University of Nevada, Reno

Evaluation of Thin Asphalt Overlays for Pavement Preservation in Nevada

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil and Environmental Engineering

by

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ABSTRACT

Over the last 35 years, state highway agencies changed their emphasis from the construction of new roads to the maintenance and rehabilitation of existing infrastructures. As a routine pavement maintenance and preservation tool, thin asphalt overlays have seen importance and were classified as extremely useful for flexible and composite pavements since 1999. Defined as simple surface lifts or part of mill-and-fill strategies typically placed at no more than 1.5 in. (38 mm) thick on a well prepared surface, their essential function is not to strengthen the existing pavement capacity, but primarily to address issues related to pavement functional performance.

The overall objective of this study was to assess the use of locally available materials in Nevada for the development of a durable fine-graded thin hot-mix asphalt (HMA) overlay mixture for pavement preservation. A comprehensive laboratory evaluation using typical materials in Nevada was conducted. The investigation considered establishing two mix designs using typical local materials for the northern and southern part of the state. An optimal asphalt binder content (OBC) was selected for each mixture based on the volumetric properties and following the Nevada department of transportation (NDOT) volumetric requirements. For each mixture, the optimal binder content was varied within the allowable tolerances to simulate the potential variation in asphalt binder content during plant production. The performance of the two thin HMA mixtures were then evaluated at the various asphalt binder contents in terms of their resistance to moisture damage using indirect tensile strength (TS), resistance to surface raveling and abrasion, dynamic modulus (|E*|) property, resistance to rutting using the flow number (FN) test, resistance to reflective cracking using the Texas Transportation Institute (TTI) overlay
tester, workability and compactability using the pressure distributor analyzer (PDA), and the developed interlayer bond strength using the Louisiana Interlayer Shear Strength Tester (LISST).

Overall, both designed fine-graded mixtures showed a very good performance and are expected to perform well when used as a thin hot-mix asphalt overlay. In particular, good stability, very good resistance to surface raveling and abrasion, and excellent resistance to reflective cracking were observed for both thin HMA overlay mixtures at all evaluated asphalt binder contents. Thin asphalt overlay mixtures behaved as ordinary mixture and are expected to last longer than chip seals for a lower net present worth costs.

Based on the findings from this study, it was recommended to construct field test sections in various parts of the state to evaluate the field performance of the developed thin HMA overlay mixtures in Nevada.

**Keywords:** Asphalt Mixture, Thin Asphalt Overlays, Allowable Tolerances, Raveling, Abrasion, Stiffness, Rutting, Reflective Cracking, Workability, Compactability Interlayer Bond Strength, Chip Seal.
DEDICATION

I dedicate this work to my father Fares, my mother Mathilda, my brother Joseph, my sister Joyce and all my friends for all their love, endless support and encouragement. I praise and thank the LORD for each of them.
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The completion of this master’s thesis was not possible without the support of several people. With boundless love and appreciation, I would like to extend my heartfelt gratitude and appreciation to the people who helped me to bring this study into reality.

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CHAPTER 1 INTRODUCTION

1.1. Introduction

Over the last 35 years, the United States departments of transportation and highway agencies changed their emphasis from the construction of new roads to maintenance and rehabilitation of existing infrastructure by using several pavement preservation techniques (1). These techniques are defined as a set of cost-effective practices designated to extend pavement life, improve safety, and save public tax dollars. Table 1.1 below shows the different possible preservation treatments typically used for flexible and rigid pavements.

Table 1.1 - Preservation Techniques for Flexible and Rigid Pavements.

<table>
<thead>
<tr>
<th>Flexible pavement treatments</th>
<th>Rigid pavement treatments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Asphalt Rejuvenators</td>
<td>Crack Sealing</td>
</tr>
<tr>
<td>Asphalt Sealers</td>
<td>Joint Resealing</td>
</tr>
<tr>
<td>Crack Sealing</td>
<td>Spall Repair</td>
</tr>
<tr>
<td>Crack Filling</td>
<td>Dowel Bar Retrofit</td>
</tr>
<tr>
<td>Scrub Seals</td>
<td>Cross Stitching (longitudinal cracks &amp; joints)</td>
</tr>
<tr>
<td>Chip Seals</td>
<td>Partial Depth Repair</td>
</tr>
<tr>
<td>Cape Seals</td>
<td>Full Depth Repair (limited number of repairs)</td>
</tr>
<tr>
<td>Slurry Seals</td>
<td>Ultra-Thin White Topping</td>
</tr>
<tr>
<td>Micro-surfacing</td>
<td>Undersealing</td>
</tr>
<tr>
<td>Ultra-thin Overlays</td>
<td>Slab Lifting</td>
</tr>
<tr>
<td>Bonded Wearing Course</td>
<td>Diamond Grooving</td>
</tr>
<tr>
<td>Profile Milling</td>
<td>Diamond Grinding</td>
</tr>
<tr>
<td>Thin Overlays (non-structural, generally ≤ 1½ inch)</td>
<td>CPR (concrete pavement restoration)</td>
</tr>
<tr>
<td>Mill &amp; Resurface (non-structural, generally ≤ 1½ inch)</td>
<td>-</td>
</tr>
<tr>
<td>Hot In-place Recycling</td>
<td>-</td>
</tr>
<tr>
<td>Cold In-place Recycling</td>
<td>-</td>
</tr>
</tbody>
</table>

The Nevada Department of Transportation (NDOT) uses various maintenance and rehabilitation repair strategies to improve the overall states’ pavement network condition (2, 3). The maintenance repair strategies for flexible pavements include work such as chip
seals, filling potholes, and patching. The rehabilitation repair strategies include work such as asphalt overlays and recycling methods. The cost and construction timing for the various repair strategies are significantly different and dependent on the pavement condition at the time of the repair (3). Some treatments are applied on a schedule basis such as the proactive treatments. A significant cost saving is anticipated when a pavement is proactively rehabilitated in fair to good condition as compared to reactively reconstructed in very poor condition. For example, a proactive asphalt overlay prevents the pavement from deteriorating to a point when more expensive major rehabilitation or reconstruction strategies are required.

Recently, NDOT expressed interest in using thin hot-mix asphalt overlays as a mean to extend the available funds for pavement maintenance and preservation and for essentially delaying the need for pavement rehabilitation. As a routine maintenance and pavement preservation tool, thin asphalt overlays were classified as extremely useful for asphalt and composite pavements according to an American Association of State Highway and Transportation Officials (AASHTO) survey done by the Lead States Team on Pavement Preservation in 1999 (1, 4). These overlays are defined as surface courses typically placed no more than 1.5 in. (38 mm) thick on a well prepared surface (5). Used as simple surface lifts or part of mill-and-fill strategies, their essential function is not to strengthen the pavement but to address functional problems (5). Thin asphalt overlays are used to protect the pavement structure, slow the rate of deterioration, correct many surface deficiencies, improve ride quality, and add a minor amount of enhancement to the existing pavement (1).
In 2014, a study was sponsored by the SOLARIS institute (Tier 1 University Transportation Center) and NDOT to assess the use of locally available materials in the state of Nevada for the development of a durable fine-graded thin hot-mix asphalt overlay mixture for pavement preservation (6).

The first task of this study consisted of a review of literature to compile information on the available research, overall characteristics and properties (benefits, applications, factors affecting the application, functional and structural characteristics, and treatment life), materials and mix design (aggregate gradations and optimal laboratory compaction), and construction of thin hot-mix asphalt overlay (surface treatment and preparation, placement and in-place compaction, quality control, and performance).

The second task consisted of identifying the properties of the designed thin asphalt overlay mixtures through a comprehensive laboratory evaluation. The designed mixtures were evaluated in terms of their resistance to moisture damage (i.e., Tensile strength ratio), surface raveling (i.e., raveling test), surface abrasion (i.e., Cantabro loss test), rutting (i.e., unconfined flow number test), and reflective cracking (i.e., Texas Overlay test). The workability and compactability of the thin hot-mix asphalt designed mixtures were evaluated using the pressure distribution analyzer (PDA) mounted recently as a permanent part in the Superpave Gyratory Compaction machine (SGC). The bond strength between the thin hot-mix asphalt overlay and the existing asphalt layer was also evaluated using the Louisiana Interlayer Shear Strength tester (LISST).

The third task of the study consisted of providing recommendations and necessary revisions for the preliminary specifications for thin asphalt overlays relative to design, materials, mix design criteria, and acceptance testing.
1.2. Problem Statement

Regardless of the anticipated benefits, the historical success of thin asphalt overlays varied among the various state highway agencies (SHAs). A 2008 AASHTO questionnaire study revealed that out of the twenty-five states that have used thin asphalt overlays before, eleven reported less than satisfactory results (7). The reported problems with thin asphalt overlays included delamination, reflective cracking, poor friction, low durability, excessive permeability, and maintenance problems once failure occurs. A high resistance to reflective cracking and moisture damage along with a good bond to the existing asphalt layer are deemed necessary in order to guarantee an extended performance life for a thin overlay. Furthermore, past experiences revealed that thin asphalt overlays can be in some cases susceptible to early rutting problems due to mix instability under heavy traffic.

Based on the varying success of using thin asphalt overlays by SHAs, the challenge of this study was to develop a stable and durable fine-grained asphalt mixture using locally available material in Nevada to fulfill the function of a thin overlay. The developed thin asphalt overlay should be able to resist the various environmental and traffic conditions encountered throughout the state of Nevada.

1.3. Objective and Scope

This study was conducted to provide NDOT with a comprehensive evaluation of the material characteristics and design of thin hot-mix asphalt overlays for the State of Nevada. For this purpose, the major tasks carried out in this research were:

- Establish a review of literature by compiling information about thin overlays and their performance all around the United States.
• Establish two Hveem mix designs for thin hot-mix asphalt overlay mixtures using local materials from the northern and southern part of the state. Select an optimal asphalt binder content with respect to NDOT volumetric requirements.

• Vary the selected optimal asphalt binder content within some allowable tolerances.

• Evaluate the performance properties of the designed mixtures at different asphalt binder contents within the allowable tolerances.

• Conduct a statistical data analysis to evaluate the variation in performance properties for each mixture at different asphalt binder contents.

• Generate conclusions and recommendations, and propose upcoming tasks to develop a specification or a standard for the thin asphalt overlay in the state of Nevada.
CHAPTER 2 REVIEW OF LITERATURE

The objective of this chapter is to provide a background of information about thin asphalt overlays in the United States. It summarizes the benefits and limitations, the factors affecting the application of thin asphalt overlays, the functional and structural characteristics provided by its use, the treatment life, the materials used for recently developed mix designs, and the construction procedures.

2.1. General Overview

Referring to a survey conducted by the National Cooperative Highway Research Program (NCHRP, synthesis 464), several limitations for the thin asphalt overlay thickness were given by many agencies in different states as shown in Table 2.1 and as illustrated in Figure 2.1 (4).

<table>
<thead>
<tr>
<th>Thickness limitations in inches</th>
<th>Percentage of answer</th>
</tr>
</thead>
<tbody>
<tr>
<td>more than 2.0</td>
<td>3 %</td>
</tr>
<tr>
<td>Between 1.5 and 2.0</td>
<td>11 %</td>
</tr>
<tr>
<td>Between 1.0 and 1.5</td>
<td>24 %</td>
</tr>
<tr>
<td>Between 0.75 and 1.5</td>
<td>25 %</td>
</tr>
<tr>
<td>Between 0.75 and 1.0</td>
<td>28 %</td>
</tr>
<tr>
<td>Less than 0.75</td>
<td>9 %</td>
</tr>
</tbody>
</table>
Figure 2.1 – Summary of States Responses for Thin Overlay Thickness.

Referring to the percentage provided before, thin asphalt overlays are defined as surface courses typically placed no more than 1.5 in. (38 mm.) for more than 86% of the survey participants. Thus, because of this limitation on thickness, a smaller Nominal Maximum Aggregate Size (NMAS) is necessary for a thin asphalt overlay mixture. The following relationship between the NMAS and the asphalt concrete pavement layer thickness is typically used:

\[ t = n \times S \]  \hspace{1cm} (2.1)

Where:

\( t \) is the thin asphalt overlay thickness.

\( S \) is the Nominal Maximum Aggregate Size (NMAS).

\( n \) is an integer scaling factor.
Previous studies provided 1 to 3 as an acceptable range in which the integer scaling coefficient \((n)\) should fit. Table 2.2 shows some examples of the desired thin overlay thicknesses (i.e., 2, 1.5, 1, and 0.75 in.) as a function of the NMAS (i.e., 1, 0.75, 0.5, and 0.375 in.).

### Table 2.2 - Examples of Integer Scaling Coefficient Calculation.

<table>
<thead>
<tr>
<th>Desired Thin Asphalt Overlay Thickness, (t) (in.)</th>
<th>NMAS (in.)</th>
<th>Integer Scaling Coefficient, (n)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>3/4</td>
<td>2.67 (\approx 2)</td>
</tr>
<tr>
<td></td>
<td>1/2</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>3/8</td>
<td>5.33 (\approx 5)</td>
</tr>
<tr>
<td>1.5</td>
<td>1</td>
<td>1.5 (\approx 1)</td>
</tr>
<tr>
<td></td>
<td>3/4</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>1/2</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>3/8</td>
<td>4</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>3/4</td>
<td>1.33 (\approx 1)</td>
</tr>
<tr>
<td></td>
<td>1/2</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>3/8</td>
<td>2.67 (\approx 2)</td>
</tr>
<tr>
<td>0.75</td>
<td>1</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>3/4</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>1/2</td>
<td>1.5 (\approx 1)</td>
</tr>
<tr>
<td></td>
<td>3/8</td>
<td>2</td>
</tr>
</tbody>
</table>

Note: the highlighted combinations are rejected.

### 2.2. Benefits and Limitations

Several agencies use thin asphalt overlays as a standard practice for pavement preservation and rehabilitation. A review of literature revealed several benefits for thin asphalt overlays and are summarized as follow (5):

- longer service life and reduced life cycle cost;
- Ability to preserve the road grade and slope in residential areas with minimal impact to drainage system, particularly with a small NMAS mixture;
- Ability to withstand heavy traffic and sustain high shear stresses;
• Assure a smoother surface and maintain surface geometrics;
• Prevent stones loss after initial construction and minimize dust;
• Minimize traffic delays and reduce the tire-pavement noise generation;
• Neglect the curing time and the binder runoff;
• Recycle and save a considerable amount of energy products;
• Ability to be easily maintained and used during the construction stage;
• Ability to restore the skid resistance
• Ability to use the roadway while reconstruction is in progress;
• Saving on the construction time (i.e., an old road can be usually improved and put into full service more rapidly than building a new road)

Not all of the benefits mentioned above are present at the same time. They have a relative importance according to the location of the project, climate, and existing traffic. Regardless of the anticipated benefits mentioned before, and referring to the AASHTO questionnaire launched among twenty five states that have used thin asphalt overlays (7), these overlays can present some limitations on which agencies should focus to treat. Several problems were reported when thin asphalt overlays were used including delamination, reflective cracking, poor friction, low durability, early severe rutting and mix instability under heavy traffic, excessive permeability, and maintenance problems once failure occurs (7).
2.3. Application of Thin Asphalt Overlay

It is well-recognized that the proper pavement preservation technique should be applied at the right time and at the right pavement condition. The existing pavement condition, the expected traffic level, and the environmental condition can widely affect the application of these thin overlays \((1, 8)\).

2.3.1. Existing Pavement Condition

Applying a thin asphalt overlay on the top of an existing pavement surface is usually influenced by the pavement condition and the need for performing a structural rehabilitation. If the existing pavement condition fit to the requirements, a thin asphalt overlay is guaranteed to be a suitable treatment.

The existing pavement should exhibit a good base condition and a uniform cross section. The visible surface distress may include moderate to severe raveling, longitudinal and transverse cracks with the first sign of raveling and secondary cracking, first sign of edge cracking, block cracking, extensive to severe bleeding or polishing, and some patching in good conditions. The pavement may also have some minor base failures and depressions \((1, 8)\). However, some limitations can prevent the use of thin asphalt overlays such as a weak base or a delaminated surface for a rutted pavement.

According to the Maintenance Division at the California Department of Transportation (Caltrans), the primary failure modes of thin asphalt overlays are as follow: delamination, raveling, cracking due to poor compaction, fast cooling and less cohesion \((9)\).
2.3.2. Traffic Level

Thin asphalt overlays were originally considered as part of the standard pavement preservation techniques on low volume roads, in general secondary roads. Latest studies indicated that this technique can also be effective and well workable for high-volume roads which leaded agencies to use it as a preservation technique for high-volume road pavements. (*1, 8*).

2.3.3. Environmental Condition

According to Liu and Gharaibeh *(2013) (8)*, the climate constitutes an important factor for the performance of thin asphalt overlays. A shorter performance life is expected in dry-freeze and wet-freeze environments. In addition, moisture may have a low impact leading to a variation in the service life time of the thin overlay.

2.4. Characteristics of Thin Asphalt Overlay

Thin asphalt overlay preservation techniques present some functional and structural characteristics and are summarized as follows.

2.4.1. Functional Characteristics

The functional characteristics provided by the use of thin asphalt overlays would be improving the ride quality by smoothening of the pavement surface, improving the skid resistance by using polish-resistant aggregates, and reducing the tire-pavement noise level by using a smaller NMAS (*1, 5*). These functional characteristics qualify this treatment as one of the most effective techniques used for pavement preservation.
2.4.1.1. Ride Quality Improvement

The possibility of improving the ride quality with thin overlays improves appreciably when a milling process precedes the overlay placement. Milling is recommended to improve smoothness because it provides an initial surface leveling, removes surface distresses, and assures a uniform surface of the overlay construction. The ride quality improvement on flexible pavements last over a dozen years before reverting to the same ride quality prior to the overlay application. Meanwhile, for composite pavements, the duration of the ride quality improvement decreases to 7 years (1, 5).

2.4.1.2. Skid Resistance

The skid resistance can be widely improved by the use of thin asphalt overlay. If an existing pavement surface was constructed with polishing aggregates, or has been subject to bleeding, it may be a candidate for improved friction. Friction improvement can be accomplished by using a skid-resistant aggregate with a specific gradation (1, 5).

2.4.1.3. Noise Level Reduction

It is well-known that the pavement-tire noise generation is largely affected by the pavement surface macro-texture. In specific, the coarser the macro-texture of the surface, the noisier the traffic passing over the pavement will be. Based on a NCAT study on the noise level emission, Figure 2.2 shows that the generated noise level decreases when a smaller NMAS is used. The proper selection of materials and the mix design approach to thin asphalt overlays are crucial to the success of the pavement performance.
2.4.2. Structural Characteristics

In general, preventive maintenance techniques are supposed to be non-structural. Their essential function is not to strengthen the existing pavement capacity but primarily to address issues related to pavement functional performance. According to the Mississippi Asphalt Pavement Association (10), slurry seals, chip seals, and micro-surfacing add no structural capacity to the existing pavement in contrary to a 0.5 to 1.0 in. (12.5 to 25.4 mm) thin asphalt overlay which should be recognized and credited for this addition.

Over time, repeated traffic loading can weaken the pavement structure (e.g., fatigue cracking) and growing traffic counts require higher pavement strength. A thin asphalt overlay can be applicable when additional strength is needed and required to preserve the structure. The micro-strain decreases and the fatigue life repetitions to failure increases when additional inches of asphalt are added on the top of the existing pavement. Based on
It can be noticed that adding an additional inch of asphalt overlay reduces the developed micro strain at the bottom of the existing asphalt layer and can double the number of fatigue life repetitions required to failure.

![Tensile Strain vs Thickness and Fatigue Life Repetitions vs Overlay Thickness](image)

Figure 2.3 - Micro strain and Fatigue Life Repetitions Variation Function of the Overlay Thickness Increment.

### 2.5. Treatment life of Thin Asphalt Overlay

The application of thin asphalt overlay has been known as one of the most cost-effective methods used as a preservation technique. It is characterized by an average life cycle cost but a long lasting reduction in cost. Table 2.3 shows that the highest average service life (i.e., 8.4 years) was reported for the thin overlay among all the existing pavement preservation techniques. The pavement life is based on qualitative perceptions rather than well-designed quantitative experimental analyses. The life expectancy of a thin overlay is generally considered for seven to ten years. Most of all studies done by NAPA 

(1) refer that thin asphalt overlays can provide expected performance of ten years or more.
when it is applied to an asphalt pavement surface and six to ten years when it is placed on top of a Portland cement concrete (PCC) pavement surface. Table 2.4 summarizes the expected treatment life for a thin asphalt overlay in different states. The minimum and maximum average expected treatment life were 7.7 and 10.4 years, respectively. The overall average expected treatment life offered by the application of thin asphalt overlay is about 9 years, which is a long duration comparing to other preservation treatment and maintenance techniques.

Given the prolonged treatment life, there has been interest in using thin asphalt overlays to extend the available funds for maintenance and preservation and increase the number of lane-miles to be resurfaced annually. The asphalt overlay life span is relatively long with very high estimates of pavement life extension. Because of the prolonged pavement life span, thin asphalt overlays are considered one of the most cost-efficient solutions compared to other pavement preservation techniques.

Table 2.3 - Average Service Life and Cost per Lane Miles of Several treatments.

<table>
<thead>
<tr>
<th>Preventive maintenance treatment</th>
<th>Average Service life (years)</th>
<th>Cost per Lane-Mile (12 ft wide)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thin overlay</td>
<td>8.4</td>
<td>$14,600</td>
</tr>
<tr>
<td>Double chip seal</td>
<td>7.3</td>
<td>$12,600</td>
</tr>
<tr>
<td>Micro-surfacing</td>
<td>7.4</td>
<td>$12,600</td>
</tr>
<tr>
<td>Slurry Seal</td>
<td>4.8</td>
<td>$6,600</td>
</tr>
</tbody>
</table>

Table 2.4 - Treatment Life of Thin Asphalt Overlay According to Several References.

<table>
<thead>
<tr>
<th>Study</th>
<th>State or reference</th>
<th>Date of publication</th>
<th>Treatment life (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NCHRP</td>
<td>N.A</td>
<td>2014</td>
<td>7 to 11 (in average 8.4)</td>
</tr>
<tr>
<td></td>
<td>FHWA (1, 11)</td>
<td></td>
<td>8 to 11</td>
</tr>
<tr>
<td></td>
<td>Florida (1)</td>
<td></td>
<td>10 to 12</td>
</tr>
<tr>
<td></td>
<td>Minnesota (1)</td>
<td></td>
<td>5 to 8</td>
</tr>
<tr>
<td></td>
<td>New York (1)</td>
<td>2009</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Ohio (1, 7)</td>
<td></td>
<td>8 to 12</td>
</tr>
</tbody>
</table>

N.A denotes “Not available”
2.6. Thin Asphalt Overlay Mix Designs

The success of the thin asphalt overlay performance is essentially based on the proper selection of raw materials. In general, the aggregates have to withstand the repeated traffic-induced loads without resulting in rutting. The use of a smaller NMAS in the mixtures’ gradation is required to be able to reach the desired thin layer thickness without crushing the aggregates under the induced high stresses from the compaction effort and the heavy traffic loads. The small size of aggregates implies a high specific area which would lead to a high asphalt binder content. This section of the report provides some information about recent mix designs developed in several states and their requirements to guarantee a good performance.

2.6.1. Review of Aggregate Gradations and Mix Design Properties

A national survey conducted by NAPA in 2009 recommended the use of 0.5 in. (12.5 mm.) NMAS for thin asphalt overlay mixtures as shown by the aggregate gradation specifications in Table 2.5 (1). The aggregate gradations used by different agencies varied from a state to another. NDOT has recently developed a preliminary aggregate gradation for their thin asphalt overlay mix designs. Table 2.6 presents a comparison between the recent aggregate gradation proposed by NDOT and the previous gradation surveyed by NAPA for the state of Nevada (1). As shown in Table 2.6, the recent gradation has a NMAS of 0.5 in. (12.5 mm).
Several factors contribute to the proper selection of the aggregates quality such as the type of the pavement designed (i.e., AC or PCC), and the expected traffic volume and speed. The aggregate quality varies from one state to another because of the required limitations for some quality and properties tests. In the following, the gradation proposed by NDOT will be maintained and compared to the aggregate gradation limits available for three states: Alabama, North Carolina, and Utah as shown in Table 2.7 and illustrated in Figure 2.4. It can be noticed that each gradation has an upper and a lower limit where any proposed gradation is qualified as acceptable if it fits between these limits. The aggregate gradation specification for the state of Nevada and Alabama are too close, while the ones for North Carolina and Utah are coarser and finer, respectively.
Several accomplished researches have established parameters and criteria for design air voids level (AV), voids in mineral aggregate (VMA), and voids filled with asphalt (VFA). Table 2.8 summarizes the aggregate qualifications and properties for several states according to the NAPA 2009 study (1). The requirements for coarse and fine aggregates vary according to locally available materials as well as traffic levels.

