td>5.34 (36.82)</td>
</tr>
<tr>
<td>9</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>Stabilized Crushed Agg</td>
<td>13.5 (0.3429)</td>
<td>2.00</td>
<td>1.700e2</td>
<td>7.050e2</td>
<td>5.25 (36.20)</td>
</tr>
<tr>
<td>10</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>Stabilized Crushed Agg</td>
<td>13.0 (0.3302)</td>
<td>2.04</td>
<td>1.736e2</td>
<td>7.166e2</td>
<td>5.35 (36.89)</td>
</tr>
<tr>
<td>11</td>
<td>Stabilized Crushed Agg</td>
<td>16.5 (0.4191)</td>
<td>Stabilized Crushed Agg</td>
<td>13.5 (0.3429)</td>
<td>2.00</td>
<td>2.909e2</td>
<td>7.184e2</td>
<td>5.18 (35.71)</td>
</tr>
<tr>
<td>12</td>
<td>Stabilized Crushed Agg</td>
<td>16.5 (0.4191)</td>
<td>Stabilized Crushed Agg</td>
<td>13.0 (0.3302)</td>
<td>2.04</td>
<td>2.910e2</td>
<td>7.323e2</td>
<td>5.28 (36.40)</td>
</tr>
<tr>
<td>13</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>Stabilized Uncrushed Agg</td>
<td>13.5 (0.3429)</td>
<td>2.00</td>
<td>2.074e2</td>
<td>7.798e2</td>
<td>5.69 (39.23)</td>
</tr>
<tr>
<td>14</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>Stabilized Uncrushed Agg</td>
<td>15.5 (0.3937)</td>
<td>1.71</td>
<td>1.956e2</td>
<td>1.956e2</td>
<td>5.35 (36.89)</td>
</tr>
<tr>
<td>15</td>
<td>Stabilized Crushed Agg</td>
<td>16.5 (0.4191)</td>
<td>Stabilized Uncrushed Agg</td>
<td>13.5 (0.3429)</td>
<td>2.00</td>
<td>2.864e2</td>
<td>2.864e2</td>
<td>5.60 (38.61)</td>
</tr>
<tr>
<td>16</td>
<td>Stabilized Crushed Agg</td>
<td>16.5 (0.4191)</td>
<td>Stabilized Uncrushed Agg</td>
<td>15.5 (0.3937)</td>
<td>1.71</td>
<td>2.930e2</td>
<td>2.930e2</td>
<td>5.23 (36.06)</td>
</tr>
<tr>
<td>17</td>
<td>Stabilized Uncrushed Agg</td>
<td>18.0 (0.4572)</td>
<td>Stabilized Uncrushed Agg</td>
<td>13.5 (0.3429)</td>
<td>2.00</td>
<td>4.003e2</td>
<td>4.003e2</td>
<td>5.74 (39.58)</td>
</tr>
<tr>
<td>18</td>
<td>Stabilized Uncrushed Agg</td>
<td>18.0 (0.4572)</td>
<td>Stabilized Uncrushed Agg</td>
<td>15.5 (0.3937)</td>
<td>1.71</td>
<td>4.007e2</td>
<td>4.007e2</td>
<td>5.35 (36.89)</td>
</tr>
<tr>
<td>19</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>Stabilized Sand</td>
<td>17.5 (0.4445)</td>
<td>1.50</td>
<td>2.456e2</td>
<td>8.232e2</td>
<td>5.78 (39.85)</td>
</tr>
<tr>
<td>20</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>Stabilized Sand</td>
<td>20.5 (0.5207)</td>
<td>1.29</td>
<td>2.347e2</td>
<td>7.791e2</td>
<td>5.39 (37.16)</td>
</tr>
<tr>
<td>21</td>
<td>Stabilized Crushed Agg</td>
<td>16.5 (0.4191)</td>
<td>Stabilized Sand</td>
<td>17.5 (0.4445)</td>
<td>1.50</td>
<td>2.915e2</td>
<td>8.372e2</td>
<td>5.54 (38.20)</td>
</tr>
<tr>
<td>22</td>
<td>Stabilized Crushed Agg</td>
<td>16.5 (0.4191)</td>
<td>Stabilized Sand</td>
<td>20.5 (0.5207)</td>
<td>1.29</td>
<td>2.963e2</td>
<td>7.791e2</td>
<td>5.12 (35.30)</td>
</tr>
<tr>
<td>23</td>
<td>Stabilized Uncrushed Agg</td>
<td>18.0 (0.4572)</td>
<td>Stabilized Sand</td>
<td>17.5 (0.4445)</td>
<td>1.50</td>
<td>4.099e2</td>
<td>8.800e2</td>
<td>5.75 (39.64)</td>
</tr>
<tr>
<td>24</td>
<td>Stabilized Uncrushed Agg</td>
<td>18.0 (0.4572)</td>
<td>Stabilized Sand</td>
<td>20.5 (0.5207)</td>
<td>1.29</td>
<td>4.106e2</td>
<td>8.144e2</td>
<td>5.30 (36.54)</td>
</tr>
</tbody>
</table>
Based on the results of Table 4.8, equivalency factors can be proposed for all four stabilized subbase types. The proposed subbase course equivalency factors for the all bituminous subbase and crushed aggregate subbase are less conservative than the established equivalency factors. On the other hand, the proposed uncrushed aggregate subbase and sand subbase equivalency factors are more conservative than the established equivalency factors. The proposed base course equivalency factors are shown in Table 4.9.

