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Optimizing Laboratory Curing Conditions for Hot Mix Asphalt to Simulate Field Behavior

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OPTIMIZING LABORATORY CURING CONDITIONS FOR HOT

MIX ASPHALT TO SIMULATE FIELD BEHAVIOR

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Dedication

I owe my success to my beautiful mother, Guadalupe Arras, who strived to help me all the way until this milestone of my career. I also dedicate this work to my beloved grandparents whose unconditional love during their life gave me the strength to pursue this career.

OPTIMIZING LABORATORY CURING CONDITIONS FOR HOT MIX ASPHALT TO SIMULATE FIELD BEHAVIOR

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Abstract

The engineering properties of asphalt mixtures change with time. Shortly after placement, asphalt concrete (AC) layers are more susceptible to rutting. As the pavement ages, the AC layer becomes stiffer, brittle, and thus more susceptible to cracking. Current protocols provide guidelines for the selection of materials, the determination of the material proportions (e.g., aggregates and binder content), and the evaluation of the engineering properties (e.g., cracking and rutting potentials) of any given asphalt mix design. However, these protocols do not consider the impact of aging on the mixture. This study investigated existing and novel laboratory methods to determine protocols that simulate the two aging states needed to design an asphalt mixture to resist rutting and cracking and provide information on how curing affects the physical and engineering performance of binders and mixtures. This study leveraged existing research studies and available performance data along with a systematic test matrix to optimize the curing conditions. The wide range of tests conducted in this study indicated that the minimum time to achieve levels of long-term aging using an optimized laboratory protocol would be close to 24 hrs. The steady state of aging for ranking different mixes may be adequately achieved after a period of 24 hrs (including 2 hrs of short-term aging) by aging loose mixture in a conventional laboratory oven. The efficiency of another aging protocol using pressure to accelerate aging has been demonstrated. This method can be used to characterize the performance of mixes more accurately under long-term aging within a period of 24 to 48 hrs.

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Chapter 1: Introduction

Problem Statement

The engineering properties of asphalt mixtures change with time. Shortly after placement, asphalt concrete (AC) layers become susceptible to rutting. As the pavement ages, the AC layer becomes stiffer, brittle, and thus more vulnerable to cracking. Most highway agencies provide guidelines for the selection of materials, the determination of the material proportions (e.g., aggregates and binder content), and the evaluation of the engineering properties (e.g., cracking and rutting potentials) of any given asphalt mix design. However, those specifications do not provide any check or guidance on the impact of aging on the mixture. This project seeks to investigate and propose protocols to simulate realistically the short- and long-term aging states required to evaluate the asphalt mixture resistance to rutting and cracking, respectively. Such conditioning protocols must be valid for all types of asphalt mixes including, but not limited to mixtures that incorporate reclaimed asphalt pavement (RAP), reclaimed asphalt shingles (RAS), recycling agents, and other additives.

This study focuses on developing and implementing optimized, representative, and practical laboratory aging protocols that improve the mix design process leading to the production and placement of long-lasting, stable, and durable asphalt mixtures. The findings from this research project are also expected to provide insight into the formation of distresses (rutting and cracking) associated with individual asphalt mixtures.

Objectives and Scope

The main goal of this project is to deliver implementable and optimized laboratory aging protocols that can be used to assess the engineering properties of mixtures after short- and long-term aging. To achieve the goal of this project, the following activities had to be conducted:

1. Conduct a comprehensive literature search and gap analysis to identify weaknesses and strengths of current practices to simulate field aging in the laboratory.
2. Assess the effectiveness and practicality of current short- and long-term aging protocols for use with laboratory performance tests (e.g., Hamburg wheel-tracking, HWT, overlay tester, OT, and indirect tension, IDT tests).
3. Formulate a robust and consistent analysis method, corresponding aging indices, and acceptance limits to determine the variation in the engineering properties of different asphalt mixtures and associated asphalt binders after simulated field aging.
4. Develop optimized aging protocols for laboratory- and plant-produced mixtures practically to represent the early age rutting potential and cracking resistance after long-term aging of different mixtures and corresponding binders more accurately.
5. Generate laboratory and field performance data to verify the simulated field-aging conditions of laboratory-produced, plant-produced, and field-compacted asphalt mixtures and concomitant asphalt binders.
6. Incorporate the proposed optimized laboratory aging protocols for asphalt materials into the balanced mix design process and specifications.

Organization of Report

To address the stated goal and objectives, the report is divided into eight chapters including the introduction. These are further categorized into three main phases:

- Phase I (*Documentation*) consists of documenting the current state of the practice aging protocols used by other agencies and recently developed methods related to this project. Several approaches were formulated, and the most promising approaches were selected. The primary outcome of this phase was identifying the candidate-aging protocols with an accompanying experiment design plan to address the technical objectives described in the problem statement. This phase consists of the following tasks:

Chapter 2 - Comprehensive Documentation of Current Practices and Specifications

Chapter 3 - Preliminary Assessment of Existing Laboratory Aging Protocols on Asphalt Performance Using Available Data

Chapter 4 - Development of Experiment Design

- Phase II (*Evaluation*) involved conducting an initial experimental evaluation of the candidate laboratory aging protocols to assess their effectiveness in simulating the field-aging behavior of asphalt concrete and binders during the production, placement, and service life of a pavement. A laboratory-validated version of the aging protocol was selected to further evaluate the influence of mix design variables on the aging and performance of asphalt mixtures. This phase included the following tasks:

Chapter 5 - Initial Evaluation of Aging Potential of Asphalt Concrete with Laboratory Aging Protocols

Chapter 6 - Extended Evaluation of Aging Potential of Asphalt Concrete with Refined Laboratory Aging Protocols

- Phase III (*Verification and Validation*) focused on the validation of the recommended laboratory aging protocols based on additional laboratory and field performance data. The findings from this phase were used to make any adjustments or refinements to the recommendations from Phase II. This phase included:

Chapter 7 - Verification of Preliminary Laboratory Aging Protocols

Chapter 8 – Conclusions and Recommendations

Chapter 2: Literature Review

Aging is considered a major environmental effect that drives changes in the engineering properties of asphalt mixtures during the design, production, placement, and service life of the flexible pavement. The aging of asphalt mixtures causes the material to stiffen over time and to become more brittle which contributes to durability problems such as cracking of the AC layer. One of the main goals of evaluating the engineering properties of an asphalt mixture is to screen for crack- and rut-susceptible mixtures. To do so accurately, one needs representative laboratory aging protocols that can produce specimens of asphalt mixtures simulating the two critical aging states, freshly placed, and long-term aged asphalt mixture.

Current mix design processes usually require characterizing the engineering properties of an asphalt mixture using specimens conditioned with short-term oven aging (STOA) protocols. These specimens are typically oven cured for 2 hrs at a specified compaction temperature related to the performance grade (PG) of the binder. In the early aging state, asphalt mixtures are more susceptible to rutting. As the pavement ages, the asphalt concrete layer becomes stiffer and more prone to cracking. Thus, the current aging protocols to simulate the field-aging behavior of asphalt mixtures do not accurately reflect their expected long-term cracking performance.

Long-term oven aging (LTOA) protocols have been implemented mainly to predict the effects of aging on the engineering properties of asphalt mixtures over time. Per AASHTO R 30, the standard practice to simulate the field aging of an asphalt mixture consists of curing compacted laboratory specimens for five days at 185ºF (85ºC). Even though several studies have demonstrated LTOA protocols could produce relatively consistent results with the same degree of aging that would take place in situ, it is impractical to vary laboratory-curing conditions for asphalt mixtures several times before molding specimens for performance testing. The implementation of optimized laboratory curing protocols is paramount to prevent early age rutting and particularly long-term cracking susceptibility during the mix design process.

In the last several years, the Texas Department of Transportation (TxDOT) has funded several studies to investigate the engineering properties of a wide range of asphalt mixtures, the effectiveness of performance test methods, and the implementation of sustainable measures such as incorporating recycled materials, warm-mix asphalt technologies, and recycling agents. These studies also focused on the development of a performance measure to evaluate asphalt mixtures, which is necessary to develop a framework to implement the concept of “balanced mix design” or more accurately performance-based mix design. Similarly, other State Highway Agencies (SHA) and national agencies have performed studies to develop and evaluate laboratory-aging methods to reflect more accurately the aging that occurs in the field.

A common observation among many of the reports has to do with dissimilarities between the results of laboratory-mixed laboratory-compacted (LMLC) and plant-mixed lab-compacted (PMLC) specimens. The difference is associated with the aging state of PMLC because specimens undergo a period of short-term aging in the plant. To prepare PMLC specimens, the mixture must go through another aging process in the laboratory. On the other hand, LMLC specimens go through the mixing and compaction process in the laboratory without initial aging. A rigorous comparative analysis of the results between comparable PMLC and LMLC specimens is necessary to bridge the gap.

Laboratory Aging Protocols and Engineering Behavior of Asphalt Binders

Extraneous factors, such as temperature, pavement thickness, air void content and distribution, dictate the spatial distribution and extent of aging in an asphalt mixture. However, it is ultimately the aging of the asphalt binder that dictates the change in the performance characteristics of the asphalt mixture. Consequently, any investigation on aging requires a focus on the aging characteristics of the asphalt binder.

The national standard specification to determine the PG of an asphalt binder, AASHTO M320, includes short- and long-term laboratory conditioning procedures to simulate the aging that occurs in asphalt binders during production and over the service life of the pavement. The conditioning procedures used in AASHTO M320 are (1) the Rolling Thin Film Oven Test (RTFO, AASHTO T240) for short-term aging and (2) the Pressure Aging Vessel (PAV, AASHTO R28) for long-term aging. These procedures were developed during the Strategic Highway Research Program (SHRP) based on previous experience and limited validation studies using mostly unmodified asphalt binders recovered from in-service pavements.

Bell (*1989*) documented several laboratory short-term protocols for asphalt binders and mixes, including the Thin-Film Oven Test (TFOT), the RTFO, the Stirred Air-Flow Test (SAFT), and the German Rolling-Flask Test (GRF). The RTFO exposes an asphalt binder film to continuous heat and airflow to induce oxidative aging and it is the most commonly used method to simulate short-term aging. Airey and Brown (*1998*) indicated that the RTFO conditions were not identical to the aging conditions induced during actual mixing but had shown a good correlation to the aging conditions observed in the conventional batch mixes. Bahia et al. (*2001*) concluded that RTFO did not adequately simulate the aging of modified asphalt binders, even though it might be satisfactory for neat binders. Anderson and Bonaquist (*2012*) evaluated the existing asphalt binder technologies to identify potential modifications for improving the short-term aging protocols. They recommended SAFT and the modified German rotating flask (MGRF, *Glover et al., 2001; Robertson et al., 2001*).

Numerous studies have used pressurized air or oxygen to accelerate aging in the laboratory, specifically for the performance characterization of asphalt binders. Bahia and Anderson (*1995*) indicated that the use of high pressure was desirable to simulate accelerated long-term aging in asphalt binders because (1) volatile loss was minimized, (2) aging could be accomplished without high temperatures, (3) large sample sizes could be accommodated, (4) field climate conditions could be approximated, and (5) laboratory use was practical. The pressure-aging vessel (PAV, AASHTO R 28), which is believed to simulate about four to ten years of aging, is the long-term aging test currently specified in the Superpave specifications. Kandhal and Wenger (*1975*) concluded that the stiffness of field cores reached an asymptotic value within 10 years of service. Mallick and Brown (*2004*) evaluated field cores extracted at different service lives. They stated that the PAV method could successfully simulate the long-term aging of the asphalt binder.

Short-Term Aging of Asphalt Concrete Mixtures

Asphalt mixture production temperatures are optimized to ensure complete drying of mineral aggregates, proper coating and bonding of the aggregate-binder system, and adequate workability and compactability for handling and compaction (*Newcomb et al., 2015*). The production and placement of mixtures require conditioning temperatures that range from 220ºF (104ºC) to 325ºF (163ºC) (*Kuennen 2004; Newcomb 2005*). Most highway agencies already have short-term aging procedures to simulate the aging and asphalt absorption of an asphalt mixture as it is produced in a plant and transported to the site. The effectiveness of the short-term aging process to simulate the mixing, production and placement process for asphalt mixtures is important to estimate accurately the mixture's durability and stability.

The standard practice for asphalt mix design in the laboratory is to simulate the aging and binder absorption that occurs during production and construction by conditioning loose mixes before compaction for a specified time and temperature. The recommended procedure in AASHTO R 30 for preparing specimens for volumetric analysis is 2 hrs at the compaction temperature. Newcomb et al. (*2015*) recommended the following basic changes to the AASHTO R 30 short-term aging protocol: 1) fixing the compaction temperatures for warm mix asphalt (WMA) at 240ºF (116ºC) and hot mix asphalt (HMA) at 275ºF (135ºC) and 2) conditioning the sample for 2 hrs at the compaction temperature regardless of whether the specimens are being prepared for volumetric analyses or performance testing. These conclusions were drawn by comparing volumetric parameters (i.e., theoretical maximum specific gravity and binder absorption), dynamic modulus and resilient modulus, and HWT test results from lab-produced specimens to plant-produced specimens.

Table 2.1 presents a summary of some of the studies conducted to investigate the influence of short-term aging conditions (temperature and time) on HMA and WMA production. In general, lower curing temperatures are proposed for WMA mixtures than those normally specified for HMA. For HMA mixtures, 2 hrs of curing is the common recommendation.

Table 2.1 Summary of Laboratory Short-Term Aging Protocols for Asphalt Mixtures

Reference	STOA Conditions	Key Findings
Aschenbrener and Far (1994)	- 2 hrs at compaction temperature	- Reheating influenced HWT test results
Rashwan and Williams (2011)	- 2 hrs at 302ºF (150ºC) for HMA - 2 and 4 hrs at 230ºF (110ºC) for WMA	- Dynamic modulus and flow number tests results were higher for HMA with higher temperature and mixtures with RAP
Bonaquist (2011)	- 2 and 4 hrs at compaction temperature	- Aggregate absorption and IDT test results comparable to field cores - Recommend 2 hrs at compaction temperature for WMA and a longer aging period for rutting and moisture susceptibility
Clements et al. (2012)	- 0.5, 2, 4, and 8 hrs at 275ºF (135ºC) for HMA - 0.5, 2, 4, and 8 hrs at 237ºF (114ºC) for WMA	- Similar DCT test results for HMA and WMA - Reduced dynamic modulus and flow number and increased rutting for WMA on HMA
Estakhri (2012)	- 2 hrs at 275ºF (135ºC) - 4 hrs at 275ºF (135ºC)	- Equivalent HWT test results for WMA and HMA - Aging time and temperature effect on HWT and overlay tester results
Epps Martin et al. (2014)	- 2 and 4 hrs at compaction temperature at 275ºF (135ºC) - 2 hrs at compaction temperature + 16 hrs at 140ºF (60ºC) + 2 hrs at compaction temperature	- Recommend 2 hrs at 275ºF (135ºC) for HMA and 2 hrs at 240ºF (116ºC) for WMA

Newcomb et al. (*2015*) proposed changes to the AASHTO R 30 protocol for conditioning HMA and WMA. Newcomb et al. considered aging that occurred during the production, construction, and in-service life of the pavement. Although Newcomb et al. focused on short-term laboratory aging to better simulate field behavior during mixing, transportation, and placement of asphalt mixtures, the long-term aging effect was also considered to reflect 1-3 years of in-service life of the pavement.

Newcomb et al. (*2015*) addressed the aging patterns of LMLC specimens with raw materials, plant-mixed plant-compacted (PMPC) specimens, and field cores. Three performance tests, namely the resilient modulus, HWT and dynamic modulus tests were used to evaluate the effects of short- and long-term aging stages. They determined that the resilient modulus test was sensitive to assessing aging progression. In addition, the resilient modulus test was used to compare the stiffness of LMLC and PMPC cores. Binder was extracted and recovered from each field core to assess aging. Specimens were evaluated using the dynamic shear rheometer (DSR) and Fourier transform infrared spectroscopy (FTIR) devices. Newcomb et al. (*2015*) validated that the STOA protocol for LMLC and PMPC specimens yielded similar aging processes between the two.

Newcomb et al. (*2015*) focused on the impacts associated with climate, type of aggregate (including aggregate absorption), asphalt type and source, RAP/RAS content, type of plant and the temperature(s) volumetric and performance parameters of asphalt mixtures after the short-term conditioning. They found that the factors affecting the performance of asphalt mixtures were the type and proportion of recycled material, aggregate absorption, and asphalt source. They observed that the mixtures that included RAP and RAS in the asphalt mixture exhibited significantly higher stiffness. Conversely, they indicated that the stiffness of a material with high aggregate absorption would be lower. They determined that many more parameters affected long-term aging.

Newcomb et al. (*2015*) conducted the study in two phases. Phase I consisted of verification of asphalt mixtures volumetric mix design, performance test, and analysis of factors affecting short-term aging. The volumetric mix design involved quantifying the theoretical maximum specific gravity, percent of absorbed binder, percent of effective binder, and effective binder film thickness of LMLC and PMPC specimens. They found a good correlation between the volumetric properties extracted from LMLC and PMPC specimens except for one section with highly absorptive aggregates. They also reported that since the lab and plant mixtures from the same source did not have the same absorption rate, their theoretical maximum specific gravity and percent of absorbed binder differed.

The performance test results from comparable LMLC and PMPC specimens yielded reasonable correlations. However, the performance test results from LMLC specimens and field cores were poorly correlated. The field cores generally provided higher rutting resistance. Newcomb et al. (*2015*) explained the difficulty of fitting the field cores into the HWTD molds as one possible factor for the lack of correlation. They reported a good relationship between the level of binder oxidation and binder stiffness.

Newcomb et al. (*2015*) proposed an AASHTO practice for conducting plant-aging studies that provide detailed instructions on how to prepare LMLC or PMPC specimens. Figure 2.1 illustrates the recommended flowchart for conducting short-term aging on asphalt mixtures. From Phase I of this study, they determined that the sources of binder, aggregate absorption and the inclusion of recycled materials affected the stiffness and rutting potential of short-term aged AC mixtures. However, regardless of plant type or production temperature, no significant effect was observed on short-term aged AC mixtures.

Phase II consisted of quantifying field aging and the correlation of field aging to LTOA protocol. Newcomb et al. (*2015*) proposed cumulative degree-days (CDD) as a field metric that captures field aging by considering climate and time. Field aging is quantified by the property ratio of an AC mixture from the sampling time to the original state (during construction). CDD can be estimated from Equation 2.1 by accounting for different dates of construction and climatic zones.

$$CDD = \sum(T_{max} - T_{base}) \qquad \textbf{(2.1)}$$

where T_{max} is the daily maximum temperature (ºF) and T_{base} is recommended as the base temperature of 32ºF (0ºC).

AC mixtures with a higher property ratio go through a higher rate of aging during the given period. They concluded that warmer climatic zones are more susceptible to aging and have higher CDD values. Conversely, cooler climatic zones experience lower levels of aging and CDD values.

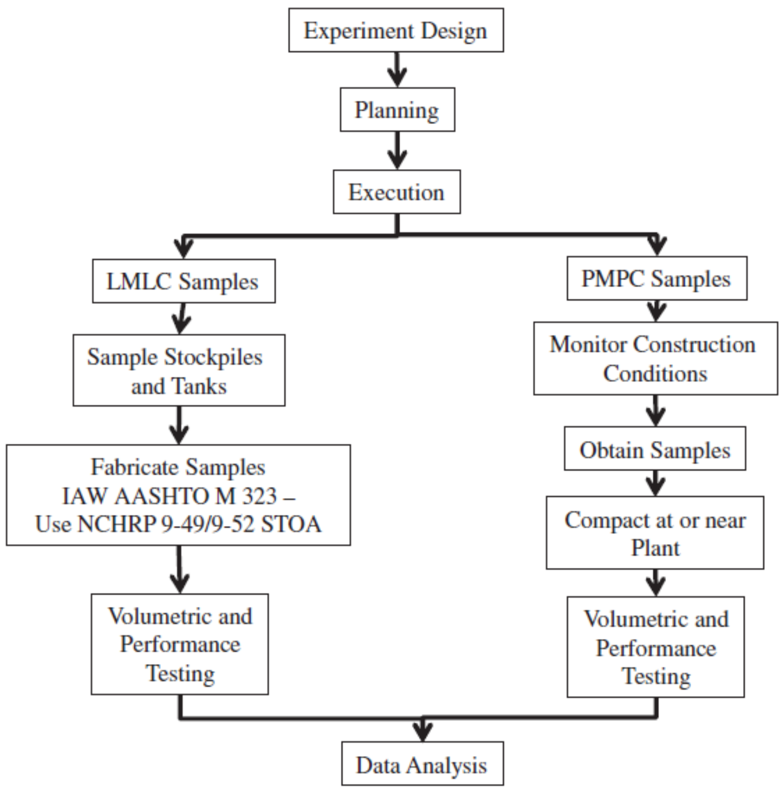


Figure 2.1 Short-Term Aging Flow Chart for Asphalt Mixtures (Newcomb et al., 2015)

Newcomb et al. (*2015*) evaluated LTOA similarly to STOA by analyzing production temperature, plant type, recycled material, and aggregate absorption parameters. They determined that including recycled material and the aggregate absorption and the inclusion of recycled materials affected the stiffness of long-term aged AC mixtures. Similar to STOA, no significant effect was observed on short-term aged AC mixtures regardless of plant type or production temperature. A significant increase in stiffness and rutting potential was reported for LTOA specimens with high absorption. They concluded that AASHTO R 30 long-term aging protocol reflected the first 1 to 3 years of in-service life of the pavement. Newcomb et al. (*2015*) validated the findings from Epps Martin et al. (*2014*) with similar behavior resulting from the LTOA protocols.

Long-Term Aging of Asphalt Concrete Mixtures

The issue of aging of asphalt binder has been recognized and investigated for almost a century (*Hubbard and Reeve, 1913; Thurston and Knowles, 1936; and Van Oort, 1956).* These previous studies confirmed that oxidation is responsible for changes in asphalt properties due to exposure to outdoor weathering. Subsequent studies have been conducted to develop aging protocols that can be implemented to condition laboratory specimens for performance characterization. Table 2.2 shows a summary of some of the relevant studies on the LTOA properties of asphalt mixtures. Two main aspects have been considered during the development and implementation of laboratory aging protocols: (1) state of material during aging (compacted specimen vs. loose mix) and (2) pressure level (oven aging vs. pressurized aging). These factors are discussed in more detail in the following sections.

Kim et al. (*2018*) deliberated on the long-term aging of asphalt mixtures to improve AASHTO R 30 protocols. They indicated that one of the biggest shortcomings of AASHTO R 30 was that only one temperature was used to simulate the various range of effects associated with different climatic zones. Kim et al. analyzed the effect of aging between loose mixtures and compacted specimens, performed a sensitivity analysis on the effects of the curing temperature, compared results from the oven aging against pressure aging, and observed the effects associated with the depth of the HMA or WMA layers, to propose a replacement of current long-term aging protocols. Asphalt mixture performance testing included the dynamic modulus tests. All HMA asphalt mixtures were STOA for 4 hrs at 275ºF (135ºC) while WMA underwent STOA for 2 hrs at 243ºF (117ºC). They selected long-term aging curing temperatures of 203ºF (95ºC) and 275ºF (135ºC) for a wide range of curing times that were dependent on the aging index properties (AIP) of field cores to match aging levels.

Table 2.2. Summary of Laboratory Long-Term Aging Protocols for Asphalt Mixtures

Reference	LTOA Conditions	Key Findings
Morian et al. (2011)	- Lab aging (3, 6 and 9 months at 60ºC) - Compacted specimens	- Increased mixture's dynamic complex modulus after LTOA - Binder source influenced the aging rate - Aggregate source had no major effect
Tarbox and Sias Daniel (2012)	- Lab aging of 2, 4 and 8 days at 85ºC - Compacted specimens from plant-produced mixtures	- Increased stiffness with LTOA regardless of mix characteristics - RAP mixes stiffen less than virgin mixes
Azari and Mohseni (2013)	- Lab aging of 2, 5 or/and 9 days at 85ºC - Compacted specimens	- Increased resistance to permanent deformation with LTOA - Relationship between results from STOA and LTOA protocols
Safaei et al. (2014)	- Lab aging of 2 and 8 days at 85ºC - Compacted specimens	- Good agreement between asphalt binder and mixture results - At extended LTOA, fatigue performance is negligible - HMA yields a higher modulus than WMA
Epps Martin et al. (2014)	- Lab aging of 1 to 16 weeks at 60ºC and 5 days for 85ºC - Compacted specimens	- Increased stiffness with higher temperatures and times for LTOA - After an extended aging time, WMA and HMA exhibit similar performance
Newcomb et al. (2015)	- Lab aging of 5 days at 85ºC and 2 weeks at 60ºC - Compacted specimens	- Mixture aging is more sensitive to aging temperature than to aging time
Rad et al. (2017)	- Lab aging at 95ºC and 135ºC - Loose mixtures	- Unaffected relationship between binder chemistry and rheology with 95ºC - Using 135ºC decreases dynamic modulus and fatigue resistance
Kim et al. (2018)	- Lab aging at 95ºC and 135ºC - Compacted specimens and loose mixture - Draft oven and PAV	- Current AASHTO R 30 practices induce an aging gradient - Loose mix state expedites aging - Loose mix aging at 95ºC is recommended for LTOA

Kim et al. (*2018*) reported that the aging of a loose mixture resulted in faster and more uniform aging when compared to compacted specimens. Increasing the curing temperature reduced the time needed for oven-curing the material. However, if the curing temperature exceeded 212ºF (100ºC) abnormal chemical reactions may occur that otherwise would not occur while the pavement is in service. A map of the US considering various climatic zones at specified depths (6 mm, 20mm and 50 mm) below the pavement surface and time of aging (4, 8 and 16 years) was proposed as part of the new AASHTO R 30 modification.

Similar to CDD proposed by Newcomb et al. (*2015*), Kim et al. developed the climatic aging index (CAI). They reported that the advantages of CAI over CDD were that CAI used pavement temperature as opposed to using air temperature and that CAI could estimate pavement temperature (hourly) at different depths. Equation 2.2 is used to determine the time required for oven curing to match field aging.

$$CAI = \sum_{i=1}^{N}[D\ x\ A\ x\ \exp\left(\frac{-E_a}{R\ x\ T_i}\right)/24] \qquad \textbf{(2.2)}$$

where *D* is the depth correction factor, *A* is the frequency factor, E_a is the activation energy (kJ/mol), *R* is the universal gas constant (kJ/mol-K) and T_i is the pavement temperature from the enhanced integrated climatic model (EICM) with respect to the target depth (K).

Kim et al. (*2018*) evaluated the influence of depth, frequency factor and activation energy on obtaining the CAI value. Table 2.3 presents the correction factors for *D*, *A* and E_a parameters. The correction factors influence the scatter of the data points and the laboratory curing duration. With the use of Equation 2.2 and the correction factors from Table 2.3, Kim et al. were able to calculate the CAI values for various locations around the US. It was concluded that the long-term aging of loose mix material expedited the aging with more uniformity than specimens that are compacted. Similar to previous studies cited earlier (*Arega et al. 2013*), they also reported that the compaction of specimens after the material has undergone LTOA did not affect the performance or compaction.

Table 2.3. CAI Correction Factors (*Kim et al., 2018)*

MIX ID	**Depth Correction Factor (*D*)**	**Arrhenius Equation, Pre-exponential Factor (*A*)**	**Arrhenius Equation, Activation Energy (E_a)**
Surface Layer (6 mm)	1.0000	1.4096	13.3121
20 mm Depth	0.4565	1.4096	13.3121
Deeper Layer (below 20 mm)	0.2967	1.4096	13.3121

Kim et al. (*2018*) proposed a standalone procedure for long-term conditioning by separating AASHTO R 30 short- and long-term conditions of HMA. They recommended a curing temperature of 200ºF ± 5ºF (95ºC ± 3 ºC) to condition the asphalt material in its loose state. They indicated that the duration of curing was dependent on age, climate, and depth. Their recommended specification will contain the US map with the time required to match either 4, 8, or 16 years of field aging for depth ranges of 6 mm, 20 mm, or 50 mm. Both studies bring suitable information for STOA and LTOA, validation of these protocols was performed as a preliminary assessment of current aging protocols.

Effect of STOA and LTOA on Stability and Durability of Asphalt Mixtures

Several researchers have studied the rutting potential of asphalt mixtures after subjecting asphalt mixtures to STOA and LTOA conditions. Houston et al. (*2005*) used the dynamic modulus test to determine the stiffening of asphalt mixtures due to long-term aging. A clear increasing trend in the stiffness of the mixtures was not found in that study. Azari (*2011*) investigated the effect of STOA and LTOA on the flow number (FN) test results. She observed an increase in FN with respect to the aging condition of the mixture, but the trend was not enough to propose an appropriate aging duration. Azari and Mohseni (*2013*) determined the effect of STOA and LTOA conditioning on the permanent deformation of asphalt mixtures utilizing the incremental repeated load permanent deformation (iRLPD) test. They implemented the minimum strain rate (MSR) parameter to quantify the change in stiffness of the asphalt mixtures after short-term aging and various long-term aging conditions. They proposed conducting the iRLPD test on plant-produced mixtures at two stages after production and before the mixture was placed down to determine better the rutting potential of asphalt mixtures.

Unlike rutting, the cracking susceptibility of asphalt mixtures increases as the mixture loses ductility due to aging. To accurately assess the cracking potential of asphalt mixtures in the most critical state (i.e., after long-term aging), appropriate protocols must be used to simulate long-term aging in the asphalt mixture. However, if the goal is to screen different asphalt mixtures based on their cracking resistance, a more effective strategy would be to utilize laboratory aging protocols (time and temperature combinations) that allow asphalt mixtures to reach a steady-state condition in the shortest and most practical time possible.

A limited number of studies have been conducted to investigate the effect of aging on the cracking potential of asphalt mixtures subjected to long-term aging conditions. Kim et al. (*2012*) evaluated the fatigue performance of asphalt mixtures at four different aging conditions including 135ºC for 4 hrs and 85ºC for 2, 4, and 8 days. Daniel (*2013*) investigated the aging potential of asphalt mixtures designed with different amounts of RAP under LTOA conditions. It was concluded that LTOA specimens yielded a higher stiffness value than STOA specimens. However, RAP content in the mixture did not increase mixture stiffness because RAP has a pre-aged asphalt binder that oxidizes at a much lower rate compared to virgin asphalt binder. Blankenship et al. (*2018*) investigated the influence of various long-term aging conditions for mixtures using the disk-shaped compact tension (DCT) test. They commented that the LTOA procedure from

AASHTO R 30 might not be harsh enough to simulate non-load-associated aging that a pavement accumulates over its service life.

State of Material (Compacted vs. Loose) During Aging

In a laboratory setting, aging compacted specimens is a matter of practical convenience to determine the performance of asphalt mixtures. Following the standard method AASHTO R 30, the short-term aged mixture is compacted and cut to specimen dimensions before placing the finished specimens into a forced draft oven for long-term aging. The long-term aging simulation is conducted at 85ºC for 120 hrs to represent presumably five to ten years of aging in the field. To simulate long-term aging, Bell et al. (*1994*) conditioned asphalt specimens in the oven for different lengths of time before conducting performance tests on laboratory long-term aged specimens and their corresponding field cores. Their recommendation for the long-term aging of compacted specimens was to age specimens at 85°C for 2, 4, and 8 days in the oven. The compacted specimens conditioned for 2 days at 85°C simulate 1-3 years of field aging whereas the 4- and 8-day yield 9-10 years. Harrigan (*2007*) documented a few potential limitations with the current standard aging method such as 1) using only one aging temperature to represent the long-term aging spectra, 2) recommending too wide of a range for design purposes, and 3) not considering the effect of air void content.

The long-term oven aging of compacted specimens leads to both radial and vertical oxidation gradients, which is a concern for performance testing because the properties throughout the specimen can vary (*Elwardany et al., 2016*). Although less common than aging compacted specimens, the laboratory aging of loose (uncompacted) asphalt mixture has been recommended in recent years to eliminate the aging gradient within specimens (e.g., *Arega et al. 2013, Partl et al. 2013, Mollenhauer and Mouillet 2011, Van den Bergh 2011, Reed 2010, Braham et al. 2009,* *Dukatz 2015, Elwardany et al. 2016*). Von Quintus (*1988*) aged loose mixture at 135°C in a forced draft oven for 8, 16, 24, and 36 hrs to simulate long-term field oxidation. Van den Bergh (*2009*) conducted an experimental program for aging loose mixes in the laboratory to replicate RAP material. Mollenhauer and Mouillet (*2011*) also conducted a study on the aging of loose mixtures to produce RAP materials. The properties of the binders extracted from laboratory-aged mixtures were compared to the properties of binders extracted from 11 to 12-year-old sections. They concluded that oxygen pressure could significantly reduce the aging time.

Rad et al. (*2017*) investigated the implications of aging loose asphalt mixtures at 135ºC. They performed FTIR and DSR testing to document the influence of the selected curing temperature. They concluded that long-term aging of a mixture at 135ºC negatively influenced the performance and determined aging temperatures at or below 95ºC as optimum.

Some advantages of aging loose mixtures over aging a compacted specimen are: (1) air and heat can easily circulate inside the loose asphalt mixture inducing uniform aging throughout the mix; (2) problems associated with compacted specimen integrity during laboratory aging may be reduced; and (3) the rate of oxidation may increase due to the larger area of the binder surface being exposed to oxygen (*Kim et al., 2018*) consequently reducing the time required to achieve a certain level of aging. On the other hand, it can be argued that compacting an aged loose mix results in different aggregate packing and internal structure, and consequently different performance as compared to aging a compacted test specimen. To address this concern, Arega et al. (2013) compared the internal structure of asphalt mixes compacted before and after long-term aging using X-ray tomography and reported that there was no significant difference in the internal structure of these mixes. They also pointed out that the rank order of fatigue cracking resistance of asphalt mixtures did not change significantly after long-term aging (*Arega et al., 2013*).

LAB MIXED LAB COMPACTED (LMLC) VS. PLANT MIXED LAB COMPACTED (PMLC) SPECIMENS

Different methods and approaches have been utilized to evaluate the performance of mixtures in a laboratory environment. Typically, testing of asphalt specimens in laboratories is conducted using laboratory-fabricated specimens as it is implicitly assumed that these specimens simulate plant conditions. However, recent studies have highlighted the differences in the rheology of laboratory- and plant-produced specimens. To best implement performance and simulation-based approaches, it is critical to understand the salient variances between differently produced specimens. The commonly used laboratory compacted specimens that were considered in this study include:

- LMLC: Specimens produced in the laboratory using methods that are intended to simulate the plant mixing conditions *(Kim et al. 2003)*.
- PMLC: Specimens produced in the plant but compacted in the laboratory by reheating the loose mix.

These two methods employ different handling, mixing, and compaction techniques, which can impact the properties of the specimens. Past research efforts in this area have suggested mixed results and the likelihood that lab-produced specimens could be stiffer than plant-produced specimens. One study assessed mixtures containing different proportions of RAP and RAS (*Johnson et al. 2010*). They observed that the dynamic moduli of PMLC specimens were lower than LMLC specimens. Xiao et al. (*2014)* assessed the binders of recovered samples from different mixtures and showed that the failure temperature of lab-produced mixtures was higher than that of plant-produced mixtures. However, McDaniel et al. *(2002)* performed frequency sweep tests on different specimens from various states and indicated that the stiffness of lab- and plant-produced mixtures were similar. Rahbar-Rastegar and Daniel *(2016)* compared the rheological characteristics of plant-produced and lab-produced specimens and the impact of different mixture variables. The mixtures produced in dissimilar conditions yielded samples with different material properties. Additionally, there were no systematic differences between the properties in either of the production conditions. Rather, the differences were more strongly influenced by binder grade, gradation etc. Yin et al. *(2015)* reported that current STOA criteria for HMA and WMA were sufficient to reproduce the changes seen in performance criteria and volumetric properties during plant production.

Influence of Pressure on Aging

The aging rate of asphalt mixtures can be accelerated by increasing the curing temperature during oven aging. This observation has led to proposals for the use of high curing temperatures such as 135ºC for loose mixtures (e.g., *Braham et al., 2009; Blankenship, 2015; Dukatz, 2015*). However, increasing curing temperatures for LTOA conditions may introduce inconsistencies in the chemical composition of the asphalt binder. The disruption of polar molecular association and subsequent sulfoxide decomposition become critical at temperatures that exceed 100ºC (e.g., *Herrington et al. 1994, Petersen 2009, Petersen and Glaser 2011, Glaser et al. 2013*). Rad et al. (*2017*) investigated the effect of two long-term aging temperatures, 95ºC and 135ºC, to age asphalt mixtures and determine their engineering properties based on dynamic modulus and damage resistance measured using the simplified viscoelastic continuum damage (S-VECD) model. They recommended an optimal aging temperature of 95ºC for loose mixtures. They indicated that the relationship between asphalt binder chemistry and rheology was unaffected if the aging temperature was below 95ºC.

As an alternative to oven aging, pressure and forced air can be used to increase the rate of aging in asphalt mixtures. Several researchers have tried pressure aging of asphalt mixtures for both compacted and loose mixture specimens. Von Quintus et al. (*1988*) performed a study on LTOA and long-term pressure aging of compacted specimens. The study consisted of LTOA compacted specimens at 60°C for two days following five days of conditioning at 107°C. Pressure aging was conducted at 60°C and 100 psi (690 kPa) for durations of five and ten days. They concluded that higher aging levels were reached for oven-aged specimens compared to the pressure-aged samples.

Bell et al. (*1994*) evaluated several pressure-aging systems as part of the SHRP project. They tried 'pressure oxidation' of compacted specimens using several pressure/temperature combinations in a pressurized vessel. Compacted samples were exposed to air or oxygen for 0, 2, or 7 days at pressures of 690 kPa or 2070 kPa and 25°C or 60°C. An unusual modulus trend was observed, i.e., as the oxidation level increased the modulus decreased.

Khalid and Walsh (*2000*) developed an accelerated pressure oven method to simulate the long-term aging of porous asphalt mixes. One of the main shortcomings of PAV aging compared to oven aging was the amount of material that could be aged simultaneously. To obtain uniform aging, a uniform thin layer of the loose mix should be placed in the PAV, which reduced the capacity of the instrument to around 1 kg (*Partl et al., 2013*). Partl et al. (*2013*) study suggested that binder oxidation continued up to nine days of conditioning, but that the rate of oxidation decreased with an increase in the duration of the conditioning.

Influence of Mix Design Variables on Aging of Asphalt Mixtures

Several variables influence the aging and performance of asphalt mixtures including mix type, binder source, performance grade of the binder, aggregate source, aggregate absorption, aggregate gradation, and recycled material and its content within the mixture.

Elliot et al. (*1991*) studied the effects of gradation on the creep stiffness, split tensile strength, resilient modulus, and air voids of asphalt mixtures. They concluded that the variations in gradation had the greatest effect, especially when the shape of the gradation curve changed. They indicated that the tensile strength was influenced more by the air void content and compaction than the variation in gradation. Abo-Quadis et al. (*2007*) evaluated different limits of American Society for Testing and Materials (ASTM) specifications for aggregate gradation. The open-graded gradation had the least amount of stripping resistance mostly due to the amount of air voids in the mix as the interlocking of the particles was poor. Guler (*2008*) found that gradation was the most influential design parameter for the mechanical properties (elastic modulus and yield stress) of HMA.