### Table 2.7. Aggregate Gradations Upper and Lower Limits for Nevada, Alabama, North Carolina, and Utah.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Sieve Size in mm</th>
<th>Nevada</th>
<th>Alabama</th>
<th>North Carolina</th>
<th>Utah</th>
</tr>
</thead>
<tbody>
<tr>
<td>37.5 mm (1 1/2&quot;)</td>
<td>37.500</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>25.0 mm (1&quot;)</td>
<td>25.000</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>19.0 mm (3/4&quot;)</td>
<td>19.000</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
<td>100.0</td>
</tr>
<tr>
<td>12.5 mm (1/2&quot;)</td>
<td>12.500</td>
<td>90.0</td>
<td>100.0</td>
<td>90.0</td>
<td>100.0</td>
</tr>
<tr>
<td>9.5 mm (3/8&quot;)</td>
<td>9.500</td>
<td>70.0</td>
<td>90.0</td>
<td>58.0</td>
<td>90.0</td>
</tr>
<tr>
<td>4.75 mm (No. 4)</td>
<td>4.750</td>
<td>50.0</td>
<td>70.0</td>
<td>28.0</td>
<td>38.0</td>
</tr>
<tr>
<td>2.36 mm (No. 8)</td>
<td>2.360</td>
<td>28.0</td>
<td>58.0</td>
<td>19.0</td>
<td>32.0</td>
</tr>
<tr>
<td>2.00 mm (No. 10)</td>
<td>2.000</td>
<td>30.0</td>
<td>50.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.18 mm (No. 16)</td>
<td>1.180</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
<td>0.425</td>
<td>12.0</td>
<td>24.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.3 mm (No. 50)</td>
<td>0.300</td>
<td>9.0</td>
<td>19.0</td>
<td>8.0</td>
<td>13.0</td>
</tr>
<tr>
<td>0.15 mm (No. 100)</td>
<td>0.150</td>
<td>5.0</td>
<td>12.0</td>
<td>5.0</td>
<td>12.0</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>0.075</td>
<td>3.0</td>
<td>8.0</td>
<td>3.0</td>
<td>8.0</td>
</tr>
</tbody>
</table>
Figure 2.4 - Aggregate Gradation Limits of Thin Asphalt Overlay Mixtures from Various States.

Table 2.8 - Aggregate Qualifications and Properties of Thin Asphalt Overlay Mixtures from Various States.

<table>
<thead>
<tr>
<th>Agency</th>
<th>Alabama</th>
<th>North Carolina</th>
<th>Nevada</th>
<th>Utah</th>
<th>New York</th>
<th>Maryland</th>
<th>Georgia</th>
<th>Ohio</th>
</tr>
</thead>
<tbody>
<tr>
<td>LA Abrasion % loss</td>
<td>48 max</td>
<td>35 max</td>
<td>37 max</td>
<td>35/40 max</td>
<td></td>
<td></td>
<td></td>
<td>40 max</td>
</tr>
<tr>
<td>Sodium Sulfate Soundness, % loss</td>
<td>10 max</td>
<td>15 max</td>
<td>12 max</td>
<td>16/16 max</td>
<td></td>
<td></td>
<td></td>
<td>12</td>
</tr>
<tr>
<td>% 2 or more fractured face</td>
<td>85 min</td>
<td>80 min</td>
<td>90/90 min</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>% 1 fractured face</td>
<td>100 min</td>
<td></td>
<td>95/90 min</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand equivalent, % (Fine aggregate)</td>
<td>45 min</td>
<td></td>
<td>60/45 min</td>
<td>45 min</td>
<td>28/40</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Un-compacted Void content, % (Fine aggregate)</td>
<td>43/45 min</td>
<td>40 min</td>
<td>43 min</td>
<td>40 min</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
2.6.2. Asphalt Binders and Thin Asphalt Overlay Types

In general, two factors, the climate and traffic level, influence the selection and specification of the asphalt binder performance grade (PG). Some states use modified asphalt binders for their mixes as shown in Table 2.9 (4).

**Table 2.9 - Asphalt Binder Types Used for Thin Overlay Mixtures in Various States.**

<table>
<thead>
<tr>
<th>States</th>
<th>Binder used</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minnesota</td>
<td>Neat asphalt binder</td>
</tr>
<tr>
<td>New Jersey</td>
<td>Polymer modified PG 76-22</td>
</tr>
<tr>
<td>New York (downstate region)</td>
<td>Polymer modified PG 76-22</td>
</tr>
<tr>
<td>New York (upstate region)</td>
<td>Polymer modified PG 64-22</td>
</tr>
<tr>
<td>North Carolina (ESAL level: highest)</td>
<td>Polymer modified PG 76-22</td>
</tr>
<tr>
<td>North Carolina (ESAL level: lowest)</td>
<td>Polymer modified PG 64-22</td>
</tr>
<tr>
<td>Ohio</td>
<td>Polymer modified PG 64-22 or PG 76-22</td>
</tr>
</tbody>
</table>

The thin asphalt overlays may vary in type from a state to another due to the variation in the treatment criteria and the asphalt binder used. Several type of mixes have been used in past constructions and are summarized in Table 2.10 (4). Past studies indicated that there is no significant difference in the rutting resistance between the main course and the fine-graded mixes; hence making the fine mixes desirable because of their workability, low permeability, resistance to segregation, and being more economical and able to be placed in thinner layers.
2.6.3. Mix Design Types

Different types of mix design can be generally used for thin asphalt overlay mixtures referring to the available material, knowledge and the state where the mix is designed. Some mix designs are well-known referring to several states such as the dense-graded mixtures where the Hveem and the Superpave method are presently used, Open Graded Friction Course (OGFC) where the Marshal Mix design is usually adopted, and some private mix designs developed by many agencies such as for the state of California and Europe.

**Dense-graded (DG) mixtures:** those mixtures are characterized by a continuously graded aggregate structure. This type of mixture is generally characterized by a low air void contents and has an abrasion and functionally impermeable wearing course. Conventional dense-graded thin overlays are generally placed on structurally sound pavements because of the little structural improvement offered by this mix type and/or the additional thickness. This type of mix design can mitigate several distresses such as: raveling, oxidation, minor cracking, minor surface irregularities, some skid problems and
a pavement water proofing \((1, 5)\). The job selection is generally based on: traffic loading, existing pavement conditions, and environment. For high volume roads, fatigue cracking and permanent deformation are in general the primary factors affecting the selection. Such type of mixtures should be used on pavements that do not present a high amount of distresses. The dense-graded thin asphalt overlays showed 2 to 10 years of service life \((1, 5)\). Table 2.11 summarizes the mix design characteristics and properties for various states.

Some agencies (e.g., Georgia, Illinois, North Carolina, Ohio, Texas, and Washington) added other performance criteria like rutting test using either the APA or Hamburg Wheel-Tracking Device to prevent possible rutting. In Texas, the Texas Overlay Tester is used to evaluate the mixture’s resistance to reflective cracking.

**Table 2.11 - Required Volumetric Properties and Characteristics for Thin Asphalt Overlay Mix Designs in Selected States.** \((1)\)

<table>
<thead>
<tr>
<th>Agency</th>
<th>Alabama</th>
<th>North Carolina</th>
<th>Nevada</th>
<th>Utah</th>
<th>New York</th>
<th>Maryland</th>
<th>Georgia</th>
<th>Ohio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of gyrations based on traffic level</td>
<td>60</td>
<td>N/A</td>
<td>50 to 125</td>
<td>75</td>
<td>50 to 65</td>
<td>50</td>
<td>50</td>
<td>75</td>
</tr>
<tr>
<td>Design Air Voids</td>
<td>3 - 6</td>
<td>3.5</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0 – 7.0</td>
<td>3.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>% VMA</td>
<td>15.5 min</td>
<td>12 - 22</td>
<td>16 min</td>
<td></td>
<td></td>
<td></td>
<td>15 min</td>
<td></td>
</tr>
<tr>
<td>% VF, range</td>
<td></td>
<td>70 - 80</td>
<td>70 - 78</td>
<td></td>
<td></td>
<td></td>
<td>50 - 80</td>
<td></td>
</tr>
<tr>
<td>Asphalt Content</td>
<td>5.5 min</td>
<td>4.6 – 5.6 min</td>
<td></td>
<td></td>
<td>5.0 – 8.0</td>
<td>6.0 – 7.5</td>
<td>6.4 min</td>
<td></td>
</tr>
</tbody>
</table>

**Open-Graded Friction Course (OGFC):** These types of mixtures are also widely used in thin asphalt overlay mixtures. This layer effectively reduces the temperature differential between the top and the bottom when used over a rigid pavement. There are also many uses of OGFC not associated with PCC or composites as well. It is also useful for the roadway noise reduction at the tire-pavement interface. It is characterized by an
open void structure aggregate gradation. The air voids content for the OGFC typically ranges between 15% and 25%. The porous nature of OGFC mixtures allows surface water to quickly drain away from the surface. Several benefits can be offered by the use of the OGFC such as: a significant reduction in splash and spray relative to asphalt mixtures, abrasion resistant wearing course, reduction in tire noise, and increase in the frictional surface characteristics. It is usually placed on structurally sounds pavements and can renew the surface in terms of functional performance. However, the OGFC overlays can exhibit several distress modes such as: permanent deformation due to heavy load and high temperature, high stress areas causing a shear failure, repeated traffic loading that cause fatigue cracking, raveling (due to oxidation, hardening of the binder, water damage, low binder content, or low compaction), and poor compaction causing delamination and the loss of permeability that causes clogging of air voids. Some limitations prevent the use of the OGFC such as: unstable pavements having an extensive cracking or other distresses, snow or icy areas where there is a possibility of aggregate stripping causing the raveling or a deterioration of the existing pavement, areas with severe turning movements so it is recommended to use modified binders in those areas, muddy areas, oil spill areas where there is a dripping of oil and fuel and mill and fill areas that are not recognized to be candidates of OGFC as a bathtub effect can be created. It can be also mentioned that deicing salts and particularly sand can be used to improve friction during freezing weather, but these treatments also add to the clogging problem and reduce the effectiveness of the OGFC.
2.6.4. Laboratory Compaction

Several agencies have conducted research studies to determine the optimal number of gyrations necessary for the aggregates to lock together. It has been recommended \((1, 5)\) that each agency determines the necessary optimum locking point of the aggregate structure in its mixtures and use that number for their \(N_{\text{design}}\), while keeping the same asphalt binder for thin asphalt overlay (i.e., the number of gyrations in a Superpave gyratory compactor differs from one state to another according to the traffic level as shown in Table 2.12).

Table 2.12 - Optimal Number of Gyrations Used in Several States While Compaction. \((4)\)

<table>
<thead>
<tr>
<th>State</th>
<th>Number of gyrations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alabama</td>
<td>60 gyrations for all Superpave designs</td>
</tr>
<tr>
<td>Georgia</td>
<td>50 gyrations for 4.75 mm mixes on lower volume roadways</td>
</tr>
<tr>
<td>Maryland</td>
<td>50 gyrations for 4.75 mm mixes on lower volume roadways and 65 gyrations for higher volume roads</td>
</tr>
<tr>
<td>New York</td>
<td>75 gyrations for 6.3 mm mixes</td>
</tr>
<tr>
<td>Utah</td>
<td>50 gyrations on lower volume roadways and 125 gyrations for higher volume roads</td>
</tr>
</tbody>
</table>

2.7. Construction of Thin Asphalt Overlays

2.7.1. Surface Preparation

To profit from a long-term service life with thin asphalt overlays, it is essential to treat well the surface before adding any overlays. The surface treatment varies from a project to another and depends on the severity of distresses and the existing pavement conditions. One of these surface treatment techniques used is: “Milling”. The pavement milling offers an economical way of removing old bad or failed pavement to a certain depth
so new overlays will last longer. The milling process restores the surface to a specified grade free of bumps, ruts, wash boarding, and other imperfections. This texture surface will directly support traffic while waiting for the final thin asphalt laying. Milling also helps to maintain surface geometrics such as bridge clearances, and curbs and gutter structures. Furthermore, it provides a good bond between the old existing pavement and the applied new overlay. Before the thin asphalt overlay is applied, the surface of the old pavement should be well treated and prepared as follows (1, 5):

- All the dirt and silt should be cleaned and removed from the old surface pavement
- A geotextile fabric can be applied above the milled surface to increase the tensile strength (i.e. shear strength) of the overlaid surface. Thus, potential reduction in the reflective cracking in thin asphalt overlays.
- A tack coat can be used to ensure a sufficient bond between the succeeding layers of a pavement because the interface is close to the additional shearing forces caused by breaking and turning traffic. A good bond prevents delamination and ensures long-term performance and lasting ride quality to the highway user.

2.7.2. Placement and Compaction of Thin Overlay Lifts

Many agencies in the United States require the use of a materials transfer vehicle (MTV) to maintain a continuous paving operation. In some cases, the MTV may also contribute to improved road smoothness. To optimize the paver performance, steady state conditions have to be defined for the operation. The ability to adjust changes and to quickly bring the paver back to a steady state is a distinct advantage. The paver should directly deal with the environment changes: either those reactive to the environment or those being
induced changes at the operator’s discretion. The synchronization should be well controlled. The number of rollers is fixed in a way to keep up with the paver and to control a steady state operation. In general the thinner the lift, the faster the paver travel speed (12, 13, and 14). Vibratory rollers are not typically used to compact thin asphalt overlays to avoid the fracture of the aggregate particles. Reaching compaction with thin asphalt overlay is generally more difficult because of the low accuracy of measuring the density and verifying the required adequate compaction. In addition, several factors make a thin asphalt overlay more difficult to compact as boundary effect of the underlying layer and the maximum effective particle size and its influence on the concentration of the coarse aggregate. For layers thicker than 1 in. (25.4 mm), the density is usually compared to the theoretical void less density. Some agencies use nuclear or nonnuclear density gauges to evaluate roadway density. In other cases, the contractor can monitor the compaction during the construction to accept the required density.

In general, thin layers cool faster than thick layers, so the time window for compaction is reduced. In addition, if the targeted air voids is not reached during compaction, the thin asphalt layer will be less cohesive and ravel or delaminate faster. All these issues can be resolved by the correct selection of the asphalt binder grade, adjustments to the gradation not limited only to the NMAS and the grain size distribution, and the optimal compaction during construction.

2.7.3. Quality Control and Performance of Thin Asphalt Overlays

Three main stages are considered as essential for the quality control (QC) of the thin asphalt overlay mixtures (1, 5): the materials before entering the plant, the mixtures
after production, and the paving process. Aggregate gradations and moisture measurements should be checked in the plant. At each step of production, a mixture sample is checked and all its volumetric properties and characteristics are measured. The asphalt content, voids in mineral aggregates, and air voids can be tracked with time and a control chart should be developed showing warning limits and action limits. The density in the final mat is so important and especially for mats that are so thin (1 inch or less). It is often best to use density gauges on this type of pavement construction to monitor the consistency in density because it is difficult to drill and trim cores and obtain an accurate in-situ density measurement \( (1, 5) \).

2.7.4. Study Cases

2.7.4.1. Georgia

The thin asphalt overlay was effective in Georgia. Georgia DOT implemented the use of SMA surface courses. They added in their use the polymer-modified asphalt binder and fiber stabilizers to improve durability. They used a combination between SMA and OGFC to reduce hydroplaning. Modifications were done by adding some European specifications.

2.7.4.2. Louisiana

Louisiana DOT used Ultra-Thin Bonded Wearing Course (UTBWC) and special paver that apply a polymer-modified emulsion tack coat ahead of the mixture. It was applied in a higher rate so it can seal the surface and assure an adequate high bond. The UTBWC still performs well after more than 6 years. It is an alternate to mill-and-fill operations.
2.7.4.3. Minnesota

Minnesota DOT used the UTBWC with a thickness of 0.375 in. (9.5 mm.) and that use assured good conditions for also more than 6 years. Some tight reflective cracks appeared 7 years after construction.

2.7.4.4. Ohio

The Ohio DOT was using thin asphalt overlays as a maintenance and preservation for their pavement. The project selection is the essential criteria to the success of those thin overlays, because applying those overlays on pavements in very poor conditions increases the risk of failure by two to four times. The snow fall plays an important role for lower life expectancy for thin asphalt overlays (1, 5, and 7). Ohio DOT obtained 10 to 12 years of service life for their thin asphalt overlays. Ohio DOT also guarantees a big improvement in smoothness: over 16 years for the smoothness level to return to the same rate as it was in the old pavement. Saving 60% of funds designated for a rehabilitation project and saving 40% of the cost of minor rehabilitation on general system routes.

2.7.4.5. Texas

A big success resulted from using thin asphalt overlay in Texas DOT. They developed the fine dense-graded mixture, the open-graded mixture, and fine-graded SMA mixture for thin asphalt overlays. The thickness of the overlay was maintained below 1 in. Table 2.13 summarizes several examples of thin asphalt overlays and their used.
Table 2.13 - Thin Overlay Final Thicknesses Done by Several Mix Designs for Different Uses. (1)

<table>
<thead>
<tr>
<th>Type of mix design</th>
<th>thickness</th>
<th>use</th>
</tr>
</thead>
<tbody>
<tr>
<td>PFC: Porous Friction Course</td>
<td>0.75 in.</td>
<td>Stephens County to combat bleeding</td>
</tr>
<tr>
<td>Dense-Graded mix</td>
<td>0.5 in.</td>
<td>Projects near Austin</td>
</tr>
</tbody>
</table>
CHAPTER 3 EXPERIMENTAL PROGRAM AND TESTS DESCRIPTION

For each part of the state of Nevada (north and south), an asphalt mixture was designed at the University of Nevada, Reno (UNR) pavement laboratory following the NDOT Hveem mix design method (15). This chapter provides some detailed information about the materials used in this study and presents the experimental program followed to complete this research. In addition, a detailed description for each performance test, conducted for the purpose of the project completion, is provided.

3.1. Materials

3.1.1. Aggregates

Two aggregate sources were evaluated in this effort; one from northern Nevada and one from southern Nevada. The aggregates from Northern Nevada were obtained from the Lockwood pit while the aggregates from southern Nevada were obtained from the Lone Mountain pit. Figure 3.1 shows the location of both pits with respect to the map of Nevada. The Lockwood aggregates are best characterized as a complex volcanic sequence consisting of Basalt, Andesite, and Rhyolite while Lone Mountain aggregates are a combination of Limestone and Dolomite. The adopted aggregate gradations for the laboratory evaluation were selected in a way to follow the preliminary aggregate gradation distribution for thin asphalt overlays proposed by NDOT as shown in Table 3.1. The aggregate stockpiles, obtained from each source, were blended to obtain all targeted gradations. The gradations of the individual stockpiles are shown in Table 3.2 and Table 3.3 for the Lockwood and Lone Mountain sources, respectively. Figure 3.2 and Figure 3.3
illustrate aggregate gradation curve of the blend for the northern (Lockwood) and southern (Lone Mountain) mixture, respectively.

Figure 3.1 - Location of the Northern and Southern Aggregate Sources.

Table 3.1 - Preliminary Aggregate Gradation Distribution for Thin Asphalt Overlays Proposed by NDOT.

<table>
<thead>
<tr>
<th>Sieve Size (mm)</th>
<th>19</th>
<th>12.5</th>
<th>9.5</th>
<th>4.75</th>
<th>2.36</th>
<th>2.0</th>
<th>1.18</th>
<th>0.425</th>
<th>0.30</th>
<th>0.075</th>
</tr>
</thead>
<tbody>
<tr>
<td>Percent Passing (NDOT 2014)</td>
<td>100</td>
<td>90-100</td>
<td>70-90</td>
<td>50-70</td>
<td>--</td>
<td>30-50</td>
<td>--</td>
<td>12-24</td>
<td>--</td>
<td>3-8</td>
</tr>
</tbody>
</table>
### Table 3.2 - Lockwood Stockpiles Gradations.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3/4” AGG.</td>
</tr>
<tr>
<td>25.0 mm (1”)</td>
<td>100.0</td>
</tr>
<tr>
<td>19.0 mm (3/4”)</td>
<td>100.0</td>
</tr>
<tr>
<td>12.5 mm (1/2”)</td>
<td>38.2</td>
</tr>
<tr>
<td>9.5 mm (3/8”)</td>
<td>4.2</td>
</tr>
<tr>
<td>4.75 mm (No. 4)</td>
<td>0.3</td>
</tr>
<tr>
<td>2.36 mm (No. 8)</td>
<td>0.3</td>
</tr>
<tr>
<td>2.00 mm (No. 10)</td>
<td>0.3</td>
</tr>
<tr>
<td>1.18 mm (No. 16)</td>
<td>0.3</td>
</tr>
<tr>
<td>0.6 mm (No. 30)</td>
<td>0.3</td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
<td>0.2</td>
</tr>
<tr>
<td>0.3 mm (No. 50)</td>
<td>0.2</td>
</tr>
<tr>
<td>0.15 mm (No. 100)</td>
<td>0.2</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>0.2</td>
</tr>
<tr>
<td><strong>Bin percentages</strong></td>
<td><strong>8%</strong></td>
</tr>
</tbody>
</table>

### Table 3.3 - Lone Mountain Stockpiles Gradations.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1/2” AGG.</td>
</tr>
<tr>
<td>25.0 mm (1”)</td>
<td>100.0</td>
</tr>
<tr>
<td>19.0 mm (3/4”)</td>
<td>100.0</td>
</tr>
<tr>
<td>12.5 mm (1/2”)</td>
<td>100.0</td>
</tr>
<tr>
<td>9.5 mm (3/8”)</td>
<td>74.6</td>
</tr>
<tr>
<td>4.75 mm (No. 4)</td>
<td>4.6</td>
</tr>
<tr>
<td>2.36 mm (No. 8)</td>
<td>1.9</td>
</tr>
<tr>
<td>2.00 mm (No. 10)</td>
<td>2.3</td>
</tr>
<tr>
<td>1.18 mm (No. 16)</td>
<td>1.5</td>
</tr>
<tr>
<td>0.6 mm (No. 30)</td>
<td>1.1</td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
<td>0.9</td>
</tr>
<tr>
<td>0.3 mm (No. 50)</td>
<td>0.9</td>
</tr>
<tr>
<td>0.15 mm (No. 100)</td>
<td>1.2</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>1.0</td>
</tr>
<tr>
<td><strong>Bin Percentages</strong></td>
<td><strong>47%</strong></td>
</tr>
</tbody>
</table>
Figure 3.2 - Thin Hot-Mix Asphalt Overlay Aggregate Gradation for the Northern Lockwood Mixture (L6428)

Figure 3.3 - Thin Hot-Mix Asphalt Overlay Aggregate Gradation for the Southern Lone Mountain Mixture (LM7622)
3.1.2. Asphalt Binders

Typical PG64-28NV and PG76-22NV polymer-modified asphalt binders, supplied by Paramount Petroleum Company and Ergon Asphalt Products, were used with the northern and southern aggregate sources, respectively. The Superpave Performance Grading (PG) binder system (AASHTO M320 (16)) was used to verify the grades of the asphalt binders. The “NV” extension indicates that the asphalt binders have been graded with the PG-plus system (Nevada, Materials: Re. Section 703 – Bituminous materials) (17) which includes the Superpave PG binder system plus the following properties:

- For PG64-28NV: ductility at 39°F (4°C) for original and rolling thin film oven (RTFO) binder, toughness and tenacity at 77°F (25°C) for original binder.

- For PG76-22NV: ductility at 39°F (4°C) for original binder, non-recoverable creep compliance and creep recovery for RTFO binder.

Table 3.4 and Table 3.5 show that both asphalt binders (i.e., PG64-28NV and PG76-22NV) met the NDOT PG-plus specifications and requirements.
# Table 3.4 - Properties of the PG64-28NV Binder.

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Results</th>
<th>Specifications</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Test on Original Binder</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flash Point, °C</td>
<td>298</td>
<td>230 min.</td>
<td>AASHTO T48</td>
</tr>
<tr>
<td>Viscosity at 135°C, Pa.s</td>
<td>0.715</td>
<td>3.00 max.</td>
<td>AASHTO T316</td>
</tr>
<tr>
<td>Dynamic Shear, G*/sinδ, Test Temp 64°C at 10 rad/s, kPa</td>
<td>1.63</td>
<td>1.00 min.</td>
<td>AASHTO T315</td>
</tr>
<tr>
<td>Ductility at 4°C, 5 cm/min, cm</td>
<td>77</td>
<td>50 min.</td>
<td>Nev. T746</td>
</tr>
<tr>
<td>Toughness at 25°C, Inch.lb</td>
<td>137.6</td>
<td>110 min.</td>
<td>Nev. T745</td>
</tr>
<tr>
<td>Tenacity at 25°C, Inch.lb</td>
<td>122.2</td>
<td>75 min.</td>
<td>Nev. T745</td>
</tr>
<tr>
<td>Solubility, %</td>
<td>NT</td>
<td>99.0 min.</td>
<td>AASHTO T44</td>
</tr>
<tr>
<td>Sieve, Particulates Retained</td>
<td>NT</td>
<td>0</td>
<td>Nev. T730</td>
</tr>
<tr>
<td><strong>Tests on Rolling Thin Film Oven (RTFO) Residue, AASHTO T240</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mass Loss, %</td>
<td>0.46</td>
<td>1.00 max.</td>
<td>AASHTO T240</td>
</tr>
<tr>
<td>Dynamic Shear, G*/sinδ, Test Temp 64°C at 10 rad/s, kPa</td>
<td>3.43</td>
<td>2.20 min.</td>
<td>AASHTO T315</td>
</tr>
<tr>
<td>Ductility at 4°C, 5 cm/min, cm</td>
<td>45.75</td>
<td>25 min.</td>
<td>Nev. T746</td>
</tr>
<tr>
<td><strong>Tests on Residue from Pressure Aging Vessel Residue, AASHTO R28 @ 100 °C</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear, G*.sinδ, Test Temp 22°C at 10 rad/s, kPa</td>
<td>1674</td>
<td>5000 max.</td>
<td>AASHTO T315</td>
</tr>
<tr>
<td>Creep Stiffness, S, Test Temp -18°C at 60 s, MPa</td>
<td>140</td>
<td>300 max.</td>
<td>AASHTO T313</td>
</tr>
<tr>
<td>Creep Stiffness, m-Value, Test Temp -18°C at 60 sec</td>
<td>0.330</td>
<td>0.300 min.</td>
<td>AASHTO T313</td>
</tr>
</tbody>
</table>
Table 3.5 - Properties of the PG76-22NV Binder.