### Table 4.9. Calculated Subbase Course Equivalency Factors

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Calculated EF</th>
<th>Established EF</th>
</tr>
</thead>
<tbody>
<tr>
<td>All-Bituminous Concrete</td>
<td>4.42</td>
<td>2.30</td>
</tr>
<tr>
<td>Stabilized Crushed Aggregate</td>
<td>2.04</td>
<td>2.00</td>
</tr>
<tr>
<td>Stabilized Uncrushed Aggregate</td>
<td>1.71</td>
<td>2.00</td>
</tr>
<tr>
<td>Stabilized Sand</td>
<td>1.29</td>
<td>1.50</td>
</tr>
</tbody>
</table>

4.4.3 Discussion

Based on the linear elastic analysis results, it was found that established equivalency factors for the stabilized base materials were overly conservative, with the established equivalency factors for the highest quality materials being the most conservative. Additionally, it was found that there is a difference of 0.10 between the proposed equivalency factors for crushed and uncrushed aggregate materials – with the proposed equivalency factors for both crushed and uncrushed aggregate exceeding the established aggregate equivalency factor. The proposed factors consider the higher quality of crushed material – thus improved engineering properties – thus reducing layer thickness when stabilized materials are used.

Unlike the base course materials, it was found that some of the established equivalency factors for subbase materials were conservative, while others were not conservative. Generally speaking, the established equivalency factors for high quality materials were found to be conservative, while the established equivalency factors for lower quality materials were found to be less conservative. The proposed equivalency factors reflect this finding.
CHAPTER 5
NON-LINEAR ANALYSIS

To provide a more rigorous analysis of those equivalency factors proposed in Chapter 4, a non-linear analysis program was used in order to test the results of the linear analysis. Unlike a linear analysis, where a constant material modulus is assumed throughout the entire material layer, a non-linear analysis more accurately represents the stress dependent responses of geomaterials.

5.1 Non-Linear Material Models

5.1.1 Stress Softening Materials

Fine grained soils show stress softening behavior. For the non-linear analysis, the modulus of the subgrade soil is represented using the bilinear (arithmetic) model. The modulus of subgrade soils are primarily dependent on the stress state of the soil, therefore the bilinear model relates deviator stress to modulus (Thompson, 1998).

Using the bilinear model, the calculated modulus values over the range of deviator stresses is approximated with two straight lines, whose slope breaks at the breakpoint modulus of the soil. A generic plot in Figure 5.1 shows the general shape and various parameters which are used to describe the properties of a soil using the bilinear model. The bilinear model can also be represented using Equations 5.1 and 5.2. It has been found by Thompson that the modulus value at the breakpoint between the two lines can be used to classify fine grained soils as being either stiff or soft.

![Figure 5.1. Standard Relationship between Resilient Modulus and Deviator Stress for Fine Grained Soils and Standard Constants used in the Bilinear Model (Thompson, 1998)](image)
\[ M_R = K_{19} + K_{21}(K_{20} - \sigma_d) \text{ when } \sigma_d < K_{20} \]  \hspace{1cm} (5.1)
\[ M_R = K_{19} + K_{22}(\sigma_d - K_{20}) \text{ when } \sigma_d > K_{20} \]  \hspace{1cm} (5.2)

Where \( M_R \) represents the modulus and \( \sigma_d \) represents the deviator stress.

It should be noted that Figure 5.1 does not explicitly show two other values which are commonly used to further define the bilinear model – the deviator stress upper and lower limits. The lower limit of the deviator stress is the x axis parameter which corresponds to the highest modulus measurement on the y axis. Similarly, the upper limit of the deviator stress is the x axis parameter which corresponds to the lowest modulus measurement on the y axis. These are referred to “SDLL” and “SDUL,” respectively, in this document.

5.1.2 Stress Hardening Materials

Coarse grain soils with low fines content and aggregates show stress hardening behavior. For the non-linear analysis, the subbase and base course moduli (both crushed and uncrushed materials in unbound and stabilized conditions) will be represented using the K-theta model. This model relates the modulus of stress hardening materials to the applied bulk stress (theta). Bulk stress is equal to the sum of the deviator (applied) stress and three times the confining pressure (which represents the confining pressure applied in the x, y, and z axis directions). However, the K-theta model does not consider the effects of shear stress on stress hardening material modulus, thus is found to represent material behaviors with more error than subsequent models (Thompson, 1998). The simplicity of the K-theta model and the frequency of the model’s use in literature were of primary concern for this study.

As can be seen in Equation 5.3, the K-theta model is a power function which compares the bulk stress and the modulus. For this study, constant \( K_1 \) has a unit of psi, while \( K_2 \) is unitless.

\[ M_R = K_1 \theta^{K_2} \]  \hspace{1cm} (5.3)

Where \( M_R \) represents the modulus and \( \theta \) represents the bulk stress.

5.2 Modeling Software

To perform the non-linear analysis, an axisymmetric finite element computer program, GT Pave was utilized. GT Pave can only model a single, static wheel load. The asphalt surface is modeled as a linear elastic layer with a single modulus value, while the underlying base, subbase, and subgrade properties are represented with non-linear models. While a number of models can be used with the program, the models mentioned above will be used for this analysis.
GT Pave is unique to similar programs in two respects. First, it is able to account for the anisotropic properties of geomaterials. However, all layers will be considered isotropic for this analysis. Secondly, it models aggregate layers as heterogeneous, thus using techniques to account for voids within the layer and limited particle contact. Additional information regarding the exact techniques used in the program, in addition to the validation of the program, is available (Tutumluer, 1995).