The type of aggregates utilized also affects the behavior of the mixture as different aggregates have different absorption levels. Abo-Quadis et al. (*2007*) found that unconditioned hot mix asphalt mixtures using limestone had better stripping resistance as opposed to basalt aggregates. However, they also found that when those hot mix asphalt samples were conditioned, the basalt mix showed better resistance than that of limestone. Braham et al. (*2007*) compared the performance of limestone and granite aggregates in HMA. They indicated that although granite provided higher fracture energy at low temperatures than limestone, a reverse trend was seen at higher temperatures. Benedetto et al. (*2014*) concluded that using basalt compared to limestone in the HMA with 20% RAP, yielded greater complex modulus and higher fatigue resistance.

Clyne et al. (*2008*) evaluated the effect of RAP in asphalt mixtures by varying the proportion and source of RAP in the mix. They determined that mixtures containing RAP had higher dynamic moduli compared to the mixes without RAP. Using the semicircular bend (SCB) test, they found that mixes containing 20% RAP demonstrated a higher fracture resistance at high and low temperatures compared to the mixes containing 40% RAP. Using single-sourced RAP and multi-sourced RAP had no effect on the dynamic moduli at low temperatures; however, they affected the dynamic moduli at elevated temperatures.

PG of binder used in the mix affects how asphalt mixtures react in different environmental conditions, as well as in the mix design in general. Abo-Quadis et al. (*2007*) found that higher adhesion was achieved when using 80/100 asphalt compared to 60/70 asphalt for HMA. Guler (*2008*) reported that the asphalt content of an HMA pavement was the second most critical variable in the design process when testing the yield stress of specimens after compaction. Varying PG also affects dynamic modulus. Clyne et al. (*2008*), concluded that a mixture with a softer binder had a higher dynamic modulus compared to a stiffer binder at low temperatures. However, it was also found that a stiffer binder resulted in higher dynamic moduli for mixes with and without RAP at high temperatures. Boriack et al. (*2014*) investigated the optimum binder content of an asphalt mixture with RAP. They found that adding 0.5% of binder to the mixtures containing 20% RAP improved the fatigue and rutting resistance but slightly decreased the stiffness. Moreover, they concluded that adding different quantities of binder to mixes with 40% RAP led to a decrease in both rutting and fatigue resistance. They also observed that the number of gyrations had to be adjusted to obtain the same air void targets, and the resistance to moisture damage of HMA with high RAP content (as high as 50%) dealt more with the compatibility of PG of the binder used with the RAP rather than the percentage of RAP in the mix. Benedetto et al. (*2014*) arrived at a similar conclusion that varying proportions of binder added to HMA mixtures with RAP, could potentially increase or decrease complex modulus and fatigue resistance depending on the performance grade of the binder used.

State of the Practice of DOTs for STOA and LTOA Curing

The current TxDOT design, production and construction specifications are described in Items 340 through 346A for all mixture types. Current TxDOT Test Procedure Tex-204-F provides general guidelines and steps to select the materials, determine the material proportions (e.g., aggregates and binder content) and evaluate the engineering properties (e.g., cracking and rutting potentials) of the asphalt mix design. TxDOT Test Procedure Tex-241-F is used to produce compacted specimens for performance tests. Table 2.4 illustrates the mixing temperatures based on the performance grade and type for lab-mixed specimens. Lab-mixed materials are prepared as per Test Procedure Tex-205-F for compaction, whereas plant-produced mixtures are handled following Test Procedure Tex-222-F.

Table 2.4. Mixing Temperatures by Grade and Type *(Tex-205-F, 2016)*

Type-Grade[1]	Asphalt Material Temp.	Mixing Temp.[2]
PG 70-28, PG 76-22	325ºF (163ºC)	325ºF (163ºC)
PG 64-28, PG 70-22	300ºF (149ºC)	300ºF (149ºC)
PG 64-22, PG 64-16	290ºF (143ºC)	290ºF (143ºC)
AC-3,5,10; PG 58-28, PG 58-22	275ºF (135ºC)	275ºF (135ºC)
RC-250	100ºF (38ºC)	165ºF (74ºC)
MC-250	100ºF (38ºC)	165ºF (74ºC)
MC-800	140ºF (60ºC)	190ºF (88ºC)
CMS-2	140ºF (60ºC)	235ºF (113ºC)
AES-300	140ºF (60ºC)	235ºF (113ºC)
Asphalt Rubber (A-R) Binder	325ºF (163ºC)	325ºF (163ºC)

1 *If using RAP or RAS and a substitute PG binder in lieu of the PG binder originally specified on the plans, defer to the originally specified binder grade when selecting the mixing temperature.*
2 *When using RAP or RAS, the mixing temperature may be increased up to 325°F to achieve adequate coating.*

Before compaction, the mixtures must be conditioned at a specified temperature and time. Table 2.5 presents the curing and compaction temperatures for asphalt mixtures. While the temperature is selected based on the asphalt PG, a curing time of 2 hrs ± 5 min is specified for both lab-produced and plant-produced mixtures. On the other hand, WMA mixtures are cured for 4 hrs at a lower temperature (usually ranging from 215ºF and 275ºF). For PMLC that requires reheating, a curing period of 1.5 hrs ± 5 min is recommended. The reheated materials should be mixed and split into specific specimen sample sizes where they are cured to reach their specified compaction temperature. As the use of RAP and RAS materials continues to increase, modifications to current procedures and protocols are being made to assure the proper short-term and long-term performance of the pavement is achieved.

Table 2.5. Curing and Compaction Temperatures (Tex-241-F, 2019)

Binder[1]	**Temperature[2]**
PG 58-28	250ºF (121ºC)
PG 64-22	250ºF (121ºC)
PG 64-28	275ºF (135ºC)
PG 70-22	275ºF (135ºC)
PG 70-28	300ºF (149ºC)
PG 76-22	300ºF (149ºC)
PG 76-28	300ºF (149ºC)
Asphalt Rubber (A-R) Binder	300ºF (149ºC)

Note: *Mixtures must be compacted at the selected compaction temperature within a tolerance of ± 5ºF (±3ºC)*
1 *If using RAP or RAS and a substitute PG binder in lieu of the PG binder originally specified on the plans, defer to the originally specified binder grade when selecting the mixing temperature.*
2 *When using RAP or RAS, the mixing temperature may be increased up to 325ºF to achieve adequate coating.*

Performance tests are conducted on specimens that are short-term oven aged regardless of the engineering property of interest, cracking susceptibility, or rutting resistance. While the aging induced through the STOA protocol can positively influence the cracking resistance of a mixture, the rutting potential of an asphalt mixture was impacted due to the stiffening behavior of the mix. Therefore, laboratory-curing protocols must be developed and implemented to determine accurately the early-age rutting and long-term cracking potentials of asphalt mixtures. For the cracking susceptibility, the change in engineering properties after the long-term performance is of great interest to avoid durability problems in the field during the service life of the pavement. The curing time and temperatures for assessing the rutting potential of an asphalt mixture must be representative and valid to prevent early-age rutting right after placement activities.

The temperature and time specified for conditioning are assumed to simulate partially the aging and binder absorption that happens during the production and compaction of an asphalt mixture. During the design process, TxDOT does not require measuring the performance of asphalt mixes after being subjected to long-term aging. The current practice of conducting performance tests on specimens after short-term aging conditions implicitly assumes that the relative performance of asphalt mixtures does not change after long-term aging. Although mixtures that exhibit acceptable engineering properties after short-term aging may still underperform in service, it is impractical to vary laboratory-curing conditions many times before producing specimens that closely simulate the field behavior of mixtures. Implementing optimized laboratory curing protocols in terms of long-term cracking and early-age rutting is paramount to determining the engineering properties of underperforming mixtures during the mix design process.

Several DOTs, including Wyoming, Florida, Nebraska, Indiana, and Virginia, currently follow the proposed STOA and LTOA curing as per AASHTO R 30. The key differences among various agencies are the mixing and compaction temperature ranges for the aging of the asphalt mixtures. The performance grade of the binder is the controlling factor for each temperature, with special consideration for using modified binders. Various other state DOTs around the United States have specifications to simulate STOA as well as LTOA that differ from AASHTO R 30. The DOTs' resources, environmental, and loading conditions heavily influence their specifications.

The Illinois Department of Transportation (IDOT) has commissioned several studies aimed at verifying and modifying the current AASHTO standards. Their modifications to AASHTO R

30 include the short- and long-term aging conditioning requirements for their volumetric mixture design. To assess the impact of short-term aging on performance, they require the HWT, Illinois Flexible Index (I-FIT, AASHTO TP-124), indirect tensile strength, and tensile strength ratio (TSR, AASHTO T-283) results. The effect of long-term aging is assessed by conducting an I-FIT test. IDOT performance tests are only applicable to laboratory-prepared loose mixtures. Plant-produced mixtures are only evaluated for quality control (QC) or quality assurance (QA) purposes.

Table 2.6 illustrates the time required for short-term aging for both HMA and WMA for IDOT’s selected performance tests. The aging process can take place either immediately after mixing but before compaction or after the mixture has cooled to room temperature. The curing period is influenced by the absorption of the aggregates. Low-absorptive aggregates are cured for 1 hr ± 5 min before compaction. High-absorptive aggregate with a combined absorption greater than 2.5% is cured for 2 hrs ± 5 min. The conditioning time before mixing is not considered as the time of mixing. IDOT specifications recommend compaction temperatures based on the performance grade of the binder identical to those temperatures provided in AASHTO R 30. The replicate I-FIT specimens are long-term aged for 3 days ± 1 hr at 200ºF ± 5ºF (95ºC ± 3ºC). The long-term aged specimens are cooled to room temperature, submerged into a water bath set at 77ºF ± 1ºF (25ºC ± 0.5ºC) for two hrs, and tested per AASHTO TP 124.

Table 2.6. Short-Term Conditioning Requirements *(IDOT Central Bureau of Materials, 2019)*

Short-Term Conditioning (Hrs)[1]						
	Lab-Produced Mix			**Plant-Produced Mix**		
Test Type	Volumetric	T 283	Hamburg / I-FIT	Volumetric	T 283	Hamburg / I-FIT
HMA	1 or 2	1 or 2	1 or 2	0	0	0
WMA	1 or 2	1 or 2	3 or 4	0	0	2

1 *When two different values are present within a single cell, the correct value is based on whether low or high absorption aggregates are used.*

The Minnesota Department of Transportation (MnDOT) has funded several studies to address the environmental aging of asphalt mixtures. Some of those studies have focused on binder oxidation by using tests such as the PAV, RTFO, bending beam rheometer (BBR) and DSR. MnDOT has modified AASHTO R 30 for STOA and LTOA of their asphalt mixtures. The mixing and compaction temperatures for the preparation of an asphalt mixture are based on the PG of the binder. MnDOT follows 4 hrs of conditioning to achieve short-term curing for LMLC specimens. MnDOT's curing temperature is specified at 290ºF ± 10ºF (143ºC ± 6ºC) to simulate the pre-compaction phase in the construction process. This curing temperature allows for proper absorption of the binder into the aggregates. For modified binders, the minimum compaction temperature should be 290ºF. However, a higher compaction temperature should be used if included in the work.

Colorado Department of Transportation (CDOT) follows CP-L 5112 and CP-L 5115, which are the procedures for performance testing with the HWT Test and preparing specimens with a Superpave gyratory compactor, respectively. Different sample preparation processes are provided for the LMLC and PMLC mixes. As shown in Table 2.7, CDOT recommends similar mixing and compaction temperatures as TxDOT. The performance grade of the binder is the controlling factor for the mixing and compaction temperature of the Superpave mixture(s).

Table 2.7. Laboratory Mixing and Compaction Temperatures *(CP-L 5115, 2016)*

Superpave Binder Grade	Lab Mixing Temperature	Lab Compaction Temperature
PG 58-28 & 58-34	310ºF (154ºC)	280ºF (138ºC)
PG 64-22 & 64-28	325ºF (163ºC)	300ºF (149ºC)
PG 70-28 & 76-28	325ºF (163ºC)	300ºF (149ºC)

Note: All Temperature in this table have a tolerance of ± 5ºF (± 3ºC)

Once the material for laboratory-produced specimens has been mixed, the mixture must undergo short-term aging in an oven at its respective compaction temperature for 2 hrs. CDOT recommends an increase in duration if it is known that the mixture in the field was exposed to higher temperatures. PMLC mixtures on the other hand follow a 3 hrs ± 0.5 hr of reheating at its respective compaction temperature. A noticeable difference between the process followed by CDOT and other DOTs is that they do not require stirring the mixture during the curing process to maintain uniform conditioning. This is likely because this step may be often skipped by laboratory technicians and to also reduce additional workload.

The California Department of Transportation (CALTRANS) follows their Test Procedure 304, for short-term aging of mixes. CALTRANS, similar to TxDOT, follows a range of mixing and compaction temperatures. However, CALTRANS allows the material to be kept in the oven until it reaches a workable temperature. Before compaction, the material must be mixed thoroughly at its respective temperature to allow for even conditioning. Typically, the material is oven cured for 2 hrs before mixing at a specified compaction temperature depending on the PG of the binder. They recommend short-term curing of 2 to 3 hrs at 295ºF ± 3ºF (146ºC ± 1.5ºC) for laboratory mixed specimens. Since plant mixes have endured the mixing process of short-term aging, 2 to 3 hrs of curing before compaction is required. The reheating of PMLC mixes should not exceed 3 hrs at a compaction temperature of 235ºF ± 3ºF (113ºC ± 1.5ºC). A compaction temperature of 305ºF ± 3ºF (152ºC ± 1.5ºC) is recommended for mixes with asphalt rubber binders.

Chapter 3: Preliminary Assessment of Existing Laboratory Aging Protocols

TxDOT Research Project 0-6658, a collaboration between The University of Texas at El Paso (UTEP) and Texas A&M Transportation Institute (TTI), resulted in a Data Storage System (DSS) that includes extensive pavement material properties and performance data for 115 test sections located throughout Texas. The collected laboratory, field and performance data, traffic, and section details have been compiled and stored in the DSS. The DSS includes new construction, overlay, rehabilitation, and seal coat sections. Since the initial stage of construction, placed materials (hot mix asphalt, binder, base, subbase, and subgrade) were collected and subjected to extensive laboratory testing and material characterization. These sections have been closely monitored (many continue to be monitored) for field performance and distress progression under in-service traffic conditions. The central purpose of assembling the DSS was to leverage the comprehensive data toward calibrating and validating mechanistic-empirical (M-E) design models. Such models include Flexible Pavement Design System (FPS), Texas M-E, Texas Overlay design system, and the AASHTO Pavement ME.

Figure 3.1 displays the DSS tool. That relational database possesses a plethora of information for the test sections, including the test section details, construction and QA/QC data, and material testing data for the hot mix asphalt (HMA), binder, base/base, and subgrade. Field testing and time-based distress history are also documented. Traffic and climate information is also included in the repository. The DSS tool is updated semi-annually through performance monitoring, adding field testing data, photographs/video, field core information and visual distress survey information. The DSS tool can presently be used as a state-level database for pavement performance diagnostics. The comprehensiveness of the DSS allows for investigating the short- and long-term performance of pavement layers based on the laboratory and field-derived material properties, while also considering the routine monitoring for damage and the documented inventory of pavement system characteristics.

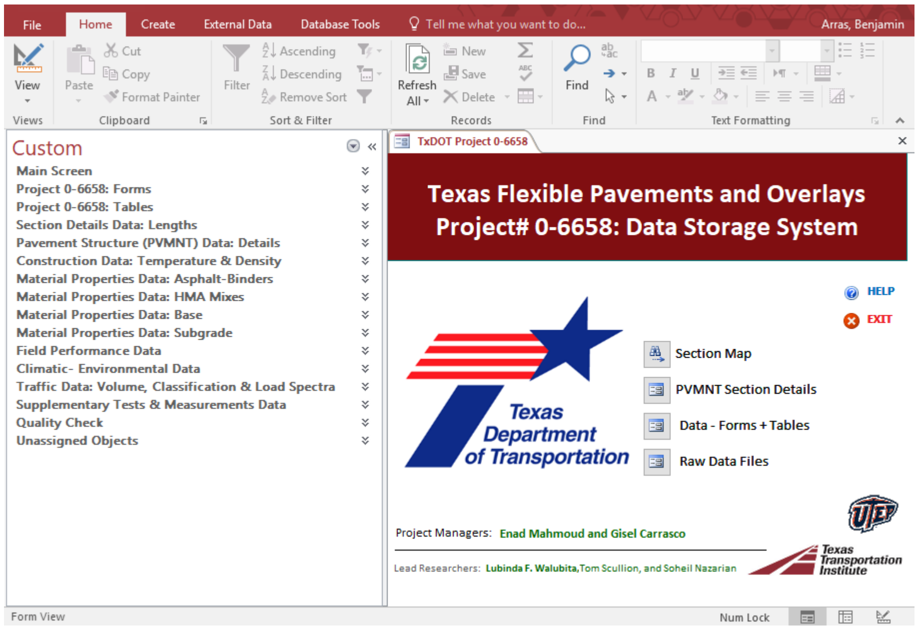


Figure 3.1. Database Storage System Graphical User Interface from TxDOT 0-6658

Figure 3.2 displays the distribution of test sections across Texas, as well as the number of test sections within each district. The selection of each section was based on the district location, climate zone, pavement structure configuration, and construction type (e.g., new construction, overlay, etc.). The DSS tool is also accompanied by global positioning system (GPS) coordinates and subsequent section mapping.

Figure 3.3 displays a schematic exhibiting the information stored for each test section in the DSS. The database contains various HMA and binder parameters of the AC layer(s) of each test section. The main relevant parameters to the scope of work of this study include surface course mix type, job mix formula (JMF), RAP and RAS information, aggregate type, asphalt content, additives information, and the short- and long-term field performance (i.e., with aging).

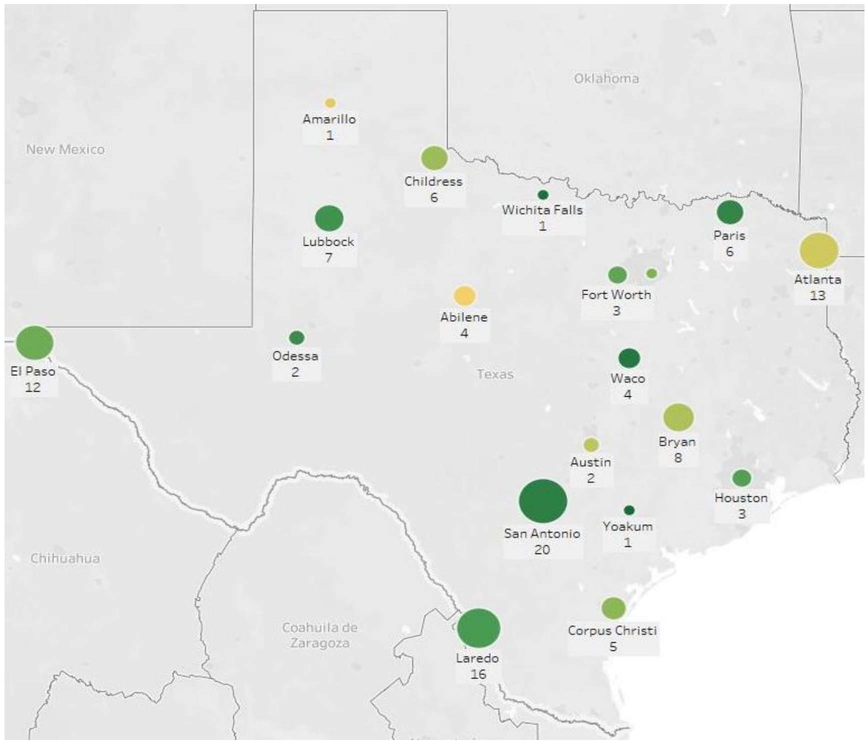


Figure 3.2. Test Locations in Texas from TxDOT 0-6658

Figure 3.3. DSS Information based on Research Project TxDOT 0-6658

The binder testing data includes results from the DSR, BBR, multi-stress creep recovery (MSCR), RTFO, specific gravity, viscosity, and PG of the binder. The laboratory performance data include HWT, OT, IDT, dynamic modulus (DM) and repeated load permanent deformation (RLPD) tests. The database also includes nondestructive testing (NDT) data from the falling weight deflectometer (FWD) and the ground penetrating radar (GPR).

Characteristics of AC Mixtures

The DSS information was analyzed to assess the impact that field aging had on the performance and service life of the test sections. Table 3.1 displays the characteristics of the test sections in the DSS by HMA item, surface course mix type, and sample size. The largest sample size was observed for dense graded mixtures (Item 341). The next largest sample size was for Stone Matrix Asphalt (SMA, Item 346). Surface courses comprising of Permeable Friction Course (PFC, Item 342), Performance Design mixes (Item 344), and Thin Overlay Mixes are also present for analysis.

Table 3.1. Hot Mix Asphalt Mixture Items and Types within TxDOT 0-6658 DSS

HMA Mix Group	HMA Mix Type	Total No. Mixes
341- Dense Graded HMA	Type B, Type C, Type D and Type F	76 Mixes
342- Permeable Friction Course	PFC	7 Mixes
344- Performance Design	SP-C and SP-D	6 Mixes
346- Stone Matrix Asphalt	SMA	14 Mixes
Thin Overlay Mixes	TOM	4 Mixes

The lowest asphalt contents (approximately 5%) were observed on the dense graded mixes. The highest asphalt contents (roughly 6.7%) were observed for the permeable friction coarse mixes. The Superpave (SP) and SMAs exhibited average asphalt contents of 5.5% and 6.2%, respectively. For the UTEP sections, different HMA mixtures contained from 0% to 23% RAP and 0 to 5% RAS content.

Figure 3.4 displays the sample size distributions of the test sections considering the facility type. The test sections within the DSS represent an array of facility types, consisting of U.S. highways, interstate highways (IH), farm-to-market roads (FM), state highways (SH), loops (LP) and spurs (SP). This bar chart is further delineated by the number of test sections managed by UTEP and those by TTI. The majority of the test sections within the DSS are on U.S. highways. Similar sample sizes are observed for test sections on IH, SH and FM roads.

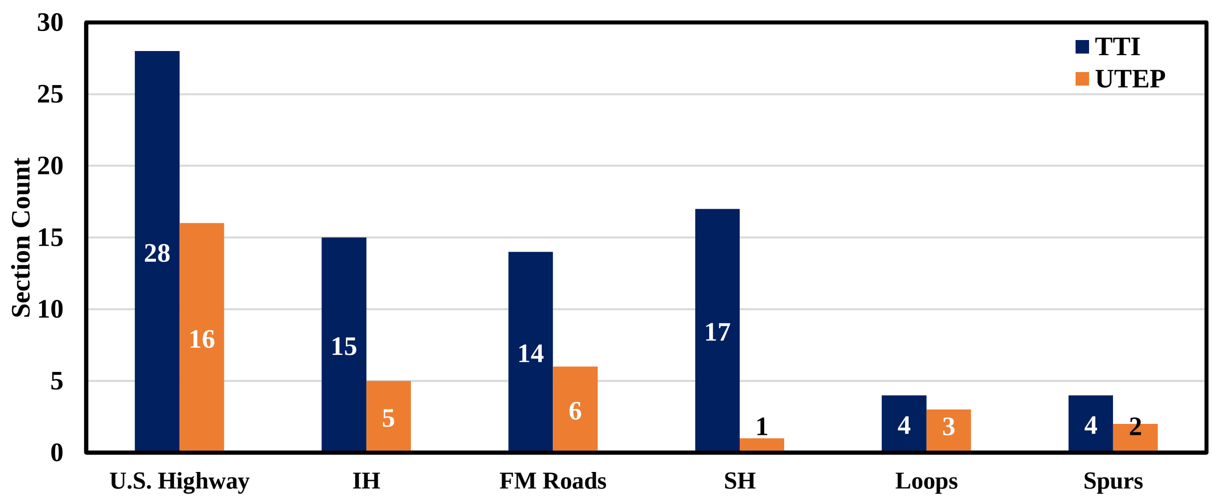


Figure 3.4. Section Distribution by Roadway Type from TxDOT 0-6658

Performance and Aging Conditions of Pavements

The DSS tool was used to analyze the performance of field-aged AC mixtures. A preliminary data analysis using the DSS rut depths from the HWT and the number of cycles to failure from the OT were evaluated to estimate the performance of AC mixtures. For the selected sections, the raw data were reanalyzed to determine the crack progression rate (CPR) and the critical fracture energy (CFE) for each mix. CPR rationally represents the rate of crack progression and CFE relates to the measures of crack initiation. The CPR and CFE have been shown to represent HMA cracking potential with a much lower coefficient of variation compared to the number of cycles (Garcia et al., *2016*).

The *short-term aging*, which should represent the cumulative effects of aging due to plant mixing, storing, as well as transporting and placing the mix in the field, typically positively impacts the rutting performance of a mix. Figure 3.5 displays the average rutting resistance indices (RRI) from 17 pavement sections from LMLC specimens. The bar chart illustrates the average rut depth of dense-graded (Type B and C), stone matrix asphalt (SMA), coarse matrix high binder (CMHB) and thin overlay mixes (TOM) while considering the different PG. The SMA and TOM mixes performed well against rutting.

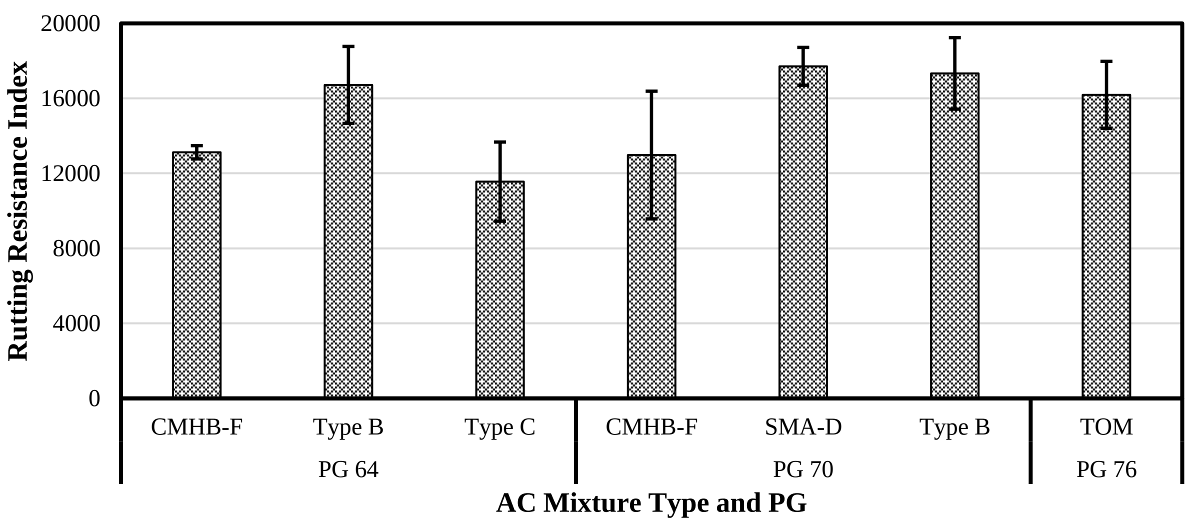


Figure 3.5. Hamburg Wheel-Tracking Test Comparison of AC Mixtures

The interaction plot shown in Figure 3.6, which is a cross plot of CPR and CFE, introduces the concept of the performance of the AC mixtures with respect to cracking. The *acceptance limit* *for CPR* is set at a value of 0.45 (*SS 3074, 2019*). Well-performing mixtures have been shown to have a CPR value lower than the acceptance limit, while poor-performing mixes exhibit values higher than the acceptance limit. CFE assesses the resistance of the mixture to crack initiation during the first loading cycle has a provisional upper limit of 3 and a lower limit of 1.

Figure 3.6 illustrates examples of two pavement sections with well- and poor-performing AC mixtures in cracking. The hollow circle that represents the initial evaluation of a well-performing mix exhibits a CPR of 0.34. The shaded circle represents CPR from a field core 44 months post-mat placement. This data point shows a CPR of 0.42; still indicating a well-performing mixture in terms of cracking. Figure 3.6 also shows the results from a pavement section that exhibited poor cracking performance in the field. The hollow triangle that represents the initial evaluation yields a CPR of 0.68. This mix would not have been accepted if the cracking criterion had been incorporated into the mixture design process. The solid triangle, with a CPR of 1.47, represents the CPR obtained on a core extracted 58 months post-mat placement. Overall, as the pavement ages, the cracking performance deteriorates.

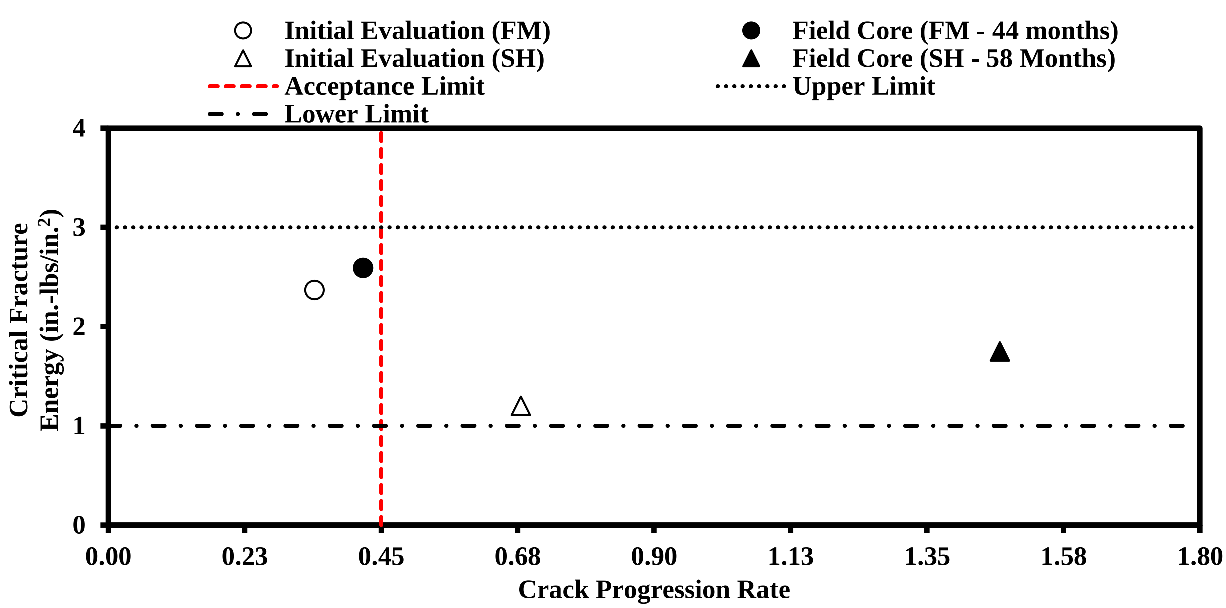


Figure 3.6. Interaction Diagram Plot for Cracking Performance

An evaluation was performed using data from the 17 sections from the DSS summarized in Table 3.2 to gain insight into the cracking performance of the AC mixtures as they aged. Based on the visual distress surveys three classes, namely, “good,” “marginal” and “poor,” were created to delineate the pavement performance with time. At that time, six of the sections were categorized as “good,” five as “marginal” and six as “poor.”

Figure 3.7a shows the initial laboratory cracking performance of the pavement sections. Seven sections failed the cracking performance criteria, whereas the remaining ten sections were within the acceptance limits. The sections that were categorized as crack susceptible were the same sections that showed “marginal” or “poor” field performance as well.

To evaluate the effect of aging over time, two sets of field cores were extracted and assessed for their respective CPR values at different ages. Figure 3.7b shows the cracking performance of the first set of cores that were extracted 2-3 years after construction. Most sections initially considered “marginal” and “poor” exhibited higher CPR values as compared to their initial results, indicating that they became more susceptible to cracking. Figure 3.7c shows the cracking performance from the second set of cores extracted from some of the sections. These cores exhibited even higher CPR values, which reinforce that as AC mixtures age they become more brittle and prone to cracking. A tendency of increasing CPR can be concluded as most of the initial “marginal” and “poor” sections have gone beyond the acceptance limit.

Figure 3.8 illustrates how the CPR value varies with time given different environmental conditions and pavement types. The number above each bar corresponds to the age of the pavement at the time of testing. A general trend of increase in CPR values with time can be observed for all pavement sections. In addition, the eight sections where a second set of cores were extracted exhibited even higher CPR values as compared to the first set of cores.

Table 3.2. Field Section Details for Cracking Performance Analysis from TxDOT 0-6658

ID	Material and Design Information					Pavement Section Information			
	Mix Type	Binder Grade	AC, %	RAP, %	NMAS, mm	Constructed Year	First Core	Second Core	Avg Daily Traffic
1	SMA-D	PG 70-28	6.3	20	12.5	2011	2014	-	3007
2	SMA-D	PG 70-28	6.0	0	12.5	2011	2014	-	612
3	SMA-D	PG 70-28	6.3	10	12.5	2011	2014	-	4837
4	SMA-D	PG 70-28	6.3	0	12.5	2012	2015	-	2103
5	SMA-D	PG 70-28	6.3	0	12.5	2011	2014	-	4600
6	SMA-D	PG 70-28	6.3	20	12.5	2012	2015	-	337
7	CMHB-F	PG 70-22	5.3	20	9.5	2013	2015	2020	579
8	CMHB-F	PG 70-22	5.3	20	9.5	2013	2015	2020	372
9	Type-C	PG 64-22	5.0	20	9.5	2013	2016	2020	343
10	CMHB-F	PG 70-22	5.0	20	9.5	2013	2015	-	-
11	Type-C	PG 64-22	5.0	20	19.0	2012	-	2020	3288
12	Type-C	PG 64-22	4.6	20	19.0	2011	2015	2020	1545
13	Type-C	PG 70-22	4.8	20	19.0	2011	2015	2020	4127
14	Type-C	PG 70-22	4.8	20	19.0	2012	2016	-	4270
15	TOM	PG 76-22	6.5	0	9.5	2012	2015	-	929
16	TOM	PG 76-22	6.5	0	9.5	2013	2016	2020	3952
17	TOM	PG 76-22	6.5	0	9.5	2013	2016	2020	2620

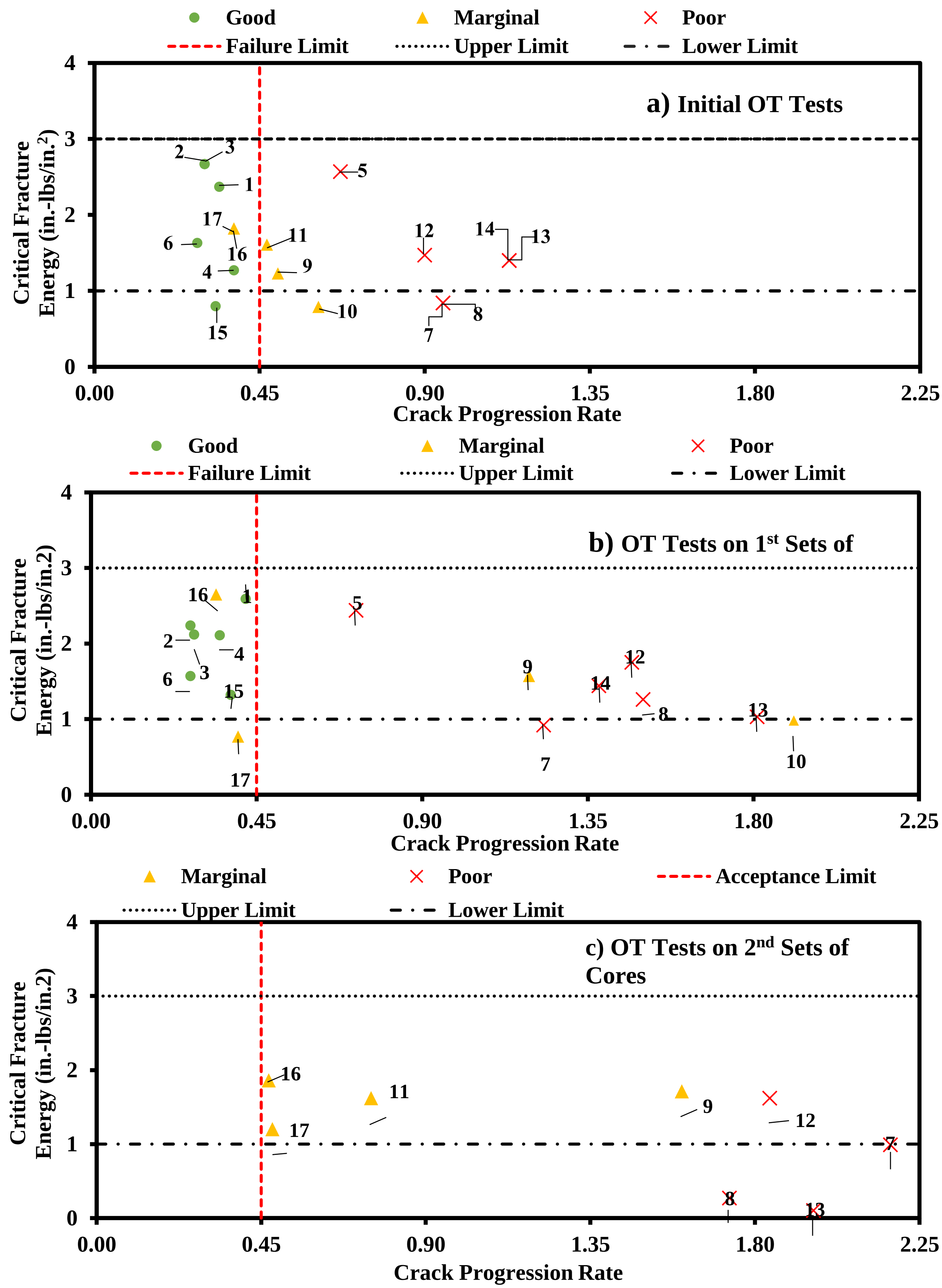


Figure 3.7. Mixture Cracking Performance Evaluation

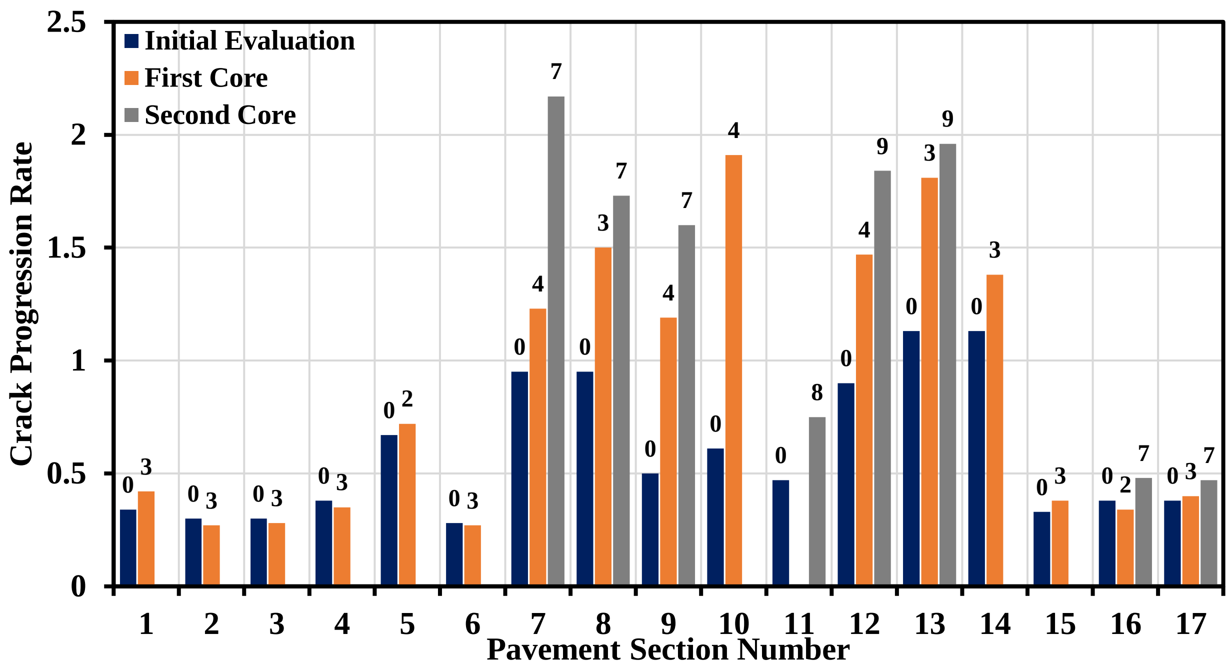


Figure 3.8. Mixture Cracking Performance Evaluation against Time

Influence of Short- and Long-Term Aging on Engineering Properties

Numerous studies have been conducted under the National Cooperative Highway Research Program (NCHRP) in relation to the objective of this study. The following relevant research projects were identified:

- NCHRP 09-36, *"Investigation of Short-Term Laboratory Aging of Neat and Modified Asphalt Binders."*
- NCHRP 09-52, *"Short-Term Laboratory Conditioning of Asphalt Mixtures."*
- NCHRP 09-54, *"Long-Term Aging of Asphalt Mixtures for Performance Testing and Prediction."*

Each of these reports has distinct objectives, but they include observations and information that are considered useful in the preparation of the experimental design plan for this project. Table 3.3 highlights the different types of testing conducted in each study, as well as the aging conditions used for the specified tests and whether they were developed to simulate short- or long-term aging.