<table>
<thead>
<tr>
<th>Property</th>
<th>Test Results</th>
<th>Specifications</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Test on Original Binder</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flash Point, °C</td>
<td>304</td>
<td>230 min.</td>
<td>AASHTO T48</td>
</tr>
<tr>
<td>Viscosity at 135°C, Pa.s</td>
<td>1.540</td>
<td>3.00 max.</td>
<td>AASHTO T316</td>
</tr>
<tr>
<td>Dynamic Shear, ( G^*/\sin\delta ), Test Temp 76°C at 10 rad/s, kPa</td>
<td>1.49</td>
<td>1.30 min.</td>
<td>AASHTO T315</td>
</tr>
<tr>
<td>Ductility at 4°C, 5 cm/min, cm</td>
<td>36</td>
<td>20 min.</td>
<td>Nev. T746</td>
</tr>
<tr>
<td>Solubility, %</td>
<td>NT</td>
<td>99.0 min.</td>
<td>AASHTO T44</td>
</tr>
<tr>
<td>Sieve, Particulates Retained</td>
<td>NT</td>
<td>0</td>
<td>Nev. T730</td>
</tr>
<tr>
<td>Polymer Content, % by mass</td>
<td>NT</td>
<td>3.0 min</td>
<td>AASHTO T302</td>
</tr>
<tr>
<td><strong>Tests on Rolling Thin Film Oven (RTFO) Residue, AASHTO T240</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mass Loss, %</td>
<td>0.06</td>
<td>1.00 max.</td>
<td>AASHTO T240</td>
</tr>
<tr>
<td>Creep Recovery, ( R_{3.2} ), Test Temp 76°C at 3.2 kPa, %</td>
<td>60.9</td>
<td>30.0 min.</td>
<td>AASHTO T350</td>
</tr>
<tr>
<td>Non-Recoverable Creep Compliance, J_{nr 3.2}, Test Temp at 76°C at 3.2 kPa, kPa(^{-1})</td>
<td>0.81</td>
<td>2.0 max.</td>
<td>AASHTO T350</td>
</tr>
<tr>
<td>Non-Recoverable Creep Compliance Difference, J_{nr diff}, %</td>
<td>88.01</td>
<td>--</td>
<td>AASHTO T350</td>
</tr>
<tr>
<td><strong>Tests on Residue from Pressure Aging Vessel Residue, AASHTO R28 @ 110 °C</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dynamic Shear, ( G^*/\sin\delta ), Test Temp 31°C at 10 rad/s, kPa</td>
<td>925</td>
<td>5000 max.</td>
<td>AASHTO T315</td>
</tr>
<tr>
<td>Creep Stiffness, S, Test Temp -12°C at 60 s, MPa</td>
<td>60</td>
<td>300 max.</td>
<td>AASHTO T313</td>
</tr>
<tr>
<td>Creep Stiffness, m-value, Test Temp -12°C at 60 sec</td>
<td>0.346</td>
<td>0.300 min.</td>
<td>AASHTO T313</td>
</tr>
</tbody>
</table>

3.1.3. Hydrated Lime

All the aggregates used for mixtures designed for the state of Nevada are required to be lime-treated. Lime, at a rate of 1.5% by dry weight of aggregate (DWA), was added to the mixtures in the form of dry hydrated lime on wet aggregate (3% moisture above the saturated surface dry condition) in accordance with NDOT specifications (18). The dried aggregates were first mixed with water for two minutes, and then dry hydrated lime was added and remixed with moistened wet aggregates for three additional minutes. The lime-treated aggregates were then marinated in a sealed plastic container for 48 hours prior to
their use in the mixing process. After marination, the samples were dried at 230°F for overnight and then mixed with the asphalt binder following the correspondent mixing procedure for hot mix asphalt (HMA) mixtures (15).

3.2. Experimental Program

As mentioned before, two aggregate sources were used in this study: Lockwood from the north and Lone Mountain from the south. Typical polymer-modified asphalt binders (i.e., PG64-28NV and PG76-22NV) were used with each aggregate source from common suppliers in the north and the south. Once the stockpiles were blended and the targeted aggregate gradations were obtained, the aggregates were lime-treated following the marination process mandated by NDOT (18). The Hveem mix design method was adopted to establish two mix designs, one for the northern part and one for the southern part of the state of Nevada. Based on NDOT volumetric requirements (18), an optimal asphalt binder content was selected for each of the developed mixtures. The optimal asphalt binder content was varied with respect to the allowable tolerances to simulate the variations in asphalt binder content during plant production (18).

The performance of the designed mixtures was evaluated in terms of their resistance to moisture damage by determining the tensile strength ratio (TSR), resistance to surface raveling and abrasion, mechanical dynamic modulus property (|E*|), resistance to rutting by conducting the confined and unconfined flow number (FN) test, resistance to reflective cracking using the Texas overlay advanced jig, the workability of the asphalt mixtures using the pressure distribution analyzer (PDA) incorporated in the updated Superpave gyratory compacter (SGC), and the developed interlayer bond strength using the Louisiana
Interlayer Shear Strength Tester (LISST). Table 3.6 presents the experimental program adopted for this study. All testing, except for the bond test, were performed at three asphalt binder contents for each mixture: selected optimal asphalt binder content, selected optimal asphalt binder content plus allowable tolerance, and selected optimal asphalt binder content minus allowable tolerance. The purpose of the evaluation at different binder contents is to ensure that the mixtures’ properties are still acceptable if any changes in the asphalt binder contents occurred during plant production within the allowable tolerances.

Table 3.6 - Summary of the Experimental Testing Program.

<table>
<thead>
<tr>
<th>Property</th>
<th>Test method</th>
<th>Number of Replicates</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile Strength ratio test (considered part of each mix design)</td>
<td>AASHTO T283 / Nev. T341D</td>
<td>8&lt;sup&gt;b&lt;/sup&gt;</td>
<td>8&lt;sup&gt;b&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>Resistance to Surface Raveling (percent of mass loss):</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Unconditioned samples</td>
<td></td>
<td>2&lt;sup&gt;a&lt;/sup&gt;</td>
<td>2&lt;sup&gt;a&lt;/sup&gt;</td>
<td>2&lt;sup&gt;a&lt;/sup&gt;</td>
</tr>
<tr>
<td>• Moisture-conditioned samples, 3 Freeze-Thaw (F-T) cycles (AASHTO T283)</td>
<td>ASTMD7196</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance to Surface Abrasion (percent of mass Loss)</td>
<td>Tex-245-F</td>
<td>2&lt;sup&gt;a&lt;/sup&gt;</td>
<td>2&lt;sup&gt;a&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>Dynamic Modulus Master Curve</td>
<td>AASHTO TP79 / PP61</td>
<td>3&lt;sup&gt;a&lt;/sup&gt;</td>
<td>3&lt;sup&gt;a&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>Resistance to Reflective Cracking</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Test temperature of 50°F (10°C) and maximum displacement opening of 0.018” (0.4572 mm)</td>
<td>Tex-248-F</td>
<td>3&lt;sup&gt;a&lt;/sup&gt;</td>
<td>3&lt;sup&gt;a&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>Resistance to Rutting:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Unconfined Flow Number (FN) at one temperature</td>
<td>AASHTO TP79</td>
<td>3&lt;sup&gt;a&lt;/sup&gt;</td>
<td>3&lt;sup&gt;a&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>• Confined Flow Number (CFN) at three temperatures</td>
<td></td>
<td>6&lt;sup&gt;a&lt;/sup&gt;</td>
<td>6&lt;sup&gt;a&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>Workability and Compactability</td>
<td>NA</td>
<td>2&lt;sup&gt;a&lt;/sup&gt;</td>
<td>2&lt;sup&gt;a&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>Interlayer Bond Strength</td>
<td>Draft AASHTO</td>
<td>6&lt;sup&gt;b&lt;/sup&gt;</td>
<td>6&lt;sup&gt;b&lt;/sup&gt;</td>
<td></td>
</tr>
</tbody>
</table>

Notes: - <sup>a</sup> denotes test conducted at optimal asphalt binder content and optimal asphalt binder content ± selected allowable tolerance.  
- <sup>b</sup> denotes test conducted at only optimal asphalt binder content.
All mixtures were evaluated at the short-term aging conditions following the Superpave recommendations because raveling, abrasion, reflective cracking, shoving and delamination are considered to be short-term distresses. The loose mixtures were subjected to 275°F (135°C) in a forced-draft laboratory oven for four hours prior to compaction in accordance with AASHTO R30 (19). In the case of the bond test, the conditioning duration was reduced to two hours in accordance with the LISST draft AASHTO procedure.

3.3. Description of Laboratory Test

This section provides a brief description of the various performance tests conducted for the thin asphalt overlay mixtures. These tests were selected to check the mixtures’ performance against the most prominent distresses for thin asphalt overlays identified in Chapter 1.

3.3.1. Raveling Test

The raveling test is conducted on thin asphalt overlay mixtures to simulate the surface raveling and abrasion under traffic and weather damage. Initially, this test is typically used to measure the resistance to raveling characteristics of emulsified asphalt mixed with field aggregates, recycled asphalt pavement (RAP) mixtures, cold-in-place recycled (CIR), and cold mix asphalt (CMA) mixtures. The small thickness of the overlay and, in particular, the dry mixture usually used in the southern part of the state make the thin asphalt overlay mixture potentially more susceptible to nearly surface raveling. According to the ASTM D7196 “Standard Test Method for Raveling Test of Cold Mixed Emulsified Asphalt Samples” (20), two methods can be followed for the test specimen preparation: method (A) for the field blended cold bituminous paving mixture, and method
(B) for the laboratory blended mixtures. Referring to method B, a quantity of 6.0 lb. (2750 g) of asphalt mixture is placed, at the compaction temperature, in the 6.0 in. (150 mm) Superpave gyratory mold and compacted for 20 gyrations to reach a height of 2.8 ± 0.2 in. (70 ± 5 mm). If the targeted height is not reached within 20 successive gyrations, the initial weight shall be adjusted. The sample is then directly removed from the compaction mold and allowed to cure at the ambient temperature of 65 to 75°F (18 to 24°C). The test specimens are then placed on a raveling test adapter and allowed to abrade for 15 minutes at the room temperature. Figure 3.4 shows the raveling test setup along with a thin hot-mix asphalt overlay specimen.

![Figure 3.4 - Installed Raveling Test Adapter with a Specimen Ready for Testing.](image)
The resistance to raveling is evaluated in terms of mass loss of the specimen at the end of the test. Initially, the specimen is weighted after the curing and just prior to the testing. This mass shall be recorded as the specimen mass prior to test. Once the test is done, the sample is well cleaned from fines with a smooth brush and weighted again. This mass shall be recorded as the specimen mass after the test. The percent of difference between these masses are considered to be the percent in mass loss as shown in Equation 3.1.

(Note: the percent of mass loss is reported to the nearest 0.1% as an average of at least two tested replicates).

\[
\% \text{Mass Loss} = 100 \times \left(\frac{A-B}{A}\right)
\]  

(3.1)

Where:

\(A\): specimen mass after curing and just prior to test

\(B\): specimen mass abraded at the end of the test

For this study, two sets of samples were prepared. The first set of specimens consisted of testing the compacted thin hot-mix asphalt overlay mixtures, once cooled down to the room temperature, without subjecting them to any additional conditioning. The second set of the thin hot-mix asphalt overlay specimens were evaluated after moisture conditioning which consisted of subjecting the compacted mixtures to three freeze-thaw (3 F-T) cycles following the conditioning procedure outlined in AASHTO T283 (21) for one freeze-thaw cycle. Moisture conditioning was done to evaluate and assess the
susceptibility of the evaluated mixtures to the combined effect of moisture and raveling damage.

3.3.2. Cantabro Loss Test

The deterioration of the asphalt surface occurs due to various forms of wear such as erosion, raveling, abrasion and various exposures. Another desired property to be evaluated for thin hot-mix asphalt overlay mixtures is its ability to resist surface abrasion. Abrasion wear occurs due to rubbing, scraping, skidding, or sliding of tires on the asphalt pavement surface. This form of wear is usually observed on the surface of pavements in general, and thin overlays in particular where friction forces are applied due to relative motions between the surfaces of the existing pavement. The resistance to surface abrasion property is highly influenced by several factors such as stiffness of the asphalt mixture, aggregate properties, surface finishing, compaction procedure, and type of topping (e.g., Open-Graded-Friction Coarse OGFC).

The Cantabro Loss test was used to assess the resistance of the mixtures to surface abrasion. The test measures the breakdown of the compacted specimens utilizing the Los Angeles Abrasion machine in accordance to Tex-245-F “CANTABRO LOSS” (22). The specimens are prepared in accordance with Tex-241-F (23). A weight of mixture between 9.25 lb. and 9.9 lb. (4200 g and 4500 g) is placed in the 6.0 in. (150 mm.) Superpave gyratory mold and compacted at the compaction temperature to a fixed height of 4.5 ± 0.2 in. (115 ± 5 mm) and a target relative density of 93 ±1 %. The specimens are then removed from the mold directly once the compaction process is done. These specimens are placed in the temperature chamber long enough to ensure a consistent temperature of 77 ± 2°F (25
±1 °C) throughout the specimen before testing. The specimens are not allowed to stay in the temperature chamber for more than 24 successive hours. The resistance to abrasion is evaluated by rolling the overlay hot-mix asphalt specimens in the Los Angeles Abrasion drum at a speed of 30 revolutions per minute, for 300 successive revolutions without including any steel ball as it is shown in Figure 3.5. Once the 300 revolutions are completed, the loose material broken of the test specimen are discarded.

![Los Angeles Abrasion Machine](image)

**Figure 3.5 - Los Angeles Abrasion Machine with a Specimen after Testing.**

The percent of loss by mass due to abrasion, called “Cantabro Loss”, constitutes an indication of the thin overlay durability and relates to the quantity and quality of asphalt binder been used. Initially, the specimen is weighted after the curing and just prior to the testing. This mass shall be recorded as the specimen mass prior to test. Once the test is done, the sample is well cleaned from fines with a smooth brush and weighted again. This
mass shall be recorded as the specimen after the test. The percent of difference between these masses are considered to be the percent in mass loss as shown in Equation 3.2. (Note: the percent of mass loss is reported to the nearest tenth as an average of at least two tested replicates)

\[ CL = 100 \times \left( \frac{A - B}{A} \right) \]  \hspace{1cm} (3.2)

Where:

- \( CL \): Cantabro Loss: percent of loss by mass due to abrasion
- \( A \): specimen mass after curing and just prior to test
- \( B \): specimen mass at the end of the 300 revolutions

3.3.3. Complex Dynamic Modulus Testing

3.3.3.1. Dynamic Modulus |\( E^* \)|

The AASHTO Pavement Mechanistic-Empirical (ME) software uses the dynamic modulus |\( E^* \)| master curve to evaluate the structural response of the hot-mix asphalt pavement under various combinations of traffic loads, speeds, and environmental conditions. To be able to achieve and complete these simulations, the |\( E^* \)| property of a hot-mix asphalt mix is evaluated under various combinations of loading frequencies and temperatures. AASHTO TP79 “Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Asphalt Mixtures Using the Asphalt Mixture Performance Tester (AMPT)” (24) and AASHTO PP61 “Developing Dynamic Modulus Master Curves for Asphalt Mixtures Using the Asphalt Mixture Performance Tester (AMPT)” (25) are
followed for the specimens preparation, for conducting the test and for developing the dynamic modulus master curves. The test was conducted using the Asphalt Mixture Performance Tester (AMPT) at frequencies of: 10, 1, and 0.1 (Hz) (the 0.01 Hertz (Hz) is additionally selected only for the highest temperature) and at temperatures of 39.2 °F, 68 °F, and 104 °F or 113°F (4°C, 20°C, and 40°C or 45°C) according to the asphalt binder used. The 104 °F (40°C) was considered for the northern mixture while 113 °F (45°C) was adopted for the southern mixture.

All the mixtures’ dynamic modulus were evaluated at the short-term aging condition where the loose mixtures were subjected to 275°F (135°C) in a forced-draft laboratory oven for four hours prior to compaction in accordance with AASHTO R30 (19). The $|E^*|$ test specimen consisted of a 4.0 in. (100 mm) diameter by 6.0 in. (150 mm) height cylinder that is cored from the center of a Superpave gyratory compacted (SGC) sample of 6.0 in. (150 mm) diameter by 7.0 in. (175 mm) height. All test specimens were compacted to 7.0 ± 0.5% air voids. The dynamic modulus set up and a specimen ready for testing are shown in Figure 3.6.
The dynamic modulus master curve is developed based on the volumetric properties of the tested sample (optimal binder content by total weight of mix, bulk specific gravity of the aggregates used, air voids, void in mineral aggregates, etc.) and the measured stress-strain relationship at all combinations of testing temperatures and frequencies. The time-temperature superposition is used to mainly construct the master curve. The data at various temperatures are shifted with respect to time until the curves merge into a single smooth function at a single temperature known as “reference temperature”. The time-temperature superposition is only applicable within the linear viscoelastic region on thermorheologically simple materials such as bituminous materials. Equation 3.3 shows a mathematical expression for the $|E^*|$ modulus. The “$*$” that affects the E notation is a
complex domain indicator, and explains the time dependency of the determined dynamic modulus.

\[
|E^*| = \frac{\sigma}{\xi} = \frac{\sigma_0 e^{j\omega t}}{\xi_0 e^{j(\omega t - \phi)}} = \frac{\sigma_0 \sin \omega t}{\xi_0 \sin(\omega t - \phi)} = E' + iE''
\] (3.3)

Where:

- \( |E^*| \): dynamic modulus, ksi or kPa
- \( E' \): elastic component of the dynamic modulus \( E^* \)
- \( E'' \): viscous component of the dynamic modulus \( E^* \)
- \( \sigma_0 \): peak amplitude stress
- \( \xi_0 \): peak amplitude strain
- \( \phi \): phase lag or phase angle
- \( \omega \): angular velocity
- \( t \): time
- \( i = \sqrt{-1} \)

As mentioned before, the \( E' \) and \( E'' \) values constitute the elastic and viscous component of the complex dynamic modulus \( (E^*) \), respectively. The loss tangent (\( \tan \phi \)) is defined as the ratio of the loss energy to the energy stored in a cycle deformation and defined as follows:

\[
\tan \delta = \frac{E''}{E'}
\] (3.4)
The absolute value of the complex modulus is defined as the maximum dynamic stress divided by the amplitude of the recoverable axial strain as following:

\[ |E^*| = \frac{\sigma_0}{\xi_0} \]  \hspace{1cm} (3.5)

The general form of the dynamic modulus master curve equation is shown in a sigmoidal model as follows:

\[ \log|E^*| = \delta + \frac{E_{max} - \delta}{1 + e^{\beta + \gamma \log f_r}} \]  \hspace{1cm} (3.6)

Where:

- \( |E^*| \): dynamic modulus, ksi or kPa
- \( \delta, \beta, \gamma \): fitting parameters
- \( f_r \): reduced frequency, Hz
- \( E_{max} \): Maximum value of the dynamic modulus, ksi or kPa

The shift factors at each temperature are calculated using the Arrhenius model (Equation 3.7).

\[ \log[\alpha(T)] = \frac{\Delta E_a}{R*\ln10} \left( \frac{1}{T} - \frac{1}{T_r} \right) \]  \hspace{1cm} (3.7)

Where:

- \( \alpha(T) \): the shift factor at temperature \( T \)
- \( \Delta E_a \): the activation energy, \( R*\ln10 = 19.14714 \)
The test temperature expressed in degree Kelvin, °K

$T_r$: Reference temperature expressed in degree Kelvin, °K

The numerical optimization is performed using the solver function in Microsoft Excel and all the data at different temperatures are shifted to a reference common temperature. The sum of the squared errors (SSE) between the average measured dynamic moduli values for each temperature and frequency and the values predicted using the mathematical model is minimized. A general statistical fit is performed based on the collected data as a final step. The standard error of the estimated value is calculated, as well as the explained variance.

3.3.3.2. Phase Angle δ

The $|E^*|$ master curve, explained and developed before, constitutes an effective way to predict the asphalt mixture $|E^*|$ beyond the testing conditions. In addition, the mechanical behavior of the asphalt mixtures is highly influenced by another parameter: the phase angle. It affects the distribution of the storage and loss moduli values known respectively as elastic and viscous components of the dynamic modulus $|E^*|$. Several approaches have been developed in order to estimate the phase angles at different temperatures and different frequencies. This section presents three of them. The main purpose of getting the phase angle master curve was to evaluate the viscous and elastic behaviors of both evaluated mixtures at different asphalt binder contents. For the rest of the study, the non-linear least-square regression model was only adopted for being the simplest and easiest method.
3.3.3.2.1. Phase Angle Master Curve Using Nonlinear Least-Squares Regression Model

An indirect approach is usually used to construct the phase angle master curve. The shift factors are obtained at each temperature from the $|E^*|$ data results. The initial sigmoidal equation is adopted in this case to predict the dynamic modulus and construct the master curve at a reference temperature (Fonseca and Witczak 1996) as shown in Equation 3.8 (26).

$$\log |E^*| = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \log T_r}}$$

(3.8)

Where:

$|E^*|$: dynamic modulus, ksi or kPa

$\delta$, $\alpha$, $\beta$, and $\gamma$: fitting parameters ($\delta$ and $\delta + \alpha$ are the minimum and maximum values of $|E^*|$, respectively)

$T_r$: reduced time of loading at reference temperature, seconds

The same shift factor model (i.e., Arrhenius) is used in the construction of the phase angle master curve. A mathematical model of the same form as the mathematical predictive model of $|E^*|$ is adopted for the prediction of the phase angle (27). Typically, the complex dynamic modulus is expressed as shown in Equation 3.9.

$$E^*(i\omega) = E'(w) + iE''(w)$$

(3.9)

Where:

$E'(w)$: real part of the complex modulus, elastic component

$E''(w)$: imaginary part of the complex modulus, viscous component
$w$: angular frequency, rad/sec

$i^2 = -1$

The relation between the real and imaginary parts of the complex modulus are given using the Kramers-Kronig equations (28).

\[
E'(w) = \frac{1}{\pi} P \int_{-\infty}^{+\infty} \frac{x_2(w^*)}{w^*-w} \, dw^*
\]  
(3.10)

\[
E''(w) = -\frac{1}{\pi} P \int_{-\infty}^{+\infty} \frac{x_1(w^*)}{w^*-w} \, dw^*
\]  
(3.11)

Where:

$P$: Cauchy principal value

$w^*$: complex angular frequency

An approximated relation between the dynamic modulus and phase angle can be expressed in Equation 3.12.

\[
\delta(w) = \pi \frac{d \log(|E^*|)}{2 \, d \log(w)}
\]  
(3.12)

By using $T_r = 1/f_r$ and $w = 2\pi f_r$ and by calculating the first derivative of the dynamic modulus with respect to the angular frequency, the modified phase angle model in terms of reduced frequency at the reference temperature is expressed in Equation 3.13.

\[
\delta(f_r) = c \frac{\alpha \gamma}{2 \left(1+e^{\beta-\gamma \log f_r}\right)^2} e^{(\beta-\gamma \log f_r)}
\]  
(3.13)
Where \( c \) is a coefficient added to modify the equation to obtain potentially more accurate predictions. This constant is also treated as a regression variable.