In addition to anisotropy, GT Pave also considers two additional parameters which were not seen in the linear analysis. First, residual stress within the geomaterial layer due to construction traffic and compaction is considered. An at rest stress coefficient is provided for each layer to account for these residual stresses. Second, GT Pave considers the stresses and strains induced in the pavement structure due to the weight of the pavement structure, in addition to the wheel load. A body force magnitude in the vertical direction is provided for each layer in order to perform these calculations. While the term “body force” is used in the program, “density” may better describe this material property, as there is no dynamic movement in this pavement modeling. The term “density” will be used to represent this parameter throughout this report.

5.3 Finite Element Mesh Generation and Load Placement

GT Pave allows the user flexibility to develop a unique finite element mesh. The mesh used for this study is depicted in Figure 5.2. The column spacing remains consistent for all runs. However, since the pavement layer thickness changes depending upon each equivalency factor, the number of rows remains constant for all runs, with a variable row thickness. Since the asphalt surface thickness does not change, the row spacing is always 0.65 inches. Additionally, the subgrade responses are calculated up to a depth of 20 inches below the bottom of the subbase (5 rows, each of which are 4 inches thick).
Wheel loading in GT Pave is represented using a circle. Thus, Equation 5.3 was used to translate the load and tire pressure information provided in Chapter 3 to calculate a radius of the loaded circular area.

\[
a = \frac{P}{\sqrt{\pi TP}} \quad (5.4)
\]

Where \(a\) is the radius of the loaded area in inches, \(P\) is the load in pounds, and \(TP\) is the tire pressure in pounds per square inch.

Using Equation 5.4 for the B-52 gear wheel loading results in a radius of 7.88 inches. The use of this radius can be seen in the first four columns from the left of Figure 5.2.

5.4 Material Properties

Non-linear material properties were determined from literature. An attempt was made to select materials which have similar properties to those materials used in the linear analysis. Each section
provides a brief description of the materials used, with additional information, as well as GT Pave input values provided in the applicable tables and figures in Appendix A. Depending on the source, some material properties had to be assumed in order to complete the GT Pave analysis. Typical values for these parameters were used – which generally corresponded to default values for each layer which are programmed into GT Pave. These assumed values are indicated in the respective tables in Appendix A.

No studies involving the use of stabilized sand were found in literature. Therefore, the non-linear analysis will only consider all bituminous and stabilized aggregate materials.

5.4.1 Subgrade

A soil from the Elliott Series in Illinois with well documented non-linear properties was used to simulate the subgrade in the following analysis runs (Thompson and Robnett, 1976). This soil was classified as an AASHTO A-6(5), and has been described as low plasticity clay, with a soaked CBR value of 4.2. This closely matches the description of the material used in the linear analysis. Additionally, the Elliot Series clay has a reported breakpoint resilient modulus of 6,180 psi, which is close to the modulus of 6,000 psi used in the linear analysis. A plot of the non-linear soil properties from laboratory testing can be found in Figure A.1. The middle curve was used, which corresponds to testing at a moisture content of 1.1% over optimum. Table A.1 shows the data used in the GT Pave analysis.

5.4.2 Subbase

Minnesota Department of Transportation (MnDOT) specified CL-3sp material was used for the subbase. Default values for this material contained in MnPave, a linear elastic pavement design program used by MnDOT, suggest that the modulus is similar to the subbase modulus used in Chapter 4. In the absence of specific CL-3sp test results, MnPave assumes a modulus for this material of 50,000 psi under winter conditions, 21,910 psi under spring conditions, and 26,610 psi under summer conditions. No plot of the non-linear soil properties based on laboratory testing can be produced, as measurements at each of the triaxial test stress states were not provided (Seyhan, 1998). However, Table A.2 shows the data used in the GT Pave analysis.

5.4.3 Aggregates

Crushed and uncrushed aggregates are represented by two aggregates from Anderson and Thompson (1995). The same high float emulsion was used to stabilize both aggregates used in this analysis. Additionally, the consistency in design, production, and testing of both materials made these two aggregates attractive for the non-linear analysis.
5.4.3.1 Uncrushed Aggregate

The uncrushed river gravel stabilized with the high float emulsion described in Section 2.2.2 was used for the uncrushed aggregate. A triaxial test was run on the aggregate in both the virgin and stabilized conditions, and results of both tests were provided in the literature. A plot of the non-linear uncrushed aggregate properties from laboratory testing can be found in Figure A.2 and A.3, for the unbound and stabilized materials, respectively. The dashed line representing the modulus value used for the linear elastic analysis is provided for reference. Table A.3 shows the data used in the GT Pave Analysis.

5.4.3.2 Crushed Aggregate

The crushed dolomitic limestone stabilized with the high float emulsion described in Section 2.1.2 was used for the uncrushed aggregate. A triaxial test was run on the aggregate in both the virgin and stabilized conditions, and results of both tests were provided in the literature. A plot of the non-linear crushed aggregate properties from laboratory testing can be found in Figure A.4 and A.5, for the unbound and stabilized materials, respectively. The dashed line representing the modulus value used for the linear elastic analysis is provided for reference. Table A.4 shows the data used in the GT Pave Analysis.