Table 3.3. Summary of Key Findings from NCHRP Studies Related to Aging

NCHRP Project No.	Test Type	Final Recommendations
09-36	- SAFT - MGRF - Rotating Cylinder Aging Test (RCAT)	- 163°C for 45 minutes - 165°C for 210 minutes - 163°C for 235 minutes
09-52	- HWT - Resilient Modulus	- 135°C and 116°C for 2 hrs for HMA and WMA respectively
09-54	- DM - Oven vs. Pressure Aging - Cyclic Fatigue Test	- 95°C/135°C for 8.9 days/16.8 hrs - 95°C and 135°C - 85°C for 8 days

The following section aims to compare the results of the different tests used within each report and to focus on differences in aging and possible differences in various experimental factors that may impact replicability for future studies or other relevant procedures. Anderson and Bonaquist (*2012*) attempted to replace both RTFOT and the PAV with a single device that would simulate short- and long-term aging. They examined the SAFT and the MGRF under varying operating conditions. They found MGRF to be an acceptable, less expensive replacement as they produced comparable results with the same aging conditions. However, they also observed that MGRF averaged about 40% of the mass change that RTFOT caused for the specimens in their study. The SAFT showed little correlation to oven-aged mixtures and took twice the amount of time as the current PAV to show equivalent aging.

Newcomb et al. (*2015*) proposed changes to the AASHTO R 30 protocol by introducing the concept of cumulative degree-days as a metric to measure the relative extent of field aging. Correlation between LMLC and PMPC specimens indicated that STOA protocols for their study were largely able to simulate plant aging and asphalt absorption that occurs during plant production. The critical in-service time when WMA equaled HMA was 23,000 CDD; approximately equivalent to 17 months in service in warm climates, and 30 months in service in colder climates. Stiffness of WMA to equal initial stiffness of HMA was accomplished with field aging of 3,000 CDD; approximately equivalent to 2 months in warm climates and 3 months in colder climates.

Most recently, Kim et al. (*2018*) validated AASHTO R 30 and proposed a standalone protocol for long-term aging. They compared the results between oven aging and pressure aging and loose mixtures and compacted specimens. The tests that included the application of pressure in compacted specimen aging were found to expedite aging. The addition to aging of loose mixes also expedited oxidation but was found to be impractical as only 500g could be aged at a time. About 8 kg of the mix is needed for a Superpave gyratory-compacted specimen. A clear difference between the fatigue performances of the asphalt mixture aged at 95℃ to 135℃ was also presented. Long-term aging at temperatures of 135℃ should be avoided as they yield poor performance results.

MATERIAL AGING

Identifying factors that contribute to the deterioration of the performance of asphalt mixtures is one of the more important objectives of this investigation. Yin et al. (*2017*) concluded that nonuniform field aging occurs on pavements in the field as the surface layer aged more rapidly when compared to the bottom. Azari and Mohseni (*2013*) studied the short- and long-term rutting performance of different AC mixtures. They concluded that the rutting performance is significantly affected by the mixture age and determined that different AC mixtures age differently. Radziszewski (*2007*) analyzed different components of AC mixtures and evaluated them on their relation to permanent deformation, which can be an indicator of aging through rutting and cracking. That study created three lists, each classifying the level of resistance to rutting and creeping after no aging, short-term aging, and long-term aging as shown in Table 3.4.

Table 3.4. Summary of Key Findings Related to Material Aging (Radziszewski, 2007)

Aging	Most Resistance	Intermediate Resistance	Least Resistance
No aging	- Superpave - Porous asphalt mixture with 15% air voids	- AC with 17% of rubber-modified binder - MNU with Modbit 30B binder - AC with 7% plastomer-modified binder	- AC with 3 and of plastomer-modified binder - Asphalt concrete with 50/70 bitumen
Short-term aging	- SMA with 35/50 binder - Superpave	- MNU with Modbit 30B binder - AC with 5% plastomer-modified binder	- AC with 35/50 bitumen - AC with 50/70 bitumen
Long-term aging	- AC with 17% of rubber modified binder - SMA with 35/50 binders - Superpave	- AC with 5% plastomer-modified binder - MNU with 17% of rubber-modified - Porous asphalt mixture with 20% air voids	- AC with 50/70 bitumen - AC with 3% elastomer-modified binder - AC with 35/50 bitumen

As part of this study, the short- and long-term aging cracking performance of different AC mixtures across Texas was assessed using OT data. The standard practices to simulate the field aging of an AC mixture were done by curing the loose mixtures following (1) AASHTO R 30 for long-term oven aging (LTOA) and (2) Tex-206-F for short-term oven aging (STOA). The curing processes for LTOA comprised 5 days at 185ºF (85ºC). Curing for STOA is 2 hrs at its respective compaction temperature (based on TxDOT standards). Table 3.5 displays the relevant mix design information of the different AC mix designs across Texas.

Table 3.5. Summary of Pavement Sections

Mix No.	Mix Type	Original Binder	Asphalt Content (%)
1	DG-C	70-28	4.6
2	DG-C	70-28	4.6
3	SP-C	76-22	5.5
4	SP-C	76-22	5.0
5	DG-D	76-22	5.2
6	DG-D	64-22	5.2
7	DG-D	64-22	5.2
8	SP D	76-22	5.5
9	SP-D	70-22	5.3
10	SP-D	70-22	5.3
11	SP-D	64-22	5.4
12	SP-D	70-22	5.4

Districts that repeat have different mix designs (change in gradation/mix type/binder source).

Per AASHTO R 30, the loose mixture was cured at a maximum thickness of 2 in. to ensure even oven aging of the material. Figure 3.9 shows the CPR values obtained for LTOA and STOA specimens. A clear trend is observed where the LTOA materials exhibited higher CPR values as the material aged except for the three mixtures that showed a marginal increase. The remaining nine sections had an increase in CPR ranging from 20% to 363% when compared to their respective STOA specimens.

Formulation of Laboratory Aging Protocol and Performance Indices

The existing AASHTO R 30 standard and Kim et al. (*2018*) attempt to simulate the aging of the asphalt mixtures equivalent to a certain number of years in service. Given that the aging kinetics of binder reaches a steady-state condition after a certain amount of aging, it is hypothesized that there is a level of long-term aging after which the rank order of aging and consequently cracking characteristics of different binder-mixtures does not change. For a cracking test to distinguish accurately between crack-resistant and susceptible mixes, it is critical but adequate that mixes are aged to achieve this steady state rate even if the mixes do not reach a state that is equivalent to 10 or 20 years of field aging. Figure 3.10 illustrates an analysis of the aging kinetics for different binder mixtures and their respective performance (e.g., cracking potential) rankings at different aging levels based on the steady-state aging condition of asphalt mixtures and binders. This study utilized this concept to develop optimized laboratory aging protocols for mixtures.

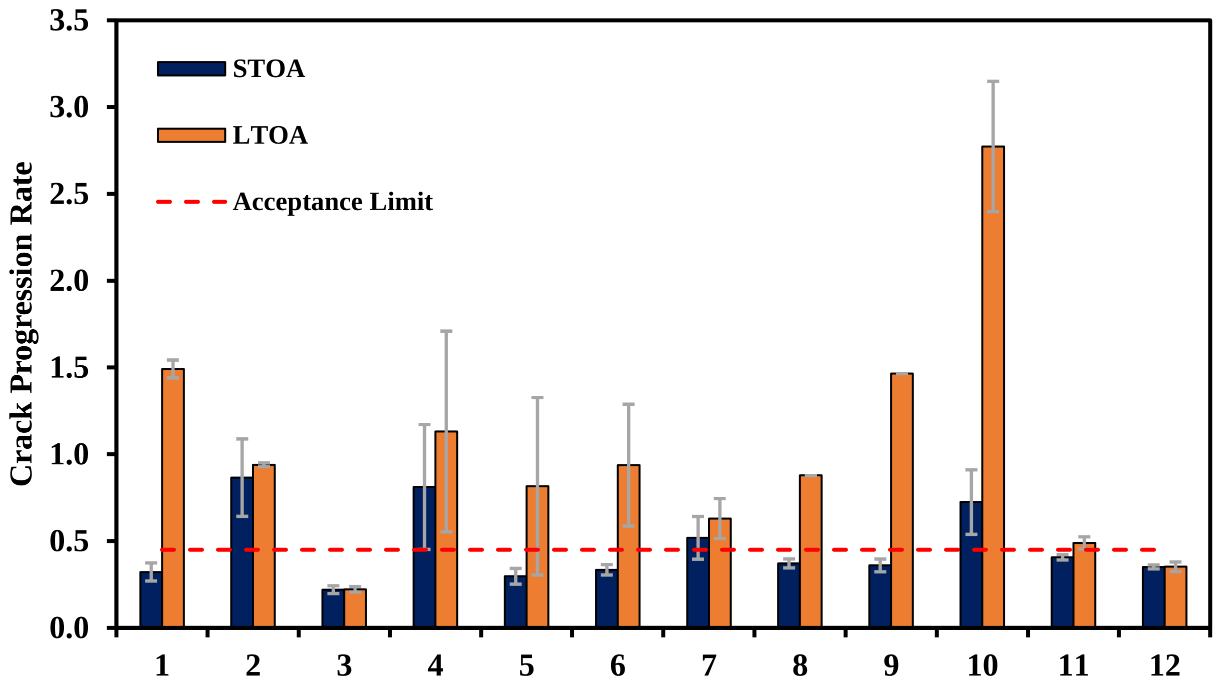


Figure 3.9. Crack Progression Rate Results for Aged Mixtures

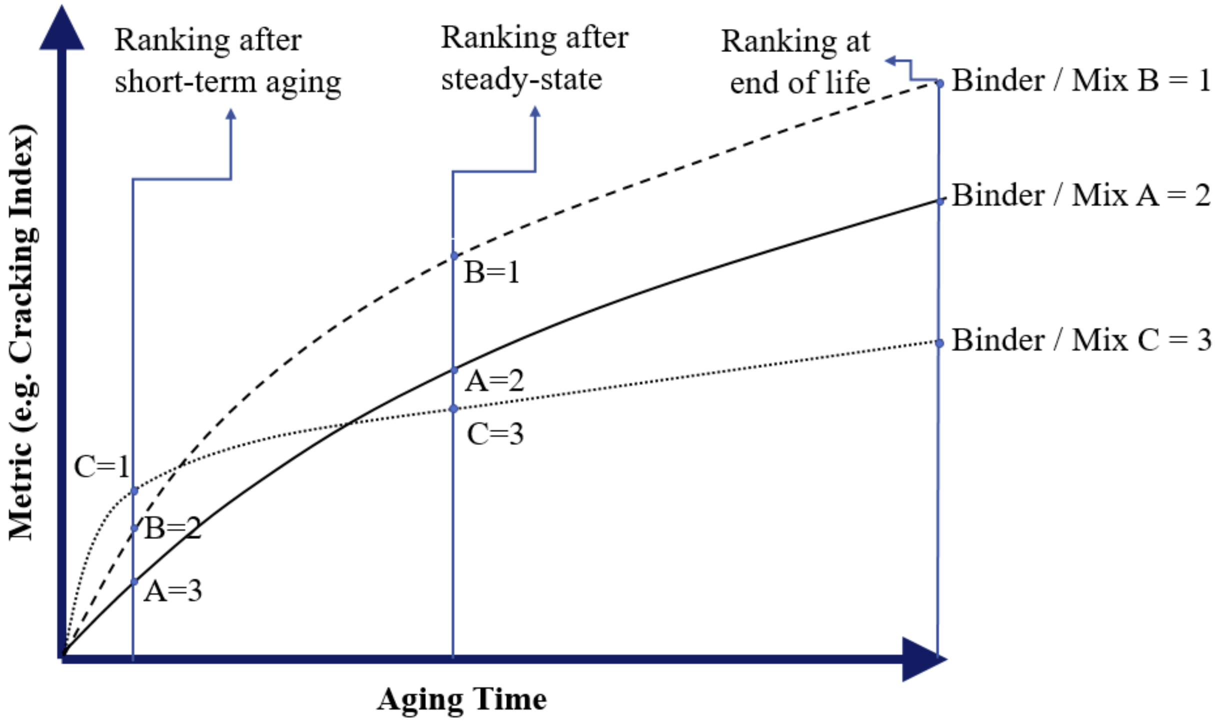


Figure 3.10. Representative Aging Kinetics of Asphalt Mixtures at Different Aging Levels

Three main factors that dictate the aging rate of any given binder are temperature, duration, and pressure. These factors were considered to select the best candidate aging protocols for further examination in the remainder of this study. AASHTO R 30 standard for traditional long-term aging conditions consists of curing compacted specimens at a temperature of 85ºC. Kim et al. (*2018*) compared the aging induced by oven-cured loose mixtures at 95ºC and 135ºC. They recommended a curing temperature of 95ºC for loose mixtures so that the chemical composition and integrity of the asphalt binder are not compromised. These studies point out that for any aging procedure to be realistic, it is important that the procedure not utilize a temperature above 95ºC.

Newcomb et al. (*2015*) documented that the curing conditions from AASHTO R 30 approximately simulate the aging conditions of pavements with 11 and 22 months in service for warmer and colder climates, respectively. Based on the literature search, there is no consistent approximation for the long-term aging prediction between the laboratory aging protocols and field aging of asphalt concrete layers. Kim et al. (*2018*) recommended aging durations based on project-specific and climate-based determination. Although the laboratory aging conditions recommended may consistently predict the aging conditions of asphalt mixtures in the field, the practices from NCHRP Project 09-54 are not optimal for routine mix design applications. Therefore, it is hypothesized that there is a point during long-term aging after which different binders in different mixtures reach a steady-state aging rate.

Metrics to Track and Quantify Aging and Impact of Aging on Performance

Based on prior results described above and previous literature, the following three approaches were used to assess the extent and impact of laboratory aging:

1. The optimum aging time was quantified based on the mixture performance results for both short- and long-term aging. In this study, actual field data and laboratory data obtained using the Kim et al. (2018) protocol were used as a benchmark for the long-term behavior of asphalt mixtures. The short-term aging condition was verified to determine if 2 hrs or 4 hrs would be optimum. Note that benchmarking using field data requires a longitudinal study that samples and measures aging for multiple years beyond the data provided.
2. The second metric is the minimum extent of aging after which the binder and mixture reach a steady-state condition and there is no change in the rate of change of rutting or cracking-resistance of the mixture. In other words, the stage after which the rank order of mixture performance does not change anymore, particularly in terms of cracking resistance.
3. An alternative method that uses all three parameters to accelerate the aging rate was explored. This method can be used to characterize the performance of mixes more accurately under long-term aging within a period of 24 to 48 hrs.

Chapter 4: Development of Experiment Design for Initial Evaluation

Various asphalt mixtures were selected and sampled to be included in a two-tier experimental design. The following parameters highlight the factors pertaining to selected asphalt mixtures that were recommended for consideration in the *initial evaluation* of aging processes:

- Frequency of use of a mix
- Pavement application
- Historical performance of the AC mixes
- Geological composition (e.g., hardness and absorption) of aggregates
- Type of asphalt binder and source
- Use and type of additives
- Availability of recycled materials

As the first candidate protocols (Protocol 1), similar to Kim et al. (2018), the AC mixtures were aged continually at 95ºC while periodically withdrawing specimens and evaluating them for their mixture performance characteristics. In addition, mixture performance was documented to determine when a steady state had been achieved. This information was then used to identify the shortest and optimal long-term aging time that can distinguish between good and poor cracking characteristics of asphalt mixtures. The process described in Protocol 1 was also used in Protocol 2 but at a higher temperature of 150ºC. While it was understood that this might not be a realistic approach to simulate aging in terms of chemical mechanisms, it might be useful to set applicable temperature-based benchmarks for the aging of mixtures. Figure 4.1 shows a sample of loose specimens in an oven used to conduct Protocol 1 and Protocol 2.

Figure 4.1. Loose Mixture Aging in Oven (*Protocol 1 and Protocol 2*)

The candidate protocols described above may still not result in laboratory aging protocols that achieve the steady-state aging condition for all mixes within a reasonable amount of time. In this case, the only other variable that can be manipulated to achieve steady-state aging in a reasonable amount of time is pressure. While the use of pressure combined with high temperatures is a standard procedure for asphalt binders, it was explored as a potential method (Protocol 3) for asphalt mixtures. A customized pressure chamber placed inside a heating oven for loose asphalt mixtures as shown in Figure 4.2 was used to implement this protocol. For this protocol, the temperature was held the same at 95ºC, and different combinations of pressure and aging duration were evaluated. To implement this protocol properly, preliminary tests were conducted to evaluate the effectiveness and optimize the conditions for this approach. Specimens were withdrawn and evaluated after 0, 2, 4, 8, 16, 32, 64, 128, and 256 hrs for Protocol 1. For Protocols 2 and 3, loose mixtures were withdrawn at 0, 2, 4, 8, 16, and 32 hrs.

Figure 4.2. Pressure Aging Chamber for Loose Mix (*Protocol 3*)

Performance Test Methods

The two main distress types associated with aging are early-age rutting and long-term cracking. The following test methods were used to assess the aging progress of AC mixtures (see **Appendix A** for a more details of the performance tests):

- Overlay Tester (Tex-248-F): The OT will assess AC mixture susceptibility to fatigue or reflective cracking. There are two major contributing factors to the mixture's cracking resistance, the critical fracture energy (CFE), and the crack progression rate (CPR). The critical fracture energy is known as the energy that is necessary to start a crack from the bottom of the specimen at the first loading cycle. CPR is known as the process in which the specimen will undergo loading cycles in the OT that allows for propagation of the crack.
- IDEAL-Cracking Test (Tex-250-F): The IDEAL-CT will estimate the stiffness properties, tensile strength, and the cracking tolerance index (CT_{Index}) of AC mixture specimens of the HMA mixtures.
- Hamburg Wheel-Tracking Test (Tex-242-F): The HWT test was adopted in this study to evaluate the rutting susceptibility and moisture damage of AC mixtures.

As shown in Figure 4.3, two typical Superpave (SP) mixtures from El Paso were selected as part of the initial evaluation phase due to the travel constraints caused by the pandemic. Both the LMLC and PMLC specimens were considered for the first mixture, while only LMLC specimens were used for the second mixture.

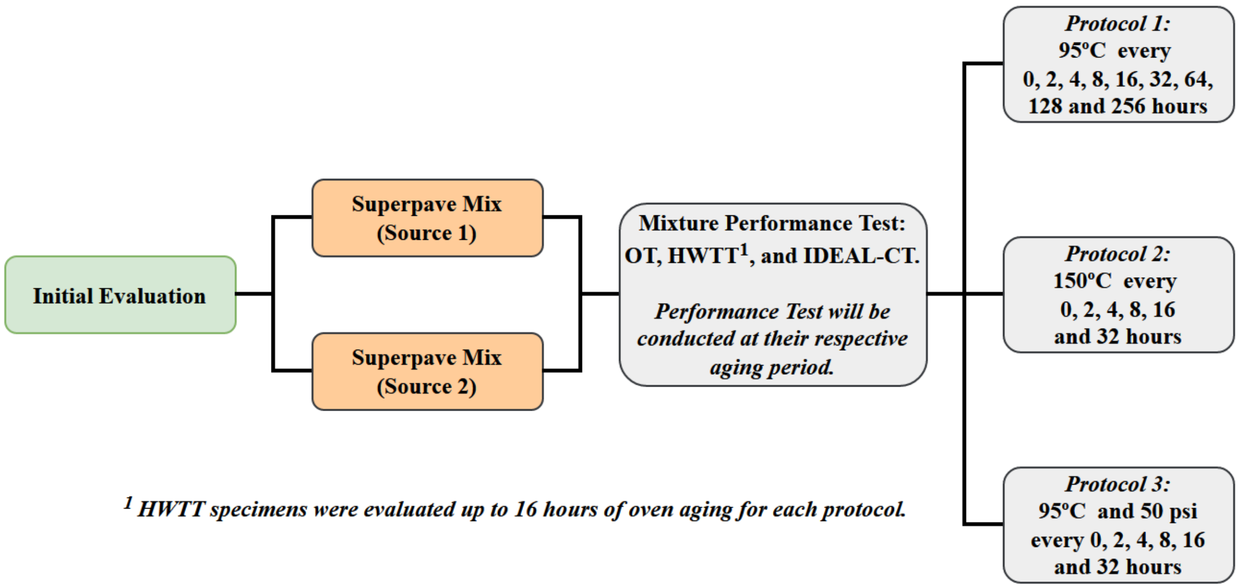


Figure 4.3. Initial Evaluation Experimental Design Plan

Chapter 5: Initial Evaluation of Aging Potential

The particle size distribution of Mix 1 and Mix 2 are shown in Figure 5.1. Mix 1 had finer gradation and contained 1.5% more RAP asphalt binder than Mix 2. The same gradation was used to prepare all specimens for each mix.

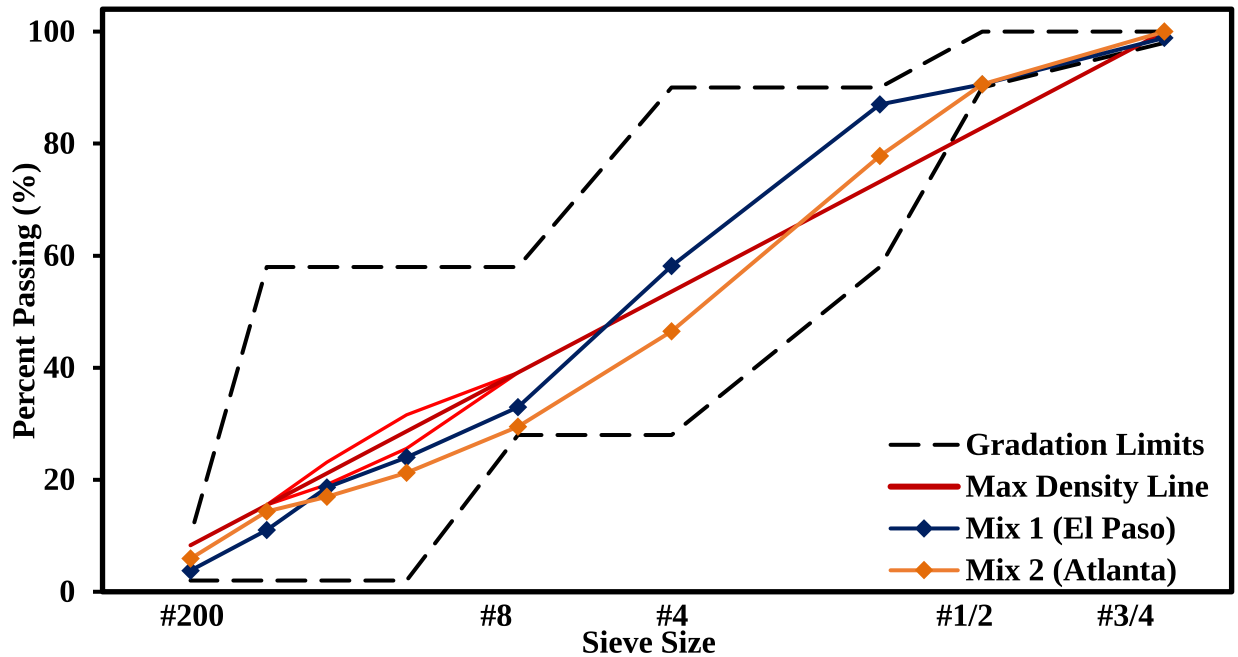


Figure 5.1. SP-C Mixture Aggregate Gradations

The volumetric properties such as optimum asphalt content (OAC), voids in mineral aggregate (VMA), RAP asphalt content, and dust/binder ratio of the AC mixes are summarized in Table 5.1. The AC mixes are both Superpave C mixtures designed with a Superpave gyratory compactor to meet a 96% target density according to Tex-241-F. The asphalt binder used was PG 70-22 for Mix 1 and PG 76-22 for Mix 2. The requirement of a 15% minimum VMA for Superpave C mixtures was met for both mixes. The average asphalt contents from four replicate samples matched the values reported for both mixes. The dust-to-binder ratio ranged from 0.6 to 1.6, that met the required limits. The RAP content for both mixes was 10% of the total mix.

Table 5.1. Volumetric Properties of AC Mixes

Mix No.	OAC (%)	VMA (%)	RAP Asphalt Content (%)	Dust/Asphalt Ratio
1	5.4	16.5	4.3	0.7
2	5.4	16.6	5.8	1.1

The results from the LMLC and PMLC specimens for Mix 1 and LMLC specimens for Mix 2 are compared in the following sections. Nine-time intervals (0 hrs, 2 hrs, 4 hrs, 8 hrs, 16 hrs, 32 hrs, 64 hrs, 128 hrs, and 256 hrs) were chosen for Mix 1 to illustrate the gradual increase in aging metrics for the initial evaluation. The time intervals for Mix 2 were shortened based on the results from Mix 1. An additional time interval of 24 hrs was added as preliminary data showed a steady-state trend on the AC tests.

Protocol 1

The normalized rutting resistance index (NRRI) from the HWT tests for Mix 1 (LMLC and PMLC) and Mix 2 (LMLC) specimens as a function of aging duration are presented in Figures 5.2 and 5.3. The rutting resistance improved with the increase in the aging period regardless of mixture type (LMLC or PMLC). Since aging the material for more than 2 hrs does not result in a notable change in the NRRI values, future work focused on early age (between 0 to 2 hrs). The differences between the PMLC and LMLC specimens' mixture performance are believed to come from differences in the length of time the mix is maintained in silos or the mixing temperature.

The OT CFE variations with aging duration are presented in Figures 5.4 and 5.5 for Mixes 1 and 2, respectively. Triplicate specimens were tested to demonstrate the repeatability of the test. The current upper and lower CFE limits of 1 and 3 lb.-in./in.^{2} advocated by TxDOT were considered for the preliminary evaluation. The standard deviations of the results that are shown as error bars are within ±0.4 for both LMLC mixes and ±0.6 for the PMLC specimens. The highest coefficient of variation (COV) was 16%. The CFE values for both the LMLC and PMLC specimens steadily increased with the aging of the mixes and were within the acceptance limit for up to 32 hrs of oven aging for Mix 1. Similarly, Mix 2 had a steady increase in CFE value until 8 hrs of aging. The increase in CFE value from 8 to 64 hrs of aging shows the mixture reached a steady state.

The variations in the OT-CPR values with the aging period are presented in Figures 5.6 and 5.7. The COVs ranged from 2% to 10% for the two LMLC mixes and from 3% to 15% for the PMLC specimens. Similar to the CFE values, the CPR values increased steadily and exceeded the acceptance limit of 0.45 after 32 to 64 hrs of oven aging.

The variations in the IDEAL CT indices obtained from triplicate specimens with the aging period are presented in Figures 5.8 and 5.9. The COVs ranged from 7% to 19% for the two LMLC mixes and 6% to 19% for the PMLC specimens. The current acceptance criterion for the CT_{Index} is 80 or greater. The CT indices from both the LMLC and PMLC specimens fell below the acceptance limit after 4 to 8 hrs of oven aging for Mix 1. However, the values fell below the acceptance criteria after 24 hrs of aging for Mix 2. For Mix 1, the LMLC specimens had lower CT_{Index} values compared to the PMLC specimens. The difference between the mixes can be attributed to the original PG of the binder used in the mixes. Mixes with excessive aging could not maintain their solid structure under the compressive load and showed fair tensile strength.

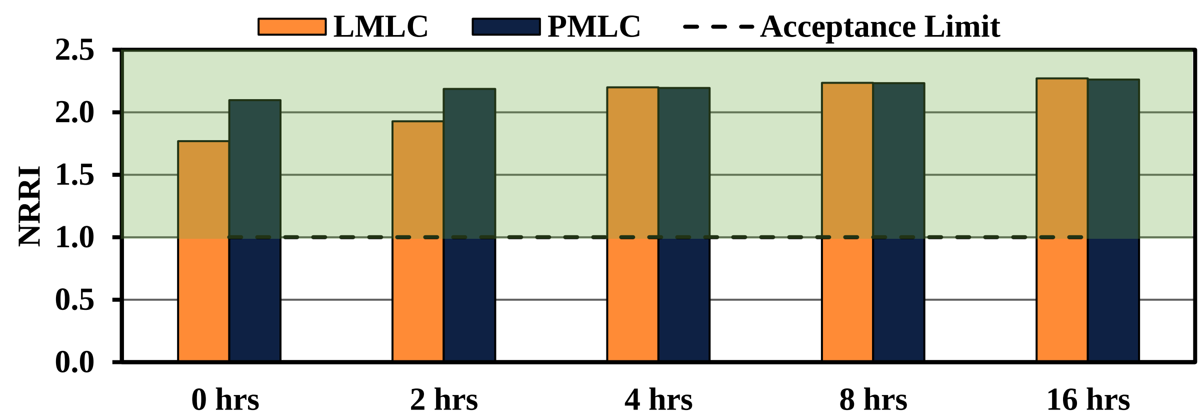


Figure 5.2. HWTT Results for Mix 1 Using Protocol 1

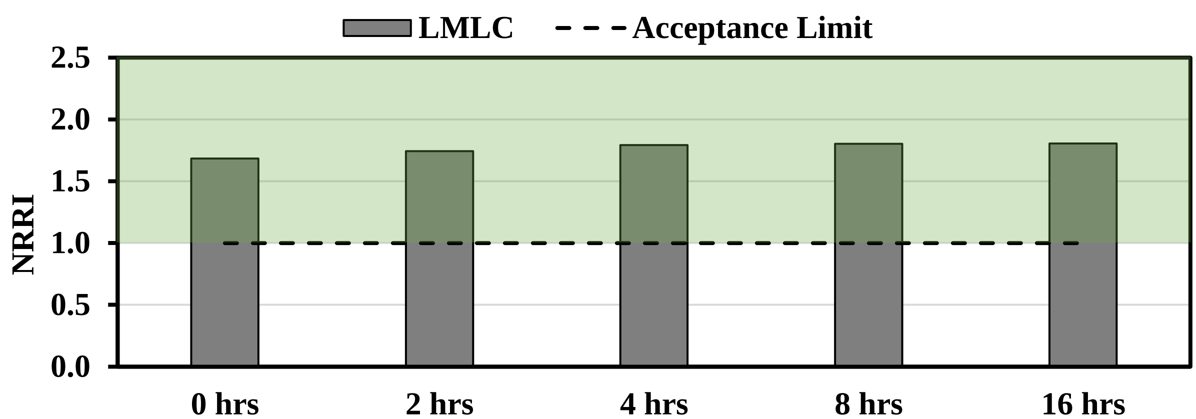


Figure 5.3. HWTT Results for Mix 2 Using Protocol 1

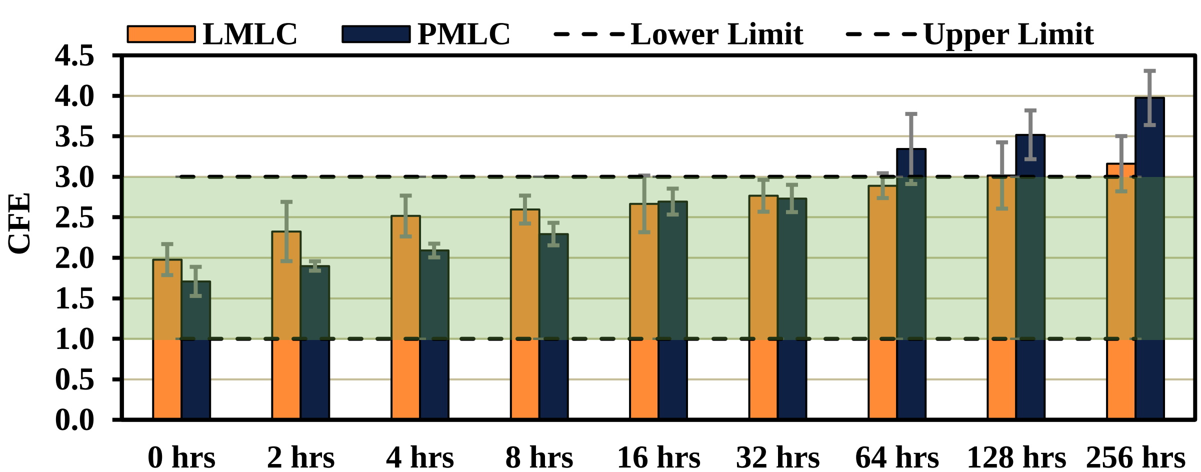


Figure 5.4. OT Critical Fracture Energy Results for Mix 1 Using Protocol 1

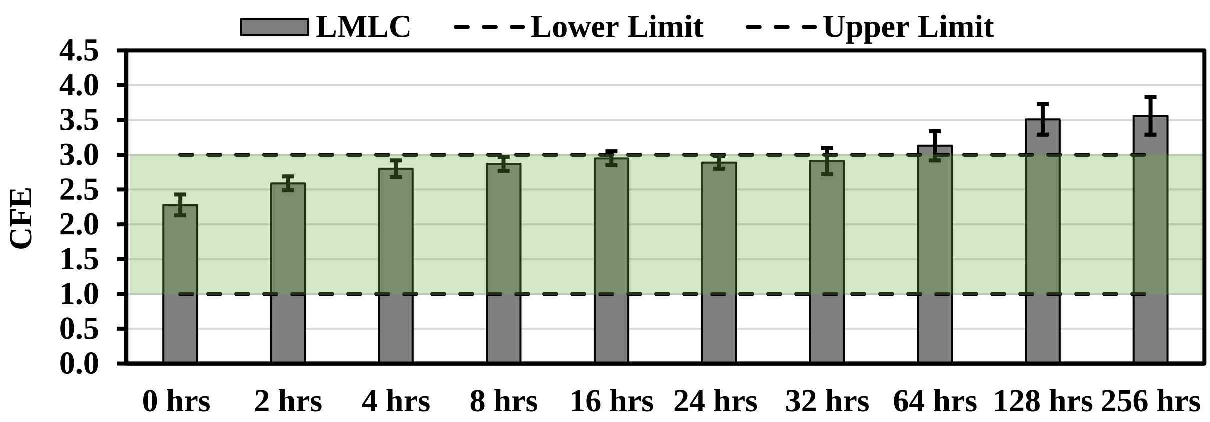


Figure 5.5. OT Critical Fracture Energy Results for Mix 2 Using Protocol 1

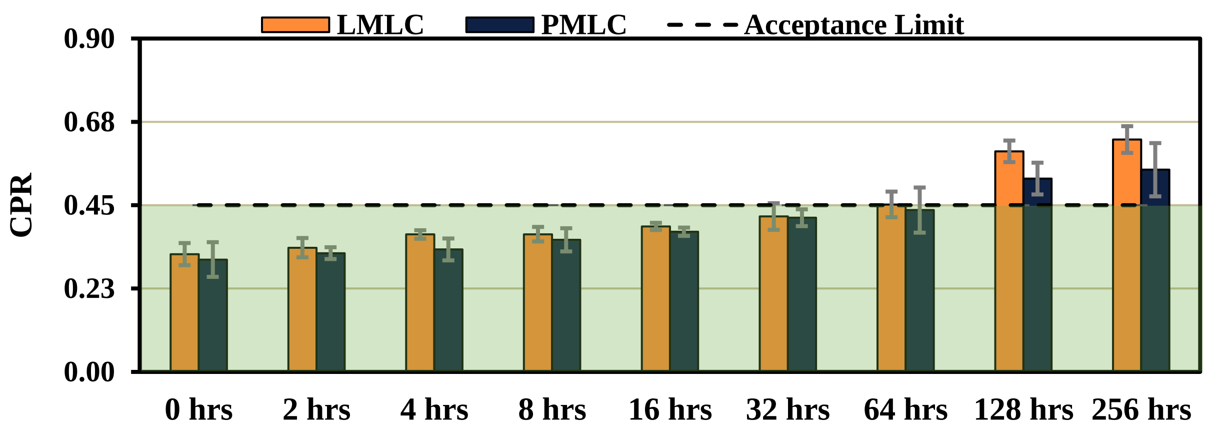


Figure 5.6. OT Crack Progression Results for Mix 1 Using Protocol 1

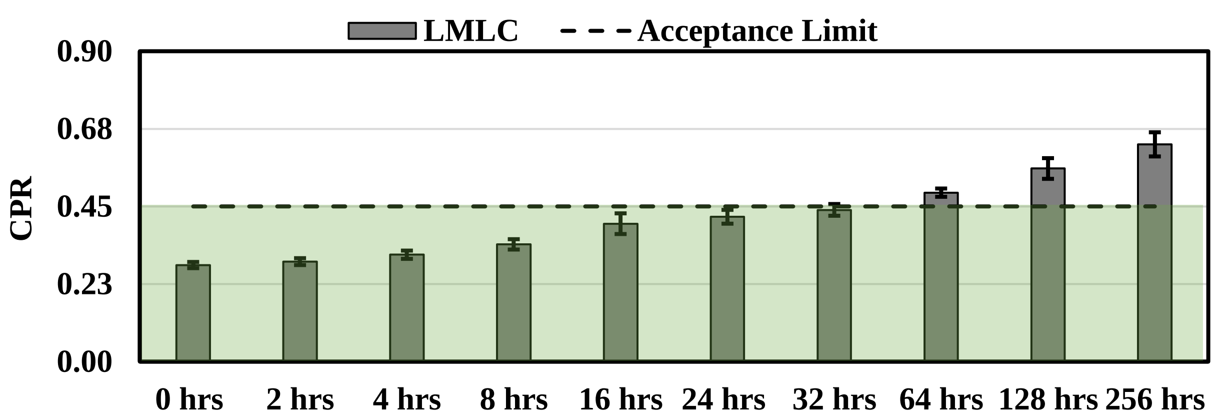


Figure 5.7. OT Crack Progression Results for Mix 2 Using Protocol 1

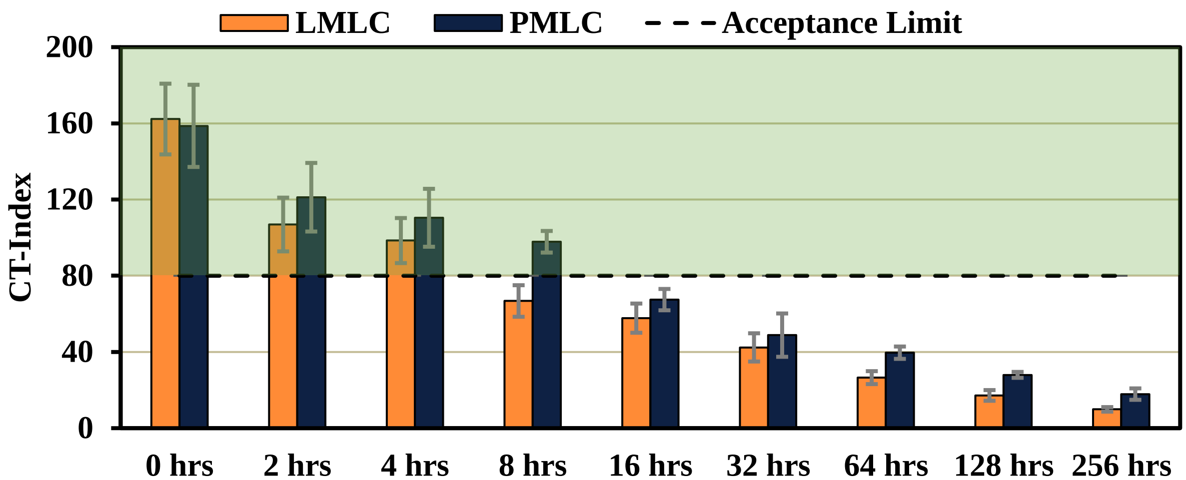


Figure 5.8. CT_{Index} Results for Mix 1 Using Protocol 1

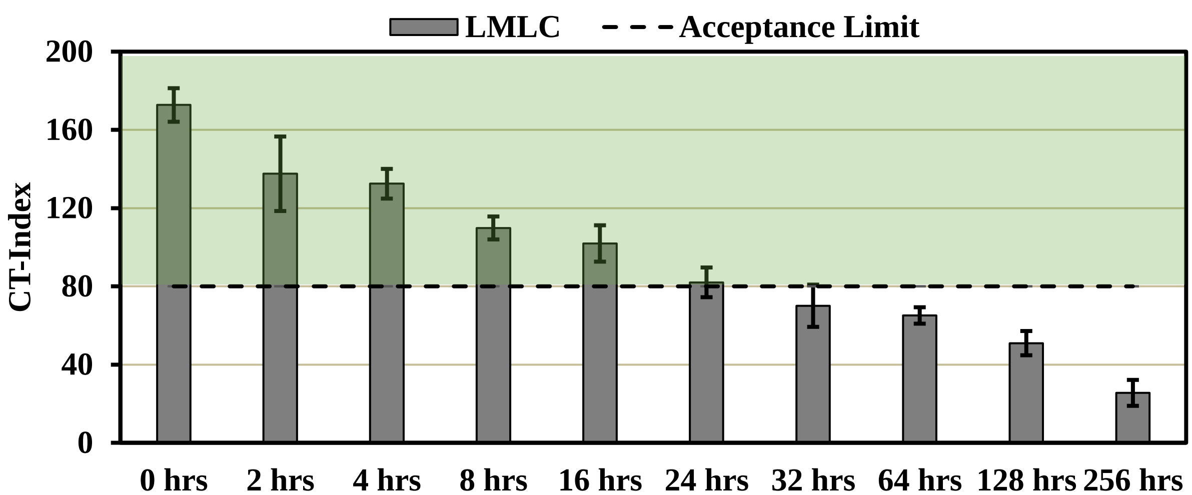


Figure 5.9. CT_{Index} Results for Mix 2 Using Protocol 1

The variations in the IDT strength with the aging period are presented in Figures 5.10 and 5.11. The COVs ranged from 3% to 10% for the two LMLC mixes and from 3% to 9% for the PMLC specimens. Considering the current TxDOT's lower and upper acceptance limits of 80 and 200, the mixtures exhibit a pattern similar to the OT results where the IDT tensile strength steadily increased and exceeded the limit after 32 to 64 hrs of aging.