When both master curves for \(|E^*|\) and phase angle are to be developed, the sum square of the deviation between the predicted and measured dynamic modulus (\(|E_{p}^*|\) and \(|E_{m}^*|\)) and the deviation between the predicted and measured phase angle (\(\delta_{p}\) and \(\delta_{m}\)) are minimized as shown in Equation 3.14.

\[
\Delta^2 = \sum_{1}^{N} \left( \frac{|E_{p}^*|^2 - |E_{m}^*|^2}{|E_{m}^*|^2} + \frac{|\delta_{p}|^2 - |\delta_{m}|^2}{|\delta_{m}|^2} \right) \tag{3.14}
\]

Since the \(|E^*|\) was analyzed previously using the Pavement ME modified sigmoidal function, the purpose of using this theory targets only the phase angle shifting master curve. For these reasons, the regression target was modified to minimize the sum square of the deviation between the logarithm of the measured \(E^*\) and the predicted one (Equation 3.15) to determine the fitting parameters (\(\alpha, \beta, \delta, \gamma, \) and \(\Delta E_a\)) and then minimize the sum square of the deviation between the phase angles (measured and predicted) to estimate the fitting parameter (\(c\)) as shown in Equation 3.16.

\[
\Delta^2 = \sum_{1}^{N} (\log(E_{measured}^*) - \log(E_{predicted}^*))^2 \tag{3.15}
\]

\[
\Delta^2 = \sum_{1}^{N} \left( \frac{\delta_{measured} - \delta_{predicted}}{\delta_{measured}} \right)^2 \tag{3.16}
\]
3.3.3.2. Phase Angle Master Curve Using the Fourier Transform

By definition, the Fourier transform is a mathematical operation that decomposes a function into its corresponding constituent frequencies known as frequency spectrum. Two representation domains can be defined: time domain representation and frequency domain representation. When the waveform depends on time, it is called time domain representation and when the frequency spectrum is a function of frequency, it is called frequency domain representation. The function gives a complex number that contains information regarding the magnitude and phase of each frequency content. From a mathematical perspective, the Fourier transform of a function $h(x)$ is expressed as follows:

$$H(x) = \int_{-\infty}^{+\infty} h(x) \ast e^{-2\pi i x \zeta} dx$$  \hspace{1cm} (3.17)$$

Where

$H(x)$: Fourier transform of the function $h(x)$

$\zeta$: frequency

$i$: complex domain

If the stress signal is denoted by $g(t)$ and the strain signal by $h(t)$, the cross correlation function of the two signals is obtained by calculating the Fourier transform of the product of $g(t)$ and $h(t)$ shown in Equation 3.18 (29).

$$\rho_{gh}(\tau) = \int_{-\infty}^{+\infty} g(t) \ast h(t + \tau)dt$$  \hspace{1cm} (3.18)$$

Where
\( \rho_{gh}(\tau) \): defined as the cross correlation

\( g(t) \): stress signal

\( h(t) \): strain signal

By definition, the Cross Power Spectrum (CPS) of two signals is the Fourier Transform of the cross correlation \( \rho_{gh}(\tau) \) and is calculated using Equation 3.19.

\[
Y_{gh}(\zeta) = \int_{-\infty}^{\infty} \rho_{gh} e^{-2\pi i x \tau} dx = \frac{G(\zeta) \cdot CH(\zeta)}{N^2}
\]

(3.19)

Where

\( G(\zeta) \): Fourier Transform of the stress signal \( g(t) \)

\( CH(\zeta) \): complex conjugate of the Fourier transform of the strain signal \( h(t) \)

\( N \): sample size of the data

In general, the cross power spectrum (CPS) identifies the strength of the signal in terms of energy as function of the frequency. Because of the CPS is a complex valued function, it can express the phase angle \( \phi \) between both signals \( g(t) \) and \( h(t) \). To be able to calculate the phase angle \( \phi \), both stress and strain signals are first transformed from time domain to frequency domain using Fourier transform and \( \phi \) is calculated from the real and imaginary components of the CPS as shown in Equation 3.20.

\[
\phi = \tan^{-1}\left(\frac{\text{imag} (Y_{gh})}{\text{real} (Y_{gh})}\right)
\]

(3.20)
3.3.3.2.3. Phase Angle Master Curve Using the 2S2P1D Model

Sometimes, studying the viscoelastic behavior of an asphalt mixture in terms of the absolute value of the complex dynamic modulus (|E*|) is not enough which will require the introduction of the phase angle conception. One of the few available physical models that have the capability of providing the elastic and viscous properties separately is the 2S2P1D. The “2S2P1D” denotes a model composed from a simple combination of 2 springs, 2 parabolic and 1 dashpot as shown in Figure 3.7.

Figure 3.7 - 2S2P1D Model Representation.

The complex dynamic modulus (E*) can be represented by the 2S2P1D model, at a given reference temperature, through the following equation (30):

\[ E^*(i\omega \tau) = E_0 + \frac{E_\infty - E_0}{1 + \delta(i\omega \tau)^{-k} + (i\omega \tau)^{-h} + (i\omega \beta \tau)^{-1}} \]  

(3.21)

Where

\( w \): the pulsation (radial frequency), \( 2\pi \times \) frequency, rad/sec

\( E_0 \): Static modulus when \( w \to 0 \)

\( E_\infty \): limit of complex modulus when \( w \to \infty \)

\( h, k \): exponents such as \( 1 > h > k > 0 \)
δ: dimensionless constant

β: dimensionless constant where \( \beta = \eta \tau^{-1} (E_\infty - E_0) \); when \( w \to 0 \) the \( E^*(i\omega \eta) \to E_0 + i\omega \eta \)

τ: characteristic time, which varies only with temperature, accounts for the Time Temperature Superposition Principle:

\[ \tau(T) = a_T(T) \tau_0 \]

\( a_T(T) \): the shift factor at temperature \( T \)

\( \tau_0 = \tau(T_r) \) determined at the reference temperature \( T_r \)

In this case, the shift factor at temperature \( (T) \) can be determined using the Williams-Landel-Ferry (WLF) model for asphalt materials expressed in Equation 3.22.

\[
\log(a_T) = \frac{-c_1 (T - T_r)}{c_2 + (T - T_r)} \quad \text{(3.22)}
\]

Using the solver function available in Microsoft Excel, seven constants (\( \delta, k, h, E_\infty, E_0, \beta, \) and \( \tau_0 \)) are determined based on the collected and measured data for both mixtures at different binder content by minimizing the sum of the square error between the log of the magnitude of the measured dynamic modulus (\( \log|E^*|_{\text{measured}} \)), and the log of the estimated one by the 2S2P1D model (\( \log|E^*|_{\text{model}} \)), at \( N \) points of frequencies. Once the nine constants are determined (seven constants for initial model and two constants for shift factor model), the Cole-Cole plot is generated representing the relationship between the loss (viscous) modulus (\( E'' \)) and the storage (elastic) modulus (\( E' \)) as shown in Figure 3.8. The Cole-Cole plot describes the linear viscoelastic behavior of an asphalt mixture.
The slopes at the left and right ends of the Cole-Cole curve are the $h$ and $k$ of the parabolic elements, respectively. The unknown constant, $E_\infty$, denotes the maximum storage modulus where the loss modulus is zero at very high frequency ($w \to \infty$). The minimum value of the storage modulus represents the rigidity of the parallel configuration of the spring $E_0$ and the dashpot $\eta$, where the loss modulus is zero at very low frequency ($w \to 0$).

3.3.4. Unconfined Flow Number Test

Permanent deformation occurs at early stages of pavement life. Permanent deformation can either be in the form of rutting or shoving. Rutting is caused by progressive movement of materials under repeated loads. The rutting resistance was evaluated by measuring the flow number (FN) of the thin hot-mix asphalt overlay mixtures in accordance with AASHTO TP79 (24).
The test is conducted unconfined with a repeated deviatoric stress of 87 psi (600 kPa) and a contact stress of 4.35 psi (30 kPa). This results in a pulse load with a repeated loading and unloading cycle. Each loading cycle consisted of 0.1 second loading followed by a rest period of 0.9 second. The specimen for the flow number test is a cylinder with 4.0 in. (100 mm) diameter and 6.0 in. (150 mm) height that is cored from the center of Superpave gyratory compacted sample having 6.0 in. (150 mm) diameter and 7.0 in. (175 mm) height. The test temperature is the design high pavement temperature at 50% reliability as determine using the Long-term Pavement Performance Binder (LTPPBind) software Version 3.1. The temperature is computed at a depth of 0.80 in. (20 mm) below the pavement surface. The pavement temperatures were determined to be 129˚F (54˚C) and 147˚F (64˚C) for northern and southern Nevada, respectively.

The axial deformation after each pulse is measured and the cumulative permanent axial strain is calculated and plotted with respect to the number of loading cycles. This relationship covers three stages: primary, secondary, and tertiary as shown in Figure 3.9. The flow number is defined as the point in which tertiary flow begins. It is the cycle that sees the least change in cumulative axial strain.
The primary stage sees a rapid increase in permanent strain with a decreasing rate of plastic deformation. This is mainly due to a rearrangement of the mixture structure with an eventual concentration of stresses in the contact surface between the loading plate and the sample due to small irregularities, predominately associated with volumetric change \((31)\). Previous research has shown that densification is unlikely with pavements well compacted during construction and its contribution is only at first working stage of asphalt pavement.

The second stage sees a constant value for permanent strain increase. Lower rate of deformation during the secondary stage suggests a more stable mixture after initial densification has been achieved, and the structure of the mix has finished its relocation due to initial traffic compaction.
The tertiary stage sees high levels of permanent axial strain associated with plastic or shear deformation under no volume change \((41, 42)\). This change is reached when the specimen begins to deform significantly and individual aggregates composing the shape of the mixture are moving past each other. The point at which the tertiary stage begins is the flow number.

The Francken mathematical model is used to compute the flow number from the confined and unconfined tests. This well suited mathematical model combines both a power model which characterizes the primary and secondary stages, and an exponential model which fits the tertiary stage \((41, 42)\). A regression mathematical analysis is conducted in order to obtain the Francken model parameters shown in Equation 3.23.

\[
\xi_p(N) = A \times N^B + C \times (e^{D \times N} - 1) \tag{3.23}
\]

Where:

\(\xi_p\): permanent axial strain

\(N\): number of loading cycles

\(A, B, C, \text{and } D\): regression constants

Once these regressions constants are determined, the first derivative of the Francken model with respect to the number of loading cycles expresses the strain rate as shown in Equation 3.24.

\[
\frac{d\xi_p(N)}{dN} = (A \times B \times N^{(B-1)}) + (C \times D \times e^{D \times N}) \tag{3.24}
\]
The rate of change of the slope of permanent strain is computed at each cycle through the second derivative expression of the Francken model as shown in Equation 3.25 (41, 42). The flow number is the point at which the rate of change of the slope changes from negative to positive (inflexion point) (41, 42).

\[
\frac{d^2 \xi_p(N)}{dN^2} = (A \ast B) \ast (B - 1) \ast N^{(B-2)} + (C \ast D^2 \ast e^{D \ast N})
\]  

(3.25)

The regression constant C indicates whether tertiary flow has started or not. Finally, the Francken model shows an accurate reflection of all three stages of deformation, including the tertiary stage (41, 42).

The flow number indicates the rutting resistance of the evaluated mixtures. The rutting model for each mixture is determined using the confined flow number data results. The testing principles, the theories used as well as the assumptions adopted are shown in details in Chapter 5.

3.3.5. Texas Overlay Test

Reflective cracking is one of the primary forms of distresses in asphalt overlays of flexible and rigid pavements. It may affects ride quality and allow the penetration of water and debris into these cracks which would accelerate the deterioration of the overlay and the underlying pavement, thus leading to a reduction in pavement serviceability (32).

The Texas overlay test was used in this study to evaluate the mixtures’ resistance to reflective cracking in accordance with Tex-248-F procedure (33). The horizontal opening and closing of joints and cracks that exist underneath a new asphalt overlay are
specifically simulated using the Overlay jig. The Overlay test jig was recently designed to increase the functionality of the Asphalt Mixture Performance Tester AMPT machine by enabling it to determine the susceptibility of asphalt mixtures to fatigue and reflective cracking.

The overlay test specimen consists of a 6 in. (150 mm) long by 3 in. (75 mm) wide and 1.5 in. (37.5 mm) thick sample that is trimmed from a 6 in. (150 mm) diameter by 4.5 in. (115 mm) height Superpave gyratory sample. The trimmed test specimens are compacted to 7.0 ± 0.5% air voids. Once prepared, each sample is glued on two metallic plates, well fixed on a mounting wide plate using epoxy. Once dried, the samples glued to the plates are mounted in the jig making the setup ready to start the test. A photo of the overlay test set up and a specimen ready for testing is shown in Figure 3.10.

![Jig Installed in the AMPT Chamber with a Specimen Ready for Testing.](image)
The test is conducted in a controlled displacement mode until failure occurs at a loading rate of 1 cycle per 10 seconds with a maximum displacement of 0.025 in. (0.635 mm) and 0.018 in. (0.457 mm) at the testing temperatures of 77 ± 1°F (25 ± 0.5 °C) and 50 ± 1°F (10 ± 0.5 °C), respectively. Based on a prior research done by Hajj et al. (2), it was decided that the test would be conducted only at a temperature of 50 ± 1°F (10 ± 0.5 °C) and at a displacement of 0.018 inch (0.457 mm), as the samples are more likely to fail at lower temperature, especially for the northern part of the state, therefore a better comparison among mixtures can be established.

Each cycle consists of 5 seconds of loading and 5 seconds of unloading. The number of cycles to failure was defined as the number of cycles to reach 93% drop in initial load which is measured from the first opening cycle. If a 93% reduction in initial load is not reached within a certain specified maximum number of cycles, the test will stop automatically. The maximum number of cycles is selected based on the research needs. A minimum value of 1,200 cycles is required in accordance to NDOT. For this study, 2,500 was selected initially as a maximum number of cycles and then increased to 5,000 cycles since no failure was observed at the lower number of cycles. At the end of the test, the trimmed specimen density, starting load, final load, percent decline in load, number of cycles to failure and, the number of observed cracks are reported.

3.3.6. Workability and Compactability Test

The term workability has been used to describe several properties related to the ease with which an asphalt mixture can be placed, worked by hand, and compacted. Satisfactory workability is important in obtaining the desired asphalt mixture smoothness and density
within a compacted pavement. It is more difficult to construct smooth pavements with mixtures having a low workability. Pavements that are under-compacted may experience significant performance problems due to high voids. If not properly compacted, the potential for permeability problems, as well as the rate of oxidative aging of the binder, increase considerably thereby reducing pavement life.

The Gyratory Pressure Distribution Analyzer (GPDA) is utilized in this study to investigate the compaction and the workability aspects of the evaluated thin hot-mix asphalt overlay mixtures. The GPDA was newly developed and incorporated to increase the functionality of the Superpave Gyratory Compaction (SGC) machine by enabling it to measure the pressure, moment, resistive effort and shear in addition to the height, density and angle of the sample at each gyration during the compaction process.

Once mixed, the samples are conditioned for two hours at the compaction temperature according to AASHTO R30 (19) and then compacted to the maximum number of gyration (function of the expected traffic) using the Superpave gyratory compaction machine according to AASHTO R35 (34). A hot-mix asphalt type 2C mix design, recently done for each part of the state delivered for Nevada department of transportation, was used as a reference mixture (35); therefore a better comparison among the workability and compactability of the mixtures can be established.

The primary objective of this section is to estimate the number of gyrations required to provide an optimum aggregate interlock, in other terms sufficient workability and best compactability. The data generated from the SGC are generally used to compute the volumetric properties such as density or air voids contents function of the number of the applied gyrations. The densification curves represent the density ($G_{mm}$) of the evaluated
mixture sample at each applied gyration. These curves are used to evaluate the tested mixture resistance to compaction energy applied by the compaction machine. Previous studies (done by Vavrik, et al.2000), suggest evaluating the mixtures’ compaction characteristics based on the locking point. The locking point describes the point at which the mixture exhibits a marked increase in resistance to densification (36). Three approaches were used in this study to determine the locking point:

- Approach 1: according to Alabama DOT, the locking point corresponds to the number of gyration at which the sample being gyrated loses less than 0.04 in. (0.1 mm) in height between two successive gyrations.

- Approach 2: according to Georgia DOT, the locking point is defined as the number of gyrations at which the same height of the gyrated sample is recorded for the third time over three successive gyrations.

- Approach 3: the compaction characteristics of the evaluated mixtures were analyzed using data from the traditional Superpave Gyratory Compactor (SGC) results. The SGC locking point (approach 3) corresponds to the number of gyrations at which the sample will show a rate of change equal to or less than 0.05 mm for three consecutive gyrations.

- Approach 4: the compaction characteristics of the evaluated mixtures were analyzed using data from the PDA recently incorporated in the Superpave Gyratory Compactor (SGC) machine. The PDA locking point (approach 4) corresponds to the number of gyrations at which the rate of change in the frictional resistance is less than 0.01.
The PDA is a simple accessory that measures the force applied to the mixture using three load-cells equally spaced at angle of 120° as shown in Figure 3.12.

![Figure 3.12 - Analysis of Forces at the Base of the Pressure Distribution Analyzer.](image)

The variation of forces during each gyration is measured and the eccentricity of the resultant force is reported. Using the moment equilibrium equations, the two eccentricities $e_x$ and $e_y$ are calculated as shown in Equation 3.26 and Equation 3.27, respectively. The resultant eccentricity is then calculated using Equation 3.28.

\[
\sum M_x = 0 \implies e_y \tag{3.26}
\]
\[
\sum M_y = 0 \implies e_x \tag{3.27}
\]
\[
e = \sqrt{e_x^2 + (r_y - e_y)^2} \tag{3.28}
\]

Where
$P_1$, $P_2$ and $P_3$ are load-cell forces (c.f. Figure 3.12)

$e_x$: x component of the eccentricity

$e_y$: y component of the eccentricity

$e$: total resultant eccentricity

$r_y$: location of the plate center point with respect to the x-axis (in the case when the PDA is permanently added to the SGC machine, $r_y = 0$)

Thus, the frictional shear resistance (FR) of each evaluated mixture at its different binder contents can be calculated using Equation 3.29.

$$FR = \frac{Re}{AH} \tag{3.29}$$

Where

$FR$: the frictional resistance

$R$: Resultant force

$e$: eccentricity

$A$: cross-section area

$H$: sample height at any gyration cycle.

The locking point is defined as the gyration number at which the mixture starts exhibiting a little or no effect in further densification process. For this research, two specimen per mixture and per binder content were tested for compactability and all the data are reported and analyzed in Chapter 4.
3.3.7. Interlayer Bond Strength Test

Usually tack coats are applied on a well prepared surface before the placement of the overlay to ensure adequate bond strength between the old and the new layer. Shear failure may occur at the interface when the surface of the old pavement cannot provide enough strength to resist stresses due to traffic and environmental loading. A poor interface bond strength can result in slippage, shoving, and surface cracks. The Louisiana Interlayer Shear Strength Tester (LISST) was used to characterize the interface shear strength between the layers of the compacted specimens. The test was conducted in accordance with the draft AASHTO procedure (37) that was developed as part of the NCHRP project 9-40 report 712 (38).

The test specimen consisted of a thin hot-mix asphalt overlay mixture compacted on top of a typical dense-graded hot-mix asphalt layer. A cylindrical base specimen (i.e., dense-graded hot-mix asphalt), 6.0 in. (150 mm) in diameter and 2 in. (50 mm) in height was compacted first until it reaches approximately 4% air voids simulating the existing asphalt pavement layer. The base specimen is then allowed to cool down in the mold to the room temperature (37, 38). Three sets of samples were prepared:

- The first set of specimens consisted of placing the thin asphalt overlay mixture at the compaction temperature directly on top of the cooled base part of the specimen without applying any tack coat.

- The second set consisted of applying a slow setting (SS) asphalt emulsion (70/30) 50% diluted with water in addition at a rate of 0.09 to 0.12 gallon per square yards on top of the base dense-graded hot-mix asphalt compacted sample.
- The third set of specimens consisted of applying a High Performance Seal (HPS) at a temperature of 350°F (177°C) on the top of the base specimen and then compacting the hot-mix asphalt overlay mixture.

In all cases, the thin hot-mix asphalt overlay was compacted to 2 in. (50 mm) height and to a target air voids level of 7±1% simulating the initial in-place air voids level. Figure 3.13 below shows a typical test specimen. Final specimens were allowed to cure for 24 hours at the room temperature prior to testing.

![Figure 3.13 - LISST Jig with a Specimen before and after Testing.](image)

The specimens were loaded in the LISST frame in such a manner that the interlayer surface (i.e. interface) is located directly in the middle of the gap between the fix reaction frame and the mobile loading one. The displacement was applied continuously (without shock) at a constant displacement rate of 0.1 in/min (2.54 mm/min) until failure. The peak ultimate load was recorded. The interlayer shear strength was calculated using Equation 3.30.

(Note: The precision and bias statements for this method have not been determined.)
\[ ISS = \frac{P_{ult}}{\pi D^2} \]  

(3.30)

Where:

*ISS*: interlayer shear strength, Pa or psi

*P*<sub>ult</sub>: ultimate load applied to specimen, N or lbs.

*D*: diameter of test specimen, m or in.
CHAPTER 4 MIX DESIGNS AND TEST RESULTS

As mentioned before, raw aggregate materials, collected from Lockwood and Lone Mountain pits, and polymer-modified asphalt binders, PG64-28NV and PG76-22NV, were used for the northern and southern mixture, respectively. This chapter presents in detail the mix design developed for each part of the state. In addition, it provides the analysis of all the test results generated from the performance evaluation of the laboratory mixtures.

4.1. Mix Designs

4.1.1. Material Preparation

All the aggregates used for developing the mix designs and evaluating the performance properties were lime treated. Marination, a process mandated by NDOT (15, 18), was explained in details in a previous section (section 3.1.3). The dried batched aggregates as well as the binder used to prepare the samples, are heated at the mixing temperature for two hours prior to mixing.

4.1.2. Theoretical Maximum Specific Gravity

Defined as the specific gravity excluding air voids, the theoretical maximum specific gravity ($G_{\text{mm}}$) is a critical hot-mix asphalt characteristic used to compute the volumetric characteristics necessary for the optimum asphalt binder content selection. Based on past experiences with asphalt mixture projects done for the state of Nevada, 6.0% and 4.5% were adopted as a starting asphalt binder content value for the theoretical maximum specific gravity calculation for the northern and southern part of the state, respectively. This step is qualified as mandatory to be able to calculate the amount of asphalt absorbed by the aggregates ($V_{ba}$) and then determine the effective asphalt content
(\(P_{be}\)). The size of the sample shall conform the minimum sample required size that depends on the nominal maximum aggregate size according to AASHTO 209 \((39)\). Once the replicates were mixed, a short-term aging of two hours was required at the compaction temperature to allow time for aggregate to absorb the binder. In addition, short-term aging usually helps in minimizing the variability in volumetric properties between replicates. After aging, each loose mixture was well separated by hand and spread on a large tray until it cools down to the room temperature. The loose mixture was then moved to a container once cooled down and its dry weight was recorded directly prior to testing. The container was then filled by water and vacuumed at a pressure of 27.5 ± 2.5 mm.Hg for a duration of 15 minutes. Finally, the weight of the mixture was measured under water and recorded for the rest of calculation. The \(G_{mm}\) value is calculated using Equation 4.1.

\[
G_{mm} = \frac{A}{A-B}
\]  

(4.1)

Where:

\(A\): dry weight of mixture at the room temperature, g or lbs.

\(B\): weight of the mixture under water at a temperature of 77 ± 1 °F (25 ± 0.5 °C), g or lbs.

Once the mix design was set up for each part of the state, the \(G_{mm}\) values were re-verified at the corresponding selected optimal asphalt binder content.

4.1.3. Mixing and Compaction

The heated aggregates were mixed with various amount of asphalt binder so that at least two binder contents were above and at least two others were below the expected
optimum asphalt binder content for each mixture. The sets of selected asphalt binder contents were “5.0%; 5.5%; 6.0%; 6.5%” and “3.5%; 4.0%; 4.5%; 5.0%; 5.5%” by dry weight of aggregates (DWA) for the northern and southern mixtures, respectively. The samples, once mixed and conditioned for 15 ± 3 hours at 140°F (60°C), were compacted using the kneading compactor at a temperature of 230°F (135°C) according to MTM_T303D_NDOT Hveem mix design standard (15). The Hveem specimen has a 4.0 in. (100 mm.) diameter and 2.50 in. (64 mm) height cylinder shape. The required mass, essentially controlled function of the specimen height, was added on two phases in the Hveem mold. At each phase, a twenty inner and twenty outer rod tamps were applied to allow a uniformity of the sample while compaction and that’s to prevent segregation. Two set of tamping blows were then applied to compact the samples following the Hveem mix design method. The first set consisted of 10 to 50 successive tamping blows (usually an average of 25 is adopted) of 250 psi (1723.7 kPa) for each blow to accomplish a semi-compacted condition of the mixture so it will not be unduly disturbed when the full load is applied. The second set consisted of 150 successive tamping blows of 500 psi (3447.4 kPa) for each blow. The sample were then conditioned for at least an hour and a half at a temperature of 140°F (60°C) prior to leveling. The leveling process consisted of applying a 12566 lbf (1000 psi) 55.9 kN (6.89 MPa) leveling off load at a head speed of 0.25 in. (64 mm) per minute (15).

4.1.4. Optimum Asphalt Binder Content

The optimum asphalt binder content was selected based on the volumetric properties (percent of air voids, percent of voids in mineral aggregate, Hveem stability,
percent of voids filled with asphalt, and unit weight) computed at each binder content of the sets mentioned before.