5.4.4 Asphalt Surface

As previously mentioned, the asphalt surface is treated as an elastic solid in GT Pave. Therefore, the parameters used in Chapter 4 will again be used to represent that asphalt surface in this scenario. The asphalt surface properties used are shown in Table A.5.

5.5 Unbound Pavement Responses

Again, similar to the procedure used in Chapter 4, an unbound pavement section will be modeled with GT Pave to obtain the critical responses under the singular wheel load and using the non-linear geomaterial properties. Due to the vastly different modeling techniques between BISAR and GT Pave, it is not expected that the magnitudes of the critical responses will be similar.

The unbound layers selected for this initial analysis were selected in an attempt to match the parameters used in Chapter 4 for the unbound baseline pavement structure. The unbound, uncrushed aggregate will be used as the base course, as it nearly matches the CBR 80, 61,000 psi modulus material used in the linear elastic analysis. The unbound subbase and subgrade materials already described are also shown to have reasonable correlation with the properties used in the linear analysis. The pavement structure for the unbound baseline pavement is shown in Table 5.1.
Table 5.1. Unbound Baseline Pavement Structure for Non-Linear Analysis

<table>
<thead>
<tr>
<th>Pavement Layer</th>
<th>Material Type</th>
<th>Thickness, in (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subgrade</td>
<td>Elliot Series</td>
<td>---</td>
</tr>
<tr>
<td>Subbase</td>
<td>MnDOT CL-3sp</td>
<td>26.5 (0.6731)</td>
</tr>
<tr>
<td>Base Course</td>
<td>Uncrushed River Gravel</td>
<td>20.0 (0.5080)</td>
</tr>
<tr>
<td>Asphalt Surface</td>
<td>Modulus = 350,000 psi</td>
<td>6.5 (0.1651)</td>
</tr>
</tbody>
</table>

Since the GT Pave analysis is only capable of a single wheel load, the pavement critical responses will not be influenced by the load distributions from other wheel loads, the critical responses will occur under the wheel load. These critical responses are provided in Table 5.2.

Table 5.2. Unbound Baseline Pavement Structure Critical Responses from Non-Linear Analysis

<table>
<thead>
<tr>
<th>Measured Value</th>
<th>Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal Strain, Bottom of AC, microstrain</td>
<td>7.431e2</td>
</tr>
<tr>
<td>Compressive Strain, Top of Subgrade, microstrain</td>
<td>5.890e2</td>
</tr>
<tr>
<td>Compressive Stress, Top of Subgrade, psi (kPa)</td>
<td>11.82 (81.50)</td>
</tr>
</tbody>
</table>

5.6 Testing Matrix

GT Pave will be used to test the equivalency factors developed using the linear analysis. The proposed base course factors will first be evaluated. Critical responses for both the established and proposed equivalency factors for each stabilized base course material will be provided for reference and comparison. Table 5.3 shows the testing matrix for evaluating the base course equivalency factors.

Table 5.3. Testing Matrix for Base Course Non-linear Analysis

<table>
<thead>
<tr>
<th>Run</th>
<th>Base Material</th>
<th>Base Thickness Based on:</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>All-Bituminous</td>
<td>Established All Bituminous EF</td>
</tr>
<tr>
<td>2</td>
<td>All-Bituminous</td>
<td>Proposed All Bituminous EF</td>
</tr>
<tr>
<td>3</td>
<td>Stabilized Crushed Aggregate</td>
<td>Established All Bituminous EF</td>
</tr>
<tr>
<td>4</td>
<td>Stabilized Crushed Aggregate</td>
<td>Proposed All Bituminous EF</td>
</tr>
<tr>
<td>5</td>
<td>Stabilized Uncrushed Aggregate</td>
<td>Established All Bituminous EF</td>
</tr>
<tr>
<td>6</td>
<td>Stabilized Uncrushed Aggregate</td>
<td>Proposed All Bituminous EF</td>
</tr>
</tbody>
</table>

Using a base course thickness determined using the proposed equivalency factors, the subbase course equivalency factors can be tested using a non-linear analysis. Again, subbases will only be tested with base courses which have superior engineering properties. No stabilized sand layers are shown in Table 5.4 due to unavailability of non-linear laboratory testing data in literature.
Table 5.4. Testing Matrix for Subbase Course Non-linear Analysis

| Run | Base Material | Subbase Material | Subbase Thickness Based on*:
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>All Bituminous</td>
<td>All Bituminous</td>
<td>Established All Bituminous EF</td>
</tr>
<tr>
<td>8</td>
<td>All Bituminous</td>
<td>All Bituminous</td>
<td>Proposed All Bituminous EF</td>
</tr>
<tr>
<td>9</td>
<td>All Bituminous</td>
<td>Stabilized Crushed Agg</td>
<td>Established Aggregate EF</td>
</tr>
<tr>
<td>10</td>
<td>All Bituminous</td>
<td>Stabilized Crushed Agg</td>
<td>Proposed Crushed Aggregate EF</td>
</tr>
<tr>
<td>11</td>
<td>Stabilized Crushed Agg</td>
<td>Stabilized Crushed Agg</td>
<td>Established Aggregate EF</td>
</tr>
<tr>
<td>12</td>
<td>Stabilized Crushed Agg</td>
<td>Stabilized Crushed Agg</td>
<td>Proposed Crushed Aggregate EF</td>
</tr>
<tr>
<td>13</td>
<td>All Bituminous</td>
<td>Stabilized Uncrushed Agg</td>
<td>Established Aggregate EF</td>
</tr>
<tr>
<td>14</td>
<td>All Bituminous</td>
<td>Stabilized Uncrushed Agg</td>
<td>Proposed Uncrushed Aggregate EF</td>
</tr>
<tr>
<td>15</td>
<td>Stabilized Crushed Agg</td>
<td>Stabilized Uncrushed Agg</td>
<td>Established Aggregate EF</td>
</tr>
<tr>
<td>16</td>
<td>Stabilized Crushed Agg</td>
<td>Stabilized Uncrushed Agg</td>
<td>Proposed Uncrushed Aggregate EF</td>
</tr>
<tr>
<td>17</td>
<td>Stabilized Uncrushed Agg</td>
<td>Stabilized Uncrushed Agg</td>
<td>Established Aggregate EF</td>
</tr>
<tr>
<td>18</td>
<td>Stabilized Uncrushed Agg</td>
<td>Stabilized Uncrushed Agg</td>
<td>Proposed Uncrushed Aggregate EF</td>
</tr>
</tbody>
</table>