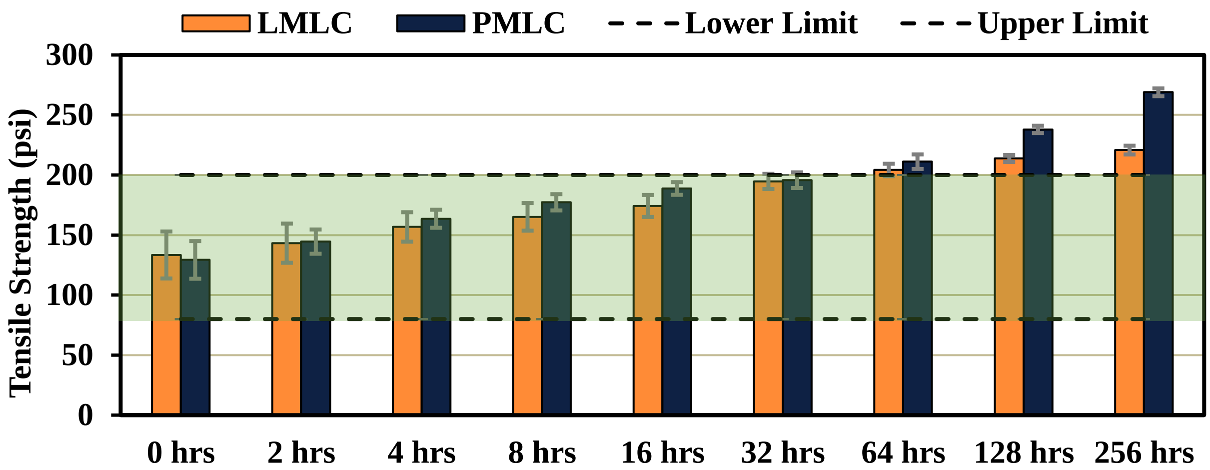


Figure 5.10. Indirect Tensile Strength Results for Mix 1 Using Protocol 1

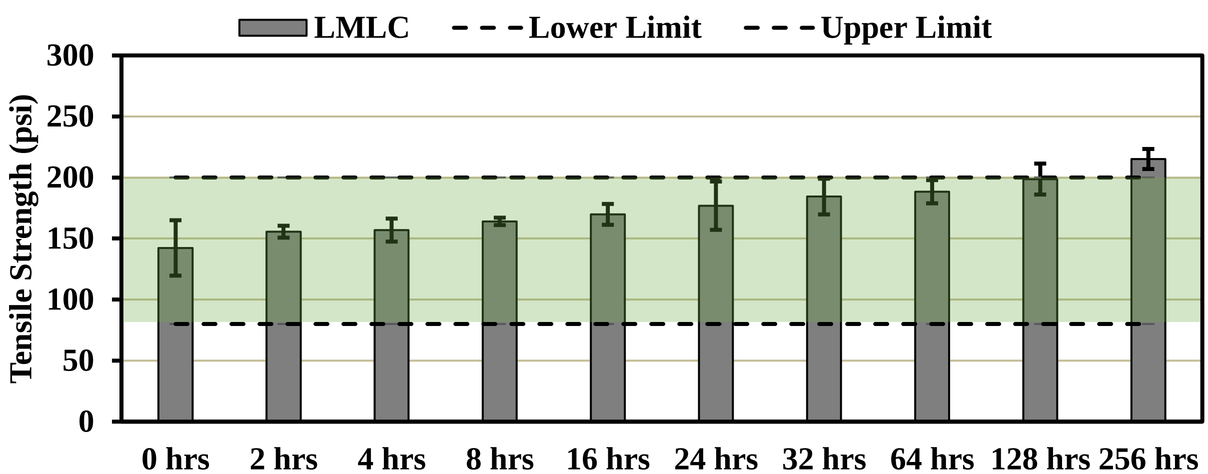


Figure 5.11. Indirect Tensile Strength Results for Mix 2 Using Protocol 1

Protocol 2

Given the increase in temperature to 150ºC, the aging durations were reduced to a maximum of 32 hrs. Even though it is known the oxidation of the binder could be compromised at higher temperatures, the results are useful to set oxidation benchmarks and evaluate the impact an increase in temperature would have on AC mixtures. The variations in NRRI from the HWT tests with the aging period are illustrated in Figures 5.12 and 5.13. Similar to Protocol 1, the mixture performance improved with the increase in the aging period. After the recommended 2 hrs of oven aging, the NRRI values are reasonably constant.

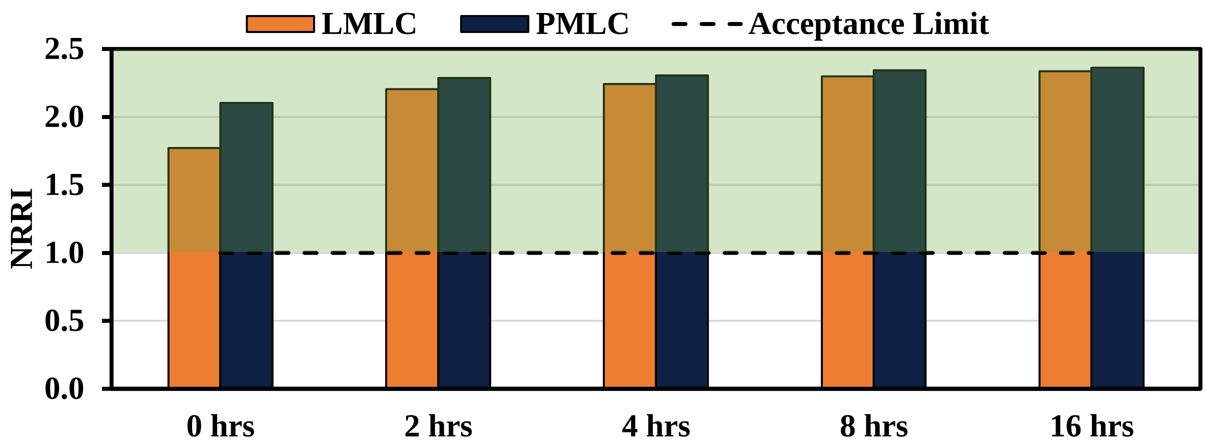


Figure 5.12. HWTT Results for Mix 1 Using Protocol 2

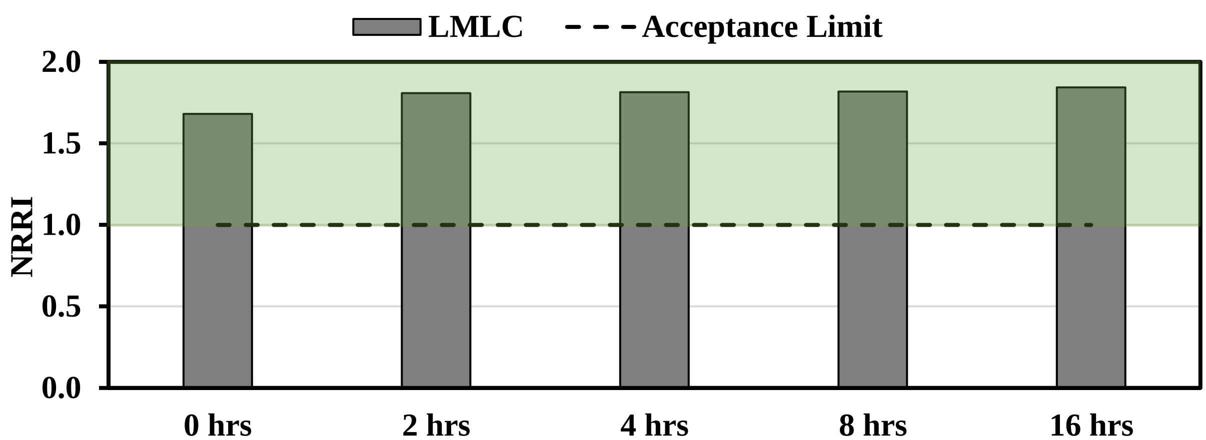


Figure 5.13. HWTT Results for Mix 2 Using Protocol 2

Figures 5.14 and 5.15 show the CFE values for Mix 1 and Mix 2, respectively. The upper acceptance limit was exceeded after 2 hrs of oven aging, much faster than the 64 hrs observed for Protocol 1.

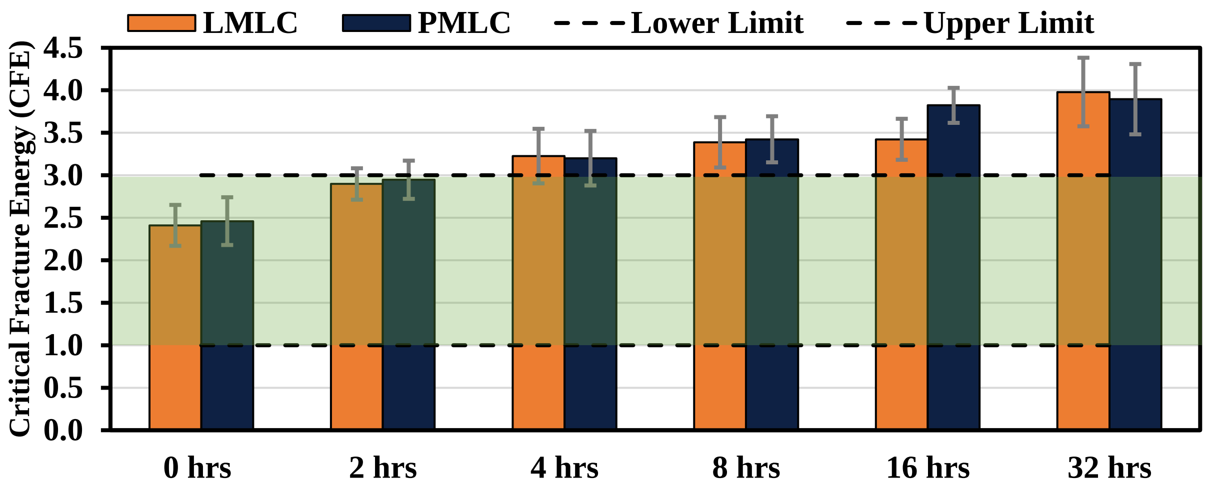


Figure 5.14. OT Critical Fracture Energy Results for Mix 1 Using Protocol 2

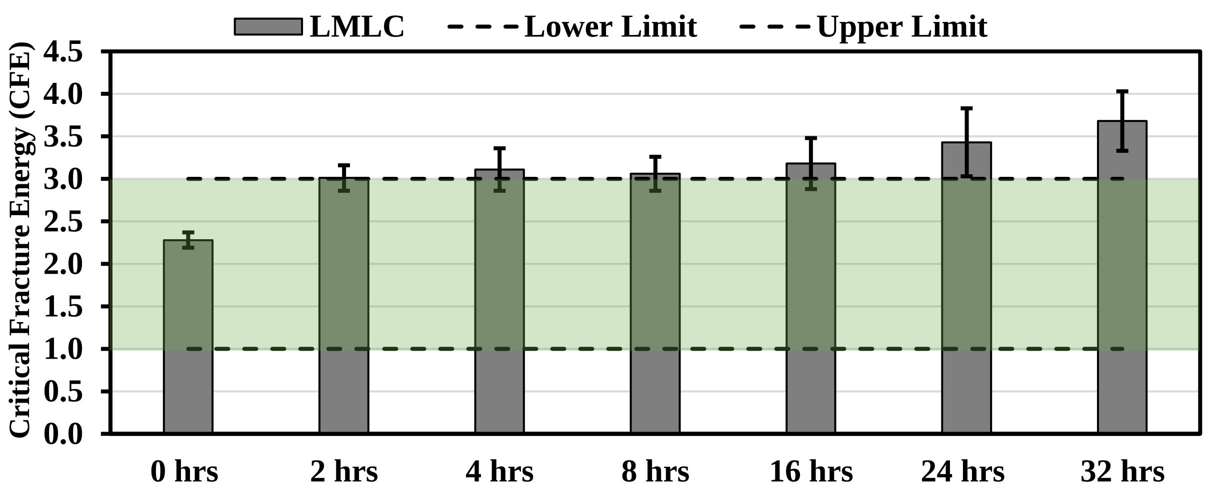


Figure 5.15. OT Critical Fracture Energy Results for Mix 2 Using Protocol 2

Similarly, as shown in Figures 5.16 and 5.17, the CPR value from the OT exceeded the acceptance limit of 0.45 after 8 hrs of oven aging, as opposed to 32 hrs per Protocol 1. The variability of the results from replicate specimens was much higher compared to the lower temperature from Protocol 1.

The CT_{Index} became less than the acceptance limit of 80 after 2 hrs of oven aging (as opposed to 8 hrs for Protocol 1) as evident in Figure 5.18 for Mix 1. In contrast, the CT_{Index} was less than the acceptance threshold after 4 hrs of oven aging for Mix 2 as shown in Figure 5.19.

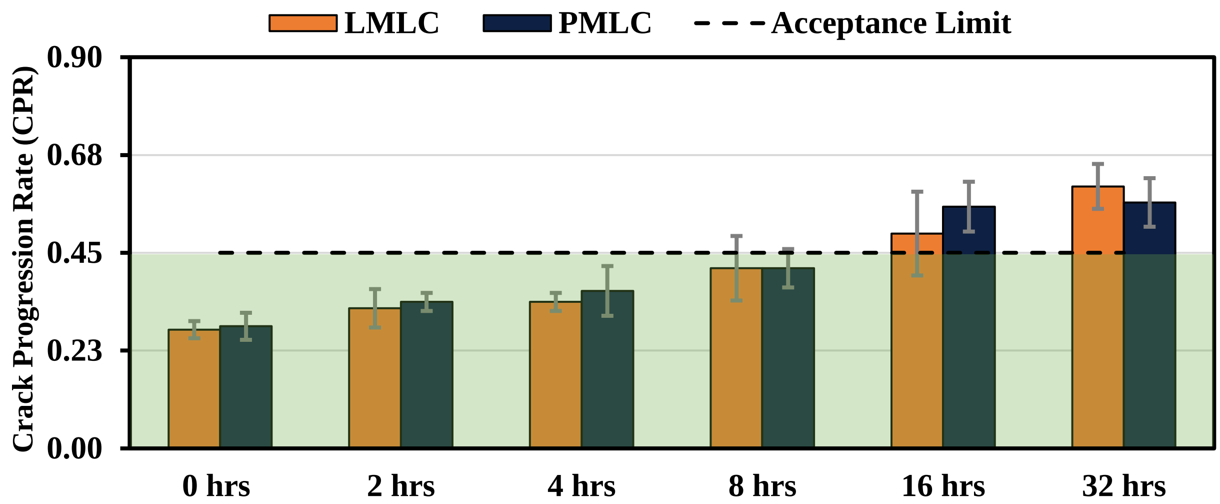


Figure 5.16. OT Crack Progression Results for Mix 1 Using Protocol 2

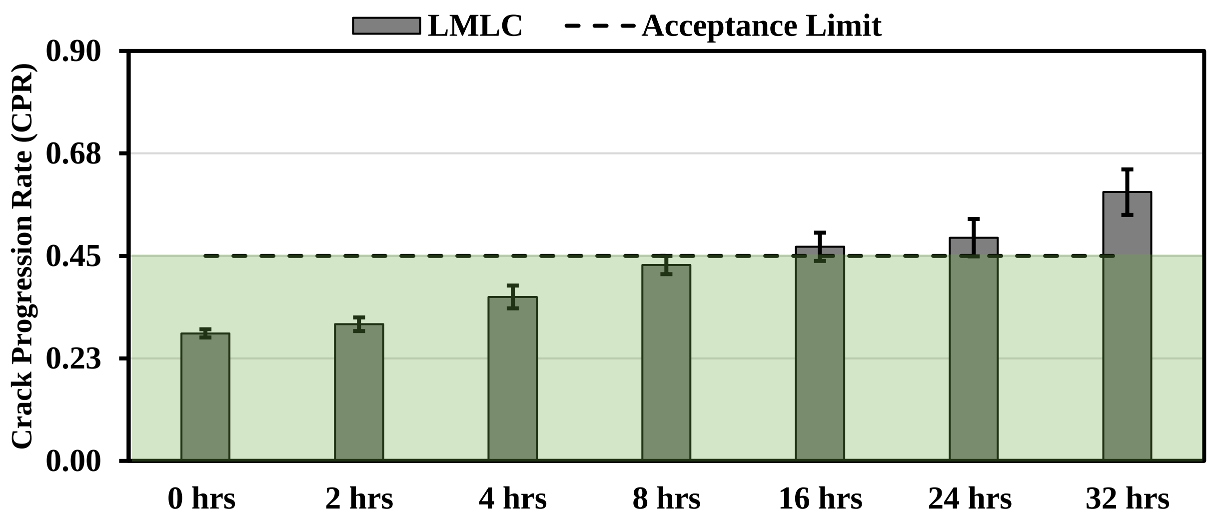


Figure 5.17. OT Crack Progression Results for Mix 2 Using Protocol 2

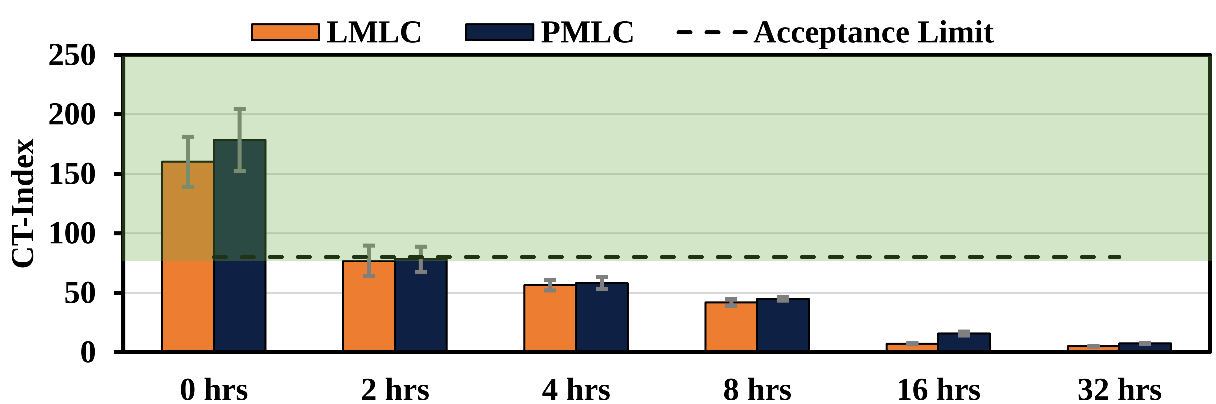


Figure 5.18. CT_{Index} Value Results for Mix 1 Using Protocol 2

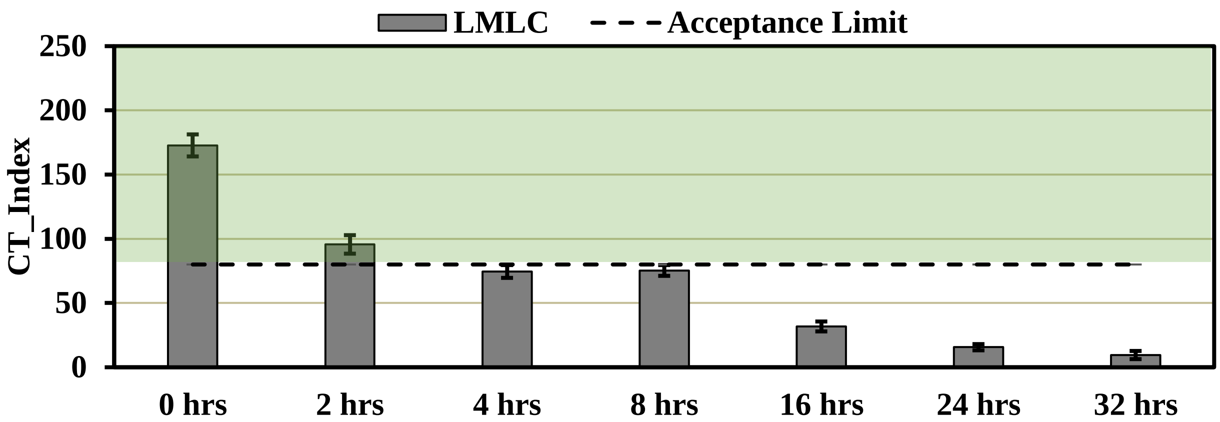


Figure 5.19. CT_{Index} Value Results for Mix 2 Using Protocol 2

Once again, the IDT strengths presented in Figures 5.20 and 5.21 exceeded the upper limit of 200 psi after 8 hrs for Mix 1. On the other hand, Mix 2 exceeded the limit after 4 hrs of oven aging. The IDT strengths increase much more rapidly as compared to the results from Protocol 1.

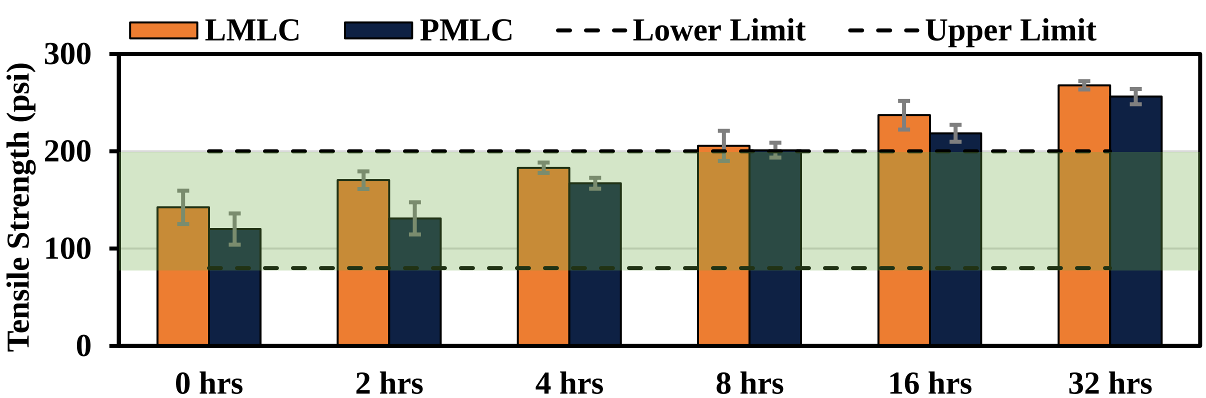


Figure 5.20. Tensile Strength Results for Mix 1 Using Protocol 2

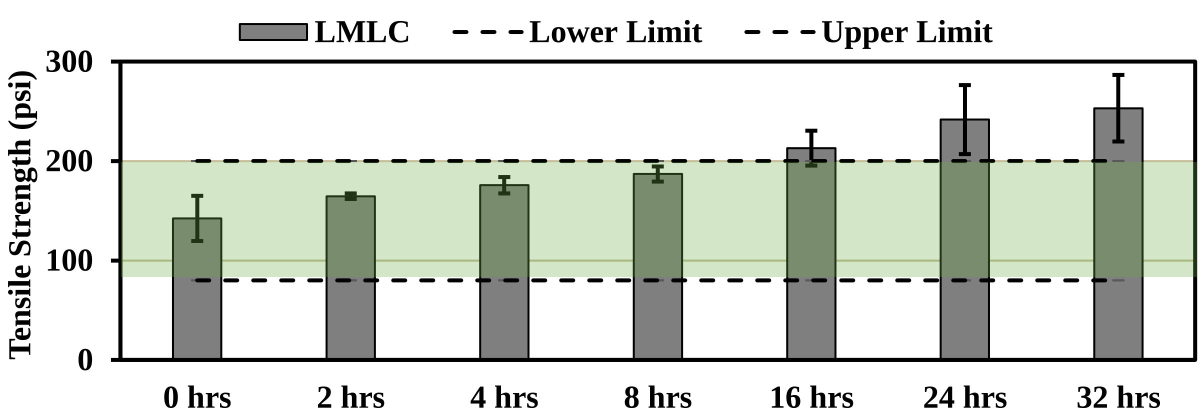


Figure 5.21. Tensile Strength Results for Mix 2 Using Protocol 2

Protocol 3

The feasibility of Protocol 3 which uses both elevated temperature and pressure to accelerate the aging process was verified through five different mixtures, two oven durations, and two different pressure levels. Figure 5.22 illustrates the pressure device setup. To assure the mixture was evenly aged, a perforated stainless-steel stand with 7/32 in. holes was used to allow airflow. The loose mixture was then put into each level in a sealed tank with constant pressure and temperature.

Figure 5.22. Stainless Steel Stand and Pressure Device Setup

Figure 5.23 illustrates the schematic and equipment used for the accelerated aging method used for this protocol. The three main stages involved in the process were (i) oven aging, (ii) applying constant pressure, and (iii) circulating airflow within the device to allow for uniform aging. The loose mix was exposed to 2 hrs of short-term aging at 135ºC while maintaining the thickness of the material to less than 2 in. The aged loose AC mixture was further aged in the pressurized device at a predefined pressure at a temperature of 95ºC. Several standalone thermocouples were used to monitor and validate the temperature inside the tank and within the loose materials.

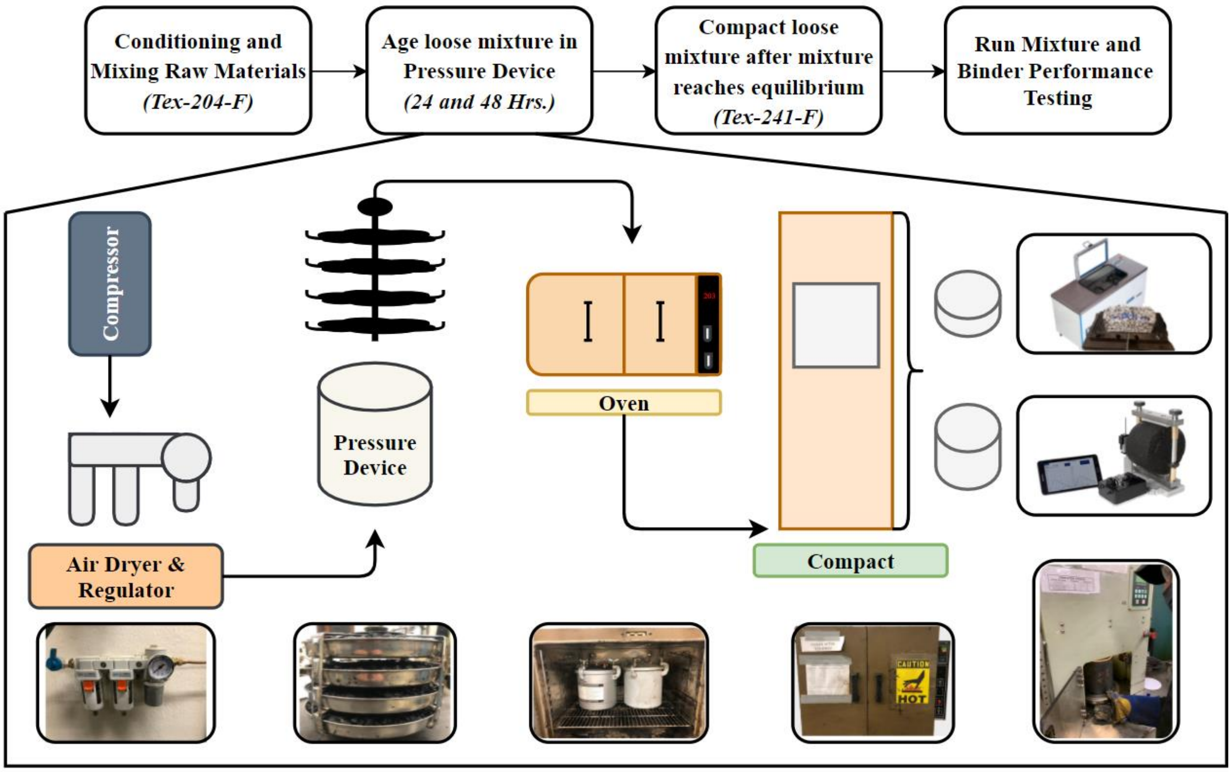


Figure 5.23. Schematic of the Accelerated Aging Procedure

Three sets of IDT specimens using Mix 1 were made to determine the uniformity of aging between the top and bottom tiers of the steel stand. Figure 5.24 illustrates that the tensile strengths from the top and bottom tiers are comparable. The aging process is repeatable since the IDT strengths vary between 109 psi and 116 psi for the top-tier specimens and between 106 psi and 115 psi for the bottom tiers.

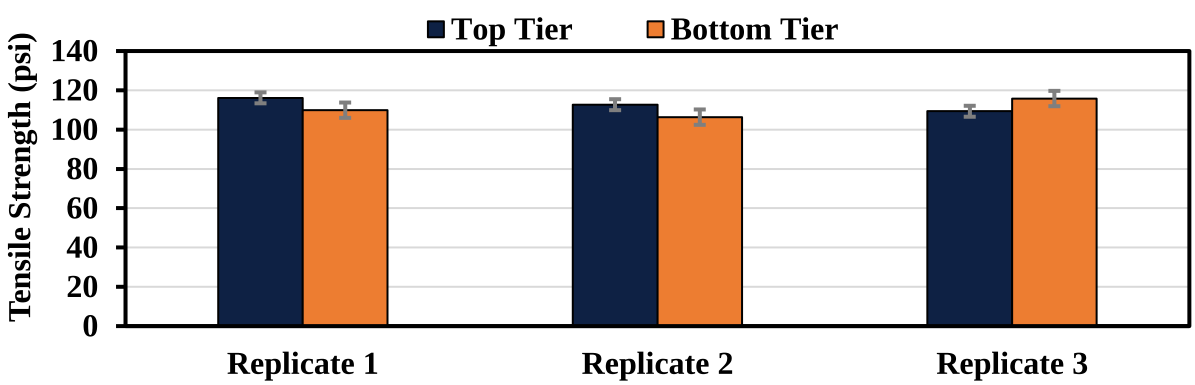


Figure 5.24. Tensile Strength Comparison Using Protocol 3

Once the pressure device demonstrated consistent aging throughout the device, a study was conducted with Mix 1 to optimize the pressure level. Figure 5.25 shows the CT indices of a few of the iterations. Compared to the 5-Day benchmark, the specimens aged at a lower pressure for 24 hrs were not adequately aged, while those aged at a higher pressure and for 48 hrs met or exceeded the benchmark CT_{Index} values.

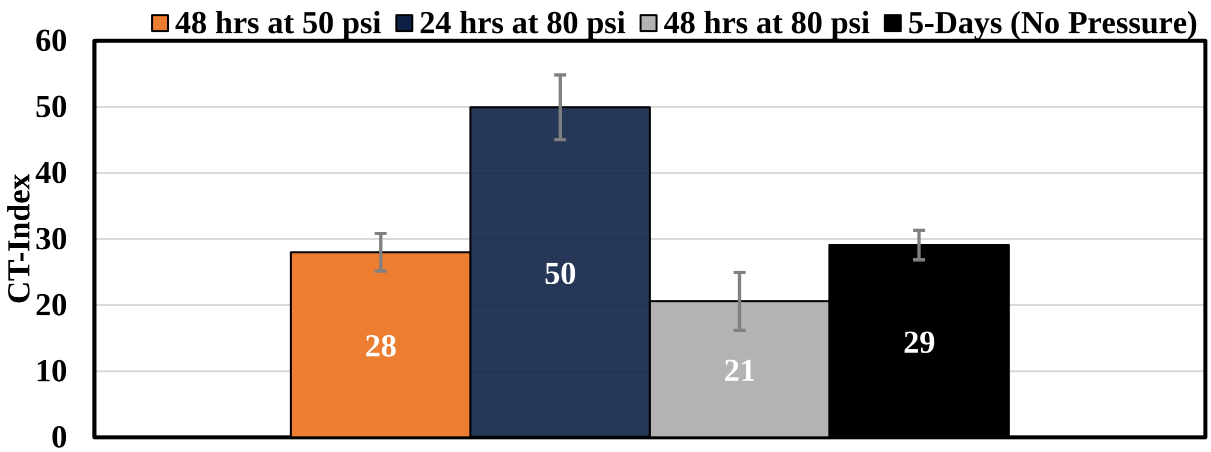


Figure 5.25. CT_{Index} Value Comparison Using Protocol 3

The IDT strengths among different aging processes are compared in Figure 5.26. Similar to the previous results the values from the lower pressure and duration were not able to meet those of the 5-Day benchmark. These initial results indicate that Protocol 3 with appropriate pressure could surpass the benchmark strength and CT_{Index} within 48 hrs.

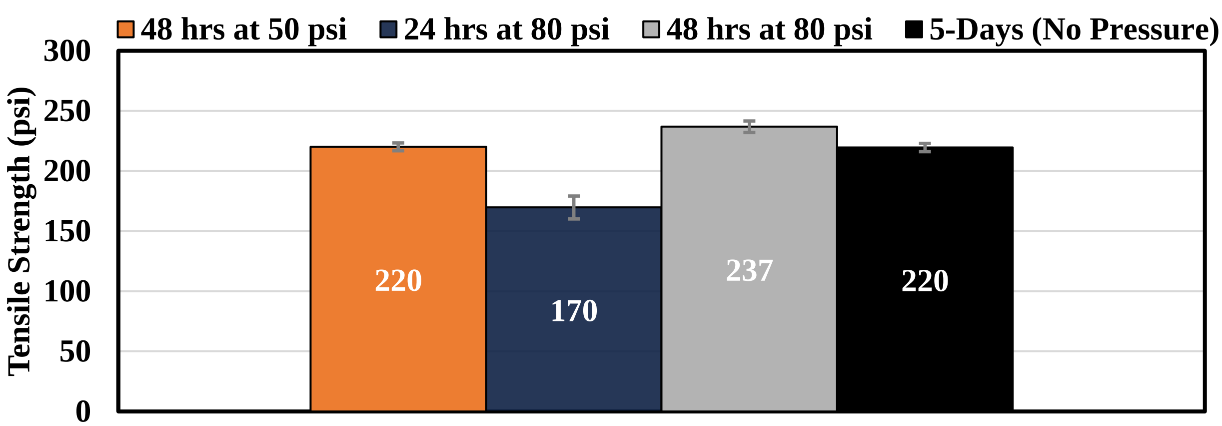


Figure 5.26. IDT Strength Comparison Using Protocol 3

An additional five mixtures were evaluated with different binder PG, aggregates sources, and admixtures to assure the feasibility of this method. Table 5.2 presents the details related to the constituents of these five mixtures. Two of the five mixtures (No. 1 and No. 5) had the same characteristics and gradations as Mix 1 and Mix 2, respectively. Each mix was subjected to aging regimes of 5 days of aging at 95ºC as a benchmark in addition to 24 hrs and 48 hrs of aging at 80 psi.