**Percent of air voids A.V (%)**: the compaction process reduces the volume of air in the hot-mix asphalt layer. Therefore, the characteristic of concern is the volume of air within the compacted sample. It is calculated by comparing the specimen’s bulk specific gravity ($G_{mb}$) with its theoretical maximum specific gravity ($G_{mm}$) assuming that the difference is only due to air for dried mixtures. The percent of air voids is calculated as follows:

$$A.V \, (\%) = \left(1 - \frac{G_{mb}}{G_{mm}}\right) \times 100 \quad (4.2)$$

Where:

- $A.V$: percent of air voids, %
- $G_{mb}$: bulk specific gravity of the compacted specimen
- $G_{mm}$: theoretical maximum specific gravity of the tested mixture at the corresponding asphalt binder content

**Percent of voids in mineral aggregates VMA (%):** defined as the intern-granular void space between the aggregate particles of a compacted paving mixture that includes the air voids and effective asphalt binder content. It is calculated by comparing the specimen’s bulk specific gravity ($G_{mb}$) affected by the percent of aggregates ($P_s$) with the bulk specific gravity of the aggregate blend ($G_{sb}$) as mentioned in Equation 4.3.
\[ VMA = 100 - \left( \frac{G_{mb} \cdot P_s}{G_{sb}} \right) \]  

(4.3)

Where:

\( VMA \): percent of voids in mineral aggregates, %

\( G_{mb} \): bulk specific gravity of the compacted specimen

\( G_{sb} \): Bulk specific gravity of the blended aggregates

\( P_s \): percent of aggregates present in the tested mixture

**Percent of voids filled with asphalt VFA (%):** defined as the percent of VMA filled with binder asphalt and calculated using Equation 4.4.

\[ VFA = 100 \cdot \left( \frac{VMA - AV}{VMA} \right) \]  

(4.4)

Where

\( VFA \): Voids filled with asphalt binder

\( VMA \): percent of voids in mineral aggregate

\( AV \): percent of air voids

The optimum asphalt binder content was determined by identifying the highest binder content which provides 4% air voids (A.V) (a range of 3% to 6%), a minimum percent of voids in mineral aggregates (VMA) of 12% (a range of 12% to 22% selected based on the expected number of traffic Equivalent Single Axle Loads ESALs), and a minimum percent of voids filled with asphalt (VFA) of 65% (a range of 65% to 75% selected based on the expected number of ESALs). Figure 4.1 and Figure 4.2 show the
volumetric properties variation function of the asphalt binder contents mentioned in the set previously specified for each mixture. Based on these volumetric plots, the optimal asphalt binder contents were selected to be 6.3% and 4.5% by dry weight of aggregate for the northern and southern mixture, respectively. For these binder contents, the volumetric properties requirements were met.
Figure 4.1 - Plots for: (a) Percent of Air voids, (b) Percent of Voids in Mineral Aggregates, and (c) Percent of Voids Filled with Asphalts Function of the Asphalt Binder Content for the Northern Mixture.
Figure 4.2 - Plots for: (a) Percent of Air voids, (b) Percent of Voids in Mineral Aggregates, and (c) Percent of Voids Filled with Asphalts Function of the Asphalt Binder Content for the Southern Mixture.

4.1.5. Stability

The stability of the hot–mix asphalt mixture, once placed, constitutes a mandatory property that has to be verified and checked for requirement, therefore the early failure of the pavement due to instability is prevented. The Hveem stabilometer provides the key
performance prediction measurement for the Hveem mix design method mandated by NDOT (15). It measures the resistance to deformation of a compacted hot-mix asphalt sample by measuring the lateral pressure developed from applying a vertical load. It is described as a closed-system triaxial test.

The compacted samples, once compacted at a temperature of 230°F (110°C) and leveled at a temperature of 140°F (60°C), were kept in oven for conditioning at a temperature of 140°F (60°C) for a duration of 2 to 4 additional hours. The samples were taken out from the mold by a plunger and then placed in the calibrated stabilometer. An increasing load was applied at the top of each sample at a rate of 0.05 in/min (1.27 mm/min). As the load was increasing, the lateral pressure was measured by the stabilometer and was read at specific applied vertical loads such as 500 lbf (2.22 kN), 1000 lbf (4.45 kN), 2000 lbf (8.90 kN), 3000 lbf (13.34 kN), 4000 lbf (17.80 kN), 5000 lbf (22.24 kN), and 6000 lbf (26.69 kN). The vertical movement was stopped when the load reached exactly 6000 lbf (26.69 kN). Immediately, the applied vertical load was reduced to 1000 lbf (4.45 kN). The corresponding displacement, noted $D$, was then determined by determining the number of revolutions required to increase the stabilometer lateral pressure from 5 psi to 100 at a rate of two hand revolutions per second. The stability value $(S)$ was calculated using Equation 4.5. A minimum Hveem stability value of 37 is required according to NDOT 2014 silver book (18). Figure 4.3 shows the stability values determined at each binder content for each developed mixture. The stability at the optimal binder content met the required criterion for both mixtures which induces a stable and durable behavior under expected traffic.
(Note: a correction of the stability value is made if the sample height is not 2.50 in. (63.5 mm)).

\[
S = \frac{22.2}{P_h \cdot D \cdot (P_v - P_h)^{-0.222}}
\]  (4.5)

Where

\(P_v\): vertical pressure, typically 400 psi (2800 kPa) corresponding to a 5000 lb f (22.24 kN) total load applied on a 4.0 in. (100 mm) diameter Hveem sample

\(P_h\): horizontal pressure corresponding to \(P_v\) in psi (kPa)

\(D\): displacement of specimen in 0.01 in. (0.25 mm) units.

---

![Graph](attachment:image.png)

\[y = -4.0994x^2 + 45.856x - 81.41\]

\(R^2 = 0.9936\)
Figure 4.3 - Plots for Hveem Stability Values Function of the Asphalt Binder Contents for: a) the Northern Mixture and b) the Southern Mixture.

4.1.6. Moisture Sensitivity

Moisture susceptibility is a primary cause of distresses in hot-mix asphalt pavements. HMA layers should not degrade substantially from moisture penetration into the mix. A mixture is considered susceptible to moisture if the internal asphalt binder to aggregate bond weakens in the presence of water. To measure the potential for moisture damage for the HMA mixtures, moisture susceptibility testing can be performed in the laboratory. The results from the moisture susceptibility test may be used to predict the potential for long-term stripping.

Several factors can have influence on the moisture damage of the HMA mixture. In general, moisture susceptibility is increased by any factor that increases moisture content in the HMA. These factors can be summarized as follows:
- **Asphalt binder characteristics**: where the viscosity is considered a very important parameter for the binder selected;

- **Aggregate characteristics**: where the aggregate categorized as hydrophilic (attract water) are more likely to strip than aggregates that are hydrophobic (repulse water). To address this, either stripping-susceptible aggregates can be avoided, an anti-stripping asphalt binder modifier can be used or a mineral anti-stripping filler (i.e. lime) can be used;

- **Air voids**: when HMA air voids exceed about 8% by volume, they may become interconnected and allow water to easily penetrate the HMA layer and cause moisture damage through pore pressure or ice expansion. To address this, HMA mix design adjusts asphalt binder content and aggregate gradation to produce design air voids of about 3 to 6%;

- **Construction weather**: cool weather construction can lead to insufficient compaction, resulting in high air voids and a relatively permeable HMA pavement. It can also increase the moisture content in the constructed HMA;

- **Climate**: wetter climates, freeze-thaw cycles and temperature fluctuations can allow more moisture into the HMA structure;

- **Traffic**: increased traffic loading can accelerate moisture damage if water was initially present in the HMA structure. In general, moisture susceptibility tests do not measure individual factors but rather attempt to quantify a HMA mixture’s ability to resist moisture damage.
For this research, the Modified Lottman was used to evaluate the vulnerability of each developed mixture to moisture. This test compares the tensile strength of unconditioned samples to samples partially saturated with water and subjected to an accelerated conditioning.

For this test, ten samples were prepared, mixed and short-term aged for 15 ± 3 hours at a temperature of 140°F (60°C) according to AASHTO T283 and Nev. T341 (21, 40). These samples were then compacted at a temperature of 230°F (110°C) to a target air void of 8 ± 0.5%. According to AASHTO T283 (21), a 4 in. (100 mm) diameter and 2.5 in. (63.5 mm) height specimen were tested because the northern and southern gradations include aggregates whose sizes are smaller than 1 in. (25.4 mm). The ten samples were split into two sub-lots of 5 specimens each: one left un-conditioned at the room temperature and the others were conditioned. The samples in each sub-lot were selected in a way that both sub-lots have a close air void average. A water saturation that ranges between 70% and 80%, following T283 method, was done for each sample using a vacuum pressure of 90 to 500 mm.Hg. Once wrapped, these samples were kept for a minimum duration of 16 hours at a temperature of 0°F (-18°C). The samples were then placed in a 140°F (60°C) water bath for 24 hours. At the end, the samples were moved to a 77°F (25°C) water bath for a minimum duration of 2 hours which allowed a testing at the room temperature.

An indirect tension test was conducted on each sample by applying a load at a constant rate of 2 in/min (50 mm/min). The tensile strength was calculated using Equation 4.6.

\[ S_t = \frac{2 \times P}{\pi \times t \times D} \]  

(4.6)

Where:

\( S_t \) is defined as the tensile strength (TS), psi (kPa)
$P$: Peak applied load, lbs (kN)

t: sample thickness, in. (mm)

$D$: sample diameter, in. (mm)

The tensile strength ratio ($TSR$) is defined as the ratio of the tensile strength of the non-conditioned samples to the tensile strength of those conditioned. Figure 4.4 and Table 4.1 resume the experiment results for the northern and southern mixtures. Following NDOT specifications, the northern and southern mixtures should be designed with a minimum dry tensile strength ($TS$) at 77°F (25°C) of 65 psi (448 kPa) and 90 psi (620 kPa) respectively and a minimum retained Tensile Strength Ratio ($TSR$) of 70%. Based on the results generated from the experiment, both mixtures met the NDOT criterion for moisture damage indicating a good resistance to moisture stripping.

Figure 4.4 - Tensile Strength Statistical Representation (Error bars represent the mean values plus or minus 95% confidence interval)
### Table 4.1 - Moisture Damage Results Summary Table for: a) the Northern Mixture and b) Southern Mixture.

<table>
<thead>
<tr>
<th>Description</th>
<th>Non - Conditioned Set</th>
<th>Conditioned Set</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample ID</td>
<td>D1 D2 D3 D4 D5</td>
<td>W1 W2 W3 W4 W5</td>
</tr>
<tr>
<td>Diameter (in)</td>
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<td>4.00 4.00 4.00 4.00 4.00</td>
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<tr>
<td>Thickness (in)</td>
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<td>2.57 2.60 2.57 2.56 2.56</td>
</tr>
<tr>
<td>Air Void (%)</td>
<td>8.9 8.8 8.3 7.7 7.7</td>
<td>9.0 8.5 8.3 8.2 7.7</td>
</tr>
<tr>
<td>Average Air Voids (%)</td>
<td>8.3</td>
<td>8.3</td>
</tr>
<tr>
<td>Saturation (%)</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Peak Applied Load (lbs)</td>
<td>1082.8 1128.1 1129.5 1117.8 1121.2</td>
<td>1131.1 1169.8 1155.7 1167.8 1350.0</td>
</tr>
<tr>
<td>Tensile Strength T.S (psi)</td>
<td>66.37 69.69 69.58 69.49 70.53</td>
<td>70.05 71.61 71.57 72.60 83.93</td>
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<tr>
<td>Cracked/ Broken aggregate or Visual moisture damage</td>
<td>1 0 1 1</td>
<td>0 0 0 1 0</td>
</tr>
<tr>
<td>Average TS (psi)</td>
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<td>71.5</td>
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<tr>
<td>TSR ratio (%)</td>
<td>102.7</td>
<td>75.0</td>
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</table>

(a)

<table>
<thead>
<tr>
<th>Description</th>
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<th>Conditioned Set</th>
</tr>
</thead>
<tbody>
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<td>Sample ID</td>
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<td>W1 W2 W3 W4 W5</td>
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<td>2.55 2.60 2.55 2.61 2.54</td>
</tr>
<tr>
<td>Air Void (%)</td>
<td>8.8 7.3 8.4 8.3 ---</td>
<td>9.1 7.2 8.4 8.3 8.6</td>
</tr>
<tr>
<td>Average Air Voids (%)</td>
<td>8.2</td>
<td>8.3</td>
</tr>
<tr>
<td>Saturation (%)</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Peak Applied Load (lbs)</td>
<td>1427.5 1540.5 1560.8 1390.6 ---</td>
<td>1114.7 1467.4 1262.0 1480.1 1304.7</td>
</tr>
<tr>
<td>Tensile Strength T.S (psi)</td>
<td>88.56 95.31 97.37 87.07 ---</td>
<td>69.51 89.79 78.71 90.07 81.64</td>
</tr>
<tr>
<td>Cracked/ Broken aggregate or Visual moisture damage</td>
<td>0 1 0 0 ---</td>
<td>0 1 1 0 0</td>
</tr>
<tr>
<td>Average TS (psi)</td>
<td>93.25</td>
<td>85.05</td>
</tr>
<tr>
<td>TSR ratio (%)</td>
<td>91.2</td>
<td>75.0</td>
</tr>
</tbody>
</table>

(b)

4.1.7. Final Mix Designs

Figure 4.5 and Figure 4.6 summarize the mix design information for the mixtures from both sources (North and south). Each figure regroups pertinent mix design data, NDOT requirements, and information on aggregate specific gravities and gradations.
Table 4.5 - Lockwood Thin Asphalt Overlay Mix Design and Aggregate Properties.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydrated Lime, %</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Mixing Temperature, °F</td>
<td>320</td>
<td>293-310</td>
</tr>
<tr>
<td>Compaction Temperature, °F</td>
<td>305</td>
<td>295-310</td>
</tr>
<tr>
<td>Coarse Aggregate Bulk Gravity, Gb</td>
<td>2.697</td>
<td>2.85 Max</td>
</tr>
<tr>
<td>Fine Aggr. Apparent Gravity, Ga</td>
<td>2.765</td>
<td>2.85 Max</td>
</tr>
<tr>
<td>Optimum Binder (OBC), % DWA</td>
<td>6.3</td>
<td>--</td>
</tr>
<tr>
<td>Optimum Binder (OBC), % TWM</td>
<td>5.9</td>
<td>--</td>
</tr>
<tr>
<td>Air Voids in Total Mix</td>
<td>4.9</td>
<td>3.6</td>
</tr>
<tr>
<td>VMA, %</td>
<td>19.5</td>
<td>--</td>
</tr>
<tr>
<td>Hveem Stability</td>
<td>45</td>
<td>37 Min.</td>
</tr>
<tr>
<td>Max. specific gravity at OBC, Gsm</td>
<td>2.422</td>
<td>--</td>
</tr>
<tr>
<td>Unconditioned Tensile Strength, psi</td>
<td>70</td>
<td>65 Min.</td>
</tr>
<tr>
<td>Conditioned Tensile Strength, psi</td>
<td>71.5</td>
<td>--</td>
</tr>
<tr>
<td>Tensile Strength Ratio, %</td>
<td>100</td>
<td>70 Min.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Aggregate Gradation (Lockwood)</th>
<th>Sieve Size</th>
<th>% Passing</th>
<th>Control Points</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Min</td>
<td>Max</td>
<td></td>
</tr>
<tr>
<td>37.5 mm (1.5&quot;)</td>
<td>100.0</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>25.0 mm (1&quot;)</td>
<td>100.0</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>19.0 mm (3/4&quot;)</td>
<td>100.0</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>12.5 mm (1/2&quot;)</td>
<td>95.0</td>
<td>90</td>
<td>100</td>
</tr>
<tr>
<td>9.5 mm (3/8&quot;)</td>
<td>85.0</td>
<td>70</td>
<td>90</td>
</tr>
<tr>
<td>4.75 mm (No. 4)</td>
<td>55.3</td>
<td>50</td>
<td>70</td>
</tr>
<tr>
<td>2.36 mm (No. 8)</td>
<td>41.7</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>2.00 mm (No. 10)</td>
<td>39.2</td>
<td>30</td>
<td>50</td>
</tr>
<tr>
<td>1.18 mm (No. 16)</td>
<td>32.3</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>0.6 mm (No. 30)</td>
<td>24.2</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
<td>19.4</td>
<td>12</td>
<td>24</td>
</tr>
<tr>
<td>0.3 mm (No. 50)</td>
<td>13.9</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>0.15 mm (No. 100)</td>
<td>7.4</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>4.9</td>
<td>3</td>
<td>8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Aggregates</th>
<th>AGG. 1</th>
<th>AGG. 2</th>
<th>AGG. 3</th>
<th>AGG. 4</th>
<th>AGG. 5</th>
<th>AGG. 6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material Description</td>
<td>3/4&quot; AGG</td>
<td>1/2&quot; AGG</td>
<td>3/8&quot; AGG</td>
<td>Cr. Fines</td>
<td>Nat. Fines</td>
<td>--</td>
</tr>
<tr>
<td>Bin Proportions</td>
<td>8%</td>
<td>14%</td>
<td>27%</td>
<td>33%</td>
<td>18%</td>
<td>--</td>
</tr>
</tbody>
</table>

Figure 4.5 - Lockwood Thin Asphalt Overlay Mix Design and Aggregate Properties.
Figure 4.6 - Lone Mountain Thin Asphalt Overlay Mix Design and Aggregate Properties.
4.1.8. Asphalt Binder Content Variation

The selected optimum asphalt binder content for each mixture was varied with respect to the allowable tolerances mentioned in the NDOT silver book 2014 to simulate the variation in asphalt binder content during production. These tolerances were selected in a way that the final plant mix product shall comply with the approved mixture design and the project control requirements recommended by NDOT. Values of ±0.4% and ±0.3% of asphalt binder content by dry weight of aggregate were selected as limits for the allowable tolerances for the northern and southern mixture, respectively. Table 4.2 summarizes the volumetric properties of the mixtures from both sources at the optimum asphalt binder content and the optimum asphalt binder content plus and minus the selected allowable tolerance.

Table 4.2 - Summary of the Volumetric Properties of Both Mixtures at their Different Asphalt Binder Contents.

<table>
<thead>
<tr>
<th></th>
<th>Northern Nevada</th>
<th>Southern Nevada</th>
</tr>
</thead>
<tbody>
<tr>
<td>Optimal Binder Content by DWA, % (Based on Volumetric Properties)</td>
<td>6.3</td>
<td>4.5</td>
</tr>
<tr>
<td>Binder Content by DWA, %</td>
<td>5.90 6.30 6.70</td>
<td>4.20 4.50 4.80</td>
</tr>
<tr>
<td>Binder Content by TWM, %</td>
<td>5.57 5.93 6.28</td>
<td>4.03 4.31 4.58</td>
</tr>
<tr>
<td>Air Voids (AV), %</td>
<td>5.90 4.90 4.20</td>
<td>5.20 4.20 3.20</td>
</tr>
<tr>
<td>Voids in Mineral Aggregate (VMA), %</td>
<td>19.6 19.5 19.6</td>
<td>14.3 14.0 13.7</td>
</tr>
<tr>
<td>Voids Filled with Asphalt (VFA), %</td>
<td>70.0 74.8 79.0</td>
<td>63.8 70.3 76.7</td>
</tr>
<tr>
<td>Hveem Stability</td>
<td>46 45 42</td>
<td>53 50 44</td>
</tr>
<tr>
<td>Aggregate Water Absorption, %</td>
<td>2.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Bulk Specific Gravity of the aggregate blend $G_b$</td>
<td>2.622</td>
<td>2.719</td>
</tr>
<tr>
<td>Effective Specific Gravity of the blend, $G_{se}$</td>
<td>2.654</td>
<td>2.788</td>
</tr>
<tr>
<td>Bulk Specific Gravity of the Asphalt binder, $G_b$</td>
<td>1.015</td>
<td></td>
</tr>
<tr>
<td>Absorbed Binder Content by weight (DWA) ($P_{ba}$), %</td>
<td>0.467 0.467 0.467 0.924 0.924 0.924</td>
<td></td>
</tr>
<tr>
<td>Effective Binder Content by weight (TWM) ($P_{be}$), %</td>
<td>5.13 5.49 5.84 3.14 3.42 3.70</td>
<td></td>
</tr>
<tr>
<td>Effective Binder Content by volume (TWM)($V_{be}$), %</td>
<td>11.58 12.45 13.29 7.65 8.38 9.10</td>
<td></td>
</tr>
</tbody>
</table>
4.2. Test Results and Data Analysis

As mentioned before, the laboratory evaluation of the laboratory-produced mixtures included tests to evaluate the thin hot-mix asphalt overlay resistance to surface raveling and abrasion, reflective cracking, rutting, and moisture damage. In addition, the interlayer shear strength capacity is evaluated using the Louisiana Interlayer Shear Strength Tester (LISST) and the workability of the thin asphalt overlay mixtures was also evaluated using the pressure distribution analyzer (PDA).

All testing, except for the interlayer bond test, were performed at three asphalt binder contents: selected optimal asphalt binder content (OBC), optimal asphalt binder content plus allowable tolerance (OBC + tolerance), and optimal asphalt binder content minus allowable tolerance (OBC - tolerance). The purpose of the evaluation at different asphalt binder contents was to ensure that mixtures with acceptable properties are still achieved if any changes in the design asphalt binder content occurred during plant production.

All mixtures were evaluated at the short-term aging condition where loose mixtures were kept at a temperature of 275°F (135°C) in a forced-draft laboratory oven for four hours prior to compaction in accordance with AASHTO R30 (19). In the case of the bond test, the conditioning duration was reduced to two hours in accordance with the LISST draft AASHTO procedure (37). This section presents the test results and the analysis of the data generated from the various laboratory evaluations of each mixture. Various statistical tools were used including F and t-tests to compare the performance data reported at the optimal asphalt binder content and the optimal asphalt binder content plus and minus the selected tolerances for each mixture. A significance level of 5% was adopted for this
statistical analysis. The following abbreviations and nomenclatures were used for the data analysis section:

- **L6428_OBC, L6428_OBC+ and L6428_OBC-**: Northern mixture (L6428) manufactured with Lockwood aggregates and PG64-28NV asphalt binder at OBC, OBC plus 0.4%, and OBC minus 0.4%, respectively.

- **LM7622_OBC, LM7622_OBC+ and LM7622_OBC-**: Southern mixtures (LM7622) manufactured with Lone Mountain aggregates and PG76-22NV asphalt binder at OBC, OBC plus 0.3%, and OBC minus 0.3%, respectively.

4.2.1. Resistance to Surface Raveling

As mentioned before, the mass loss of the specimen after conducting the test was adopted to evaluate the resistance to raveling of the evaluated mixture. While no standard criterion has been implemented for dense-graded asphalt mixtures, a maximum percent of mass loss of 20% for un-aged specimens and 30% for aged specimens has been typically used for open-graded mixtures (41). The same criterion was conserved for the non-conditioned and moisture conditioned samples. Figure 4.7 shows some non-conditioned and moisture conditioned samples before and after being tested for the surface raveling. A review of the percent of mass loss by raveling data presented in Figure 4.8 reveals the following observations:

- Regardless of the asphalt binder content and moisture conditioning state, both evaluated mixtures exhibited an extremely low (less than 0.4%) percent of mass loss by raveling indicating a very high resistance to surface raveling.
The LM7622 mixture exhibited a higher mass loss than the L6428 mixture. This may be attributed to the lower asphalt binder content for the LM7622 mixture when compared to the L6428 mixture.

Figure 4.7 – (a) Un-Conditioned and (b) Moisture-Conditioned Samples before and After the Raveling Test.

It has been reported in previous studies, that the use of polymer-modified binders for asphalt mixtures in general can help to improve durability, resistance to cracking and especially resistance to surface raveling. The structure of a modifying polymer (i.e.
elastomer) within the asphalt means the mixture will be more elastic under traffic, less sensitive to temperature fluctuations and also helps to reduce the tendency of the pavement to ravel once it has aged. It has been reported also that the lime treatment of HMA mixtures, a mandated procedure by NDOT for all designed asphalt mixtures, can significantly improve their resistance to raveling and moisture damage.

**Figure 4.8 - Percent of Mass Loss at 77°F (25°C) by Raveling for Unconditioned and Moisture-Conditioned Samples (Error bars represent the mean values plus or minus 95% confidence interval).**

4.2.2. **Resistance to Surface Abrasion**

As mentioned before, the mass loss of the specimen after conducting the test known as “Cantabro loss” was adopted to evaluate the resistance to abrasion of the tested mixture. A good resistance to surface abrasion is considered for mixtures whose maximum percent
of Cantabro loss does not exceed 20% for un-aged samples and 30% for the aged ones (41). Figure 4.9 shows samples before and after testing for surface abrasion for both mixtures.

The following observations can be made from the plots of Figure 4.10:

- Both mixtures exhibited a Cantabro loss less than 20% for all evaluated asphalt binder contents (i.e., at OBC and OBC±selected tolerances) indicating a good resistance to surface abrasion.

- A decrease in the Cantabro loss was observed for both mixtures with the increase in asphalt binder content. However, no statistical significant difference at the 5% significance level was observed between the mixtures at higher (OBC+) and lower (OBC-) asphalt binder contents when compared to the corresponding mixture at OBC.
Statistically, significantly higher Cantabro loss values (5 to 8 times) were observed for the LM7622 mixture when compared to the L6428 mixture at all asphalt binder contents. The higher susceptibility to surface abrasion for the LM7622 mixture can be attributed to the lower asphalt binder content and the difference in aggregate sources. Historically, limestone southern aggregates have been known to exhibit high losses in tests such as LA abrasion and wear tests.