It should be noted that the all bituminous material is tested with GT Pave in both the base and subbase positions, even though GT Pave only considers this material to be linear elastic. While the all bituminous layers are considered linear elastic, the non-linear responses of underlying layers can still be assessed in this analysis, thus providing insight on the effects of non-linear pavement foundation layers on equivalency factors.

5.7 Results

5.7.1 Base

The results for various stabilized base course materials are provided in Table 5.5. The critical responses using the proposed equivalency factors from the linear analysis (Runs 2, 4 and 6) are found to provide an adequate level of conversancy compared to the unbound baseline pavement structure using non-linear modeling methods.

Table 5.5. Non-linear Analysis Base Course Results

<table>
<thead>
<tr>
<th>Run</th>
<th>Base Material</th>
<th>Thickness, in (m)</th>
<th>Equivalency Factor</th>
<th>Tensile Strain, Bottom of AC, microstrain</th>
<th>Compressive Strain, Top of Subg, microstrain</th>
<th>Compressive Stress, Top of Subg, psi (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>All-Bituminous</td>
<td>17.5 (0.4445)</td>
<td>1.15</td>
<td>1.351e2</td>
<td>1.643e2</td>
<td>8.18 (56.40)</td>
</tr>
<tr>
<td>2</td>
<td>All-Bituminous</td>
<td>11.5 (0.2921)</td>
<td>1.74</td>
<td>2.318e2</td>
<td>2.273e2</td>
<td>8.34 (57.50)</td>
</tr>
<tr>
<td>3</td>
<td>Stabilized Crushed Aggregate</td>
<td>20.0 (0.5080)</td>
<td>1.00</td>
<td>4.734e2</td>
<td>3.178e2</td>
<td>9.61 (66.26)</td>
</tr>
<tr>
<td>4</td>
<td>Stabilized Crushed Aggregate</td>
<td>16.5 (0.4191)</td>
<td>1.21</td>
<td>4.812e2</td>
<td>3.491e2</td>
<td>9.77 (67.36)</td>
</tr>
<tr>
<td>5</td>
<td>Stabilized Uncrushed Aggregate</td>
<td>20.0 (0.5080)</td>
<td>1.00</td>
<td>5.123e2</td>
<td>3.560e2</td>
<td>9.98 (68.81)</td>
</tr>
<tr>
<td>6</td>
<td>Stabilized Uncrushed Aggregate</td>
<td>18.0 (0.4572)</td>
<td>1.11</td>
<td>5.357e2</td>
<td>4.227e2</td>
<td>10.34 (71.29)</td>
</tr>
</tbody>
</table>
It was not expected to have a close comparison in critical response magnitudes between the linear and non-linear analyses. This was indeed confirmed by the analysis results, where the compressive subgrade strains were found to be considerably lower using non-linear methods, while the subgrade compressive stresses were found to be considerably higher than those obtained from the linear analysis. The asphalt surface tensile strain was found to be similar between analysis methods, as both consider the layer to be linear elastic.

When comparing the stabilized crushed and uncrushed aggregate with equal base course thicknesses (Runs 3 and 5), lower critical response magnitudes are produced under load when the stabilized crushed material is used. This is expected, as the stabilized crushed aggregate is considered to have superior engineering properties.

5.7.2 Subbase

The results for various subbase equivalency factors are provided in Table 5.6. Unlike the base course results, some of the proposed subbase equivalency factors are found to produce a pavement section which is less conservative than the unbound baseline pavement section when using stabilized crushed and uncrushed aggregate. The liberal critical response results are indicated with red text and bold font in Table 5.6. Based on this analysis, however, the proposed all bituminous subbase equivalency factor is adequate.

In addition, it is also interesting to note that under the stress states of the subbase position, the stabilized uncrushed aggregate used in this test actually produces a stronger pavement layer. When comparing the critical response magnitudes between Runs 11 and 16, it can be seen that lower stress and strains are produced within the pavement structure under loading when a 13.5 inch stabilized uncrushed aggregate base is used, compared to the 13.5 inch stabilized crushed aggregate base.

5.7.3 Discussion

The proposed base and subbase equivalency factors, which were based on the linear analysis, were tested using non-linear analysis methods and material responses based on laboratory testing reported in literature. It was found that the proposed base course equivalency factors were found to maintain their conservancy when non-linear material properties were considered. While the proposed all bituminous subbase equivalency factor was found to have an adequate level of conservancy, the non-linear methods identified that the stabilized crushed aggregate and stabilized uncrushed aggregate equivalency factors were not conservative.