Table 5.2. AC Mixtures for Validation Using Protocol 3

No.	Aggregate Type	Binder Grade	Mix Design Characteristics
1	Limestone	PG 70-22	SP-C: 10% RAP, HydroFoam IEQ, 5.4% AC
2	Limestone	PG 70-22	SP-C: 20% RAP, HydroFoam IEQ, 5.1% AC
3	Igneous	PG 70-22	SP-C: 15% RAP, N/A, 5.3% AC
4	Igneous	PG 76-22	SP-C: 10% RAP, N/A, 5.4% AC
5	Limestone/Igneous	PG 70-22	SP-C: 15% RAP, Evotherm, 5.5% AC

The CT indices and IDT strengths at the different aging durations are compared in Figures 5.27 and 5.28, respectively. Four of the five mixes met or exceeded the benchmark values in 48 hrs or less. The abnormal behavior from Mix 4 could be from the increase in the binder's high PG.

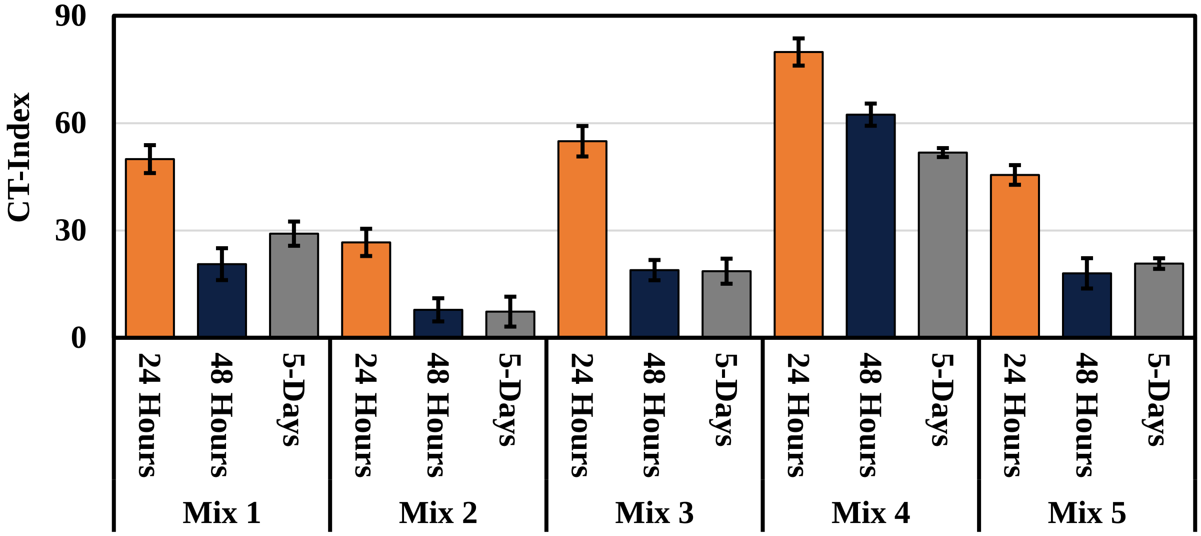


Figure 5.27. CT_{Index} Value Comparison Using Protocol 3

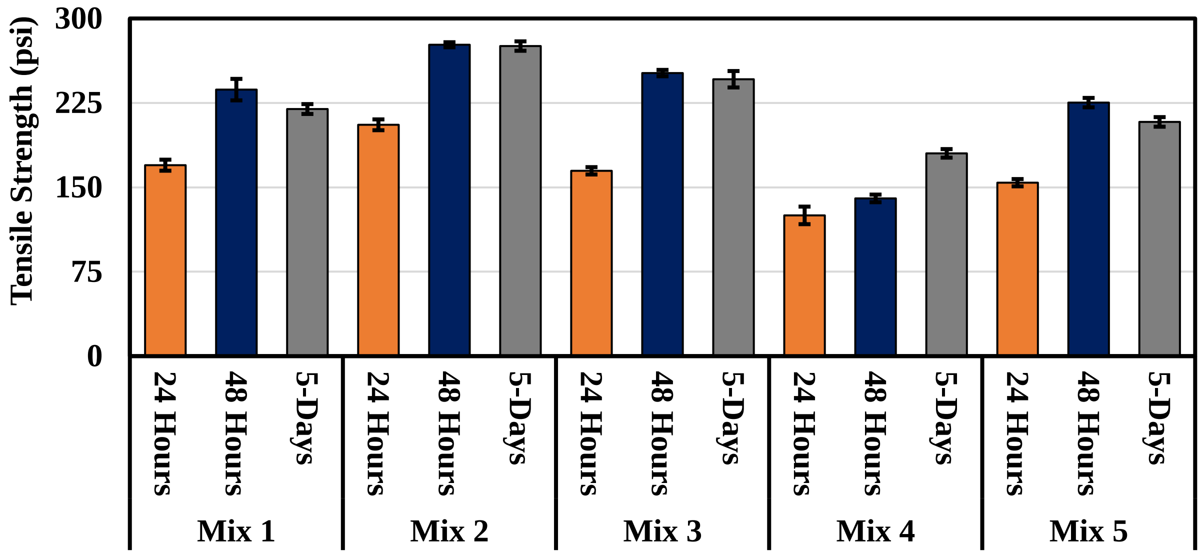


Figure 5.28. IDT Strength Comparison Using Protocol 3

Initial Evaluation of Steady State of Mixtures

Four asphalt mixtures with four RAP contents (0%, 10%, 20%, and 30%) with a PG 70-22 binder were oven-aged following Protocol 1 for different periods. The IDT strengths on triplicate specimens between different RAP content are compared in Figure 5.29. The variations in the OT's CPR and CFE values with the RAP contents are shown in Figures 5.30 and 5.31. As hypothesized, the rank order of samples at 24 hrs was identical to the rank order at 120 hrs for both IDT and OT results. These results indicate the likelihood of a steady state level and provide encouraging results for further testing. This hypothesis will be further explored in the next chapter that considered more mixture variables to formulate more robust and generalizable conclusions at the mixture level.

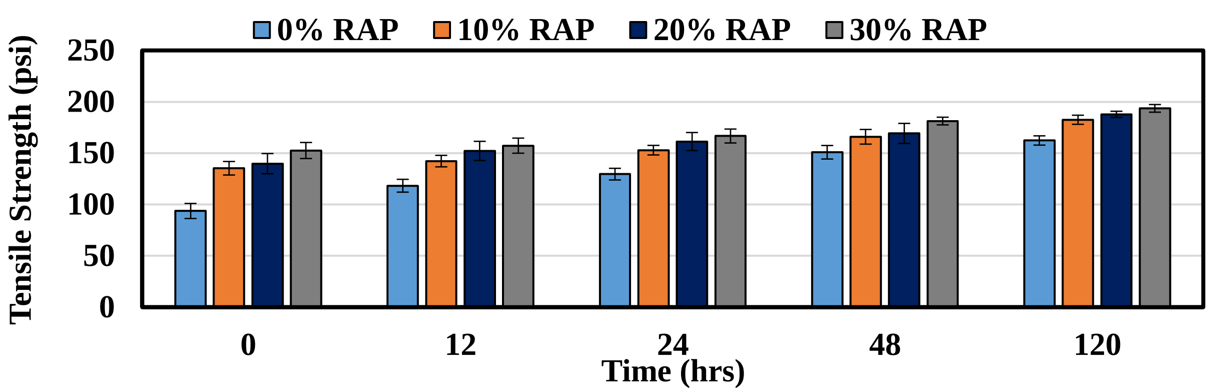


Figure 5.29. Rank Order Comparison for IDT Tensile Strength

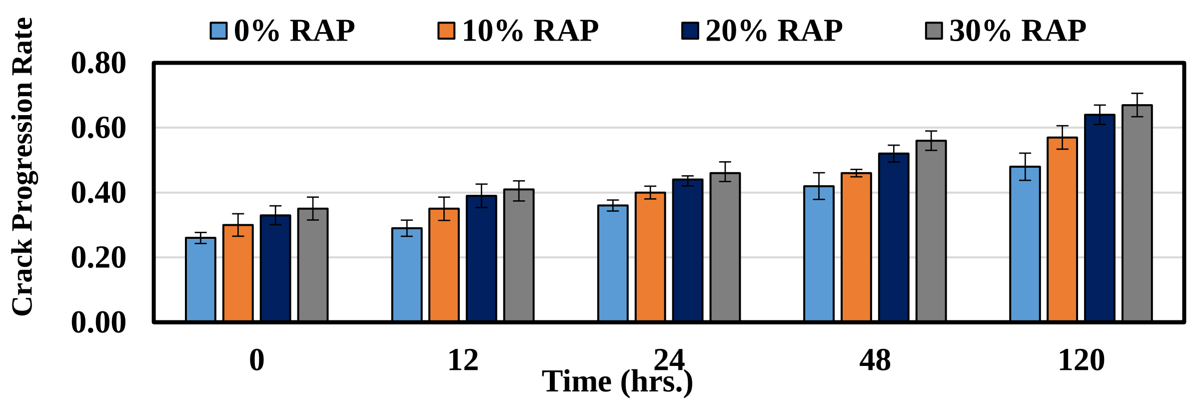


Figure 5.30. Rank Order Comparison for OT Crack Progression Rate

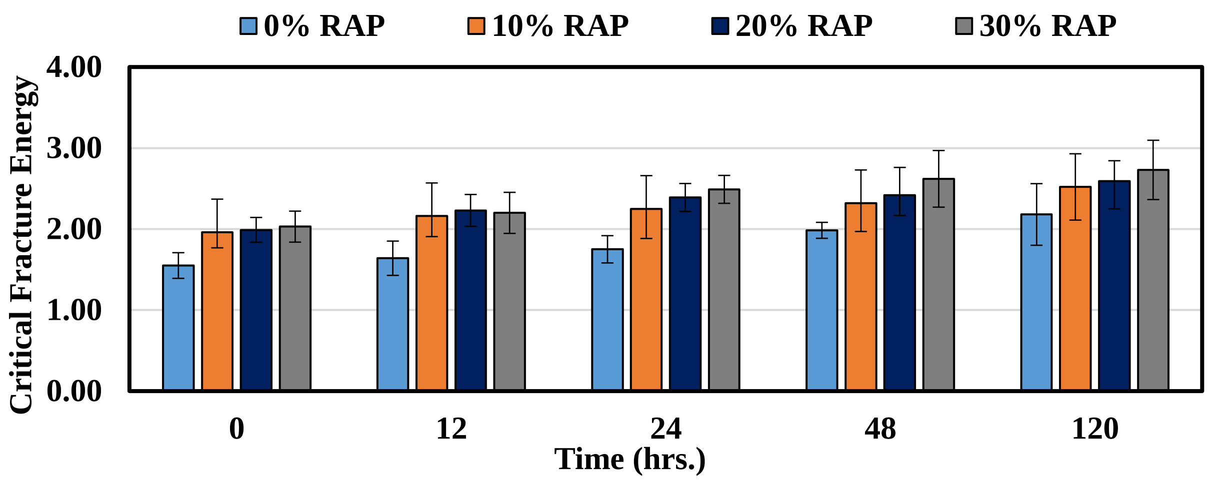


Figure 5.31. Rank Order Comparison for OT Critical Fracture Energy

The initial evaluation focused on identifying the aging-related kinetics of asphalt mixtures by investigating different aging protocols and identifying the most optimal method for aging mixtures in the laboratory. The main findings of the initial evaluation can be summarized in the following manner:

- A reduction in aging durations for more practical testing durations and the addition of 24 hrs testing period,
- Mixture performance tests showed a consistent increase in aging-related metrics with increased testing time. A preliminary analysis indicated that the minimum time for mixing to achieve a steady state may be a period of 24 to 32 hrs under Protocol 1.
- Protocol 2 was not considered further in this study as the results were unsatisfactory and the technique might not be scientifically sound for routine testing.
- Protocol 3, using the pressurized aging device, showed promise as an accelerated technique and was further up scaled and evaluated in the extended evaluation.
- The rate of change of rutting characteristics appears to be minimal beyond 2 hrs of short-term oven aging. Therefore, the short-term duration of 2 hrs was adopted and considered in the total oven aging duration.

Chapter 6: Extended Evaluation of Aging Potential

The extended evaluation was limited to the two protocols summarized in Table 6.1. Protocol 2 was mainly used to provide the long-term accelerated aging of mixtures within a shorter period. Based on the previous results of the study and for overall convenience, the curing time were changed to 10, 22, 46 and 118 hrs to account for the 2 hrs of short-term aging undergone by the mixes before long-term aging using the different protocols.

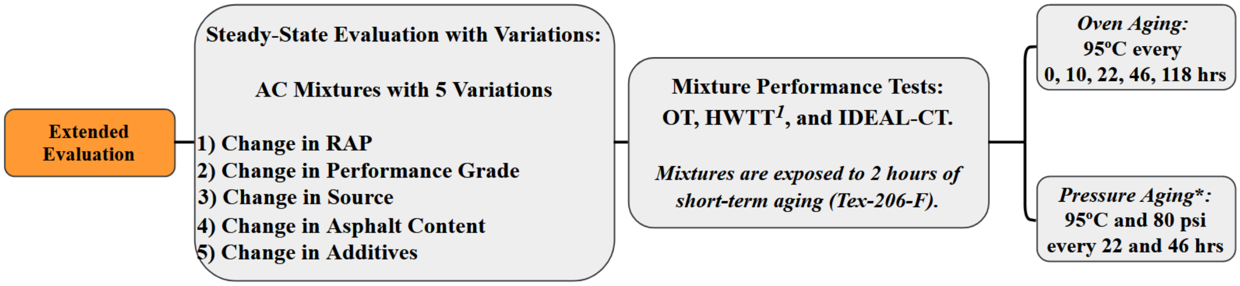
Table 6.1. Optimized Laboratory Aging Protocol

Protocol No.	**Curing Condition**		**Curing Times**
	Temperature	*Pressure*	
1	Cured at 95°C	**-**	0, 10, 22, 46, 118 hrs
2	Cured at 95°C	80 psi	22 & 46 hrs

The *extended evaluation* was informed by the outcomes of the initial evaluation. The most promising protocol(s) were redefined and further evaluated by analyzing the following variables:

- Mix type,
- Aggregate sources and surface aggregate classification (SAC),
- Binder PG and sources,
- Recycled material sources and quantity,
- Asphalt content,
- Additives.

To assess further the early-age rutting and long-term cracking potential of mixtures the evaluation of AC mixtures and different variables were explored as shown in Figure 6.1.



[1]HWTT specimens are evaluated for early age rutting between 0 hours to 4 hours of aging for each protocol.
**Alternative accelerated aging methods are subject to change based on results.*

Figure 6.1. Extended Evaluation Overview

The most common AC mixes in Texas are Type C and Type D, dense-graded (DG), Superpave (SP) mixes, and stone-matrix asphalt (SMA) mixes. A variety of typical mixtures including DG-D, SP-C, and SP-D was evaluated for the following reasons:

- DG-D was compared to SP mixes to assess the impact of changes in aggregate sizes, gradation, and asphalt content.
- SP mixes were compared further to the DG mix to delineate the influence of gradation and aggregate structure. Furthermore, to evaluate the impact of different compaction methods and ensure the process applies for different mix types.

Mixture Design Variables

Five variables were considered to study the influence of mix design variables on aging behavior including change in RAP (content and source), PG (source and grade), aggregate (type and SAC), asphalt content, and additives as recommended by the manufacturer. Table 6.2 documents the different variations evaluated for each of the control mixtures. As part of the extended evaluation, one DG and four SP mixtures were evaluated. Each had different variations within the control mix.

Table 6.2. Mix Design Breakdown and Variations

				5 Recommended Variations			
Mix No.	1: RAP	2a: PG Source	2b: PG Grade	3a:Aggregate Gradation	3b:Aggregate Source	4:Asphalt Content	5:Additive
1	10/20/30						
2	10/20/30		76/70/64				1.5x & 2.0x Dose
3		3 Sources			SAC A to B		
4				MDL		-0.5 & 1.0	
5	0 to 30						

To delineate the influence of aggregate type and properties on the aging potential of asphalt mixtures, absorptive and non-absorptive aggregates were considered with different Surface Aggregate Classification (SAC) per TxDOT Tex-499-A. The SAC describes the aggregate quality based on friction and durability. Limestone-Dolomite, Gravel, and Igneous aggregate sources like those used in Reyes et al. (*2008*) and Garcia et al. (*2020*) were used, because of the wealth of data that already existed in terms of mix design and performance.

Three asphalt binder grades (PG 64-22, PG 70-22, and PG 76-22) were considered to understand and document the influence of binder grades on aging. For the same PG binder, different source locations across Texas were also tested to document the variability in aging due to changes in the binder source.

Ten mix designs with different RAP contents were used to produce the asphalt mixtures. The amount of RAP ranged from 0% to 30% of the total mix. The aging behavior of binders can be influenced by the thickness of the binder film in the mixtures. The asphalt content was also changed to evaluate its impact on aging potential.

Two rejuvenators' agents were also evaluated. The performance of the rejuvenated mix was compared to the performance of a mix without additives to delineate the influence of additives. The dosages used followed those recommended by the manufacturers.

Table 6.3 shows the details related to the constituents of these five control mixtures and the targeted variations. All the mixtures were LMLC and were subjected to long-term aging at 95ºC following the short-term aging condition of 2 hrs.

Table 6.3. Mixtures Constituents for Protocol 1

Mix No.	Aggregate Type	Binder Grade	Mix Design Characteristics
1	Dolomitic Limestone	PG 70-22	SP-D: 20% RAP, Evotherm, 6.1% AC
2	Dolomitic Limestone	PG 64-22	SP-D: 30% RAP, Evotherm, 6.0% AC
3	Gravel	PG 70-22	SP-C: 10% RAP, ZycoTherm, 5.3% AC
4	Igneous	PG 70-28	DG-D: 8% RAP, N/A, 5.6% AC
5	Gravel	PG 70-22	SP-C: 20% RAP, N/A, 5.8% AC

Hamburg Wheel-Tracking Test Results

During the initial evaluation phase, oven aging beyond 4 hrs showed no significant loss in the mixture's rutting resistance and the rutting characteristics of the mix improved. In this section, the main focus was on understanding the impact rutting resistance had the most effect on the changes in the mix design variables. Figures 6.2 to 6.4 present the impact of the RAP content for Mixes 1, 2, and 5. The results are in good agreement with similar studies in literature. The results suggest that NRRI consistently increased with the oven aging period as the RAP content increased in the mixture. Increments in RAP percentage and aging time prevent the permanent deformation of asphalt mixtures. It could be inferred that a strong correlation exists between aging time versus NRRI as well as RAP and NRRI. Increasing the aging time by 1 hr might add 0.02 to the NRRI value, considering the same amount of RAP is added to Mix 5. These marginal effects are lower in comparison to Mixes 1 and 2. This suggests that the impact of RAP percentage and aging time might be mixture dependent. These results suggest that the impact of RAP might be more meaningful beyond a 10% inclusion.

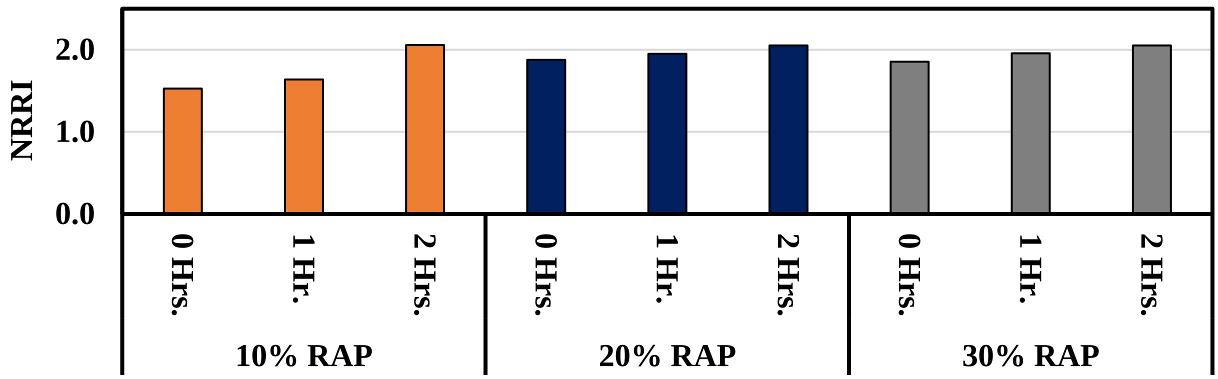


Figure 6.2. NRRI Results for Change in RAP (Mix 1)

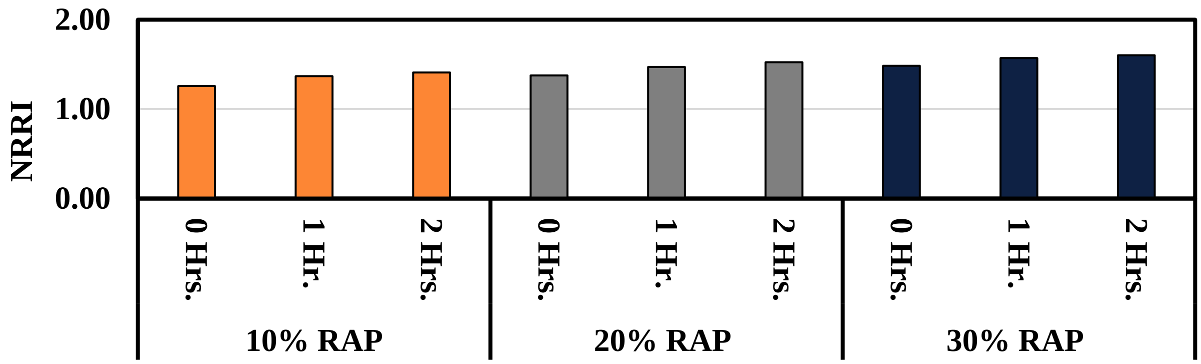


Figure 6.3. NRRI Results for Change in RAP (Mix 2)

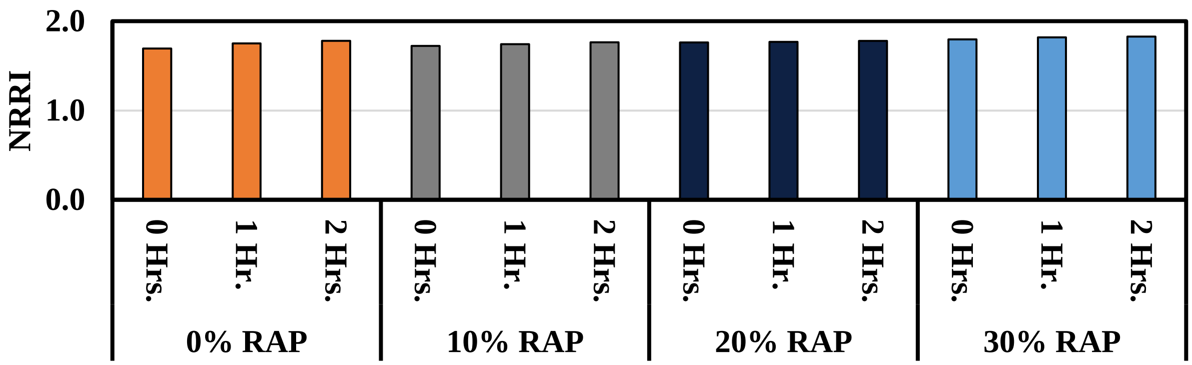


Figure 6.4. NRRI Results for Change in RAP (Mix 5)

Figures 6.5 and 6.6 illustrate the impact of the change in the concentration of additives for different binders on the rutting performance of a mix. The manufacturer recommended dosages was the control at a 3.0%, dosage and a half at 4.5%, and double dose at 6%. A drop in NRRI as a function of aging duration was observed, regardless of the binder type and additive dosage. This is somewhat counter-intuitive because generally rutting resistance is expected to increase with an increase in aging duration. It is speculated that this anomalous behavior could be due to volatilization or degradation of the additive.

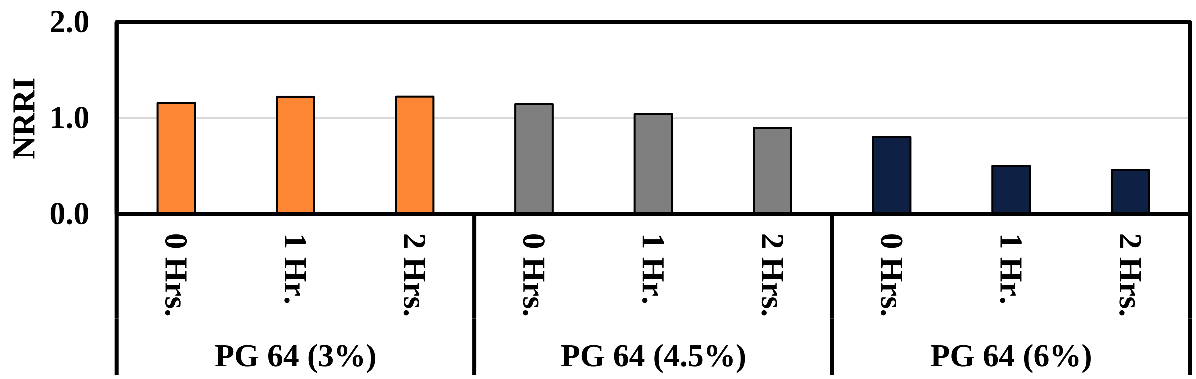


Figure 6.5. NRRI Results with PG 64 for Change in Additives (Mix 2)

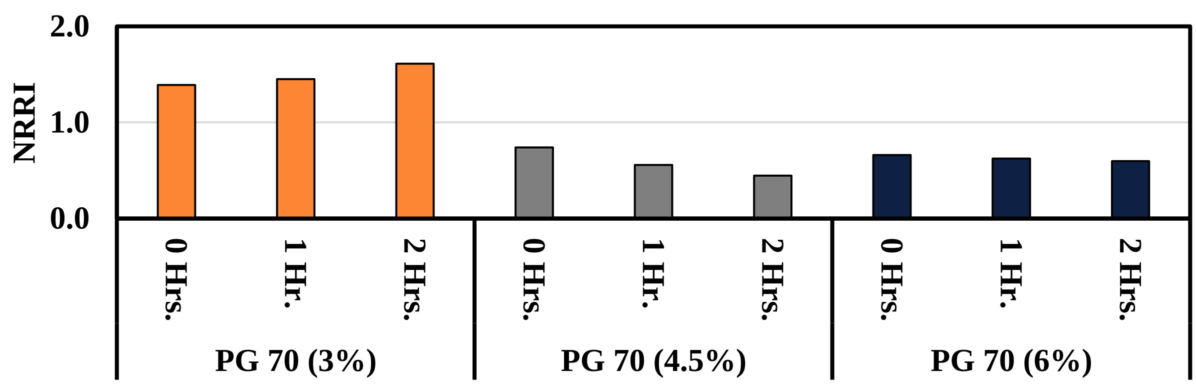


Figure 6.6. NRRI Results with PG 70 for Change in Additives (Mix 2)

Figure 6.7 confirms that changes in the binder's PG impact the rutting results. As the binder grade increased so did the rutting resistance of the mixture regardless of the oven aging period. The combined effects of the binder source and SAC are illustrated in Figure 6.8. As expected, the binder type impacted the rutting performance. The rates of change of NRRI values beyond 2 hrs of short-term oven aging for the given variations are minimal. The average NRRI results for Binder A, B, and C were 2.22, 2.19, and 2.00, respectively. Not much difference is observed between the performance of Binder A and Binder B, but a noticeable drop in performance is detected for Binder C. Reasonably, mixes with SAC A behaved better than mixes with SAC B. The average NRRI results for SAC A and SAC B mixes were 2.19 and 2.08, respectively. The longer AC mixtures are exposed to oven aging, the higher the NRRI will be. All mixtures met the HWTT specifications established by TxDOT. Nevertheless, all factors showed to impact the rutting performance of AC mixture specimens.

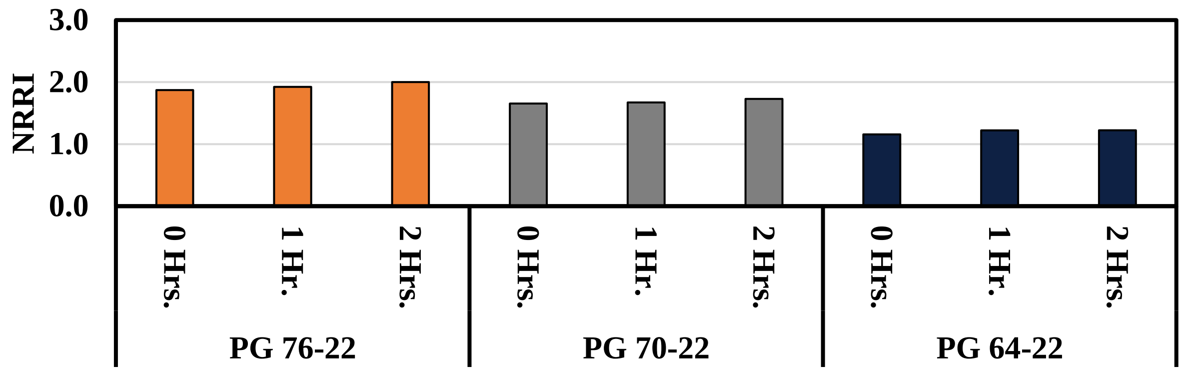


Figure 6.7. NRRI Results for Change in Performance Grade (Mix 2)

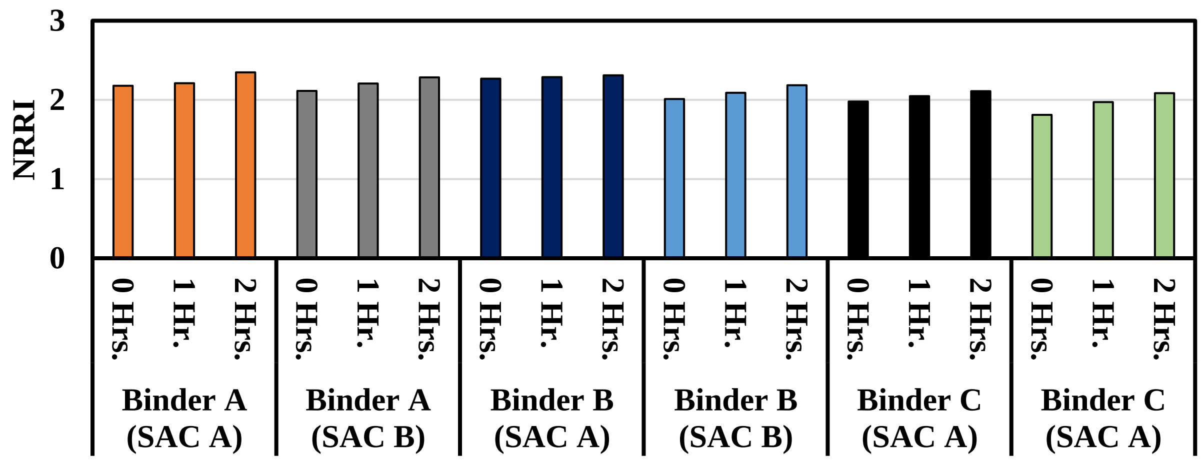


Figure 6.8. NRRI Results for Change in Binder Source and SAC (Mix 3)

Figure 6.9 shows the impact a coarser or finer mix may have on the rutting performance while Figure 6.10 illustrates the results for change in asphalt content. Both parameters exhibit a minimal change in NRRI with curing time.

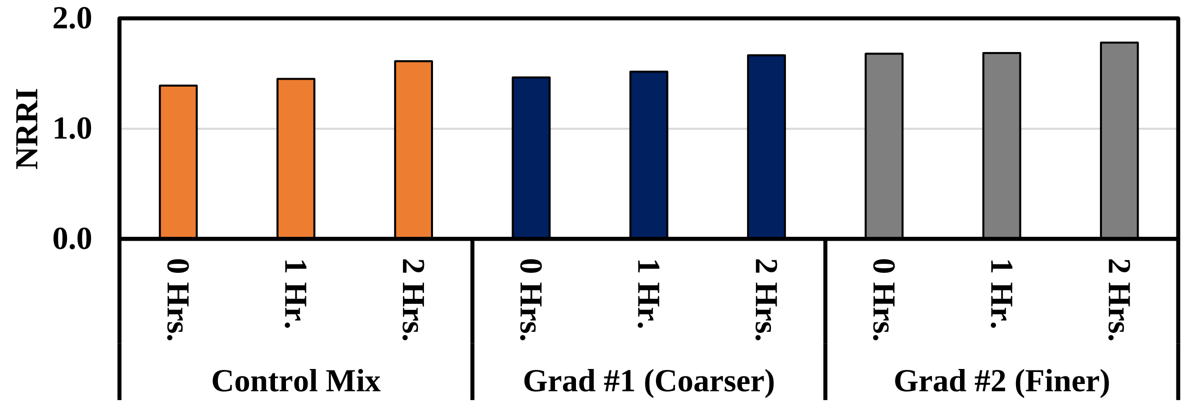


Figure 6.9. NRRI Results for Change in Gradation (Mix 4)

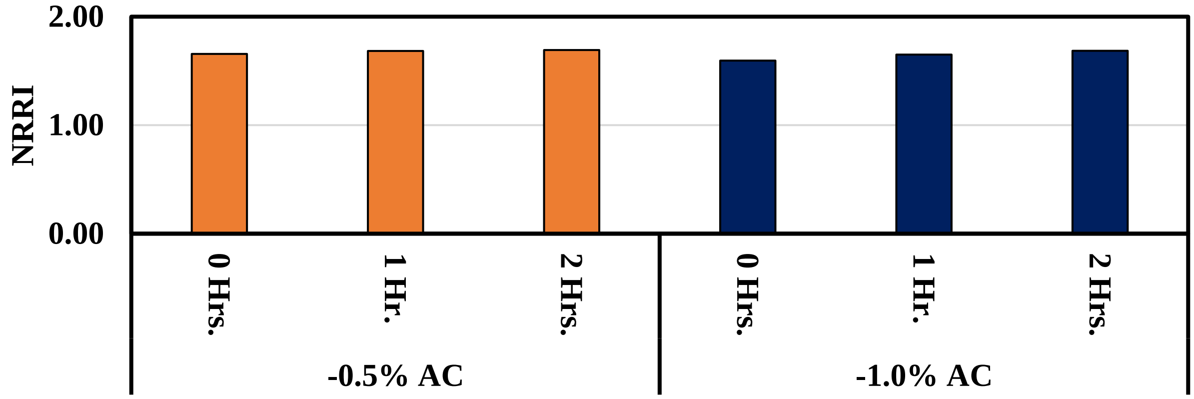


Figure 6.10. NRRI Results for Change in Asphalt Content (Mix 4)

Indirect Tensile Test Results

The changes in tensile strength and CT_{Index} of mixture design variables, as summarized in Table 6.2 are presented in Figures 6.11 to 6.18. The dashed lines in the figures represent the respective 5-day benchmark value for each test.

Figures 6.11 to 6.13 present the cracking performance for change in RAP content for Mixes 1, 2, and 5. The allowable tensile strength range established by TxDOT is between 85 and 200 psi for unaged material. Figure 6.11 shows that increasing the RAP content in the mixtures yields an increase in the tensile strength whereas the CT_{Index} decreases.