![Figure 4.10 - Percent of Mass Loss by Abrasion for Un-Conditioned Samples from Both Mixtures.](image)

4.2.3. Complex Dynamic Modulus Property

4.2.3.1. Dynamic Modulus $|E^*|$

The $E^*$ property provides an indication on the overall quality of the hot-mix asphalt mixture. The magnitude of the $E^*$ property depends on several properties of the HMA mix including aggregate properties, gradations, binder grade, volumetric and age. In addition,
the magnitude of $E^*$ also depends on temperature and rate of loading (i.e. frequency). Typically, a temperature of 68°F (20°C) has been selected to represent an intermediate effective temperature for the state of Nevada and a frequency of 10 Hz has been selected to represent standard highway loading corresponding to the 0.1 second pulse duration generated during the testing using the AMPT machine.

In general terms, an $E^*$ property above 300 ksi (2068.4 MPa) at 68°F (20°C) and 10 Hz indicates a good stable HMA mix. On the other hand, an $E^*$ property at 68°F (20°C) and 10 Hz above 1,500 ksi (10342.1 MPa) indicates a HMA susceptible to cracking (i.e. fatigue and reflective cracking) (2). In addition, a temperature of 104°F (40°C) and 113°F (45°C) have been also selected for the northern and southern mixtures respectively, representing an effective high temperature at which the mixture is susceptible to rutting. For these temperatures, the same frequency of 10 Hz has been conserved. A review of the $|E^*|$ data presented in Figure 4.11 and Figure 4.12 reveals the following observations:

- Both evaluated mixtures (northern and southern) exhibited a dynamic modulus property similar to the ones observed for the corresponding dense-graded HMA mixtures in Nevada indicating a good stability under traffic loading.
- In the case of the northern Lockwood mixture, no significant difference was observed between the $|E^*|$ property at OBC and OBC-. Slightly lower $|E^*|$ values were observed at the OBC+.
- In the case of the southern Lone Mountain mixture, no significant difference was observed in the $|E^*|$ property for different asphalt binder contents.
• The combination of aggregate source and asphalt binder grade had a significant impact on the magnitude of the |E*| property. Higher |E*| values were observed for the southern mixture when compared to the northern mixture regardless of the asphalt binder content.

However, all mixtures have E* properties at 68°F (20°C) and 10 Hz that are above 300 ksi (2068.4 MPa) and well below 1,500 ksi (10342.1 MPa) which indicates that all mixtures are expected to be stable under traffic loading without any susceptibility to either fatigue or thermal cracking.

![Figure 4.11 - |E*| Values at 10 Hz and Different Temperatures for Both Mixtures](image)

*Figure 4.11 - |E*| Values at 10 Hz and Different Temperatures for Both Mixtures (Error bars represent the mean values plus or minus 95% confidence interval).*
Figure 4.12 - Dynamic Modulus $|E^*|$ at 68°F (20°C) for Short-Term Aged: a) Northern Lockwood Mixtures and b) Southern Lone Mountain Mixtures.
4.2.3.2. Phase Angle $\delta$

The phase angle ($\delta$) provides an indication about the elastic and viscous behavior of the evaluated mixture at different temperatures and loading rates. The magnitude of the $\delta$ property depends on several properties of the HMA mixture including aggregate properties, gradations, binder grade, volumetric and age. In addition, the magnitude of $\delta$ also depends on temperature and rate of loading (i.e., frequency or time). Typically, a temperature of 68°F (20°C) has been selected to represent an intermediate effective temperature for the state of Nevada and a frequency of 10 Hz has been selected to represent standard highway loading corresponding to the 0.1 second pulse duration generated during the testing using the AMPT machine. In addition, a temperature of 104°F (40°C) and 113°F (45°C) have been also selected for the northern and southern mixtures respectively, representing an effective high temperature at which the mixture is susceptible to rutting which may affect the variation in viscous and elastic behavior for both evaluated mixtures at their different binder contents. For these temperatures, the same frequency of 10 Hz has been conserved. The following sub-section provides plots and values determined, calculated, and estimated for the phase angle ($\delta$) of each mixture (i.e., north and south) at their different asphalt binder contents using “the nonlinear least-squares regression model”.

Appendix A (Chapter 8) provides a detailed presentation of how the phase angle master curve shifting process was done using the nonlinear least-square regression model. Figure 4.13 and Figure 4.14 illustrate the regression model and the variation in phase angles at different temperatures for both evaluated mixtures at their different binder contents.
Figure 4.13 – Phase Angle Master Curves Plots at 68°F (20°C) for Northern and Southern Short-Term Aged Mixtures Using the Least-Squares Regression Model.

Figure 4.14 – Phase Angle Values at 10 Hz and Different Temperatures for Both Mixtures Using the Least-Squares Regression Model (Error bars represent the mean values plus or minus 95% confidence interval).
A review of the phase angle (δ) data reveals the following observations:

- Both evaluated mixtures (northern and southern) exhibited a phase angle property similar to the ones observed for the corresponding dense-graded HMA mixtures in Nevada indicating a normal elastic and viscous behavior and acceptable variation of the phase angle (δ) function of the loading rate and temperature.

- In the case of the northern Lockwood mixture, a slight difference was observed between the phase angle (δ) property at OBC and OBC-: the phase angle of the OBC mixture is slightly higher for the 0.0001 – 100 Hz frequency range, meanwhile it becomes slightly lower for the rest of the frequency spectrum indicating a similar elastic and viscous behavior of the OBC and OBC- northern mixtures. For the L6428NV_OBC+ mixture, significant higher phase angles and a flatter curve were observed at different frequencies.

- In the case of the southern Lone Mountain mixture, no significant difference was observed in the phase angle (δ) property for different asphalt binder contents.

- The combination of aggregate source and asphalt binder grade had a significant impact on the magnitude of the phase angle (δ) property as the one concluded for the dynamic modulus |E^*| property. Higher phase angle (δ) values were observed for the northern mixture when compared to the southern mixture regardless of the asphalt binder content.
However, all mixtures have phase angle (δ) properties at 68°F (20°C) and 10 Hz frequency lower than 45° which indicates that all mixtures are expected to be more elastic under traffic loading.

4.2.4. Flow Number Test Results

According to AASHTO TP79 (24), the thin hot-mix asphalt overlay mixture should meet the following flow number criteria at the representative test temperature based on the expected traffic level during the design period:

- No minimum FN value is required for a traffic level less than 3 million equivalent single axle loads (MESALs).
- A minimum FN value of 53 is required for a traffic level between 3 and 10 MESALs.
- A minimum FN value of 190 is required for a traffic level between 10 and 30 MESALs.
- A minimum FN value of 740 is required for a traffic level greater than 30 MESALs.

These criteria are considered to be under the same loading conditions as the one adopted for evaluating the thin asphalt overlay mixtures at their different asphalt binder contents (i.e., load duration and rest period, confinement and deviatoric stress levels). Figure 4.15 shows the flow number for the various short-term aged mixtures each one at its corresponding temperature according to the LTPP Bind software Version 3.1. (129°F (54°C) and 147°F (64°C) for the northern and southern mixtures, respectively). The following observations can be revealed form the data reported:
• Overall, the L6428 mixture met the minimum FN value of 190 for a traffic level up to 30 MESALs with the L6428_OBC being slightly lower than the criterion value. On the other hand, the LM7622 met the minimum FN value of 53 for a traffic level up to 10 MESALs.

• A slightly lower and higher FN value was observed, respectively, for the L6428 and LM7622 mixture at OBC, but not statistically significant, when compared to the corresponding mixture at OBC- and OBC+.

• The L6428 mixture exhibited higher FN values than the LM7622 mixture regardless of the asphalt binder content. However, it should be noted that the L6428 was tested at a temperature that was 18°F (10°C) lower than the test temperature of the LM7622.

Figure 4.15 -Flow Number Values for the Short-term aged Northern and Southern Mixtures (Error bars represent the mean values plus or minus 95% confidence interval).
4.2.5. *Texas Overlay Test Results*

None of the evaluated mixtures reached the 93% drop in initial load indicating an excellent resistance to reflective cracking. However, different percent of drops in initial load were observed at 2,500 and 5,000 cycles for the various evaluated mixtures at 50°F (10°C) as shown in Figure 4.16 and Figure 4.17, respectively. Overlapping of the confidence intervals implies the similarity in the measured percent of drop in initial load after 2,500 and 5,000 cycles between the different mixtures and between the different asphalt binder contents of each mixture. Based on the test results, the following observations can be made.

- Regardless of the asphalt binder content, both evaluated mixtures showed a percent of drop in initial load after 2,500 and 5,000 cycles lower than 93% at 50°F (10°C) indicating an excellent resistance to reflective cracking.

- For each of the northern and southern mixtures, the difference in the resistance to reflective cracking was not statistically different for different asphalt binder contents. In other words, the percent drops in the initial load were statistically similar at OBC, OBC- and OBC+ for each of the mixture’s source.

- A significantly lower percent of drop in initial load was observed for the L6428 mixture when compared to LM7622 mixture at all evaluated asphalt binder contents indicating a higher resistance to reflective cracking for the L6428 mixture. This observation can be attributed to the higher binder content for the northern mixture when compared to the southern mixture. However, a better comparison would be
established by comparing the mixtures at a same stiffness level. That may require the increase (i.e. for the southern mixtures) or decrease (i.e. for the northern mixture) of the testing temperature allowing for a similar $E^*$. It can be noticed that the aggregate arrangements and dispositions and the internal repartition of the air voids especially in the middle part of the trimmed sample can affect the results. In all cases, a good resistance to reflective cracking was observed for both mixtures.

![Figure 4.16 - Percent Drop in Initial Load for Short-term Aged Mixes after 2,500 Cycles (Error bars represent the mean values plus or minus 95% confidence interval).](image_url)
4.2.6. Workability and Compactability Test results

As mentioned before, four approaches were used to determine the locking point in accordance with (1) Alabama DOT method, (2) Georgia DOT method, (3) SGC results, and (4) Pressure Distributor Analyzer (PDA) results. A typical NDOT dense-graded HMA (Type 2C), previously developed for both parts of the state, was used as a reference mixture for comparison purpose.

Figure 4.18, Figure 4.19, and Figure 4.20 show the densification curves, frictional shear resistance (FR), and locking points for the thin asphalt overlay mixtures at different binder contents along with the typical NDOT HMA mix. The mixtures were compacted in the SGC at 300°F (149°C) and 320°F (160°C) for the northern and southern mixtures, respectively. While no standard criterion has been implemented for the locking point of

![Figure 4.17 - Percent Drop in Initial Load for Short-term Aged Mixes after 5,000 Cycles (Error bars represent the mean values plus or minus 95% confidence interval).](image-url)
dense-graded asphalt mixtures, this test was conducted on both mixtures at different binder asphalt contents only for the purpose of comparison. Figure 4.21 and Figure 4.22 show the variation of the locking point, determined using the four different approaches, function of the corresponding air void level for the northern and southern evaluated mixtures, respectively, at their different binder contents. The following observations can be revealed form the data reported:

- Regardless of the mixture and the asphalt binder contents, significantly higher locking point values were observed when the Georgia DOT approach was followed. This can be attributed to the stricter criterion adopted, and slightly lower locking point values were observed in the case of the PDA approach.

- Both evaluated mixtures (northern and southern), regardless of the asphalt binder content, exhibited a locking point value slightly close to the ones observed for the typical NDOT dense-graded HMA mixtures used in Nevada.

- In the case of both the northern and southern mixtures, no significant difference was observed in the locking point values at their different asphalt binder contents.

- The L6428 mixture exhibited higher locking point values than the LM7622 mixture regardless of the asphalt binder content. However, it should be noted that even though the L6428 was compacted at a temperature that was 20°F (11°C) lower than the compaction temperature of the LM7622, the target asphalt binder viscosity was the same for both evaluated mixtures.

- Higher locking point values correspond to a lower air void level requiring a higher compaction effort. It should be mentioned that the PDA locking point values correspond to air void levels which fit within the NDOT common range 3 to 6%.
The combination of aggregate source and asphalt binder grade had a significant impact on the locking point values. Higher values were observed for the northern mixture when compared to the southern mixture regardless of the asphalt binder content which can induce the need of a higher compaction effort for the northern mixtures in comparison to the southern ones.

Figure 4.18 - Densification Curves Plots function of the Number of Gyration for: a) Northern Mixtures, and b) Southern Mixtures.
Figure 4.19 - Frictional Resistance (FR) function of the Number of Gyrations for: a) Northern Mixtures, and b) Southern Mixtures.
Figure 4.20 - Locking Point Values of different Mixtures at Different Binder Contents following Different Approaches (Error bars represent the mean values plus or minus 95% confidence interval).

Figure 4.21 - Air voids Level Variation Function of the Locking Point Values Calculated Following Several Approaches for the Northern Mixture at Different Asphalt Binder Contents.
4.2.7. Bond Strength Test results

As mentioned before, the test was conducted on thin hot-mix asphalt overlay specimens that were mixed and prepared at the optimal asphalt binder contents. Figure 4.23 shows a typical specimen of each set after conduction the test. While no standard criterion has been implemented for the developed interlayer shear strength (ISS) of dense-graded asphalt pavement layers, this test was conducted on both mixtures at the optimal asphalt binder content for the purpose of comparison only. Based on the LISST test results in Figure 4.24, the following observations can be made.

- In the case of the L6428 mixture, no significant difference was observed between the ISS developed when no tack coat was used and that developed when an asphalt emulsion SS tack coat was applied. On average, a significant higher ISS value (1.9
times) was observed when the HPS tack coat was applied in comparison to the case where a tack coat was not used.

- In the case of the LM7622 mixture, significantly higher ISS values (1.6 and 2.3 times) were observed when a SS and HPS tack coat was used in comparison to the case where no tack coat was applied.

- For both L6428 and LM7622 mixtures, the highest ISS value was observed for the HPS tack coat.

- The combination of aggregate source and asphalt binder grade had a significant impact on the magnitude of the developed interlayer shear strength. Significantly higher ISS values were observed for the LM7622 mixture when compared to the L6428 mixture. This can be attributed to the higher asphalt binder content for the L6428 mixture when compared to the LM7622 mixture.

Applying the right tack coat at the right rate is very important for the thin hot-mix asphalt overlay construction. Since the overlays are developed and designed primarily for functional and not structural purposes, a full stress transmission should be verified between the new overlay and the old existing pavement so a non-failure of the overlay due to fatigue, shoving and delamination can be guaranteed.
Figure 4.23 - Samples after Testing using the LISST Jig with and without Tack Coats.

Figure 4.24 - Developed Interlayer Shear Strength function of Different Tack Coats Used (Error bars represent the mean values plus or minus 95% confidence interval).
CHAPTER 5 RUTTING MECHANISTIC ANALYSIS

Defined as a surface depression in the wheel path, rutting may occur on the top of a pavement due to mix or subgrade rutting. Mix rutting occurs when the subgrade does not rut, yet the pavement surface exhibits wheel path depressions as a result of compaction or mix design problems. Subgrade rutting occurs when the subgrade exhibits wheel path depressions due to loading. In this case, the pavement settles into the subgrade ruts causing surface depressions in the wheel path (42).

The purpose of this chapter is to provide a summary of the procedures adopted to evaluate the resistance to rutting of each mixture designed in the previous chapter, and determine the hot-mix asphalt parameter inputs used to predict rutting for the Pavement Mechanistic-Empirical (ME) software.

5.1. Confined Flow Number

5.1.1. Testing Description

The rutting model parameters were determined by conducting a confined flow number test on the thin hot-mix asphalt mixtures at their different asphalt binder contents in accordance with AASHTO TP79 (24).

The test was conducted by applying a repeated deviatoric stress of 70 psi (482 kPa), a confining pressure of 10 psi (69 kPa), and a contact stress of 3.50 psi (24 kPa). The contact stress, which represents 5% of the deviatoric stress, was applied to assure a full contact of the specimen from both sides before starting the test. The deviatoric stress results in a pulse load with a repeated loading and unloading cycle. Each loading cycle consisted of 0.1 second loading followed by a rest period of 0.9 second. The specimen for
the confined flow number test were produced with the same geometry and procedure as the one used to conduct the unconfined flow number test: a cylinder, with 4.0 in. (100 mm) diameter, 6.0 in. (150 mm) height, and 7% air voids, cored from the center of Superpave gyratory compacted sample having 6.0 in. (150 mm) diameter and 7.0 in. (175 mm) height.

The axial deformation after each pulse was measured and the cumulative permanent axial strain was calculated and plotted with respect to the number of loading cycles. This relationship also covers three stages: primary, secondary, and tertiary. This test may take more time before its completion. The sample was considered failed when the vertical strain reached a maximum value of 50,000 micro strain. If 20,000 applied cycles were not enough to let the sample fail, the test stops directly and the generated data are considered enough to predict the rutting behavior of the mixture at the corresponding temperature.

5.1.2. Testing Temperature

The hot-mix asphalt rutting is highly affected by temperature. It is well-known that the properties of viscoelastic materials change with temperature: a higher rutting is expected for a higher temperature since a higher deformation can be generated under the same loading. Being an essential component of the rutting model, the test is conducted at different temperatures.

Two test temperature options are available for conducting the confined flow number test (31): option (A) known as the multiple test temperature option, and option (B) known as the equivalent test temperature option.

The option (A), known as multiple test temperature, consists of conducting the test at three different temperatures, which are defined as: 68°F (20°C) specified as the lowest
testing temperature, high pavement temperature determined at 50% reliability using the Long-term Pavement Performance Binder LTPPBind software Version 3.1 minus 5°C (The temperature is computed at a depth of 0.80 in. (20 mm) below the pavement surface), and the middle temperature between the first two. Meanwhile, the option (B) consists of using one testing temperature defined as the equivalent temperature that will result in the same level of rutting at the end of the design period with the rutting predicted using temperatures defined for that climate and structure.

For this study, the testing temperature option (A) was adopted for the structural design of thin asphalt overlays. The testing temperatures were 68°F (20°C), 93.2°F (34°C), and 118.4°F (48°C) for the northern mixture and 68°F (20°C), 104°F (40°C), and 140°F (60°C) for the southern one. The number of test specimens is dependent on whether the multiple temperature or equivalent temperature option is selected. For this study, two specimens per mixture, per asphalt binder content, and per temperature were tested, therefore a total of six specimens (at least) were required per mixture and per binder content.

5.2. Rutting Evaluation Procedures

The following section develops two evaluation procedures used to determine the coefficient of the rutting model serving as a level 1 input for the Pavement-ME software. Both procedures use three testing temperatures mentioned in option (A) above. Equation 5.1 below expresses the plastic strain relationship included in the MEPDG to predict rut depth in the HMA layer increments called "Kaloush-Witczak vertical resilient strain transfer function".
\[ \xi_p = \xi_r \ast K_z \ast \beta_{r_1} \ast 10^{k_{r_1}} \ast (T)^{k_{r_2}} \ast \beta_{r_2} \ast (N)^{k_{r_3}} \ast \beta_{r_3} \]  \hspace{1cm} (5.1)

Where

\( \xi_p \): incremental plastic strain computed at the mid-depth of a thickness increment

\( \xi_r \): resilient strain calculated at the mid-depth of a thickness increment

\( T \): temperature at the mid-depth of a thickness increment

\( N \): number of axle load applications of a specific axle type and load interval within a specific time interval

\( \beta_{r_1}, \beta_{r_2}, \beta_{r_3} \): Local calibration coefficients which vary between a state and the other

\( k_{r_1} \): plastic deformation factor or coefficient

\( k_{r_2} \): plastic deformation factor related to the temperature

\( k_{r_3} \): plastic deformation factor related to the effect of wheel load

\( K_z \): depth function

\[ K_z = (C_1 + C_2 D)(0.328196)^D \]  \hspace{1cm} (5.2)

\[ C_1 = -0.1039H_{HMA}^2 + 2.4868H_{HMA} - 17.342 \]  \hspace{1cm} (5.3)

\[ C_2 = 0.0172H_{HMA}^2 - 1.7331H_{HMA} + 27.428 \]  \hspace{1cm} (5.4)

Where

\( D \): depth to the mid-depth of the thickness increment, in

\( H_{HMA} \): total thickness of the HMA layer

5.2.1.  \textit{NCHRP 9-30- Option (A): Multiple Temperature Option}

Once the test was conducted and the cumulative permanent deformations were computed function of the applied loading cycle, the post testing steps are then explained in
details. The rut depth transfer function represents the second stage and is defined using Equation 5.5.

\[ \xi_p = a \cdot N^b \quad \text{(5.5)} \]

where

\( a \): intercept on a logarithm scale plot

\( b \): slope on a logarithm scale plot

\( N \): load cycle term

5.2.1.1. The m-value determination

The m-value denotes the laboratory-derived slope of the steady-state or secondary region for each test specimen. A region is defined as steady-state when the slope becomes constant between the number of loading cycles and the cumulative permanent strain. This usually occurs in the secondary stage, where the behavior seems to be linear. The principle of moving a decade of loading of cycles (i.e. 1 to 10, 2 to 20; …; 10 to 100, 20 to 200; …; 100 to 1000, 200 to 2000; …) is adopted to define the average steady-state slope. This value serves as the “\( b \)” value in the rut depth transfer function. Special cases may occur while finding the m-value such as: a) the slope continues to decrease over the entire number of loading cycle: for this case the average slope should be determined for the region prior to that decrease, and b) the slope which increases at an increasing rate: for this case the test data should be excluded and the specimen should be discarded.
Once the m-value is determined for each specimen at different temperature, the laboratory-derived representative slope from all test specimens at different test specimens is determined. The methodology included in the Pavement ME software assumes that the steady-state slope is independent of the temperature. The change in temperature is only accounted for in the temperature exponent of the Kalouch transfer function and/or through the effect of temperature change on the dynamic modulus value, in another word on the laboratory intercept variation. Two cases are available: a) if the slope does not consistently change with the variation of the testing temperature, just the resultant value is the average of all the slopes, an b) if the slope is consistently changing with the test temperature variation, the representative slope is then considered only at the equivalent temperature defined before in the testing temperature option (B).

5.2.1.2. The laboratory-derived intercept

The laboratory-derived intercept \(a\) is a test-temperature dependent determined based on the laboratory data generated from the test specimen and used as the coefficient of the transfer function. The representative laboratory slope is then determined at the equivalent temperature defined in option (B).

5.2.1.3. The field-matched parameters

The field-matched parameters are determined for the HMA mixture and structure using the charts illustrated in Figure 5.1 and Figure 5.2.
Figure 5.1 - Determining the Field-Matched Slopes from Laboratory-derived Values from Repeated-Load Triaxial Tests for Kaloush’s Function.

The temperature exponent for the Kaloush transfer functions is not adjusted from the laboratory-measured values. The field-matched intercepts for each transfer function are multiplied by the thickness adjustment factors grouped in Table 5.1.

Referring to Kaloush’s transfer function, $k_{r1}$ is equal to the representative field matched intercept, $k_{r2}$ is equal to the slope of the variation curve of the laboratory-intercept
function of the corresponding temperature, and $k_{ri}$ is equal to the representative matched slope corrected for the layer thickness.

**Table 5.1 - Thickness Adjustment or Shift Factors for Determining the Intercept Value of the Transfer Functions (42).**

<table>
<thead>
<tr>
<th>HMA Mixture Application</th>
<th>HMA Layer Thickness, in.</th>
<th>Transfer Function</th>
<th>Kaloush</th>
<th>Modified Leahy</th>
<th>Westrack</th>
</tr>
</thead>
<tbody>
<tr>
<td>HMA overlays of PCC or semi-rigid pavements</td>
<td>&lt; 3.0</td>
<td>0.83</td>
<td>1.25</td>
<td>1.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3.0 to 4.0</td>
<td>0.90</td>
<td>1.10</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 4.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>HMA overlays of flexible pavements</td>
<td>&lt; 4.0</td>
<td>1.4</td>
<td>1.2</td>
<td>1.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4.0 to 6.0</td>
<td>1.2</td>
<td>1.2</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 6.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>New construction, unbound aggregate base or full depth</td>
<td>&lt; 4.0</td>
<td>1.05</td>
<td>1.2</td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4.0 to 6.0</td>
<td>1.02</td>
<td>1.1</td>
<td>1.05</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.0 to 8.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 8.0</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
</tbody>
</table>

5.2.2. **Linear-Elastic Theory**

The linear elastic theory consists of specifying all the stresses applied during the test (i.e., confining and normal stresses) and computing all the corresponding strains (i.e., total strain, permanent strain, and resilient strain) using the general Hooke’s law.