For the base and subbase combinations which had critical response parameter values which exceeded the unbound baseline pavement structure, additional GT Pave runs were performed to find the
subbase thickness which provided acceptable critical response magnitudes. The results are provided in Table 5.7. The values denoted with red text and italics indicate the controlling critical response.

Based on the materials used in the analysis, and the calculated equivalency factors listed in Table 5.7, the proposed subbase equivalency factor for stabilized uncrushed aggregate calculated with the linear analysis are too high. Stabilized aggregate materials are stress hardening – thus when in the subbase position they are subjected to lower stresses and the material has a lower modulus. The equivalency factors calculated based in Runs 20 and 21 are lower than the established equivalency factors presented in AFI 3-260-01. Based on these materials and using similar equivalency factor selection logic seen in Chapter 4, these results suggest that an equivalency factor of 1.23 would be appropriate for stabilized uncrushed aggregate.
### Table 5.6. Non-linear Analysis Subbase Course Results

<table>
<thead>
<tr>
<th>Run</th>
<th>Base Material</th>
<th>Base Thickness, in (m)</th>
<th>Subbase Material</th>
<th>Subbase Thickness, in (m)</th>
<th>Subbase Equivalency Factor</th>
<th>Tensile Strain, Bottom of AC, microstrain</th>
<th>Compressive Strain, Top of Subg, microstrain</th>
<th>Compressive Stress, Top of Subg, psi (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>2.30</td>
<td>8.377e1</td>
<td>3.291e2</td>
<td>8.96 (61.78)</td>
</tr>
<tr>
<td>8</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>All Bituminous</td>
<td>6.0 (0.1524)</td>
<td>4.42</td>
<td>1.377e2</td>
<td>4.777e2</td>
<td>10.34 (71.29)</td>
</tr>
<tr>
<td>9</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>Stabilized Crushed Agg</td>
<td>13.5 (0.3429)</td>
<td>2.00</td>
<td>2.033e2</td>
<td>5.098e2</td>
<td>10.26 (70.74)</td>
</tr>
<tr>
<td>10</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>Stabilized Crushed Agg</td>
<td>13.0 (0.3302)</td>
<td>2.04</td>
<td>2.038e2</td>
<td>5.219e2</td>
<td>10.33 (71.22)</td>
</tr>
<tr>
<td>11</td>
<td>Stabilized Crushed Agg</td>
<td>16.5 (0.4191)</td>
<td>Stabilized Crushed Agg</td>
<td>13.5 (0.3429)</td>
<td>2.00</td>
<td>4.825e2</td>
<td>8.637e2</td>
<td>13.30 (91.70)</td>
</tr>
<tr>
<td>12</td>
<td>Stabilized Crushed Agg</td>
<td>16.5 (0.4191)</td>
<td>Stabilized Crushed Agg</td>
<td>13.0 (0.3302)</td>
<td>2.04</td>
<td>4.829e2</td>
<td>8.866e2</td>
<td>13.41 (92.46)</td>
</tr>
<tr>
<td>13</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>Stabilized Uncrushed Agg</td>
<td>13.5 (0.3429)</td>
<td>2.00</td>
<td>1.908e2</td>
<td>5.044e2</td>
<td>10.27 (70.81)</td>
</tr>
<tr>
<td>14</td>
<td>All Bituminous</td>
<td>11.5 (0.2921)</td>
<td>Stabilized Uncrushed Agg</td>
<td>15.5 (0.3937)</td>
<td>1.71</td>
<td>1.882e2</td>
<td>4.598e2</td>
<td>10.02 (69.09)</td>
</tr>
<tr>
<td>15</td>
<td>Stabilized Crushed Agg</td>
<td>16.5 (0.4191)</td>
<td>Stabilized Uncrushed Agg</td>
<td>13.5 (0.3429)</td>
<td>2.00</td>
<td>4.770e2</td>
<td>8.192e2</td>
<td>13.24 (91.29)</td>
</tr>
<tr>
<td>16</td>
<td>Stabilized Crushed Agg</td>
<td>16.5 (0.4191)</td>
<td>Stabilized Uncrushed Agg</td>
<td>15.5 (0.3937)</td>
<td>1.71</td>
<td>4.754e2</td>
<td>7.387e2</td>
<td>12.82 (88.39)</td>
</tr>
<tr>
<td>17</td>
<td>Stabilized Uncrushed Agg</td>
<td>18.0 (0.4572)</td>
<td>Stabilized Uncrushed Agg</td>
<td>13.5 (0.3429)</td>
<td>2.00</td>
<td>4.509e2</td>
<td>6.936e2</td>
<td>12.32 (84.94)</td>
</tr>
<tr>
<td>18</td>
<td>Stabilized Uncrushed Agg</td>
<td>18.0 (0.4572)</td>
<td>Stabilized Uncrushed Agg</td>
<td>15.5 (0.3937)</td>
<td>1.71</td>
<td>4.500e2</td>
<td>6.272e2</td>
<td>11.97 (82.53)</td>
</tr>
</tbody>
</table>

### Table 5.7. Non-linear Analysis Subbase Course Equivalency Factors Needed to Maintain Strain and Stress Values Below Those Observed in the Unbound Baseline Pavement Structure