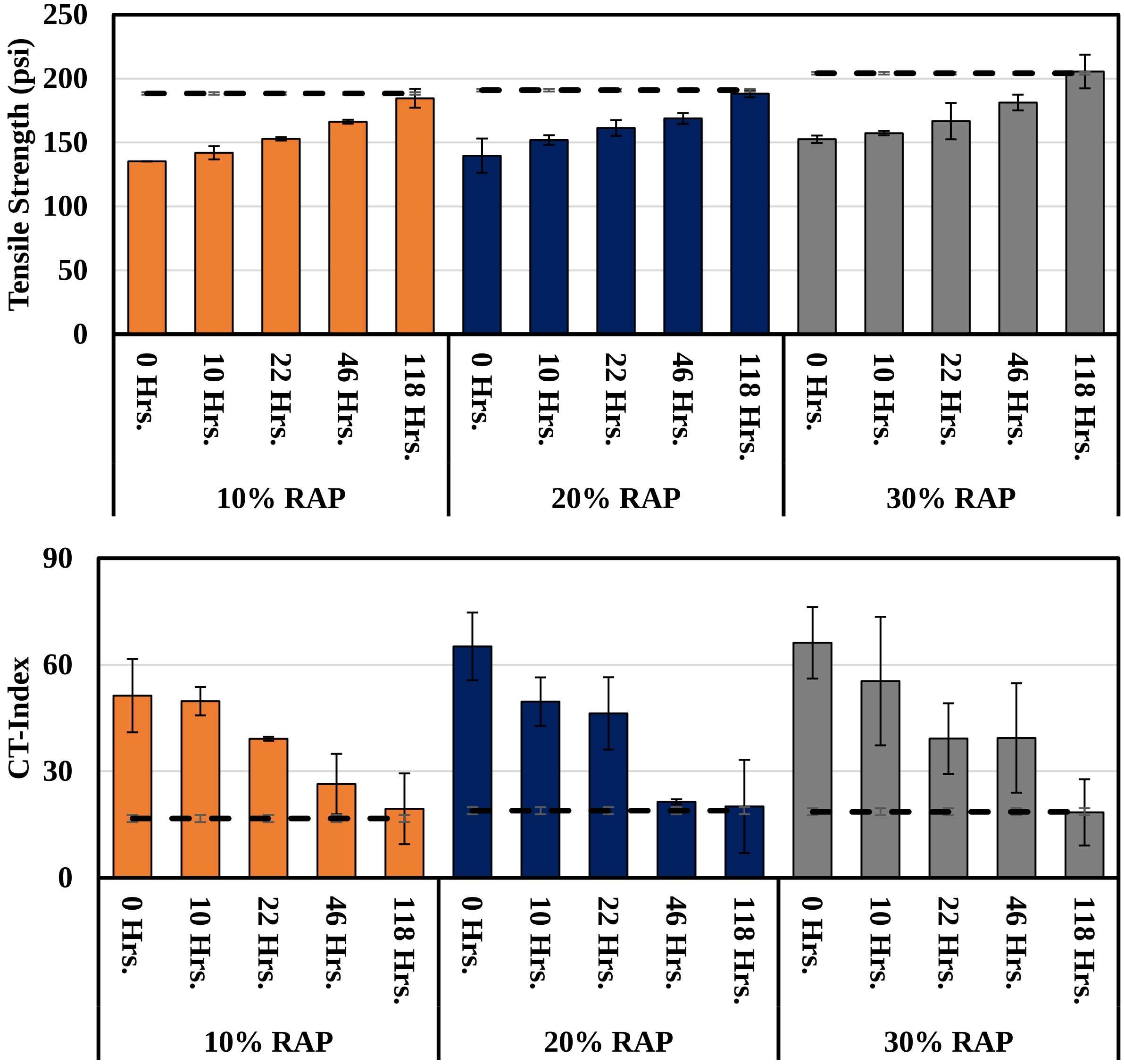


Figure 6.11. Tensile Strength and CT_{Index} Results for Change in RAP (Mix 1)

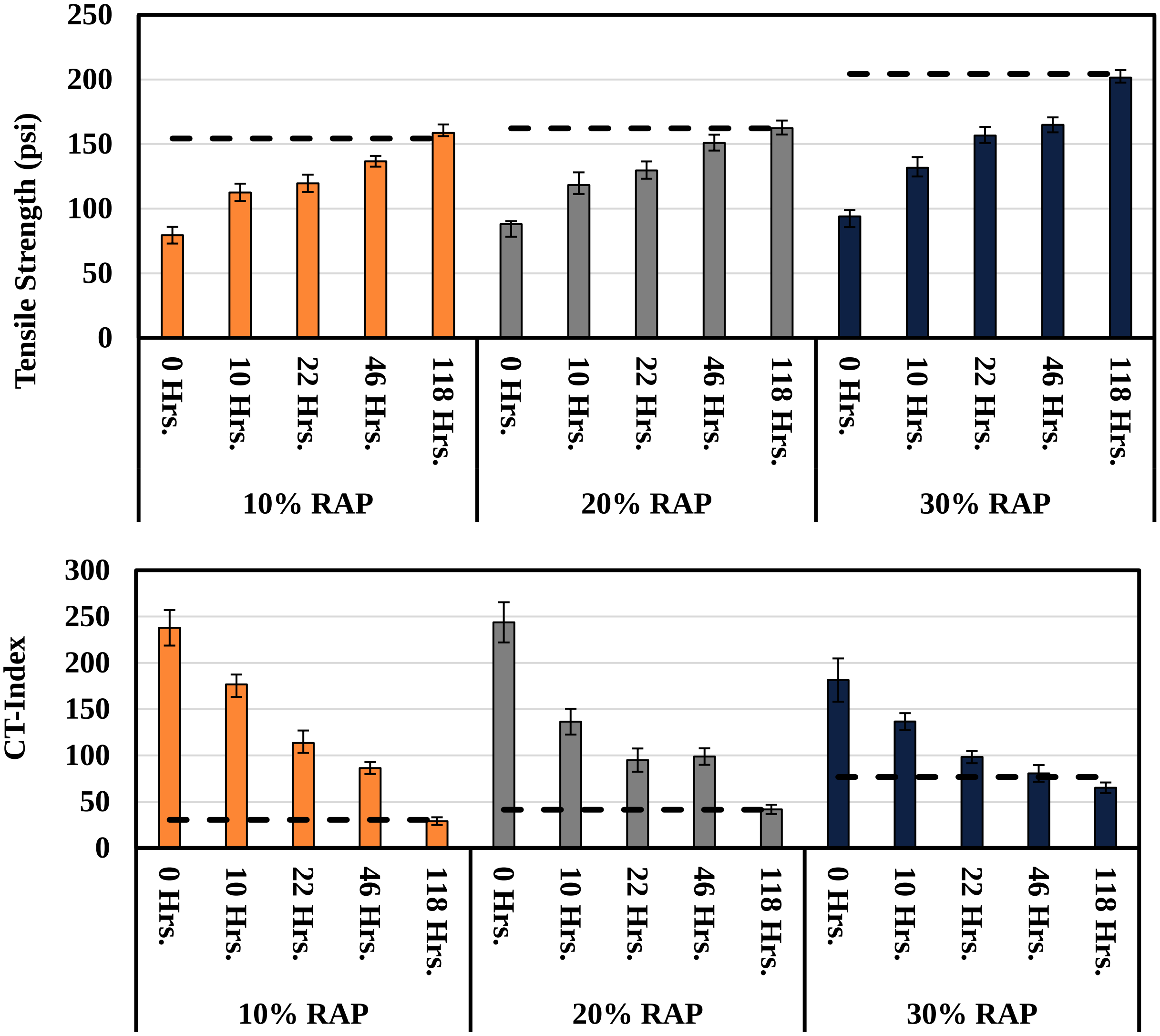


Figure 6.12. Tensile Strength and CT_{Index} Results for Change in RAP (Mix 2)

Figure 6.12 shows that the mixture with 30% RAP and 120 hrs of aging exhibited the highest tensile strength of 201 psi. The CT_{Index} values also suggest that adding more RAP material and aging period would be detrimental to the cracking performance.

Figure 6.13 shows that similar to the rutting results, the difference between 0 and 10% RAP seems insignificant. A sharp decrease in CT_{Index} is noticeable between unaged and aged materials. This outcome highlights the importance of aging asphalt mixture before testing to ensure their performance in the field. The variability levels were higher for the CT_{Index} results than those from the strength results. Overall, the results appear conclusive across all the mixture RAP contents. The results suggest that with the increase in oven aging duration, the mixtures become more crack-susceptible, regardless of the amount of RAP. As the RAP material and aging period increased, the tensile strength increased, and the CT_{Index} decreased. Most asphalt mixture combinations evaluated did not meet the minimum required CT_{Index} of 80.

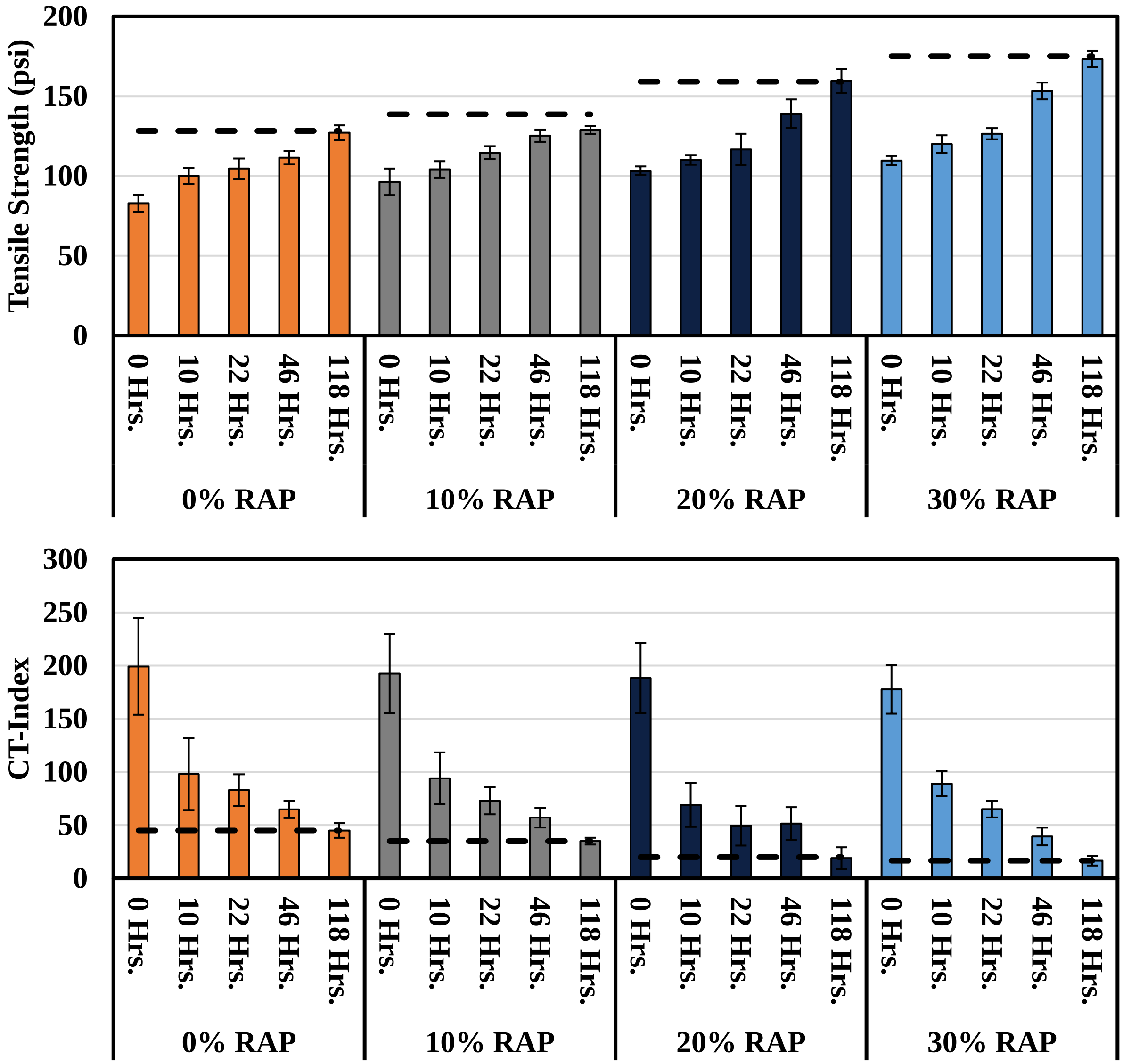


Figure 6.13. Tensile Strength and CT_{Index} Results for Change in RAP (Mix 5)

Figures 6.14 and 6.15 show the impact of the change in additives given different binders on the cracking resistance of Mix 2. Regardless of binder grade the same negative trend for strength and positive gain in CT$_{Index}$ were obtained.

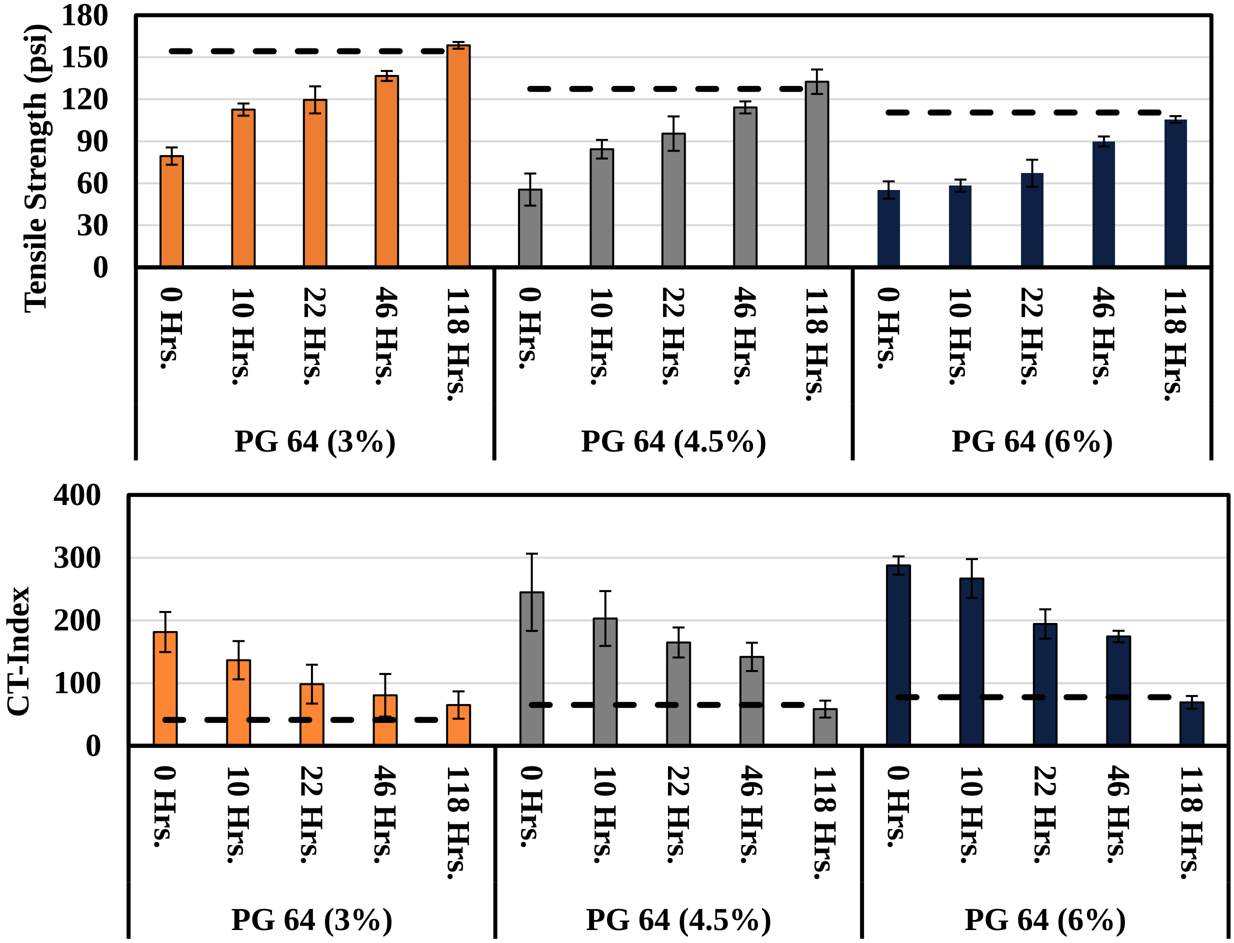


Figure 6.14. Tensile Strength and CT$_{Index}$ Results for Change in Additives (Mix 2)

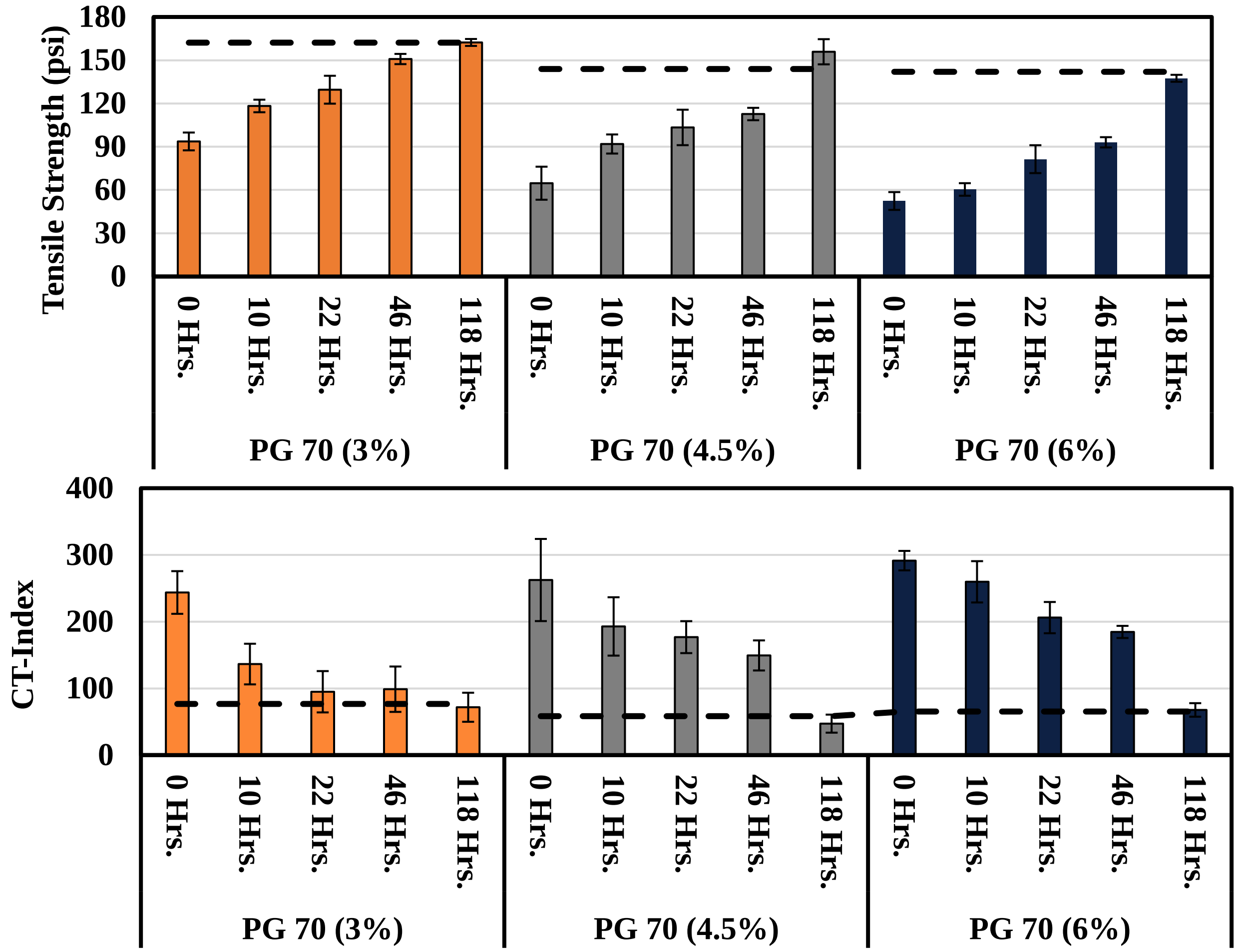


Figure 6.15. Tensile Strength and CT_{Index} Results for Change in Additives (Mix 2)

Figure 6.16 illustrates the change in the binder's PG on the cracking resistance of Mix 2. The mix with the PG 70-22 binder showed a higher tensile strength and a lower CT_{Index} compared to the mix with PG 64-22.

Figure 6.17 shows the cracking performance of mixes with SAC A and SAC B aggregates with different binder sources. Mixes with the SAC A aggregates had higher strengths when compared to the mixes with the SAC B aggregates regardless of binder sources.

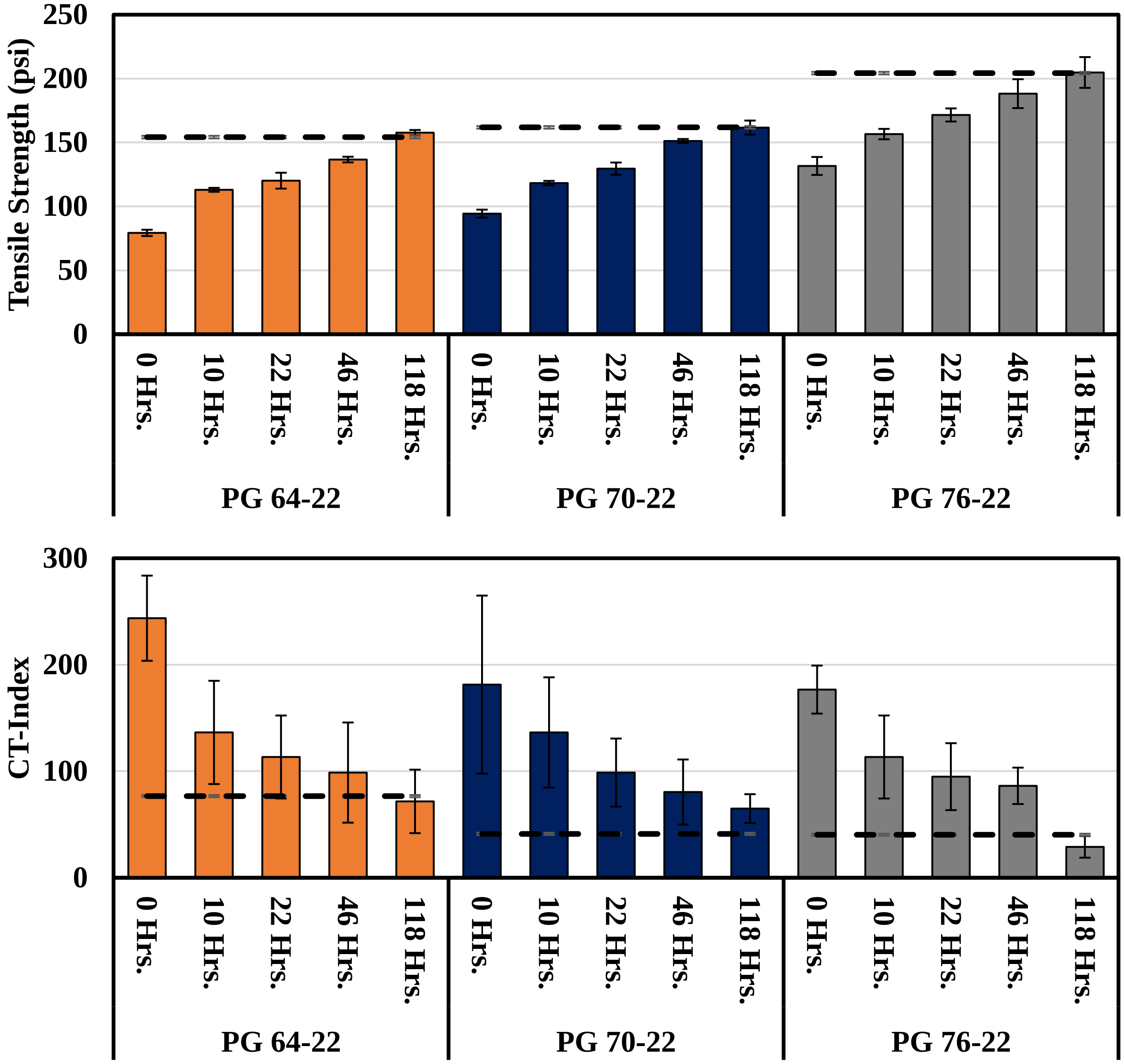


Figure 6.16. Tensile Strength and CT_{Index} Results for Change in PG (Mix 2)

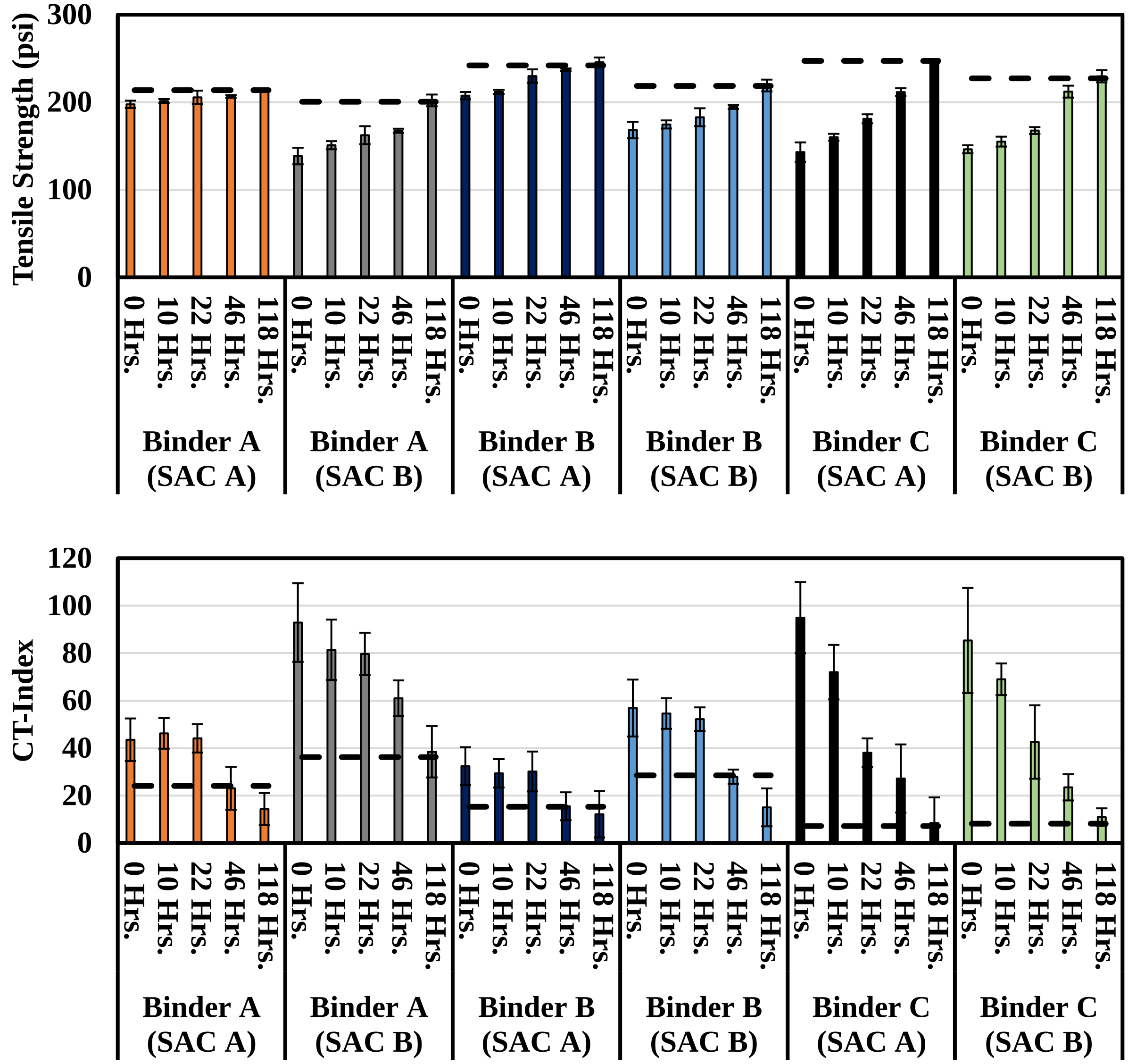


Figure 6.17. Tensile Strength and CT_{Index} Results for Change in Binder Source and SAC (Mix 3)

Figure 6.18 shows the impact a change in gradation may have on the cracking performance while Figure 6.19 illustrates the results of a change in asphalt content. The change in aggregate gradation shows the importance of having proper interlocking of material to maximize the cracking performance. When decreasing the asphalt content of a mixture the strength performance is negatively impacted.

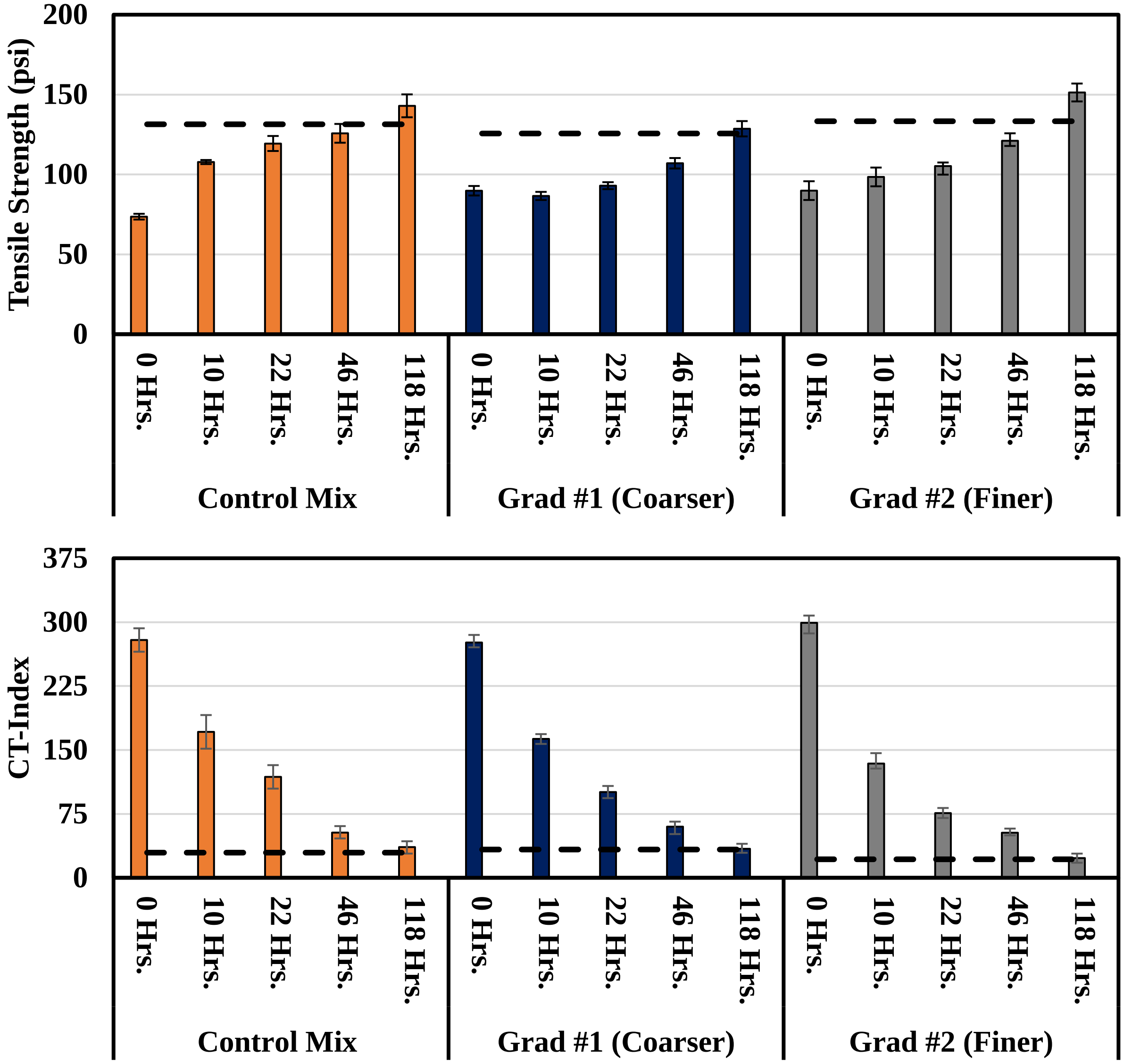


Figure 6.18. Tensile Strength and CT_{Index} Results for Change in Gradation (Mix 4)

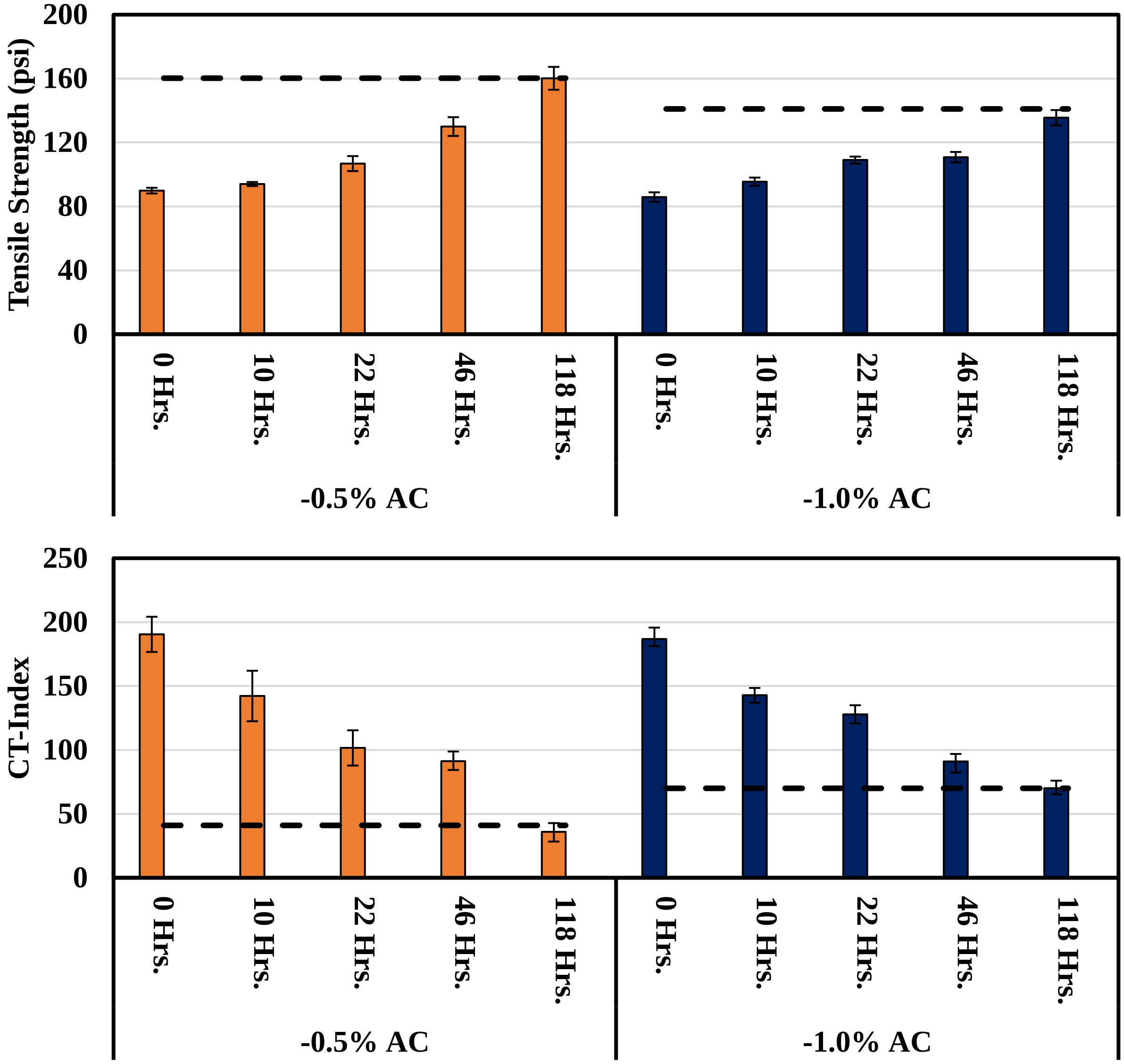


Figure 6.19. Tensile Strength and CT_{Index} Results for Change in Asphalt Content (Mix 4)

Overlay Tester Results

The CPR and CFE values with the change in RAP content are presented in Figures 6.20 to 6.22. A steady increase in CPR and CFE values was observed with the increase in the RAP content as shown in Figure 6.20. The cracking performance for Mixture 2 using a different source of RAP was evaluated as shown in Figure 6.21. The CPR and CFE results obtained indicate that increasing RAP and aging period will lead to higher cracking susceptibility, which is in good agreement with the indirect tensile and CT-Index results. The OT data obtained for Mixture 5 is presented in Figure 6.22. The experimental results corroborate that RAP and the aging period contribute synergistically toward increasing the potential cracking damage of asphalt mixtures. However, the OT protocol seems to discriminate better between the material containing 0% and 10% RAP. Additionally, the unaged and aged results seem to correlate, meaning that OT might distinguish more reasonably different aging levels. The CPR and CFE increased with the increase in the mixtures' aging time regardless of the mixture's RAP content.

From Figures 6.23 and 6.24, adding a 6% dose of additive yields a lower CPR and CFE than 3% and 4.5% doses. As expected, the increased dosage made the mixture more flexible and less susceptible to cracking.