Referring to the confined flow number testing procedure, the confining stress applied is fixed equal to 10 psi (69 kPa). This pressure designates the minor principal stress in a triaxial setup ($\sigma_{2i} = \sigma_{3i} \approx 10$ psi) and it is measured and checked at each loading cycle. The deviatoric stress has also a specified value approximately equal to 70 psi (482 kPa); the major principal stress applied is calculated using Equation 5.6.

$$\sigma_{1i} = \sigma_{2i} + \sigma_{di} + \sigma_{ci} \quad (5.6)$$

Where

$\sigma_{1i}$: major principle stress computed at each loading cycle $i$
\( \sigma_{2,i} \): confining minor principle stress applied at each loading cycle \( i \)

\( \sigma_{d,i} \): deviatoric stress applied at each loading cycle \( i \)

\( \sigma_{c,i} \): contact stress applied at each loading cycle \( i \)

Referring to the Hooke’s law and the elastic theory, the total strain, at a specific testing temperature (\( T \)), were computed using Equation 5.7 using the corresponding dynamic modulus value at a specific temperature. A frequency of 10 Hz was adopted to select the \( |E^*| \) value and which represents the 0.1 second pulse duration.

\[
\xi_{t,i} = \frac{1}{E(T)} \times (\sigma_{1,i} - 2 \times \nu \times \sigma_{2,i})
\]

(5.7)

Where

\( \xi_{t,i} \): total vertical strain computed at a loading cycle \( i \)

\( E(T) \): dynamic modulus at the corresponding temperature \( T \) and frequency of 10 Hz, ksi or kPa

\( \sigma_{1,i} \): major principle stress computed at each loading cycle \( i \), ksi or kPa

\( \sigma_{2,i} \): confining minor principle stress applied at each loading cycle \( i \), ksi or kPa

The total vertical strain is the sum of the recovered strain known as resilient strain and the non-recoverable strain known as the permanent strain illustrated in Figure 5.3 and expressed in Equation 5.8.
Figure 5.3 - Strains under Repeated Loads.

\[ \xi_{t,i} = \xi_{r,i} + \xi_{ip,i} \]  \hspace{1cm} (5.8)

Where

\( \xi_{t,i} \): total vertical strain computed at a loading cycle \( i \)

\( \xi_{r,i} \): resilient vertical strain computed at a loading cycle \( i \)

\( \xi_{ip,i} \): permanent strain increment computed at a loading cycle \( i \)

The increment in the permanent strain is computed using the Flow number mathematical regression model \( FN(N_i) \) as mentioned in Equation 5.9.

\[ \xi_{ip,i} = FN(N_i) - FN(N_i - 1) \]  \hspace{1cm} (5.9)

Where

\( \xi_{ip,i} \): permanent strain increment computed at a loading cycle \( i \)
$FN(N_i)$ and $FN(N_i - 1)$: cumulative permanent strain computed at cycles $N_i$ and $N_i - 1$, respectively using the Franken model

Once the total strain and the increment in the permanent strain are computed, the resilient strain at each loading cycle can be easily determined. Once, the resilient strain, the permanent strain, the temperature, the cycle numbers were determined from the laboratory test, a multi regression mathematical analysis is performed using the data analysis feature in the Microsoft Excel to determine the coefficient $k_{r_1}$, $k_{r_2}$, and $k_{r_3}$ mentioned in Equation 5.10. Therefore, the corresponding local lab-field calibration factors (i.e., $\beta_{r_1}, \beta_{r_2}, \beta_{r_3}$) were then applied to calibrate the rutting model for the Pavement ME software as in Equation 5.1 developed before (42).

$$\xi_p = \xi_r * 10^{k_{r_1}} * (T)^{k_{r_2}} * (N)^{k_{r_3}}$$ (5.10)

5.3. Data Results and Analysis

As mentioned before, two procedures were adopted to predict the rutting model of each mixture at its different asphalt binder content. A general comparison will be generated based on the found data.

5.3.1. Rutting model with respect to NCHRP 9-30 A

Table 5.2 and Table 5.3 regroup all the parameters found and used to determine the models predicting rutting using the NCHRP 9-30 (A) option such as: laboratory and field matched representative slope, laboratory and field matched representative intercept, plastic
deformation factors relative to temperature and influence of the live traffic in the field calibration, local calibration factors used for the state of Nevada, and finally laboratory and field calibrated rutting model. Figure 5.4 through Figure 5.7 below illustrate the permanent to resilient strain ratio \((\xi_p/\xi_r)\) plots function of the number of applied loading cycles) of both mixtures at different temperatures and at their different binder contents. These rutting models were compared to the ones having the appropriate calibration factors used when designing rehabilitated flexible pavements in northern Nevada (District II) and in southern Nevada (District I). Figure 5.8 and Figure 5.9 below show the permanent deformation \((\xi_p)\) variation of the secondary stage part function of the number of applied loading cycles for the northern and southern mixture, respectively at their different asphalt binder contents.

5.3.2. Rutting model using the elastic theory

Table 5.4 below regroups all the parameters found and used to determine the models predicting rutting using the elastic theory approach. Figure 5.10, Figure 5.11, Figure 5.12 and Figure 5.13 illustrate the variation of permanent to resilient strain ratios \((\xi_p/\xi_r)\) function of the number of applied loading cycles computed using the model generated from the elastic theory approach and compare them to the models having the appropriate calibration factors used when designing rehabilitated flexible pavements in northern Nevada (District II) and in southern Nevada (District I).
Table 5.2 – Rutting Model Parameters Following the NCHRP 1-37A Approach.

<table>
<thead>
<tr>
<th>Type</th>
<th>Mixture</th>
<th>OC (%)</th>
<th>Representative Slope</th>
<th>Representative Intercept</th>
<th>Thickness adjustment factor for the field matched intercept</th>
<th>k1</th>
<th>k2</th>
<th>k3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Laboratory *10^3</td>
<td>Field matched</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Northern (20, 34,</td>
<td>5.9</td>
<td>0.188650</td>
<td>0.2412</td>
<td>4.852</td>
<td>-0.908</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>and 48°C)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-1.27127</td>
<td>1.0718</td>
<td>0.2412</td>
</tr>
<tr>
<td></td>
<td>6.3</td>
<td>0.1853940</td>
<td>0.2394</td>
<td>4.107</td>
<td>-1.068</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-1.49520</td>
<td>0.4283</td>
<td>0.23942</td>
</tr>
<tr>
<td></td>
<td>6.7</td>
<td>0.1751678</td>
<td>0.2342</td>
<td>6.452</td>
<td>-0.621</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-0.86933</td>
<td>0.5418</td>
<td>0.23422</td>
</tr>
<tr>
<td>Southern (20, 40,</td>
<td>4.2</td>
<td>0.2907505</td>
<td>0.2991</td>
<td>1.731</td>
<td>-1.804</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>and 60°C)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-2.52539</td>
<td>1.0859</td>
<td>0.2991</td>
</tr>
<tr>
<td></td>
<td>4.5</td>
<td>0.2183300</td>
<td>0.2569</td>
<td>4.602</td>
<td>-0.959</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-1.34329</td>
<td>0.0432</td>
<td>0.25693</td>
</tr>
<tr>
<td></td>
<td>4.8</td>
<td>0.2580849</td>
<td>0.2795</td>
<td>3.057</td>
<td>-1.137</td>
<td>1.40</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-1.87164</td>
<td>0.27127</td>
<td>0.2795</td>
</tr>
<tr>
<td>Global Calibration</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Effort in NCHRP Project 1 – 40 D</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>-3.35412</td>
<td>1.56060</td>
<td>0.4791</td>
</tr>
</tbody>
</table>

Table 5.3 – Lab and Field Calibrated Rutting Models for Both Mixtures at Different Binder Contents following the NCHRP 9-30 (A) in Comparison with the Performance Models Calibrated to Nevada’s Local Conditions.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Plastic Deformation Factor</th>
<th>Lab Model</th>
<th>NV Calibration Factors</th>
<th>Field Calibrated Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>OC (%)</td>
<td>k1</td>
<td>k2</td>
<td>k3</td>
</tr>
<tr>
<td>Northern</td>
<td>5.9</td>
<td>-1.27127</td>
<td>1.0718</td>
<td>0.2412</td>
</tr>
<tr>
<td>(20, 34, 48°C)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Southern</td>
<td>4.2</td>
<td>-2.52539</td>
<td>1.0859</td>
<td>0.2991</td>
</tr>
<tr>
<td>(20, 40, 60°C)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NV D II*</td>
<td>4.5</td>
<td>-1.34329</td>
<td>0.0432</td>
<td>0.25693</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NV D I**</td>
<td>4.8</td>
<td>-1.87164</td>
<td>0.2990</td>
<td>0.27950</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- NV D II* denotes State of Nevada district II (Northern part of Nevada, Reno): the corresponding rutting model has the appropriate calibration factors to be used when designing rehabilitated flexible pavements in District II.
- NV D I** denotes State of Nevada district I (Southern part of Nevada, Las Vegas): the corresponding rutting model has the appropriate calibration factors to be used when designing rehabilitated flexible pavements in District I.
(a) Number of loading cycles

\[ \frac{\xi_p}{\xi_r} \]

- $T = 20^\circ C$, OBC: 5.9%
- $T = 34^\circ C$, OBC: 5.9%
- $T = 48^\circ C$, OBC: 5.9%

(b) Number of loading cycles

\[ \frac{\xi_p}{\xi_r} \]

- $T = 20^\circ C$, OBC: 6.3%
- $T = 34^\circ C$, OBC: 6.3%
- $T = 48^\circ C$, OBC: 6.3%
Figure 5.4 – $\xi_p/\xi_r$ Function of the Number of Loading Cycles for the Northern Mixture at: a) OBC-, b) OBC, c) OBC+, and d) using NV D II Model.
Figure 5.5 – $\xi_p/\xi_r$ Function of the Number of Loading Cycles for the Northern Mixture at its Different Binder Contents using the NCHRP 9-30 (A) Models and NV D II Model.
Number of loading cycles

(b)

(c)
Figure 5.6 - $\xi_p/\xi_r$ Function of the Number of Loading Cycles for the Southern Mixture at: a) OBC-, b) OBC, c) OBC+, and d) using NV D I model.

Figure 5.7 - $\xi_p/\xi_r$ Function of the Number of Loading Cycles for the Southern Mixture at its Different Binder Contents using the NCHRP 9-30 (A) Models and NV D I Model.
Figure 5.8 - Permanent Deformation Model Plot of the Northern Mixture at: a) OBC-; b) OBC, and c) OBC+.
Figure 5.9 - Permanent Deformation Model Plot of the Southern Mixture at: a) OBC-; b) OBC, and c) OBC+. 
Table 5.4 - Lab and Field Calibrated Rutting Models for Both Mixtures at Different Binder Contents following the elastic theory.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Plastic Deformation Factor</th>
<th>Lab Model</th>
<th>NV Calibration Factors</th>
<th>Field Calibrated Model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>k1</td>
<td>k2</td>
<td>k3</td>
<td>( \xi_p = \xi_r + 10^{k1} \cdot (T)^{k2} \cdot (N)^{k3} )</td>
</tr>
<tr>
<td>Northern (20, 34, and 48°C)</td>
<td>5.9</td>
<td>2.12795</td>
<td>1.72114</td>
<td>0.16218</td>
</tr>
<tr>
<td></td>
<td>6.3</td>
<td>2.06586</td>
<td>1.68214</td>
<td>0.16241</td>
</tr>
<tr>
<td></td>
<td>6.7</td>
<td>2.34640</td>
<td>1.22361</td>
<td>0.15091</td>
</tr>
<tr>
<td>NV D II*</td>
<td>-3.2605</td>
<td>2.0055</td>
<td>0.3161</td>
<td>( \xi_p = \xi_r + 10^{-3.2605} \cdot (T)^{2.0055} \cdot (N)^{0.3161} )</td>
</tr>
<tr>
<td>Southern (20, 40, and 60°C)</td>
<td>4.2</td>
<td>-1.84652</td>
<td>1.17866</td>
<td>0.19553</td>
</tr>
<tr>
<td></td>
<td>4.5</td>
<td>-0.93252</td>
<td>0.95610</td>
<td>0.22766</td>
</tr>
<tr>
<td></td>
<td>4.8</td>
<td>-1.88737</td>
<td>1.36895</td>
<td>0.21957</td>
</tr>
<tr>
<td>NV D I*</td>
<td>-2.9708</td>
<td>1.7435</td>
<td>0.3547</td>
<td>( \xi_p = \xi_r + 10^{-2.9708} \cdot (T)^{1.7435} \cdot (N)^{0.3547} )</td>
</tr>
</tbody>
</table>

- NV D II* denotes State of Nevada district II (Northern part of Nevada, Reno): the corresponding rutting model has the appropriate calibration factors to be used when designing rehabilitated flexible pavements in District II.
- NV D I* denotes State of Nevada district I (Southern part of Nevada, Las Vegas): the corresponding rutting model has the appropriate calibration factors to be used when designing rehabilitated flexible pavements in District I.
Number of loading cycles

$T = 20^\circ C, \text{OBC} : 5.9\%$
$T = 34^\circ C, \text{OBC} : 5.9\%$
$T = 48^\circ C, \text{OBC} : 5.9\%$

(a)

$T = 20^\circ C, \text{OBC} : 6.3\%$
$T = 34^\circ C, \text{OBC} : 6.3\%$
$T = 48^\circ C, \text{OBC} : 6.3\%$

(b)
Figure 5.10 - $\xi_p/\xi_r$ Function of the Number of Loading Cycles for the Northern Mixture at: a) OBC-, b) OBC, and c) OBC+

Figure 5.11 - $\xi_p/\xi_r$ Function of the Number of Loading Cycles for the Northern Mixture at its Different Binder Contents Using the Elastic Theory Models and NV D II model.
(a) Number of loading cycles: $T = 20\,^\circ\text{C}, \text{OBC} : 4.2\%$, $T = 40\,^\circ\text{C}, \text{OBC} : 4.2\%$, $T = 60\,^\circ\text{C}, \text{OBC} : 4.2\%$

(b) Number of loading cycles: $T = 20\,^\circ\text{C}, \text{OBC} : 4.5\%$, $T = 40\,^\circ\text{C}, \text{OBC} : 4.5\%$, $T = 60\,^\circ\text{C}, \text{OBC} : 4.5\%$
Figure 5.12 - $\xi_p/\xi_r$ Function of the Number of Loading Cycles for the Southern Mixture at: a) OBC-, b) OBC, and c) OBC+

Figure 5.13 - $\xi_p/\xi_r$ Function of the Number of Loading Cycles for the Northern Mixture at its Different Binder Contents Using the Elastic Theory Models and NV D I Model
5.3.3. Analysis and Comparisons

Based on the data generated in the previous two sections (5.3.1 and 5.3.2), the following observations can be revealed:

- Overall, for the NCHRP 9-30 (A) approach or the elastic theory approach, the L6428 mixture showed a significantly higher rutting susceptibility than the LM7622 mixture which can be attributed to the higher asphalt binder content for the northern mixture in comparison to the southern one. However, it should be noted that the L6428 rutting capacity was evaluated at lower temperatures in comparison to the testing temperatures of the LM7622 mixture (20, 34, and 48°C for L6428 mixture while 20, 40, and 60°C for LM7622 mixture) which may also explain the better resistance to rutting of the northern mixture in comparison to the southern one, taking into consideration the PG grading of each binder used in each part of the state, regardless of the asphalt binder content variation for each mixture.

In addition, it should be noted that the local calibration factor $\beta_1$ for the northern part of Nevada is much higher than the one used for the southern part.

For the L6428 mixtures:

- Significantly higher, slightly higher, and significantly lower permanent to resilient strain ratios ($\xi_p/\xi_r$) were observed at OBC-, OBC+, and OBC, respectively when the NCHRP 9-30 (A) approach was used in comparison with local model calibrated for the northern part of Nevada (District II).

- Slightly higher, constant, and slightly lower secondary stage slope (steady-state slope) was observed at OBC-, OBC+, and OBC, respectively when the NCHRP 9-
30 (A) approach was used in comparison with the local model calibrated for the northern part of Nevada (District II).

- Significantly higher permanent to resilient strain ratios ($\xi_p/\xi_r$) were observed at OBC+ but lower at OBC- and OBC when the elastic theory approach was used in comparison with the local model calibrated for the northern part of Nevada (District II).

- A lower secondary stage slope (steady-state slope) was observed when the elastic theory approach was compared to the national calibrated model slopes at the three asphalt binder contents.

- Similar permanent to resilient strain ratio ($\xi_p/\xi_r$) were observed at OBC- and OBC, meanwhile a lower one was observed at OBC+.

For the L7622 mixtures:

- Significantly lower permanent to resilient strain ratios ($\xi_p/\xi_r$) were observed at OBC-, OBC+, and OBC, respectively when the NCHRP 9-30 (A) approach was used in comparison with local model calibrated for the southern part of Nevada (District I).

- Lower secondary stage slope (steady-state slope) was observed at OBC-, OBC+, and OBC when the NCHRP 9-30 (A) approach was used in comparison with the local model calibrated for the southern part of Nevada (District I).

- Higher permanent to resilient strain ratios ($\xi_p/\xi_r$) were observed at OBC-, OBC, and OBC+ at the 40 and 60°C testing temperatures while it remains higher at 20°C for OBC- and OBC+ but become slower at OBC. All of that was when the elastic
theory approach was used in comparison with the local model calibrated for the southern part of Nevada (District I).

- A lower secondary stage slope (steady-state slope) was observed when the elastic theory approach was compared to the national calibrated model slopes at the three asphalt binder contents.
- Similar permanent to resilient strain ratio ($\xi_p/\xi_r$) were observed at OBC and OBC+, meanwhile a lower one was observed at OBC-.

### 5.4. Upcoming Tasks

It should be noted that the rutting model developed in the previous sections were excluded from the $K_z$ factor which takes into consideration the AC sub-layer thickness and the depths of the studied point in this sub-layer. For an overlay, this factor varies when the existing pavement section varies in thickness and material properties.

The following major tasks will be carried out in future to complete the advanced mechanistic and rutting analysis for this research:

- Assume three different typical pavement sections with typical material properties usually locally used in each part of the state of Nevada. The first, second, and third sections have to have an AC layer thickness of 4 inches, 8 inches, and 12 inches, respectively.
- For each section, consider a thin HMA layer of 1.5 inches thicknesses applied on the top of the existing pavement after a mill and fill process.
• Compute the rut depth using the NCHRP 9-30 (A) or the elastic theory approach rutting models determined previously for the thin HMA overlay mixtures and the local calibrated model for the remaining AC pavement section.

• Launch an advanced mechanical analysis and provide further conclusions concerning the rutting behavior of the thin HMA overlay when applied on thin and thick existing pavements.
CHAPTER 6 LIFE CYCLE COST ANALYSIS

6.1. Introduction

Providing a safe and comfortable ride for the road users requires several critical steps. The maintenance of highway facilities constitute one of these critical steps. Slowing down the deterioration process to avoid significant failure is considered the fundamental purpose of maintenance. Typically, the cost of maintenance is 15 to 20% of the expected cost to repair the ultimate failure that will occur without the application of any maintenance activities. For example, national data indicate that every $1 spent on maintaining the pavement surface condition saves $5 on major rehabilitation that will be required if the maintenance activities are not conducted. This concept holds true for all highway maintenance activities. (43)

It is well known that the main objective of a pavement maintenance activity is to maintain the current condition of the pavement and/or slow down the rate of deterioration. In this respect, maintenance activities are not applied to strengthen the pavement capacity rather than fixing and improving the functional behavior. (43)

As mentioned in Chapter 1, several preservation techniques can be applied and used as maintenance for flexible and rigid pavements. Thin asphalt overlays remain one of these techniques. The review of literature developed in Chapter 2 qualified the thin asphalt overlays as the most cost-effective preventive maintenance applied on pavements. This chapter provides a life-cycle cost analysis (LCCA) study between the thin asphalt overlay and the chip seal pavement preservation treatments. A pavement preservation technique, and in particular the chip seal, was selected for this analysis for the following two main
reasons: (1) The thin asphalt overlay is meant to be incorporated and implemented as part of the pavement preservation program for Nevada DOT; and (2) Chip seal is one of the most used techniques for preventive maintenance in the state of Nevada.

6.2. Pavement Maintenance Terminology

Defined as a proactive approach in maintaining existing highways, a pavement preservation program has three primary components in accordance with the Federal Highway Administration (FHWA): minor non-structural rehabilitation, preventive maintenance, and some routine maintenance activities (44). Table 6.1 below supports the idea that the main purpose of applying a pavement preservation activity is to restore the functionality of the existing pavement.

<table>
<thead>
<tr>
<th>Type of Activity</th>
<th>Increase capacity</th>
<th>Increase strength</th>
<th>Reduce aging</th>
<th>Restore serviceability</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Construction</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Reconstruction</td>
<td>X</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Major Rehabilitation (Heavy)</td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
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<td>X</td>
</tr>
<tr>
<td>Minor (Light) Rehabilitation</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Preventive Maintenance</td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Routine Maintenance</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Corrective Maintenance (Reactive)</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>Catastrophic Maintenance</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
</tr>
</tbody>
</table>

As defined before, pavement preservation is a set of cost-effective practices designated to extend the pavement life, improve the ride safety, and save some public dollar
taxes under the famous philosophy of applying the right pavement preservation treatment on the right pavement condition at the right time. (44)

Typically applied to pavements in good condition and having a remaining service life, preventive maintenance is a strategy of extending the service life by treating the surface or near the surface of structurally sound pavements (e.g., for Nevada’s flexible pavements: crack sealing, chip sealing, slurry sealing, micro-surfacing, and thin hot-mix asphalt overlay; while for Nevada’s rigid pavements: joint sealing, diamond grinding, dowel-bar retrofit, and full- depth concrete slab repairs). (44)

Defined as structural enhancements which purpose is extending the service life of the existing pavement and improving its load carrying capacity, the pavement rehabilitation technics are several and can be divided into two categories: minor rehabilitation known as non-structural enhancements added to the category of pavement preservation, and major rehabilitation consisting of structural enhancements. (44)

6.3. Life Cycle Cost Analysis

6.3.1. Methods Description

The life-cycle cost analysis (LCCA) is defined as a tool to determine the most cost effective alternative among different competing pavement preservation techniques when each of them is equally appropriate to be implemented on existing pavement sections. Several items are qualified as requirements to accomplish a life cycle cost analysis such as discount rate, analysis period, unit cost for treatment, and estimated life of treatment.

Defined as the difference between the interest rate and the inflation rate, the discount rate has a significant effect on the LCCA. A $1 today is not worth $1 a year from
now due to interest and inflation. Long historical trends are usually reflected by the discount rate concept. The discount rate is calculated using \textbf{Equation 6.1}.

\[ \text{Discount Rate} = \frac{\text{Int.} - \text{Infl.}}{1 + \text{Infl.}} = \text{Int.} - \text{Infl.} \]  \hspace{1cm} (6.1)

Where:

\textit{Int.}: Interest Rate (Treasury note)

\textit{Infl.}: Inflation Rate (Consumer Price Index)

Defined as the length time that agency would like to plan (e.g. 10, 15, 20 \ldots \text{ years}), the analysis period should be long enough to reflect the cost differences and force the use of each maintenance or rehabilitation alternative. An analysis period of 20 to 30 years is commonly used for flexible pavements while 30 to 40 years are adopted for rigid pavements. The analysis period is considered as the duration over which several preservation and rehabilitation techniques are compared. However, the design period remains to be the duration over which the pavement remains performing well and providing a condition index higher than the threshold lower acceptable limit.

Two methods can be used to establish a LCCA: the Present worth (PW) method and the Equivalent uniform annual cost (EUAC) method. The alternative which provides the least PW or EUAC is usually selected.

The present worth (PW) value is calculated using \textbf{Equation 6.2}.

\[ PW = \frac{F}{(1+i)^n} \]  \hspace{1cm} (6.2)
Where

\( F \): a future sum of money at the end of “N” years

\( i \): discount rate

\( n \): number of years (time in future when an alternative is applied)

The equivalent uniform annual cost is calculated using Equation 6.3.

\[
EUAC = PW \times \frac{\{i(1+i)^n\}}{(1+i)^n-1}
\]  \hspace{1cm} (6.3)

Where

\( EUAC \): Equivalent uniform annual cost

\( PW \): Present worth cost

\( n \): analysis period, years

\( i \): discount rate

Various alternatives will not end up at same level of serviceability. Thus, a salvage value for each alternative should be calculated relative to where it ends up at the end of the analysis period. The salvage value is presented as a percentage of the cost of the last applied treatment (Equation 6.4).

\[
SV = \left(1 - \frac{L_A}{L_E}\right) \times C
\]  \hspace{1cm} (6.4)

\( SV \): Salvage value

\( L_A \): actual life used out of the performance life
\[ L_E: \text{Expected performance life} \]
\[ C: \text{cost of the used treatment in today’s dollars} \]

6.3.2. Data results

The Nevada Department of Transportation (NDOT) uses chip seals as a preventive maintenance for the majority of the state’s flexible pavement. The study completed by University of Nevada, Reno (UNR) to find out the expected life for the treatments in Nevada, showed that the life expectancy for chip seals on average is 3 to 6.5 years when applied on state routes and 2.5 to 4.5 years when applied on interstate routes (43). However, they found much more interest in using thin hot-mix asphalt overlays, as a functional preventive maintenance at early time, to extend the available funds for preservation techniques since it was qualified as the most cost-effective treatment among all the pavement preservation techniques.