<table>
<thead>
<tr>
<th>Run</th>
<th>Base Material</th>
<th>Base Thickness (in)</th>
<th>Subbase Material</th>
<th>Subbase Thickness (m)</th>
<th>Subbase Equivalency Factor</th>
<th>Tensile Strain, Bottom of AC, microstrain</th>
<th>Compressive Strain, Top of Subg, microstrain</th>
<th>Compressive Stress, Top of Subg, psi (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>19</td>
<td>Stabilized Crushed Agg</td>
<td>16.5 (0.4191)</td>
<td>Stabilized Crushed Agg</td>
<td>22.0 (0.5588)</td>
<td>1.20</td>
<td>4.781e2</td>
<td>5.677e2</td>
<td>11.77 (81.15)</td>
</tr>
<tr>
<td>20</td>
<td>Stabilized Crushed Agg</td>
<td>16.5 (0.4191)</td>
<td>Stabilized Uncrushed Agg</td>
<td>21.5 (0.5461)</td>
<td>1.23</td>
<td>4.722e2</td>
<td>5.523e2</td>
<td>11.78 (81.22)</td>
</tr>
<tr>
<td>21</td>
<td>Stabilized Uncrushed Agg</td>
<td>18.0 (0.4572)</td>
<td>Stabilized Uncrushed Agg</td>
<td>17.0 (0.4318)</td>
<td>1.56</td>
<td>4.495e2</td>
<td>5.829e2</td>
<td>11.73 (80.88)</td>
</tr>
</tbody>
</table>
CHAPTER 6
SUMMARY AND CONCLUSION

6.1 Summary

Equivalency factors are used within the airfield pavement design process described in UFC 3-260-02. These factors are used to substitute stabilized base courses and subbase courses into pavement structures designed using the CBR design method, as CBR nomographs were not developed to consider the superior engineering properties of stabilized layers. In this study, mechanistic analysis methods were used to evaluate these established equivalency factors.

To perform a mechanistic analysis, material properties from laboratory testing are required. For the linear analysis, reasonable moduli for asphalt stabilized materials were determined based on a literature review. Poisson’s ratios were selected based on material type recommendations outlined in UFC 3-260-02.

To provide comparison between a pavement structure which contains entirely unbound materials and a pavement structure with stabilized layers, standard layer thicknesses and gear loading had to be established. The B-52 gear was used in the analysis, as this aircraft is categorized as one of the Air Force’s most damaging to airfield pavements. Additionally, due to the unique gear configuration and spacing, there is no overlap in loading from the two landing gears deep within the pavement structure. The standardized pavement structure, also referred as the unbound baseline pavement structure, was designed for the Air Force heavy traffic pattern and a Type C pavement area. The standardized pavement structure was designed using the LED process using PCASE, to provide a more direct material property correlation between the pavement design method and the linear analysis method.

With the pavement structure determined, the critical responses – tensile strain at the bottom of the asphalt surface, compressive strain at the top of the subgrade, and compressive stress at the top of the subgrade – were determined for the baseline unbound pavement structure under the B-52 gear load using BISAR 3.0. The calculated critical responses were assumed to be acceptable levels of stress and strain within the applicable pavement layer. Stabilized materials then replaced the unbound base and subbase courses. The stabilized layer thicknesses were adjusted until the critical response magnitudes under the stabilized pavement structure were similar to those seen under the unbound baseline pavement structure. Proposed equivalency factors were determined by calculating the ratio between the unbound pavement structure layer thickness and the stabilized pavement structure layer thickness – when both pavement structures had comparable critical response magnitudes when loaded with the standard gear load.

To further test the proposed equivalency factors determined from the linear analysis, a non-linear analysis was also undertaken. Again, non-linear material properties were determined from literature. There
was an attempt to use non-linear materials which were similar, in an engineering sense, to those materials used in the linear analysis. GT Pave was used for the non-linear analysis, which can only handle a single wheel load. Thus, the load and diameter of a single wheel within the B-52 gear was used as the standardized load. Again, critical responses were determined when the unbound baseline pavement structure was loaded. Stabilized layer thicknesses, determined from both the established and proposed equivalency factors, were then substituted into the pavement structure and critical responses were determined. It was found that the proposed equivalency factors for base courses were adequate in the non-linear analysis, for the materials and load combinations used. Additionally, the non-linear analysis found the proposed equivalency factor for the all bituminous subbase to be adequate. The proposed equivalency factors for stabilized crushed and uncrushed aggregate subbases were found to be unconservative when non-linear responses were considered. Stabilized sand subbases were not tested due to lack of available literature outlining non-linear properties of that material.

6.2 Conclusions

1. Mechanistic procedures can be used to analyze and refine an empirically based airfield pavement design procedure. Both the CBR design procedure and the work completed in the 1970s to develop stabilized layer equivalency factors are primarily based on empirical data from field testing. Mechanistic procedures can be used in concert with empirical design aspects to evaluate the resulting pavement structure. These mechanistic results can then be interpreted and applied to improve empirical methods without significant procedural change.

2. Base course equivalency factors currently suggested in UFC 3-260-02 appear to be overly conservative when analyzed using both linear and non-linear analyses. This was hypothesized given an equivalency factor of 1.00 being assigned to stabilized aggregate materials in the base position – as this suggests no decrease in layer thickness is warranted when using an asphalt stabilized aggregate.

3. Under single wheel load conditions, and with the non-linear materials used in this study, subbase layer thicknesses suggested by the linear and non-linear analyses are significantly different. This is due to the lower stress states within the subbase delivering a lower modulus with stress hardening materials. However, the single wheel load method used might calculate an artificially low stress due to load in the subbase level due to the inability of the program to consider multiple wheel loads.