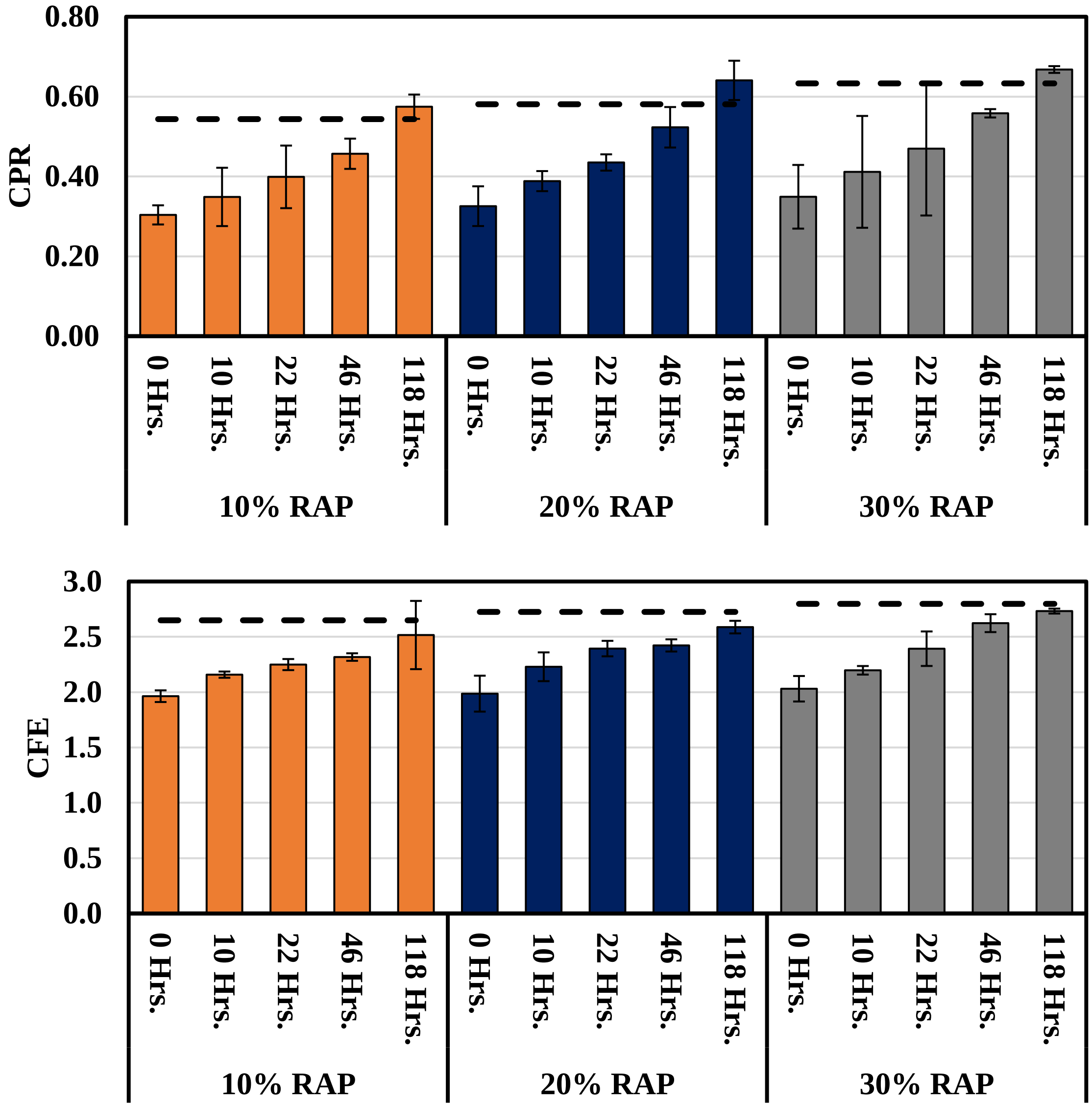


Figure 6.20. CPR and CFE Results for Change in RAP (Mix 1)

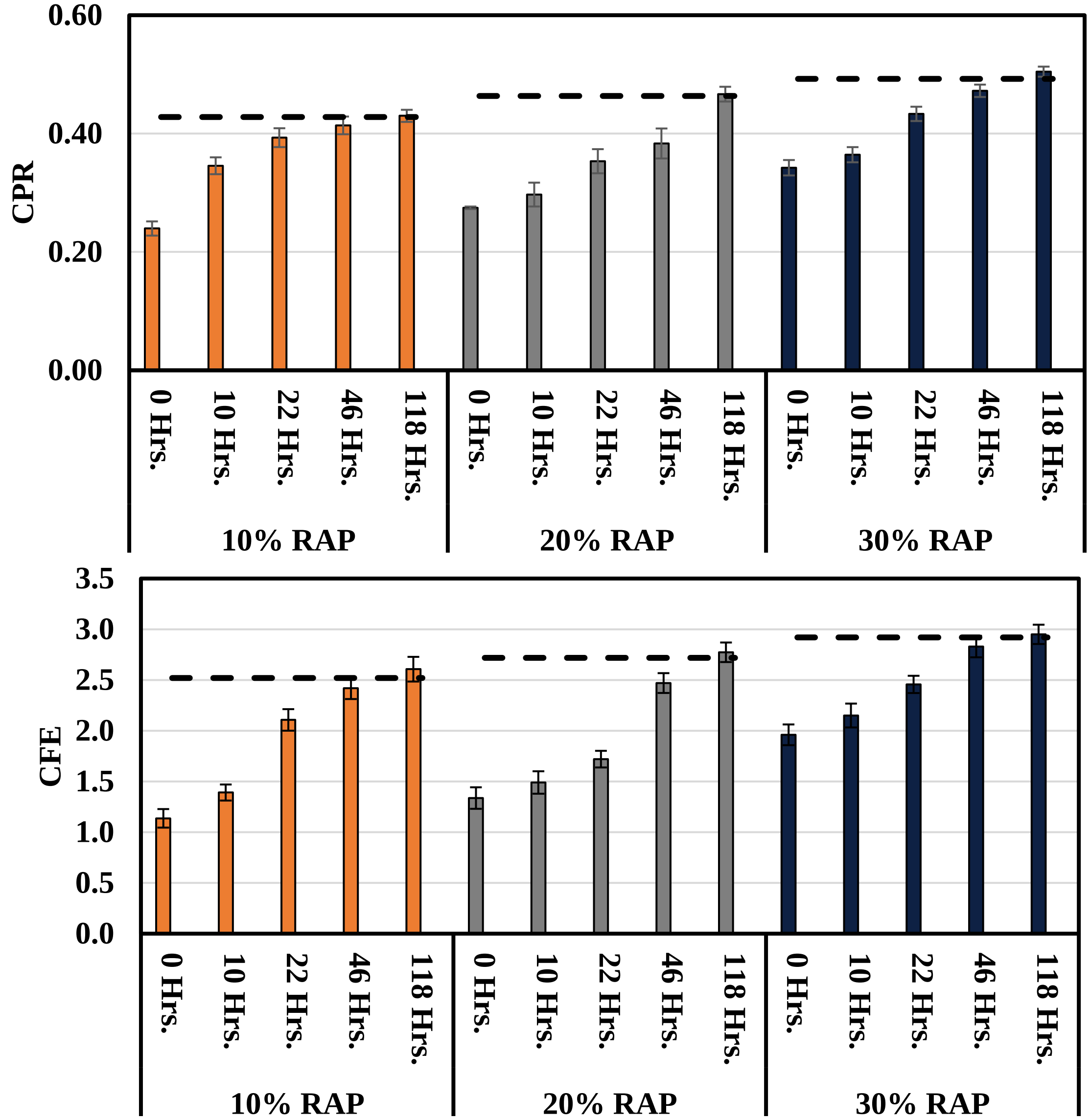


Figure 6.21. CPR and CFE Results for Change in RAP (Mix 2)

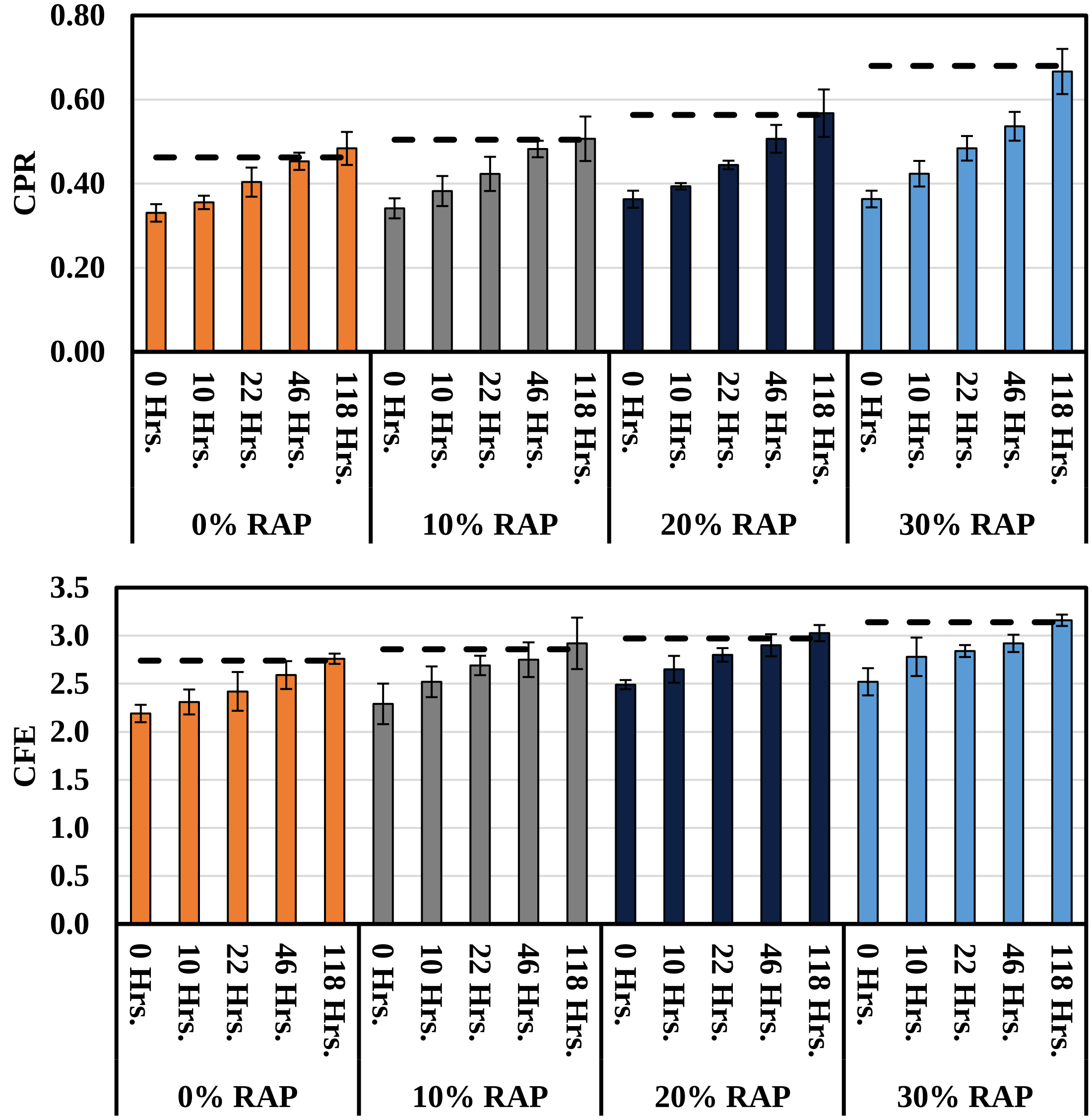


Figure 6.22. CPR and CFE Results for Change in RAP (Mix 5)

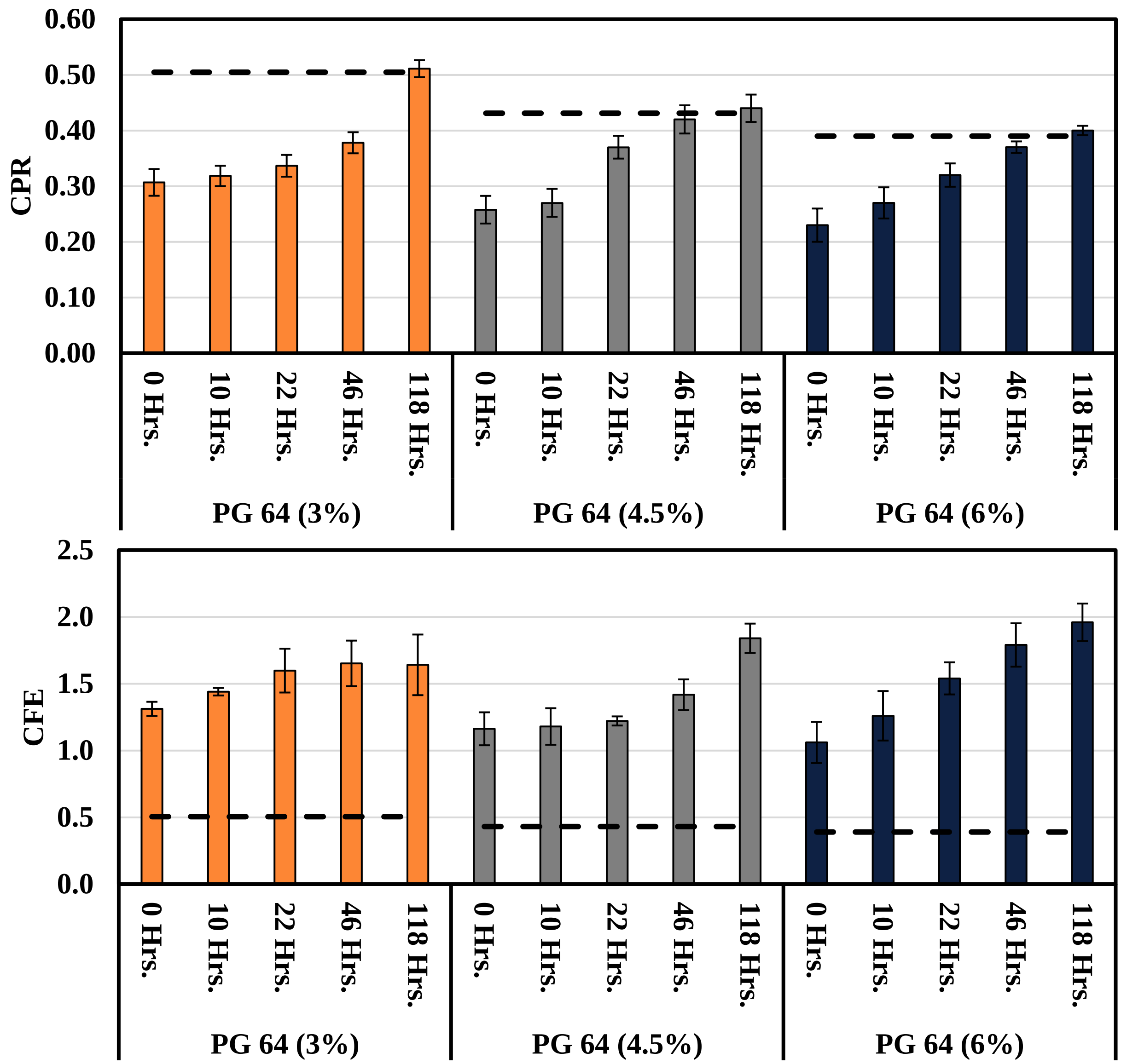


Figure 6.23. CPR and CFE Results for Change in Additive (Mix 2)

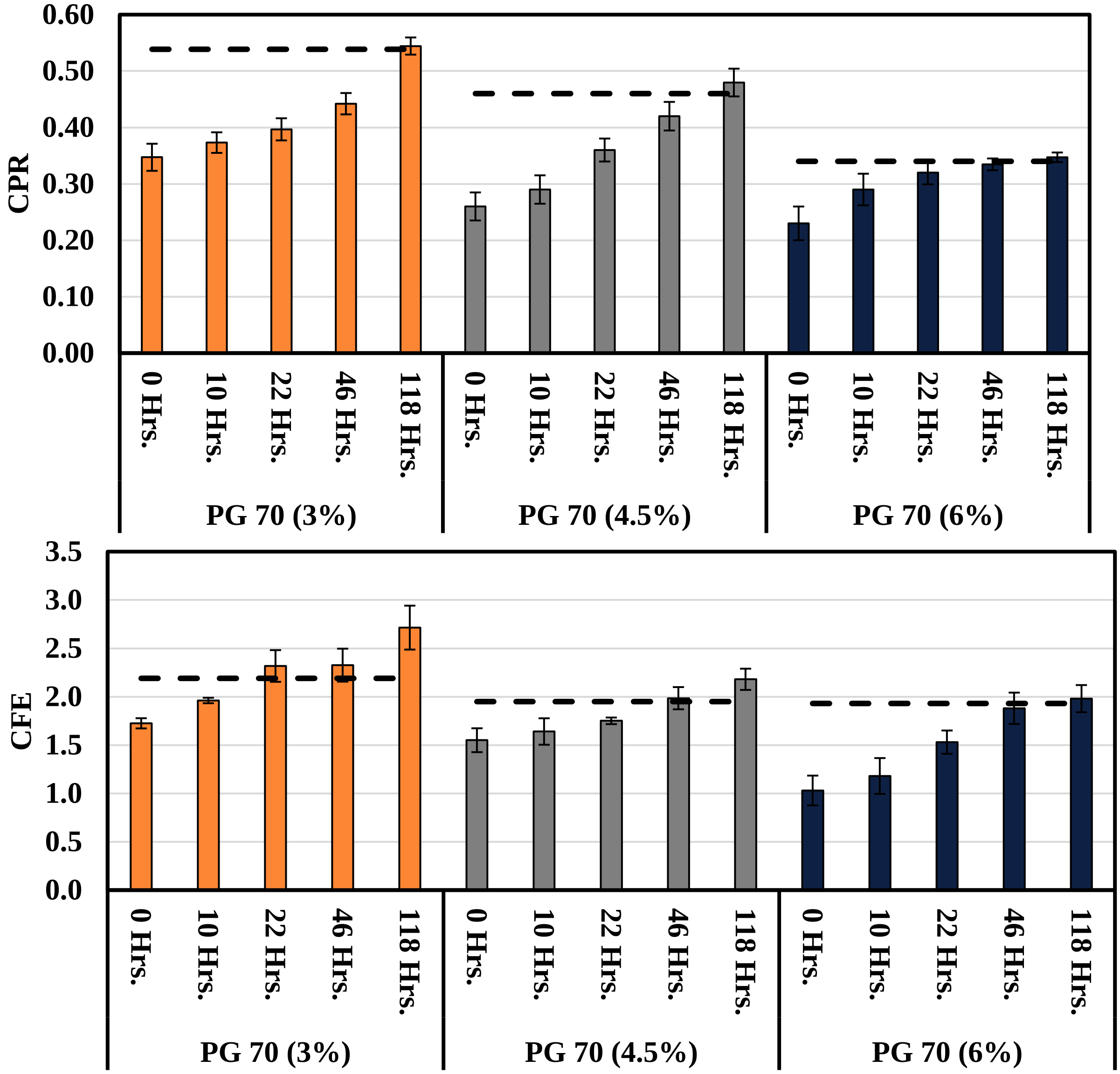


Figure 6.24. CPR and CFE Results for Change in Additive (Mix 2)

Figure 6.25 illustrates the cracking performance for Mixture 2 with different binder PGs. Increments in the aging period increased the CPR and CFE regardless of the binder grade. However, mixtures prepared with a high PG binder (PG 76-22) yield higher CFE and CPR than the mixtures with lower PG binders (PG 70-22 and 64-22). The observed trends for both cracking parameters are reasonable and in good agreement with the IDEAL-CT results. As the aging duration increases, the asphalt mixtures are more susceptible to cracking. All the asphalt mixtures evaluated seem to experience the same decrease in performance caused by the aging protocol applied. An aspect that might require a more careful analysis is that in some cases the asphalt mixture combinations do not meet the CPR criterion but meet the CFE requirements, or vice versa. The extent of this behavior should be further evaluated, especially for the aging period of 120 hrs.

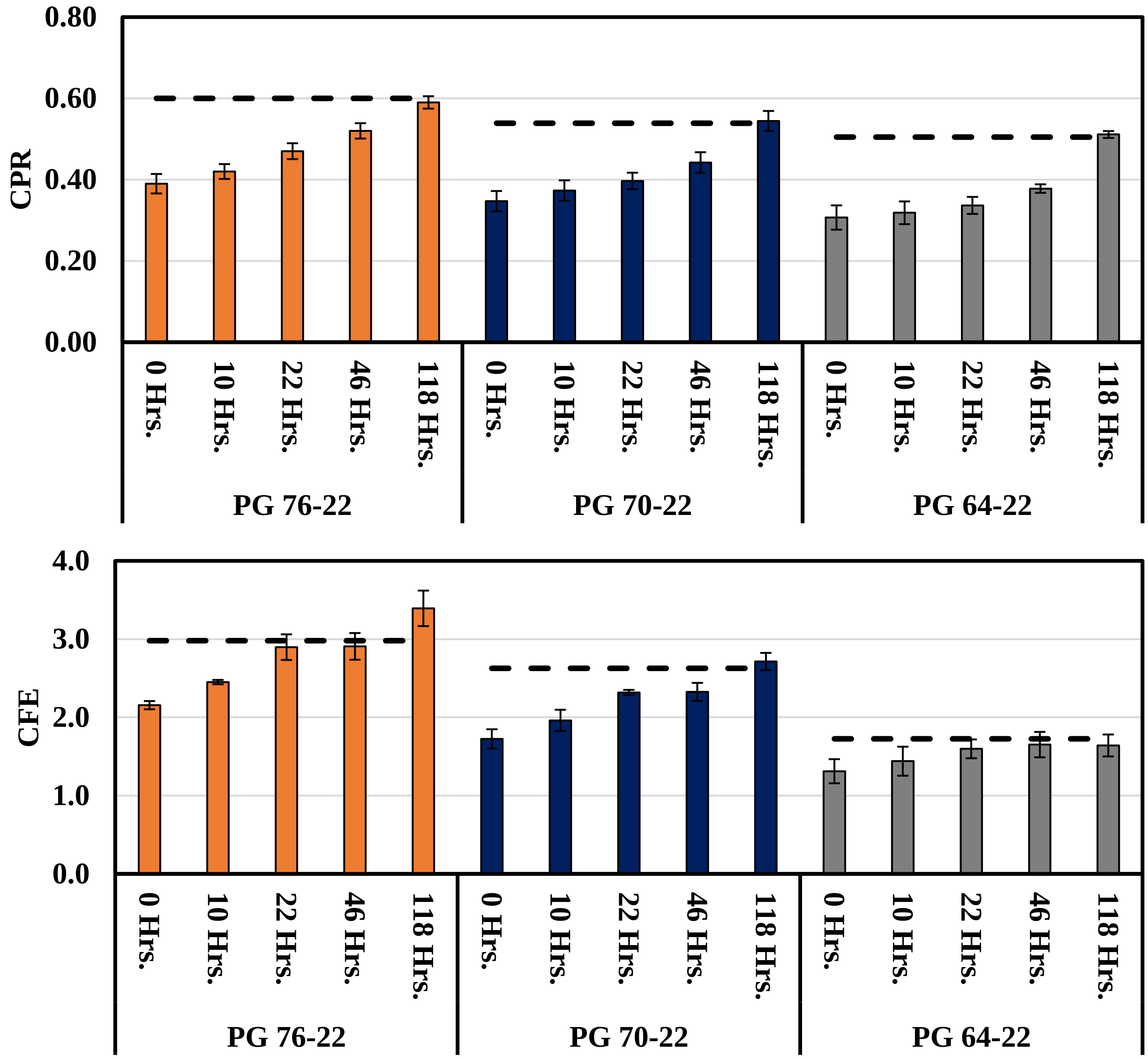


Figure 6.25. CPR and CFE Results for Change in PG (Mix 2)

Figures 6.26 and 6.27 show the cracking performance of Mix 3 prepared with different asphalt binder sources, aggregate types, and aging levels. When SAC A aggregates were utilized, specimens with Binder A exhibited lower cracking susceptibility. But, when SAC B aggregates were incorporated, specimens with Binder A exhibited the most cracking susceptibility. These results might suggest an interaction between binder and aggregate type.

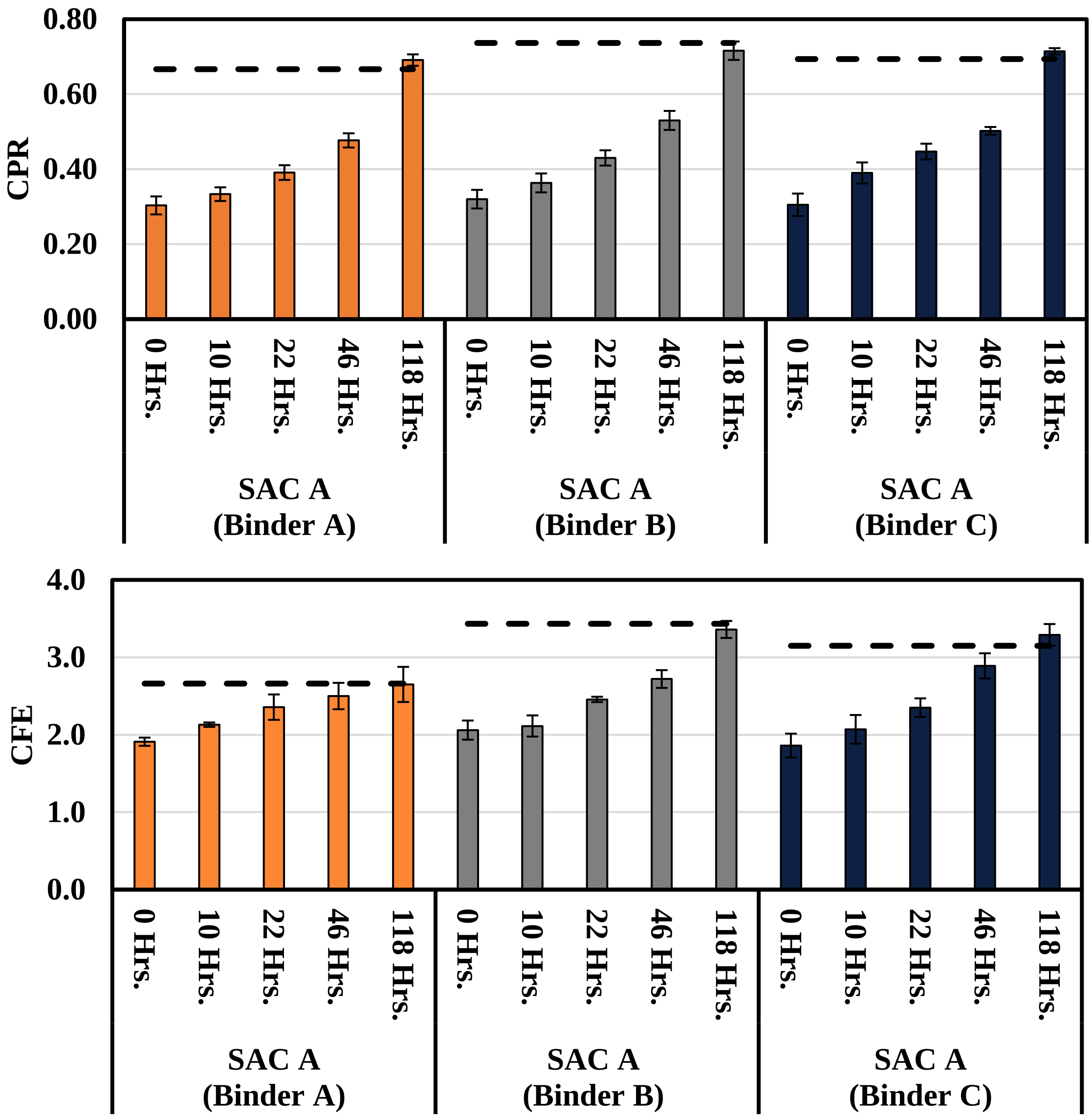


Figure 6.26. CPR and CFE Results for Change in Binder Source with SAC A (Mix 3)

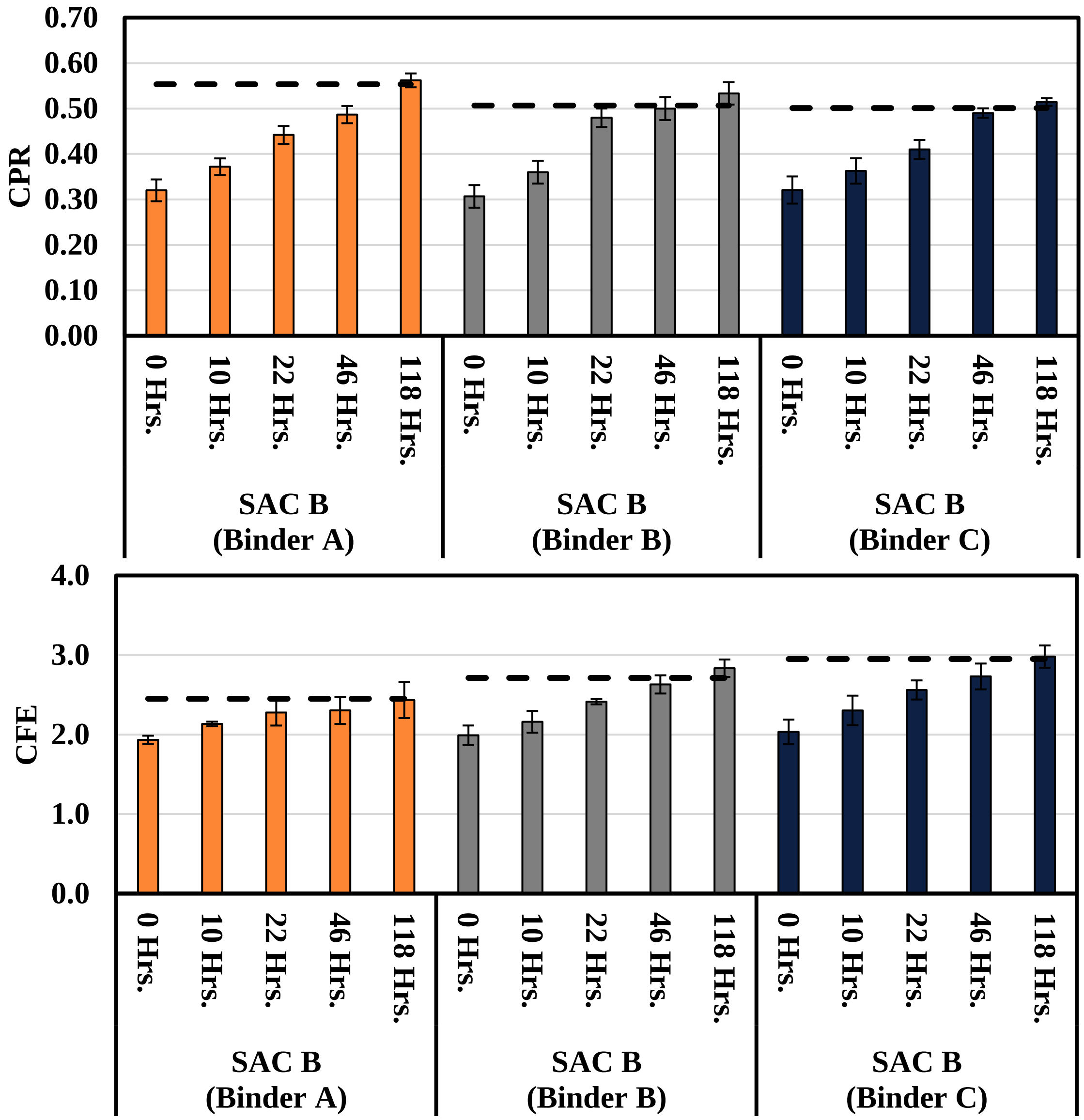


Figure 6.27. CPR and CFE Results for Change in Binder Source with SAC B (Mix 3)

Figure 6.28 illustrates the influence of changes in gradation on the cracking susceptibility of asphalt mixtures at varying aging periods. The coarser mixture performed the best and the finer mixture performed the worst. This outcome is reasonable as finer particles represent more aggregate surface area, thus more binder absorption and less effective binder availability. Lower effective binder levels lead to a poorer cracking performance, which seems to become poorer at longer aging periods.

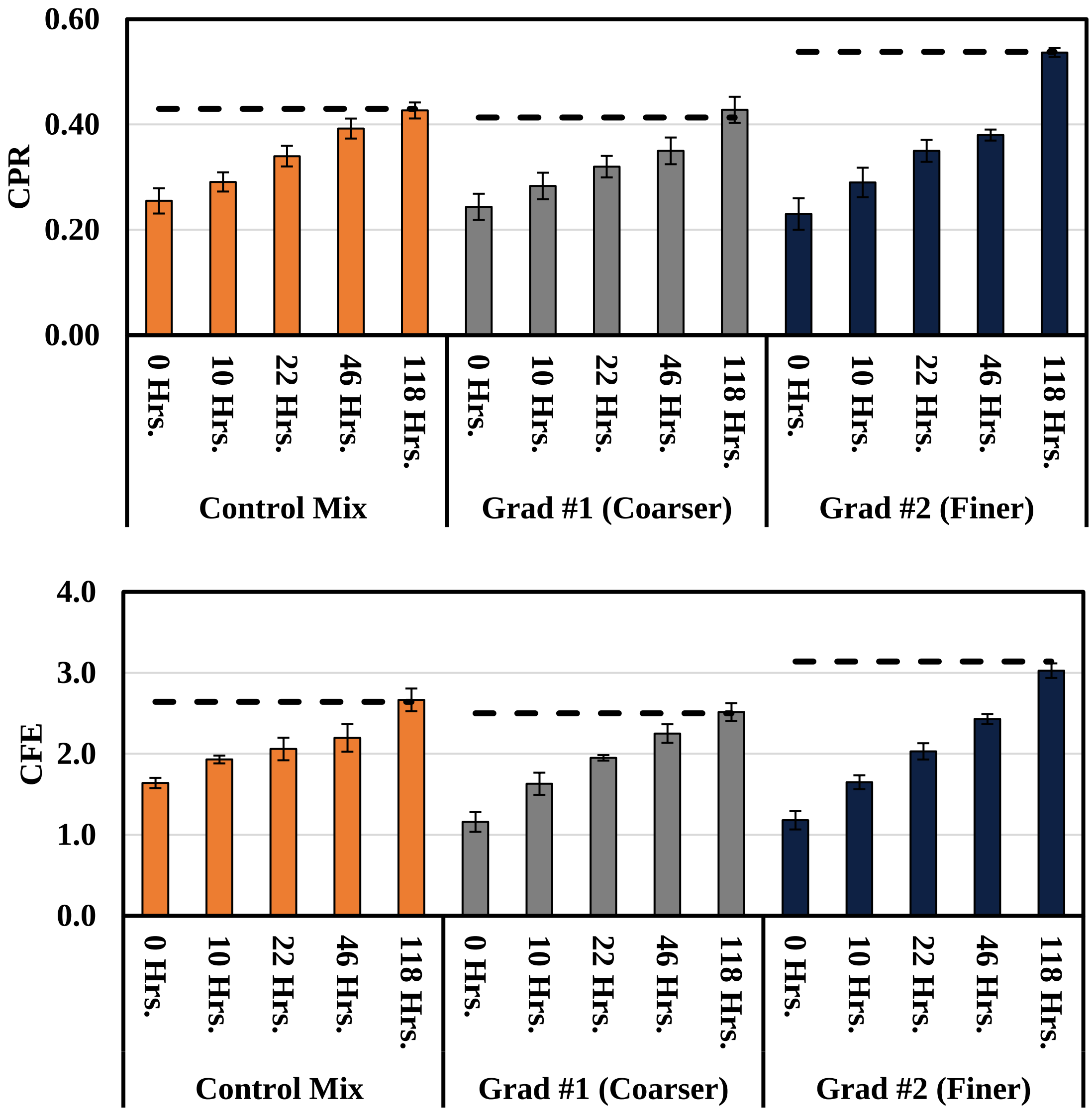


Figure 6.28. CPR and CFE Results for Change in Gradation (Mix 4)

Figure 6.29 depicts the relationship between asphalt content and cracking performance. Lower binder contents in specimens increased the cracking potential. The results indicate that reducing the asphalt binder content by 0.5% might yield similar results to having an optimum binder content, especially at shorter aging periods. However, decreasing the binder by 1.0% might drastically impact the cracking performance. This diminished performance will intensify as the mixture is aged for a longer period.

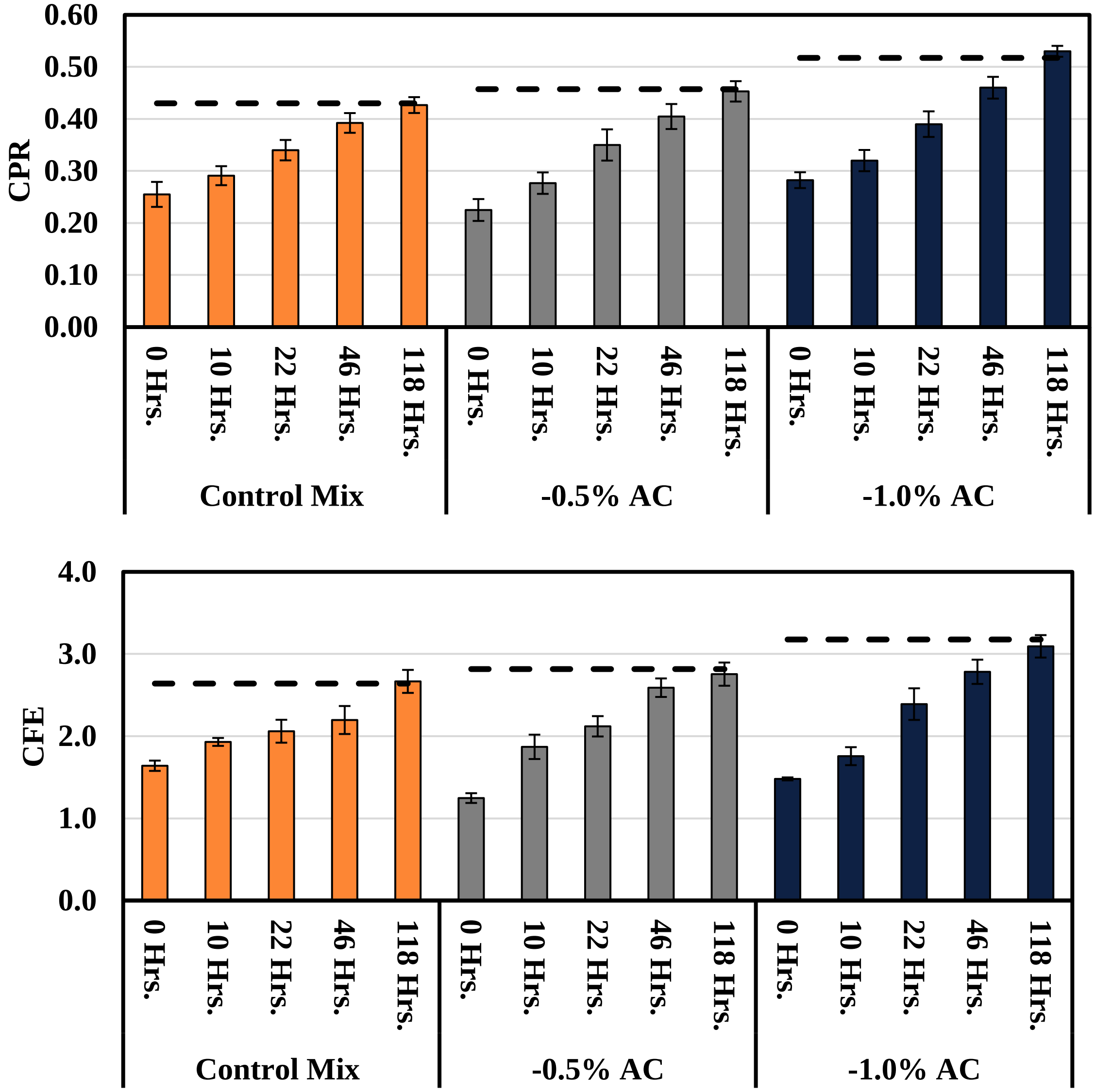


Figure 6.29. CPR and CFE Results for Change in Asphalt Content (Mix 4)

Accelerated Aging Methods: Up-Scaled Pressure Device

Work was conducted for this protocol to transform this method from its pilot stage to an advanced stage for day-to-day implementation. Figure 6.30 illustrates the pressure device along with the up-scaled setup. Changes adopted included using a dehydrating rack to maximize airflow and a quick-connect system that is more practical for increasing the amount of loose mixture that can be aged simultaneously.

Figure 6.30. Up-Scaled Pressure Device and Aging Racks

RESULTS AND DISCUSSION

A study was first performed to validate the uniformity of aging between the top and bottom tiers using three replicate samples. For that purpose, three replicate samples were exposed to 2 hrs of short-term aging at their respective compaction temperature and long-term oven aging at a constant temperature of 95°C for 22 hrs. Figure 6.31 illustrates the IDT strengths and Figure 6.32 shows the CT_{Index} values. The top and bottom tier specimens were similar in strength since the top specimens' IDT strengths ranged between 137 psi and 139 psi, and 136 psi and 140 psi for the bottom specimens. The CT_{Index} values show comparable results to the IDT strength.

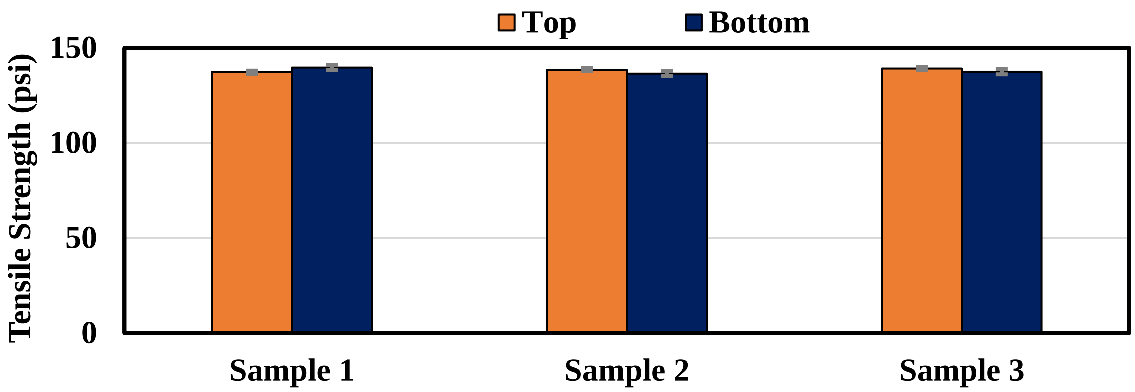


Figure 6.31. Strength Comparison Using Protocol 2

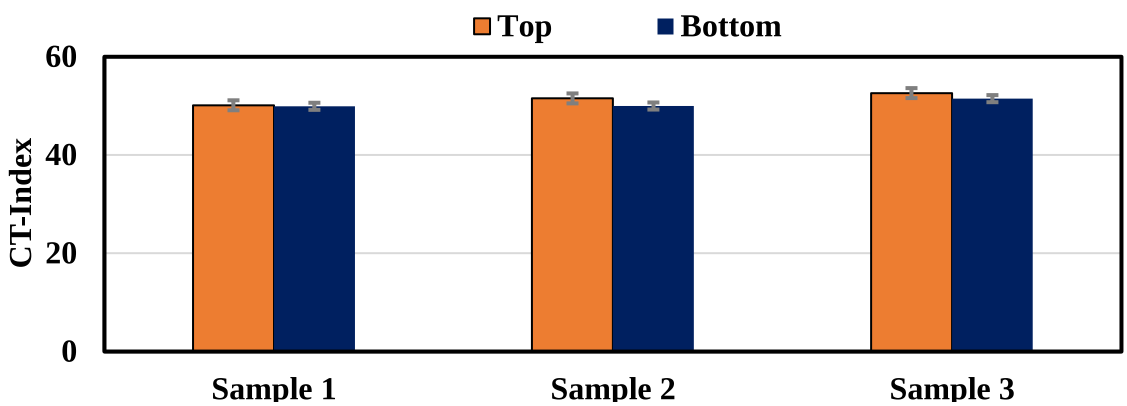


Figure 6.32. CT_{Index} Comparison Using Protocol 2

The five AC mixtures from Table 6.3 were evaluated to further validate the up-scaled setup. The Protocol 1 values from their respective performance tests were considered the benchmark values and compared to the results obtained after 22 and 46 hrs of pressure aging.