The major objective of this section was to compare between the life cycle costs of using chip seal or thin hot-mix asphalt overlay as a preventive maintenance for the state of Nevada. The chip seal was applied 3 years after the pavement construction while two cases were considered for the thin asphalt overlay: application at year 6 or year 8 after pavement construction. The time of application for the chip seal was selected based on the recent findings from a study completed at UNR (43) which showed three years to be the optimal timing for the application of slurry seal in northern Nevada. The LCCA analysis was conducted for different performance lives of the chip seal and thin asphalt overlay pavement preservation treatments. The treatment life for the chip seal was varied between 3 and 6 years while that of the thin asphalt overlay was varied between 4 and 12 years in a
1 year increment. Table 6.2 shows the different alternatives considered in the LCCA based on the treatment life and the year of application. Table 6.3 summarizes the present worth costs corresponding to each treatment function of the performance life and year of application. The following assumptions were used in the LCCA analysis:

- An analysis period of 20 years was adopted.
- A total of $11,334 per lane-mile (cost range: $1.61 – 2.80 $/yd², therefore $1.61*4*1760 = 11,334 $ per lane-mile) was considered as an average initial cost for the chip seal based on the NDOT 2014 records (45).
- Based on information compiled for the literature review (Table 2.3), a total of $14,600 per lane mile was adopted as a typical cost for thin asphalt overlays on 2006 (45) which provides an approximate cost projection of 19,050 $ per lane-mile. Referring to the office of management and budget (executive office of the president) (46), a value of 3.1% was adopted as a nominal interest rate for a 20 years analysis period. Figure 6.1 and Figure 6.2 compare the net present value (PSV) of the chip seal applied at year 3, the thin asphalt overlay applied at year 6, and the thin asphalt overlay applied at year 8 as function of the treatment performance life. Cost savings or additions based on the difference between each chip seal performance life and the various possible performance life of the thin asphalt overlay are shown in Figure 6.3 and Figure 6.4. Positive values indicate savings, while negative values denote additions in required costs for thin asphalt overlays.
<table>
<thead>
<tr>
<th>Preservation Technique</th>
<th>Treatment performance life (years)</th>
<th>Time of application (year) of the i\textsuperscript{th} treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1\textsuperscript{st} 2\textsuperscript{nd} 3\textsuperscript{rd} 4\textsuperscript{th} 5\textsuperscript{th} 6\textsuperscript{th}</td>
<td></td>
</tr>
<tr>
<td>Chip Seal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>6</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>7</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
<td>8</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>9</td>
</tr>
<tr>
<td>Thin Asphalt Overlay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>applied at year 6 after</td>
<td></td>
<td></td>
</tr>
<tr>
<td>construction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>6</td>
<td>10</td>
</tr>
<tr>
<td>5</td>
<td>6</td>
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</tr>
<tr>
<td>6</td>
<td>6</td>
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</tr>
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<td>12</td>
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<tr>
<td>Thin Asphalt Overlay</td>
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<td></td>
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<tr>
<td>applied at year 8 after</td>
<td></td>
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<tr>
<td>construction</td>
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<td>4</td>
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<tr>
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<tr>
<td>12</td>
<td>8</td>
<td>20</td>
</tr>
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</table>
Table 6.3 - Net Present Value (NPV) of Pavement Preservation Alternatives function of the Treatment Performance Life.

<table>
<thead>
<tr>
<th>Preservation Technique</th>
<th>Treatment performance life (years)</th>
<th>1&lt;sup&gt;st&lt;/sup&gt;</th>
<th>2&lt;sup&gt;nd&lt;/sup&gt;</th>
<th>3&lt;sup&gt;rd&lt;/sup&gt;</th>
<th>4&lt;sup&gt;th&lt;/sup&gt;</th>
<th>5&lt;sup&gt;th&lt;/sup&gt;</th>
<th>6&lt;sup&gt;th&lt;/sup&gt;</th>
<th>Salvage Value ($)</th>
<th>Net Present Value ($ per lane mile)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chip Seal</td>
<td>3</td>
<td>10,342</td>
<td>9,437</td>
<td>8,611</td>
<td>7,857</td>
<td>7,170</td>
<td>6,542</td>
<td>2,052</td>
<td>47,908</td>
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<tr>
<td></td>
<td>4</td>
<td>10,342</td>
<td>9,153</td>
<td>8,101</td>
<td>7,170</td>
<td>6,346</td>
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<td>5</td>
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<td>7,621</td>
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<td>29,691</td>
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<td>8,611</td>
<td>7,170</td>
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<td>---</td>
<td>---</td>
<td>1,026</td>
<td>25,097</td>
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<tr>
<td>Thin Asphalt Overlay</td>
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<td>14,038</td>
<td>12,424</td>
<td>10,996</td>
<td>---</td>
<td>---</td>
<td>5,172</td>
<td>48,148</td>
</tr>
<tr>
<td>applied at year 6</td>
<td>5</td>
<td>15,861</td>
<td>13,616</td>
<td>11,688</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>2,069</td>
<td>39,097</td>
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<tr>
<td>after construction</td>
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<td>13,207</td>
<td>10,996</td>
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<td>6,897</td>
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<td>12,424</td>
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<td>Thin Asphalt Overlay</td>
<td>4</td>
<td>14,922</td>
<td>13,207</td>
<td>11,688</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>0</td>
<td>39,817</td>
</tr>
<tr>
<td>applied at year 8</td>
<td>5</td>
<td>14,922</td>
<td>12,810</td>
<td>10,996</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>6,207</td>
<td>32,521</td>
</tr>
<tr>
<td>after construction</td>
<td>6</td>
<td>14,922</td>
<td>12,424</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>0</td>
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<tr>
<td></td>
<td>7</td>
<td>14,922</td>
<td>12,051</td>
<td>---</td>
<td>---</td>
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<td>---</td>
<td>2,956</td>
<td>24,017</td>
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<tr>
<td></td>
<td>8</td>
<td>14,922</td>
<td>11,688</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>5,172</td>
<td>21,438</td>
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<tr>
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<td>9</td>
<td>14,922</td>
<td>11,337</td>
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<td>6,897</td>
<td>19,362</td>
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<tr>
<td></td>
<td>10</td>
<td>14,922</td>
<td>10,996</td>
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<td>---</td>
<td>---</td>
<td>8,276</td>
<td>17,642</td>
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<tr>
<td></td>
<td>11</td>
<td>14,922</td>
<td>10,665</td>
<td>---</td>
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<td>9,404</td>
<td>16,183</td>
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<tr>
<td></td>
<td>12</td>
<td>14,922</td>
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<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>0</td>
<td>14,922</td>
</tr>
</tbody>
</table>
Figure 6.1 - Net Present Value (NPV) of Chip Seal and Thin Asphalt Overlay Applied at Year 6 function of the Treatment Performance Life.

Figure 6.2 - Net Present Value (NPV) of Chip Seal and Thin Asphalt Overlay Applied at Year 8 function of the Treatment Performance Life.
Figure 6.3 – Cost Savings / Additions of Using Thin Asphalt Overlay Applied at Year 6 after Construction compared to Each Chip Seal Performance Life.

Figure 6.4 - Cost Savings / Additions of Using Thin Asphalt Overlay Applied at Year 8 after Construction compared to Each Chip Seal Performance Life.
6.3.3. Analysis and Conclusion

Based on the LCCA data (i.e., Net Present value and Cost Savings/Additions), the following observations can be made:

a) For the case of thin asphalt overlay applied after 6 years from construction:

- Chip seals of 3, 4, 5, and 6 years of performance life remain more cost-effective than the thin asphalt overlays whose performance life is expected to be 4, 5.5, 7, and 8 years, respectively. However, applying a thin asphalt overlay whose performance life is expected to be higher than 8 years shows a permanent cost-effectiveness in comparison to chip seal application.

b) For the case of thin asphalt overlay applied after 8 years from construction:

- Chip seals with a performance life of 3 years is always less cost-effective in comparison with the thin asphalt overlay applied at year 8 after construction. Chip seals of 4, 5, and 6 years of performance life remain more cost-effective than the thin asphalt overlays whose performance life is expected to be 4.5, 5.5, and 6.5 years, respectively. However, applying a thin asphalt overlay whose performance life is higher than 8 years remains as a permanent cost-effectiveness in comparison to chip seal application.

c) General Conclusion

Therefore, comparing the relative costs and outcomes of using chip seals or thin asphalt overlay for the state of Nevada, the last is showing a better cost effectiveness as well as this cost-effectiveness remains more important when it is applied at a later
stage on structurally sound pavements (i.e., after 8 years of reconstruction, not 6 years).

It should be mentioned that the traffic level remains qualified as a very important factor affecting the chip seal and the thin asphalt overlay application.
CHAPTER 7 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1. Overall Objective

The overall objective of this study was to assess the use of locally available materials in Nevada for the development of durable fine-graded hot-mix asphalt overlay mixtures. For this purpose, a comprehensive evaluation of the material characteristics and design of thin hot-mix asphalt overlays for the State of Nevada was conducted. In summary, the major tasks carried out in this research were:

- Establish a review of literature by compiling information about thin overlays and their performance all around the United States.
- Establish two Hveem mix designs for thin hot-mix asphalt overlay mixtures using local materials from the northern and southern part of the state.
- Select an optimal asphalt binder content according to NDOT volumetric requirements.
- Vary the selected optimal asphalt binder content within some allowable tolerances.
- Evaluate the performance properties of the designed mixtures at different asphalt binder contents within the allowable tolerances.
- Conduct a statistical data analysis to evaluate the variation in performance properties for each mixture at different asphalt binder contents.
- Generate conclusions and recommendations, and propose upcoming tasks to develop a specification or a standard for the thin asphalt overlay in the state of Nevada.
7.2. Work Plan Overview and Summary of Findings

In summary, one mix design for each part of the state was conducted following the NDOT Hveem mix design requirements to determine the optimal asphalt binder content (OBC). Except for the interlayer bond strength test, the performance of each of the northern and southern Nevada mixtures was evaluated at three asphalt binder contents: OBC, OBC + selected tolerance, and OBC - selected tolerance. The purpose of the evaluation at different asphalt binder contents was to ensure that mixtures with acceptable properties are still achieved if minor changes in the design asphalt binder content occurred during plant production. The thin hot-mix asphalt overlay mixtures were evaluated in terms of their stability and resistance to moisture damage, surface raveling and abrasion, dynamic modulus $|E^*|$, rutting, reflective cracking, workability, and interlayer shear bond strength.

Based on the data generated from this study, the following summarizes the findings from this study:

- High stability values were observed for both northern and southern mixtures indicating a stable behavior under heavy traffic loading.
- Both mixtures met the NDOT criterion for moisture damage indicating a good resistance to moisture stripping.
- Very low values of mass loss were observed when mixtures were tested to surface raveling and abrasion. The excellent resistance to surface raveling and abrasion was observed at both, the non-conditioned and moisture-conditioned (i.e. 3 F-T) states.
• Both evaluated mixtures showed \(|E^*|\) values similar to the corresponding dense-graded HMA mixtures typically used in Nevada indicating a potentially good field performance.

• Both evaluated mixtures showed phase angle \((\delta)\) values similar to the corresponding dense-graded HMA mixtures typically used in Nevada indicating a good elastic behavior at different temperatures and loading rates.

• Lower FN values were reported for the southern mixture in comparison to the northern mixture. The northern and southern mixtures were found to be applicable for a traffic level of up to 30 and 10 MESALs, respectively.

• Significantly higher susceptibility for rutting was observed for the thin HMA overlay mixture for the northern part of Nevada when compared to the rutting potential provided by the locally calibrated model applied for all the traditional HMA mixtures developed for this part of state which may require a new calibration based on the field section since these mixtures showed excellent stability values under heavy expected traffic. However, the susceptibility for rutting remained significantly lower for the southern part of Nevada indicating an excellent rutting resistance in comparison to the traditional dense-graded HMA mixtures developed for this part of the state.

• An excellent resistance to reflective cracking was observed for both mixtures at all evaluated asphalt binder contents. A higher resistance to reflective cracking was observed for the northern mixture when compared to the southern mixture.

• An excellent workability and compactibility were observed for both mixtures at all evaluated asphalt binder contents when compared to the compaction effort required
for traditional dense-graded HMA mixtures which guarantees not having compaction issues and problems for reaching the required densities.

- In general, a higher interlayer shear bond strength was detected with the use of tack coat; the HPS tack coat resulted in a significantly higher bond strength. The type of the overlay mixture influenced the interlayer shear bond strength (a higher bond strength was observed for the southern mixture when compared to the northern mixture).

7.3. Summary of Recommendations

In summary, based on the findings from this study, it is concluded that thin HMA overlay mixtures with acceptable laboratory performance can be designed for the state of Nevada using locally source materials. In addition, the designed mixtures will conserve their mechanical properties in case of a variation of asphalt binder content, within the allowable selected tolerances, occurs in the plant during the production. It is recommended to construct field test sections to evaluate the field performance of the thin HMA overlay in Nevada. PMS data and visual distress survey are recommended to be done to monitor the ride quality and drivers’ satisfactions. The test sections should be monitored for long-term field performance in order to allow for the development of future specifications.

7.4. Upcoming Tasks and Draft Proposed Specifications

This research project constituted the first step for the incorporation and implementation of thin hot-mix asphalt overlays for the state of Nevada. The material division of Nevada DOT provided the Pavements/Materials program by a preliminary aggregate gradation specification for the thin overlays applied in the state of Nevada. This
aggregate specification was developed based on random mix trial taking into consideration the other states’ specification and the required asphalt binder content. The Western Regional Superpave Center (WRSC) at the University of Nevada, Reno (UNR) has completed the mix design of the experimental thin hot mix asphalt overlay for the northern and the southern part of Nevada using locally materials. Review of Literature for the thin overlays all around the United States was provided, mix designs for each part of the state were established, optimal asphalt binder contents were selected, variation in asphalt binder content was applied to evaluate the performance sensitivity of these mixtures, and performance properties were evaluated based on the most common distresses affecting a thin overlay on the top of an old pavement during a maintenance procedure.

The second step consists of constructing true field sections. Available aggregate stockpiles and asphalt binders, collected from the corresponding part of the state where these sections are designated to be constructed, will be provided by NDOT for the UNR laboratory. A new corresponding Hveem mix design is redeveloped and the required performance properties for a thin overlay mixture is re-verified according to the preliminary draft specification shown in Table 7.1 and Table 7.2.

The aggregates shall meet the state’s specification for a traditional Hveem asphalt mix design. Both mixtures showed very low mass loss values by raveling which lead to not introduce the mass loss by raveling as required criteria for thin asphalt overlays in the property requirements table. It should be noticed that raveling can be checked and maximum mass loss values of 20% and 30% remain acceptable for un-aged and aged evaluated mixtures, respectively. Higher mass loss values by abrasion were observed for the evaluated thin overlay mixtures. While the same criterion was adopted, 20% used
usually for OGFC was dropped down to 10% for both part of the states when thin overlays were evaluated. An excellent resistance to reflective cracking was observed based on the data generated. However, the same NDOT criterion was kept: 1,200 as a minimum number of cycles at failure. No additional restriction and limits are provided for the dynamic modulus, flow number test, bond, workability, and compactability, since these mixtures are verified to perform 100% similar to the traditional existing asphalt mixtures in Nevada. The only difference remains in the aggregate gradation specification (i.e., finer) inducing a higher asphalt binder content.

A pavement condition review is provided for the section where the thin overlay is designated to be applied. The pavement condition review consists of historical records and reviews attached to these sections, some subjective opinions based on the initial site visit, and a visual distress field survey. Field cores can be taken for a laboratory advanced testing when it is needed. The falling weight deflectometer (FWD) testing is conducted on the targeted section before milling and overlaying and periodically after paving.

The milling process and the surface preparation should precede the thin overlay placement. The tack coat application rates are fixed based on the supplier requirements. Paving works and application rates are monitored while mixing, placing, and compacting the thin asphalt overlays on the top of the corresponding sections. Field loose mixtures are shipped to the pavement laboratories at UNR where advanced performance testing are conducted and checked. The following describes the required future laboratory testing done on the field mixtures:

- Binder extraction and recovery which allows the determination of the binder properties being used. These properties should match with the properties of the raw
initial asphalt binder submitted by the supplier. The resultant dried aggregates are then sieved and the aggregate gradation of the field mixtures is then checked with the one provided when the mix designs were established at the laboratory. Job mix formulas have to be implemented at this stage.

- The theoretical maximum specific gravity ($G_{mm}$) of the field mixture is determined for purpose of checking and determining the air void levels of the tested specimens.

- All required performance tests are conducted on specimens compacted to the in-place air voids level (approximately 7%).

- Data analyses and comparisons are established between the data provided for the laboratory and the field mixtures.

- Modifications and recommendations are added to the preliminary thin asphalt overlay specification for the state of Nevada.

**Table 7.1 - Thin Asphalt Overlay Gradation Limits and Recommendations for Aggregate Properties.**

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing by Mass</th>
</tr>
</thead>
<tbody>
<tr>
<td>19 mm (3/4 inch)</td>
<td>100</td>
</tr>
<tr>
<td>12.5 mm (1/2 inch)</td>
<td>90 – 100</td>
</tr>
<tr>
<td>9.5 mm (3/8 inch)</td>
<td>70 – 90</td>
</tr>
<tr>
<td>4.75 mm (No.4)</td>
<td>50 – 70</td>
</tr>
<tr>
<td>2.0 mm (No.10)</td>
<td>30 – 50</td>
</tr>
<tr>
<td>0.425 mm (No.40)</td>
<td>12 – 24</td>
</tr>
<tr>
<td>0.075 mm (No.200)</td>
<td>3 – 8</td>
</tr>
</tbody>
</table>
Table 7.2 - Thin Asphalt Overlay Property Requirements.

<table>
<thead>
<tr>
<th>Test</th>
<th>Replicates</th>
<th>Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hveem stability @ 60˚C, 100 mm mold(^1)</td>
<td>3</td>
<td>37 min.</td>
</tr>
<tr>
<td>Cantabro Loss (Tex-245-F), 300 L.A revolutions (30 rpm), 7±1% air voids, 18 to 24˚C</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PG64-28NV</td>
<td>3</td>
<td>10% CL max.</td>
</tr>
<tr>
<td>PG76-22NV</td>
<td>3</td>
<td>10% CL max.</td>
</tr>
<tr>
<td>Texas Overlay Test, AMPT machine (Tex-248-F), 7±1% air voids, 10˚C, 0.018 inch maximum displacement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PG64-28NV</td>
<td>6(^2)</td>
<td>1,200 cycles min.</td>
</tr>
<tr>
<td>PG76-22NV</td>
<td>6(^2)</td>
<td>1,200 cycles min.</td>
</tr>
</tbody>
</table>

\(^1\) Compact Hveem stability specimens to design air voids ± 0.5%.

\(^2\) Discard the samples with the highest and lowest reflective cracking life, i.e. average the middle four results.

REFERENCES


38. AASHTO T209, *Theoretical Maximum Specific Gravity (G_{mm}) and Density of Hot Mix Asphalt (HMA)*, American Association of State Highway and Transportation Officials, 2014.


46. https://www.whitehouse.gov/omb/management/budget
CHAPTER 8 APPENDIX A: PHASE ANGLE MASTER CURVE
DETERMINATION USING THE NONLINEAR LEAST-SQUARES
REGRESSION MODEL

This chapter provides a detailed example explaining how the phase angle master curve can be obtained using the non-linear least-squares regression model. The dynamic modulus $|E^*|$ data and the phase angle ($\delta$) values of the southern mixture at OBC-(LM7622_OBC-) were used in this appendix to provide a clear explanation.

8.1. Raw Data

The raw dynamic modulus $|E^*|$ and phase angle ($\delta$) data are summarized in Table 8.1. It should be noted that these data were obtained following the AMPT procedure according to AASHTO TP79 (24). Figure 8.1 illustrates the raw $|E^*|$ and phase angle ($\delta$) data at different testing frequencies before being shifted.
Figure 8.1 - LM7622_OBC- Raw Data Plots Function of the Testing Frequencies: a) Dynamic Modulus $|E^*|$, and b) Phase Angle ($\delta$)
Table 8.1 - Dynamic Modulus $|E^*|$ and Phase Angle ($\delta$) Raw Data for LM7622_OBC-

<table>
<thead>
<tr>
<th>Temperature (°C)</th>
<th>Temperature (°F)</th>
<th>Sample</th>
<th>Frequency (Hz)</th>
<th>Dynamic Modulus $E^*$ (kisI)</th>
<th>Phase Angle $\delta$ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>39.2</td>
<td>S1</td>
<td>10</td>
<td>1812.0</td>
<td>16.40</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>1253.0</td>
<td>16.41</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>0.1</td>
<td>752.0</td>
<td>41.20</td>
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<td>1730.0</td>
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<td>24.05</td>
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<td>10</td>
<td>1414.0</td>
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<td>287.8</td>
<td>31.26</td>
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<td>0.1</td>
<td>135.3</td>
<td>30.53</td>
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<td></td>
<td></td>
<td>S3</td>
<td>10</td>
<td>548.1</td>
<td>27.42</td>
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<td></td>
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<td>45</td>
<td>113</td>
<td>S1</td>
<td>10</td>
<td>135.0</td>
<td>34.33</td>
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<td>55.2</td>
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<td>0.01</td>
<td>18.2</td>
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<td>10</td>
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<td>0.1</td>
<td>22.6</td>
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<td>0.01</td>
<td>14.4</td>
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<td>91.3</td>
<td>34.02</td>
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<td>37.8</td>
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<td>0.1</td>
<td>19.5</td>
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<td></td>
<td></td>
<td>0.01</td>
<td>13.7</td>
<td>21.14</td>
</tr>
</tbody>
</table>
8.2. Calculations

8.2.1. Dynamic Modulus $|E^*|$ Shifting

The original sigmoidal function, mentioned in Equation 3.8, was used for the dynamic modulus prediction and shifting process. The Arrhenius model (Equation 3.7) was used for the temperature shifting. A total of five fitting parameters (i.e., $\delta$, $\alpha$, $\beta$, $\chi$, and $\Delta E_a$) are required to determine the linear viscoelastic behavior of the considered material at a given temperature. The numerical optimization was performed using the solver function in Microsoft Excel and all the data at different temperatures are shifted to reference common temperature ($T = 68^\circ F (20^\circ C)$). The sum of the squared errors between the average measured dynamic moduli at each temperature and frequency and the values predicted using the mathematical model was targeted to be minimized. Table 8.2 provides the values of the fitting parameters after optimization. Figure 8.2 illustrates the shift factor variation function of the testing temperature. A general statistical fit was generated based on the collected data as a final step as shown in Figure 8.3.

<table>
<thead>
<tr>
<th>Fitting Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\delta$</td>
<td>0.755027</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>2.869693</td>
</tr>
<tr>
<td>$\beta$</td>
<td>-0.43394</td>
</tr>
<tr>
<td>$\chi$</td>
<td>0.487685</td>
</tr>
<tr>
<td>$\Delta E_a$</td>
<td>178571.9</td>
</tr>
</tbody>
</table>
Figure 8.2 - Arrhenius Temperature Shift factors to 68°F (20°C) Plots Functions of the Testing Temperatures for LM7622_OBC-

Figure 8.3 - Dynamic Modulus $|E^*|$ Master Curve at 68°F (20°C) for Short-Term Aged LM7622_OBC-
8.2.2. *Phase Angle $|\delta|$ Shifting*

Once the dynamic modulus data were shifted to the reference temperature, the modified proposed model for the phase angle master curve was introduced (Equation 3.13). A fitting parameter ($c$) was then determined by minimizing the sum square of the deviation between the phase angles (measured and predicted). A value of 1.064 was estimated which implied a 6.4% increase in the phase angle value to the one determined when using the non-modified model for the phase angle master curve. Figure 8.4 shows the phase angle values at the corresponding reduced frequencies as well as the modified phase angle master curve fit.

![Figure 8.4 - Phase Angle Values and Master Curve Fit for LM7622_OBC-](image_url)
8.3. Verifications

Shifting the phase angle ($\delta$) values and obtaining the correspondent master curve using a statistical method (i.e. Nonlinear least-squares regression model) may induce a high error because of the data limitation since a high amount of data is required to provide a better fit. Several checking can be done to evaluate the usability of the model developed and the feasibility of the generated data. Figure 8.5 shows a very good correlation between the measured and predicted dynamic modulus $|E^*|$ values. Meanwhile, a good correlation was generated between the measured and predicted phase angle ($\delta$) values as shown in Figure 8.6. An acceptable correlation (Figure 8.7) was observed between the master curve fit determined using the modified nonlinear least squares regression model and the elastic and viscous components of the complex dynamic modulus $|E^*|$ determined using Equation 8.1 and Equation 8.2.

$$E' = |E^*| \times \cos(\delta) \quad (8.1)$$

$$E'' = |E^*| \times \sin(\delta) \quad (8.2)$$

Where

$E^*$: measured dynamic modulus, ksi or kPa

$\delta$: measured Phase angle, °

$E'$: elastic component of the measured dynamic modulus $|E^*|$, ksi or kPa

$E''$: viscous component of the measured dynamic modulus $|E^*|$, ksi or kPa
Figure 8.5 - Correlation Plot between the Predicted and the Measured Dynamic Modulus $|E^*|$ for LM7622_OBC.

Figure 8.6 - Correlation Plot between the Predicted and the Measured Phase Angle ($\delta$) for LM7622_OBC.
8.4. Usage

Several softwares such as the 3D-Move analysis software Version 3.2 used the phase angle (δ) values in addition to the dynamic modulus |E*| values as level 1 inputs for their dynamic analysis. Table 8.3 provides an input example of the phase angle (δ) for the 3D-Move software for the LM7622_OBC- mixture. It should be noticed that this procedure was repeated for both mixtures at their different binder contents.

Table 8.3 - Sample of 3D-Move Phase Angle Input Using the LM7622_OBC- Data

<table>
<thead>
<tr>
<th>Temperature, °F</th>
<th>25 Hz</th>
<th>10 Hz</th>
<th>5 Hz</th>
<th>1 Hz</th>
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