6.3 Recommendations for Implementation
Based on the results of the linear analysis and the non-linear test, it is recommended that the proposed base course equivalency factors be implemented in the next revision of UFC 3-260-02. It is also suggested that the proposed equivalency factor for the all bituminous subbase be implemented in the next revision.

Due to the significant differences between the critical responses observed in the linear and non-linear evaluations, recommendations for the stabilized uncrushed and crushed aggregate are not as straightforward. If the performance of airfield pavements with asphalt stabilized subbases has been satisfactory under the established equivalency factors, then their use should be continued. However, if pavement performance using these materials has been marginal, more conservative equivalency factors can be implemented based on the limited non-linear testing in this document or additional evaluation of asphalt stabilized subbase materials completed through other means.

Since the non-linear properties of stabilized sand were not available in literature, it is recommended that the more conservative proposed equivalency factor established in the linear analysis be utilized, until further research is conducted.

6.4 Areas for Further Research

Based on the work performed in this study, the following topics are identified as areas which could benefit from additional research.

6.4.1 Asphalt Stabilized Design Guidance

The design guidance provided in UFC 3-250-11 is vague in terms of sample preparation and testing. As evidenced in the literature review, significant variations in laboratory sample preparation, curing, and testing are seen in the field – and these variations can cause significant differences in measured material properties. Creating more standardized material preparation and testing procedures will allow for a more consistent evaluation of the emulsion and aggregate being proposed for the stabilized mixture.

While the Marshall Stability test may provide adequate engineering property definition for a material to be used in the CBR design method, it does not provide useful information for mechanistic analyses or mechanistic pavement design procedures – to include the LED design method and those modeling programs used in this analysis. It may be beneficial for UFC 3-250-11 to outline appropriate tests and procedures to obtain parameters needed by mechanistic analyses in order to facilitate further investigation into the use of asphalt stabilized materials and to encourage utilization of the more strenuous LED design method. Appropriate test procedures and values for the Marshall Stability test can continue to
be referenced in guidance, should more field expedient/lower technology testing method for this material be needed.

6.4.2 Multi-Load Non-Linear Analysis Software

Readily available software available for performing non-linear analysis on pavements either cannot facilitate multiple wheel loads or perform some simplification of the pavement system in order to evaluate multiple wheel loads. For example, the ILLI-Pave non-linear, axi-symmetric, finite element evaluation program accounts for multiple wheel loads using the “engineering approach” (ILLI-Pave user’s guide). In this approach, ILLI-Pave solves the non-linear analysis under a single wheel load. The program then subdivides the pavement structure into multiple rows, 2 to 3 inches in depth, and assigns the average row modulus value determined through the non-linear analysis. After subdividing the pavement structure and assigning average modulus values, it then performs a linear analysis, in which the multiple wheel loads are considered.

To overcome this limitation, a program which can perform true non-linear analysis with multiple wheel loads could be used. This could be accomplished with the Displacement Method Analyzer (DIANA), developed at Delft University, Netherlands or through the commercially available ABACUS software with user defined stress dependent material responses for pavement layers. These tools will provide a more robust analysis – especially in the case of the subbase layer – to confirm whether the observations seen with subbase materials in GT Pave are valid in slightly higher stress environments (as load induced stress from multiple wheel loads will likely overlap in the subbase layer, producing a higher stress environment, which will in turn increase the layer modulus, and may improve the performance of stabilized materials in this position.
REFERENCES


Figure A.1. Non-Linear Properties of Elliot Subgrade (Thompson, 1976)
Figure A.2. Non-Linear Properties of Unbound Uncrushed Aggregate (after Anderson and Thompson, 1995), Red Line Denotes Modulus used in Linear Analysis

Figure A.3. Non-Linear Properties of HFE Stabilized Uncrushed Aggregate (after Anderson and Thompson, 1995), Red Line Denotes Modulus used in Linear Analysis
Figure A.4. Non-Linear Properties of Unbound Crushed Aggregate (after Anderson and Thompson, 1995), Red Line Denotes Modulus used in Linear Analysis

Figure A.5. Non-Linear Properties of HFE Stabilized Crushed Aggregate (after Anderson and Thompson, 1995), Red Line Denotes Modulus used in Linear Analysis
### Table A.1. Non-Linear Properties of Elliot Subgrade used in GT Pave Analysis (after Thompson, 1976)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tr>
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<tr>
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*Values not provided in literature. Typical values for the material were used.

### Table A.2. Non-Linear Properties of CL-3sp Subbase used in GT Pave Analysis (after Seyhan, 1998)

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<td>K1 (psi)</td>
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<td>n</td>
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*Values not provided in literature. Typical values for the material were used.

### Table A.3. Non-Linear Properties of Uncrushed Aggregate used in GT Pave Analysis (after Anderson and Thompson, 1995)

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<tr>
<th>Parameter</th>
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<td>Density (pci)</td>
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<td>K1 (psi)</td>
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*Values not provided in literature. Typical values for the material were used.
Table A.4. Non-Linear Properties of Crushed Aggregate used in GT Pave Analysis (after Anderson and Thompson, 1995)

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<td>Body Force in Vertical Direction (pci)</td>
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*Values not provided in literature. Typical values for the material were used.

Table A.5. Asphalt Surface Properties used in GT Pave Analysis

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