Figures 6.33 and 6.34 show the cracking performance indicators using tensile strength and CT_{Index}. Similar to previous results, the mixtures exhibited higher crack susceptibility as the aging period increased. The results from 22 hrs of pressure aging seem more mixture dependent than the results from 46 hrs of pressure aging. For most mixtures, the 22 hrs of pressure aging is below the benchmark method threshold. However, Mix 3 at 22 hrs of pressure aging matched the benchmark value of about 132 psi. The DG mixture (Mix 4) demonstrated the highest strengths at all aging periods. However, the most significant outcome is that the pressure device can replicate the benchmark strength and CT$_{Index}$ results within 46 hrs. Significantly reducing the time required to expose a loose mixture to a long-term aging condition.

Figures 6.35 and 6.36 illustrate the CPR and CFE results for the OT, respectively. These results suggest that 46 hrs of pressure aging tend to age the mixture to a greater extent in comparison to the benchmark method. Generally, the benchmark method provides an aging level between both pressurized procedures. However, the magnitude of the difference between the three aging methods varies depending on the mixture type. As an example, the CFE values for all mixes are about the same for the pressurized and benchmark methods. This means that the three aging protocols had the same level of stiffness. On the other hand, the CPR values obtained for Mixes 3 to 5 were similar for 22 hrs of pressure and benchmark. However, for Mix 2 the benchmark results related better to the 46 hrs of pressure aging. While Mix 1 showed more distinctive values between the three aging methods. Mix 4 had the lowest CPR and CFE values across all oven aging levels, suggesting that IDEAL-CT and OT parameters might discriminate DG mixtures differently. Having the highest tensile strength and a low CT$_{Index}$, do not correlate well with the low CRP and CFE results. The IDEAL-CT parameters suggest that it is more crack susceptible. Whereas the CPR and CFE values obtained show less susceptibility to cracking.

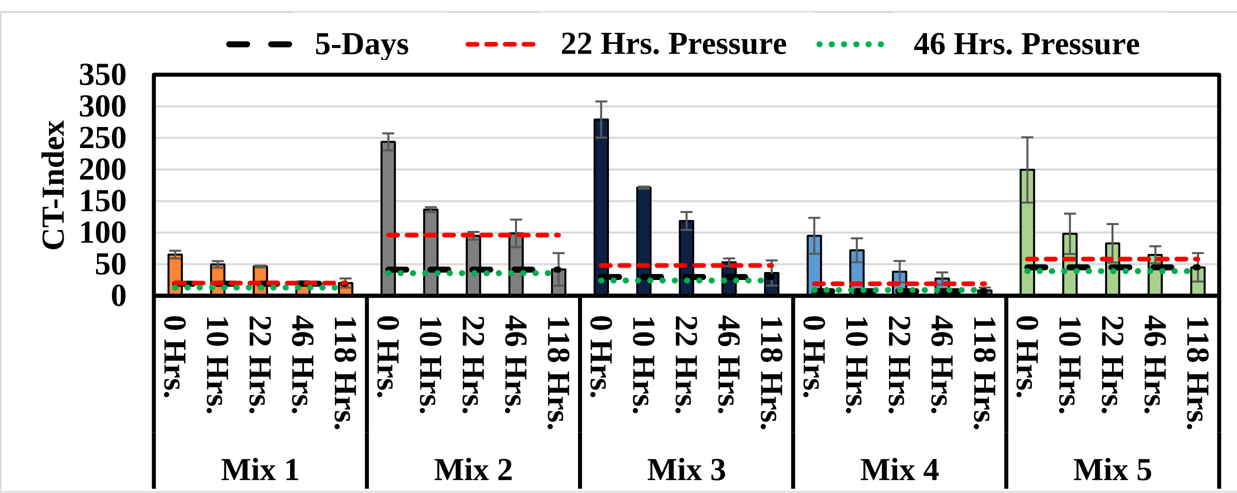


Figure 6.33. CT$_{Index}$ Verification Using Protocol 2

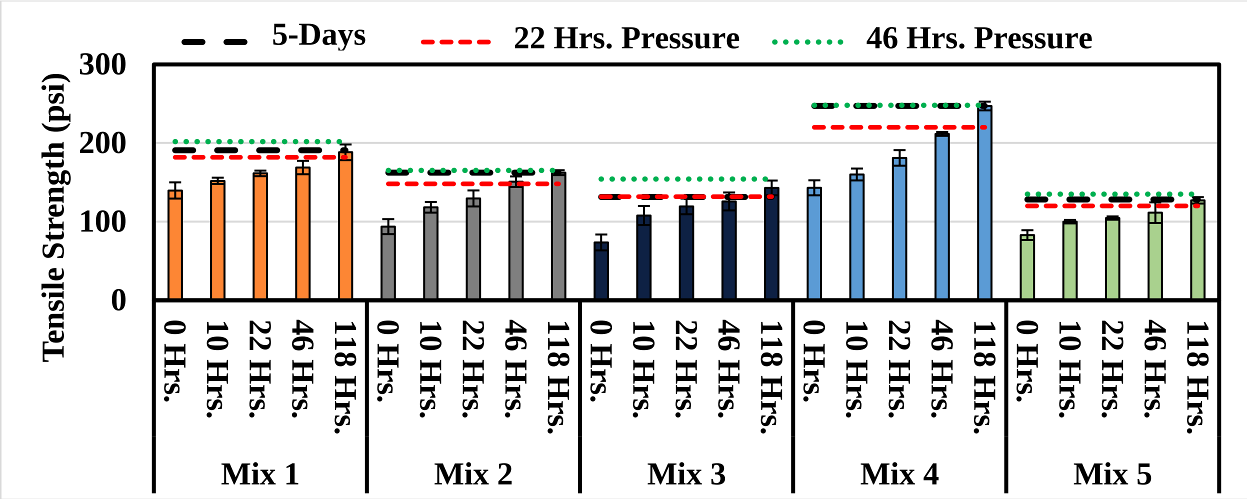


Figure 6.34. Strength Verification Using Protocol 2

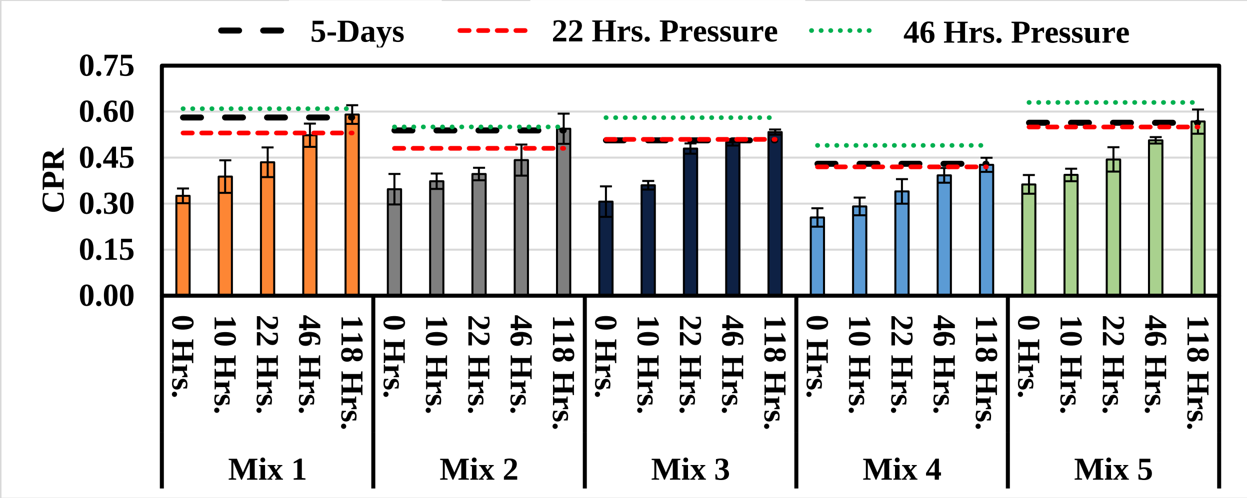


Figure 6.35. CPR Verification Using Protocol 2

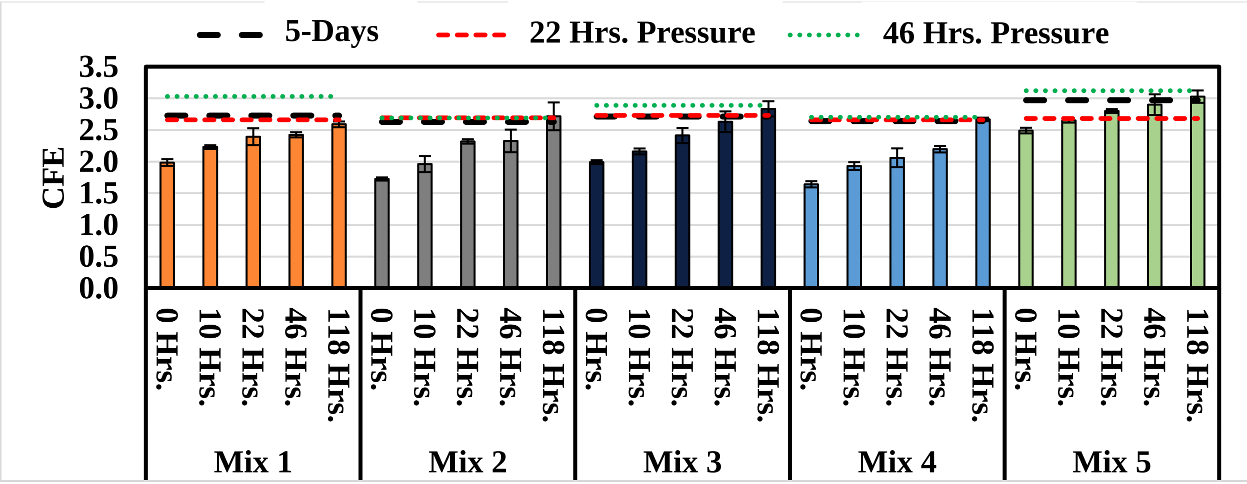


Figure 6.36. CFE Verification Using Protocol 2

SUMMARY

This chapter documented the findings from the *extended evaluation*. The most optimal aging protocols identified earlier in the project were further evaluated by studying the influence of various mix design variables on aging characteristics. Temperature, duration, pressure, and aging environment varied among these protocols. The main findings of this study based on the range of materials tested can be summarized in the following manner:

- Even though Protocol 1 is reasonable, it is impractical for day-to-day implementation due to the prolonged duration of the test to determine a mixture's susceptibility to long-term cracking.
- The aging trajectory remained generally the same when considering the use of different mix designs and variables especially in terms of tensile strength, CPR, and CFE.
- The efficiency of Protocol 2, pressure aging, has been demonstrated and its use may be considered as a technique for simulating long-term aging within 46 hrs.
- The rate of change of rutting characteristics was verified at 2 hrs of oven aging.

Chapter 7: Verification of Preliminary Laboratory Aging Protocols

Quantification of Aging Conditions

For the verification of aging protocols, the field performance of AC layers from several relevant construction projects was evaluated. To develop a database with field and lab performance data, a representative number of pavement sections relevant to the concept of this research project was used. A field evaluation and verification were conducted during the pre-construction, construction, and post-construction stages to document every phase of the project. After the completion of the field evaluation activities, the pavement sections were revisited in six-month intervals as allowed by the length of this project.

Figure 7.1 illustrates examples of the performance of three pavement sections. The hollow symbols, representing the pre-construction evaluation of the mixes, have CPR values of about 0.30 and fall within the balanced region. The black symbols, which represent the evaluation during construction, yield similar CPR values. The gray symbols, which represent the post-construction evaluation, yield higher CPR values than those during pre-construction, ranging from 0.35 to 0.45. If the performance diagram criterion were incorporated in the aged mixture design, Mix 3 would not have been accepted.

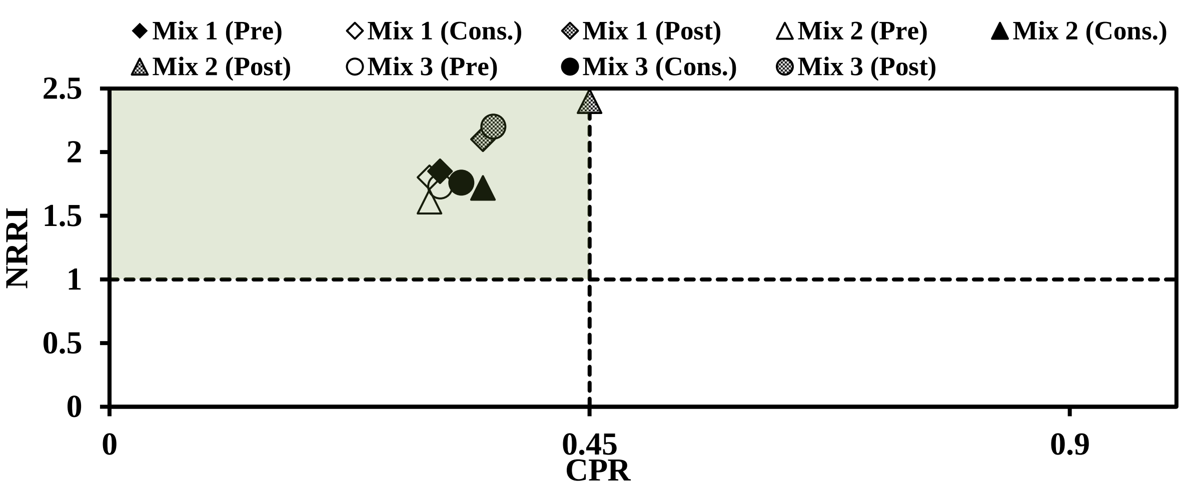


Figure 7.1. Expected Interaction Performance Plot with Balanced Mixes

Additional three mixtures were evaluated throughout a 25-month period using the OT performance test. The cross plot between the CPR and CFE shown in Figure 7.2 demonstrates examples of three balanced pavement sections with "good" crack-performing mixtures and their progression as they aged. The hollow symbols represent the initial evaluation of the mixes that exhibit a satisfactory performance for CPR and CFE. The shaded symbols represent CPR from field cores extracted 12 months post-material placement. These data points showed slightly worse CPR values but still fell within the acceptance criteria in terms of cracking. The patterned symbols represent the CPR and CFE values obtained on cores extracted 25 months post-material placement. Overall, as the pavement ages, the cracking performance deteriorates. However, these balanced mixtures do not undergo major CPR changes as the pavement ages.

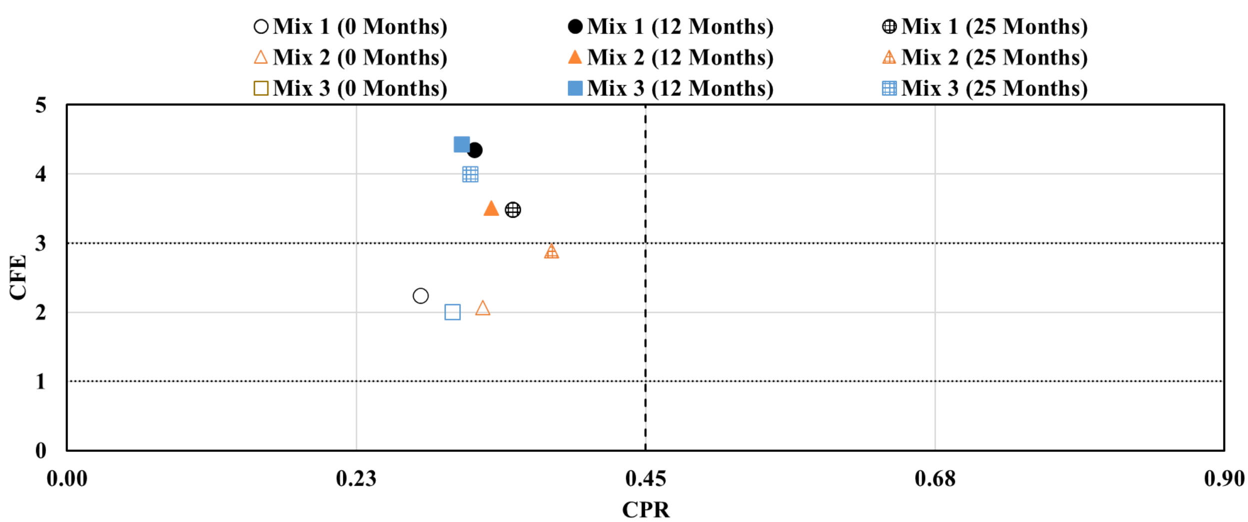


Figure 7.2. Expected Interaction Diagram for Balanced Mixes

Table 7.1 summarizes the six sections selected for field validation. Relevant data were extracted from Garcia et al. (*2020*), and pre-construction information was considered as the trial batch of the mix verification process. The pre-construction and construction data available for the projects selected are presented in Table 7.2 and Table 7.3, respectively. However, many of the sections were only placed recently, so long-term evaluation was not possible at this stage of the project.

Table 7.1. Overview of AC Mixtures

Mix No.	**Mix Type**	**Binder Grade**	**Binder Percent**	**Additives**	**RAP**
1	SP-C	76-22	5.4	N/A	20%
2	SP-D	70-22	6.1	Evotherm	20%
3	SP-D	64-22	6.0	Evoflex	30%
4	SP-C	70-22	5.3	N/A	10%
5	DG-D	70-28	5.6	Blackledge	8%
6	SP-C	70-22	5.8	N/A	20%

Table 7.2. AC Mixture Pre-Construction Performance Data

Mix No.	IDT		OT		HWTT	
	CT Index	Tensile Strength, psi	CPR	CFE, in.-lbs/in^2	NRRI	No. Cycles
1	256	95	0.29	1.83	1.69	20000
2	31	158	0.30	1.92	1.78	20000
3	226	79	0.28	1.34	1.33	20000
4	30	166	0.36	2.87	1.73	20000
5	99	119	0.30	1.71	1.74	20000
6	52	117	0.39	1.80	2.05	20000

Table 7.3. AC Mixture Construction Performance Data

Mix No.	IDT		OT		HWTT	
	CT Index	Tensile Strength, psi	CPR	CFE, in.lb-s/in^2	NRRI	No. Cycles
1	71	110	0.32	2.69	1.73	20000
2	27	175	0.35	2.04	1.91	20000
3	144	88	0.34	1.80	1.90	20000

Figure 7.3 illustrates an example of the proposed methodology for calibration of aging protocols. The hollow symbols represent a mixture used under the initial evaluation and its CPR progress. The cross symbols represent the field cores from during construction (first point) and 11-month post-material placement (second point). If the performance diagram criterion were used in the aged mixture design, this mixture would be equivalent to roughly 8 hrs of conventional oven aging.

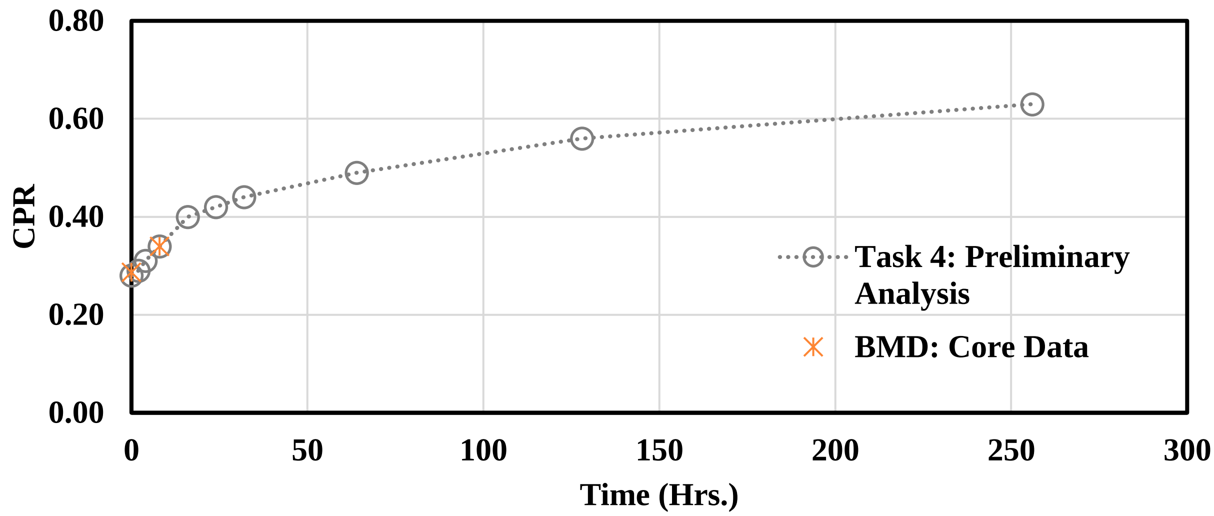


Figure 7.3. Laboratory and Field Cores Proposed for Calibration

Verification of Pressure Device

An additional 15 mixtures were evaluated with different binder PG, aggregate types, and admixtures to ensure the feasibility of the pressure. Table 7.4 presents the details related to the constituents of these 15 mixtures. Each mix was subjected to aging regimes of 5 days of aging at 95ºC as a benchmark, 22 hrs, and 46 hrs of pressure aging at 80 psi. Performance testing was conducted with IDT and OT as discussed below.

The CT_{Index} and IDT strengths at different aging conditions are compared in Figures 7.4 and 7.5, respectively. The tensile strength and CT_{Index} trends are in good agreement and consistent. The mixture types with higher tensile strength results exhibit lower CT_{Index} values.

The performance parameters from the 46 hrs of pressure aging correlate better with the comparable benchmark results than the 22 hrs of pressure aging performance parameters. However, 46 hrs of pressure aging tends to slightly age more the paving material compared to the benchmark. The variability for CT_{Index} appears to be higher when the average value is above 80.

The CPR and CFE results at different aging conditions are compared in Figures 7.6 and 7.7, respectively. Mixture 12 proved to have the highest CPR value after 46 hrs of pressure aging.

However, Mixture 9 exhibited the highest CPR value after 22 hrs of pressure aging. The differences in the performance parameters and variability between comparable specimens pressure aged for 22 hrs and 46 hrs seem mixture dependent. However, pressure aging of the specimens for 46 hrs is relatable to the benchmarked aging protocol regardless of the mixture type. Once again, all 15 mixes met or exceeded the benchmark values in 48 hrs or less.

Table 7.4. Mixtures for Validation of Protocol 3

Mix	Aggregate Type	Binder Grade	Mix Design Characteristics
1	Limestone	PG 70-22	SP-C: 10% RAP, N/A, 5.6% AC
2	Limestone	PG 70-22	SP-C: 20% RAP, N/A, 5.4% AC
3	Igneous	PG 64-22	SP-C: 10% RAP, N/A, 5.4% AC
4	Igneous	PG 70-22	SP-C: 15% RAP, N/A, 5.3% AC
5	Igneous	PG 76-22	SP-C: 10% RAP, N/A, 5.4% AC
6	Limestone/Igneous	PG 70-22	SP-C: 15% RAP, Evotherm, 5.5% AC
7	Limestone Dolomite	PG 70-22*	SP-D: 10% RAP, Evotherm, 6.1% AC
8	Limestone Dolomite	PG 70-22*	SP-D: 20% RAP, Evotherm, 6.1% AC
9	Limestone Dolomite	PG 70-22*	SP-D: 30% RAP, Evotherm, 6.1% AC
10	Limestone Dolomite	PG 64-22*	SP-D: 30% RAP, EvoFlex, 6.0% AC
11	Limestone Dolomite	PG 64-22	SP-D: 30% RAP, EvoFlex, 6.0% AC
12	Gravel *(SAC A)*	PG 70-22*	SP-C: 10% RAP, ZycoTherm, 5.3% AC
13	Gravel *(SAC B)*		SP-C: 10% RAP, ZycoTherm, 5.3% AC
14	Gravel *(SAC A)*	PG 70-22*	SP-C: 10% RAP, ZycoTherm, 5.3% AC
15	Gravel *(SAC B)*		SP-C: 10% RAP, ZycoTherm, 5.3% AC

**NOTE: Mixes 7 to 13 have three different binder sources.*

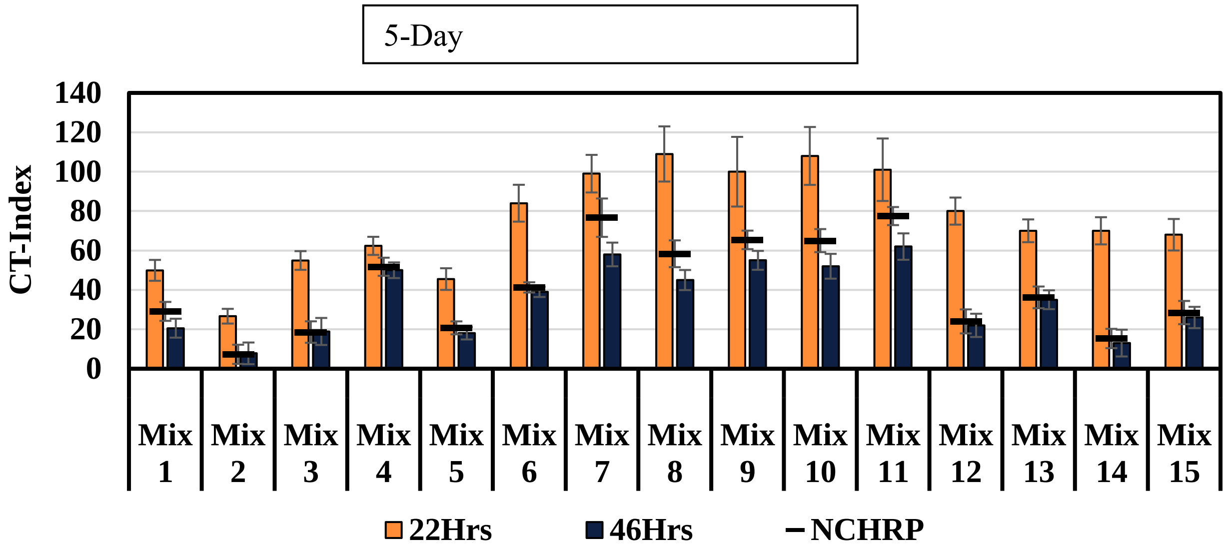


Figure 7.4. CT_{Index} Extended Evaluation Comparison Using Protocol 2

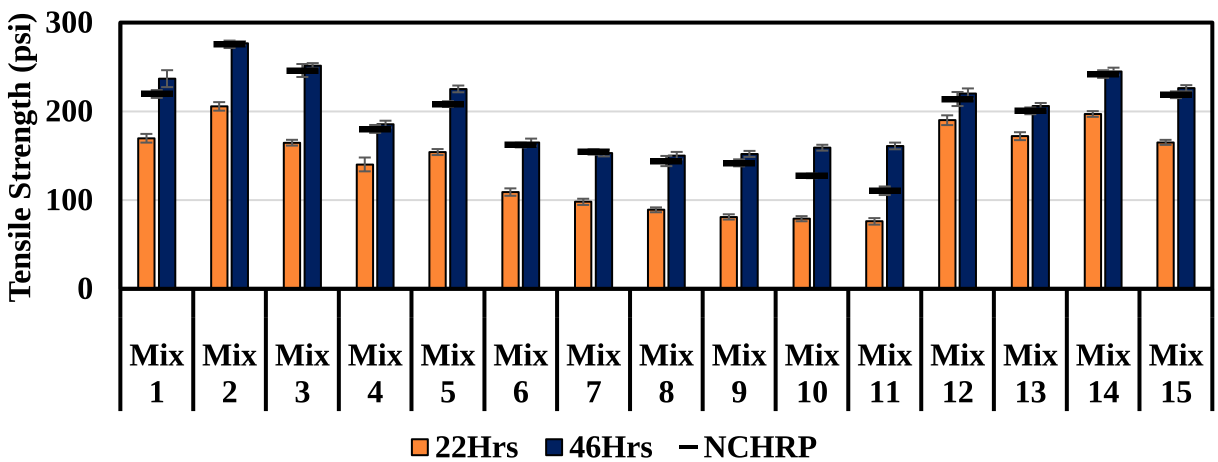


Figure 7.5. IDT Strength Extended Evaluation Comparison Using Protocol 2

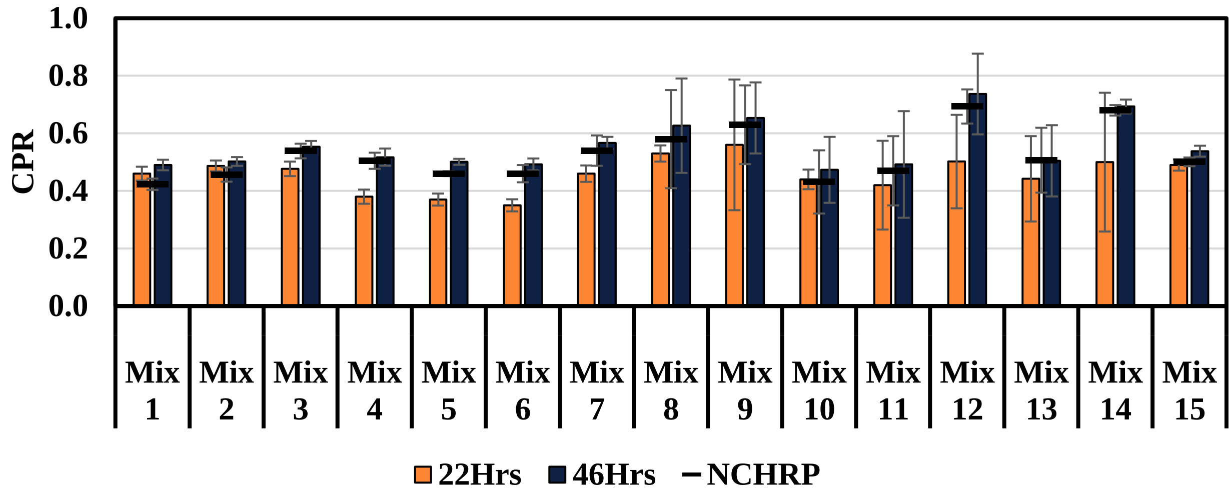


Figure 7.6. CPR Extended Evaluation Comparison Using Protocol 2

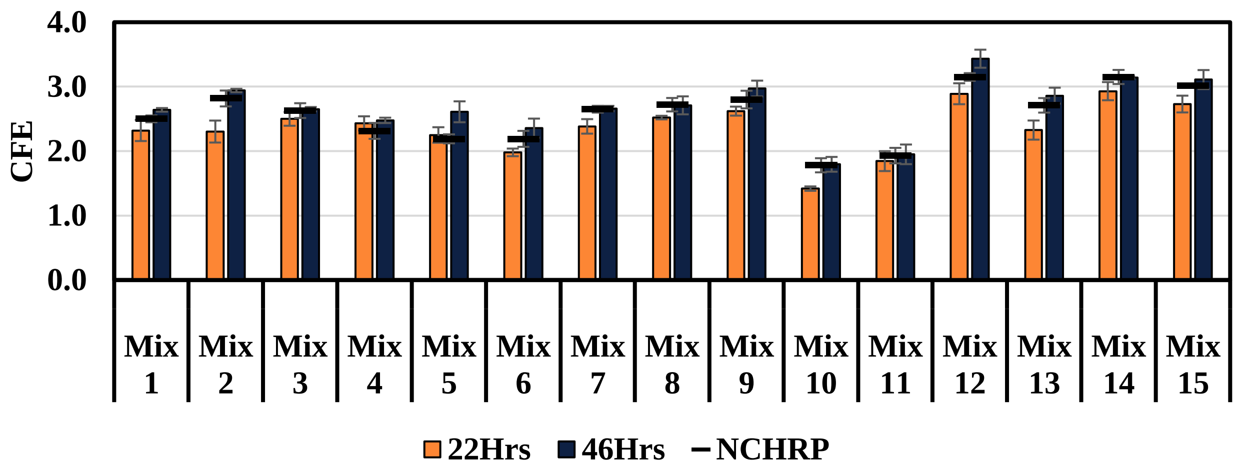


Figure 7.7. CFE Extended Evaluation Comparison Using Protocol 2

Verification of Rank-Order in Mixtures

More relevant to Protocol 1, it was hypothesized that the aging kinetics of mixtures reaches a steady-state condition after a certain amount of aging, and hence there could be a level of long-term aging after which the rank order of aging of different mixtures does not change. This will benefit a cracking test to distinguish accurately between crack-resistant and -susceptible mix.

The rank order was evaluated using all the mixtures and variations as listed in Table 6.2. Performance results from tensile strength, CPR, and CFE best captured the steady-state concept in all the variations considered including change in RAP, additives, PG binder location and grades, aggregate SAC and source, and asphalt content. A consistent rank order was preserved at different RAP contents as shown in Figure 7.8. The rank order throughout the testing period was maintained at 1, 2, and 3 in the case of RAP content incorporated into the mixture. For other variables, the rank order was unstable was not kept before 24 hrs of aging, but it then stabilized in a similar fashion to the RAP ranking order. In contrast, the CT_{Index} parameter properties were correlated to a lesser extent due to the high variability in the COV. In the case of the CT_{Index}, an inconsistent pattern was obtained throughout the aging period regardless of the variation in the mix as shown in Figure 7.9.

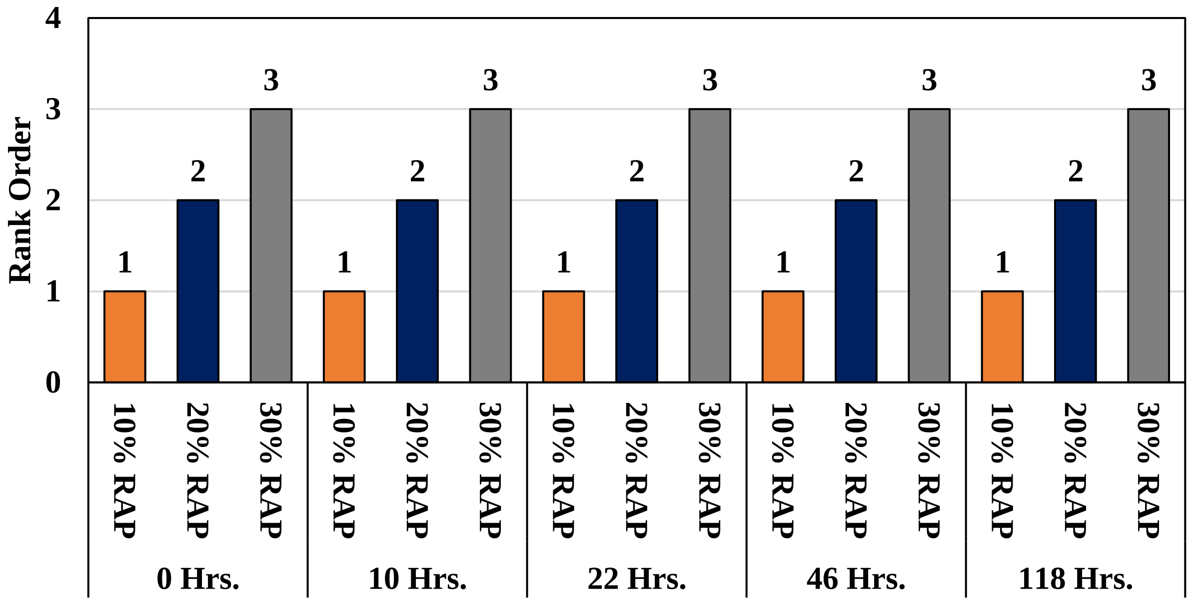


Figure 7.8. Rank-Order Pattern Considering Change in RAP for CPR, CFE, and Tensile Strength

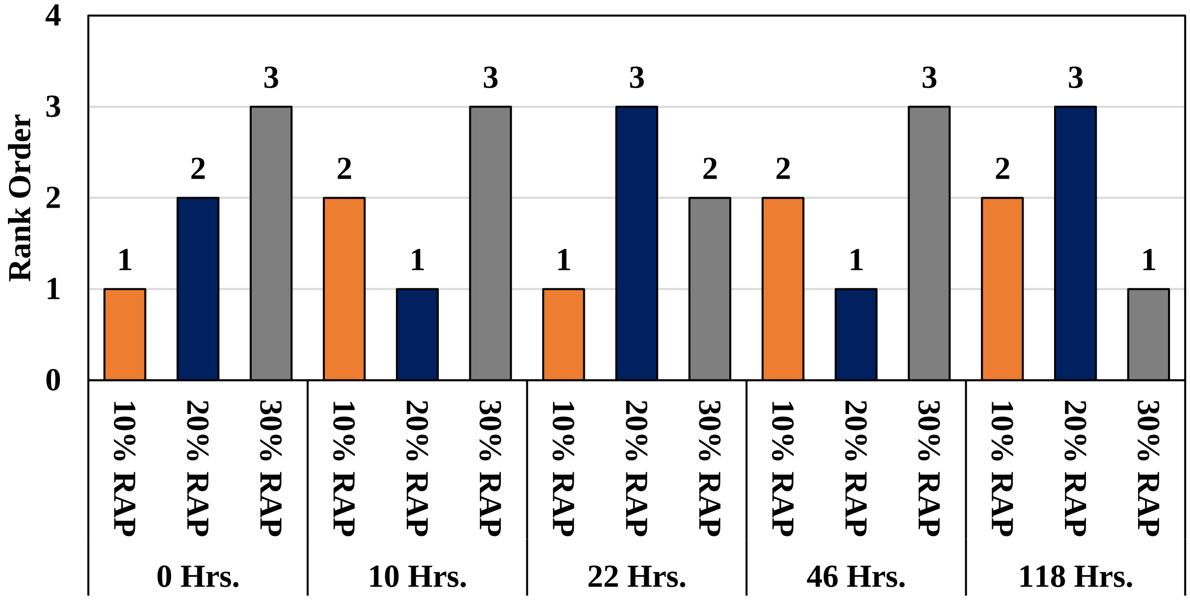


Figure 7.9. Rank-Order Pattern Considering Change in RAP for CT_{Index}

Chapter 8: Conclusions and Recommendations

Conclusions

The main focus of this study was on long-term aging and its impact on cracking performance. The following aging protocols were selected for evaluation in this study:

- Protocol 1: Loose mixture aging in a laboratory oven at a temperature of 95°C.
- Protocol 2: Loose mixture aging using a pressurized device.

Based on the extensive analysis conducted in this study, the following conclusions were drawn:

- Protocol 1 is a promising protocol for routine testing, but it is highly impractical to wait prolonged periods to determine a mixture's susceptibility to cracking.
- For Protocol 1, mixture tests showed a consistent increase in aging-related metrics with increased testing time.
- The steady state of aging i.e., the aging time and duration after which the relative rank order of cracking performance does not change for different mixes is about 24 hrs (including 2 hrs of short-term aging) under Protocol 1.
- The efficiency of Protocols 2, pressure aging, has been clearly demonstrated and its use may be considered a more efficient alternative for the day-to-day operation of realistically simulating long-term aging for rigorous evaluation.

Recommendations

- Although the work performed on this study gathered extensive data there are several other variables to fully understand the aging phenomenon of asphalt pavements in the field and capture this behavior using the recommended laboratory methods. The variables that should be further studied are additional types of mixtures (i.e., TOM, SMA, and PFC), rejuvenators, plant-produced mixtures, warm mix additives and other sustainable asphalt mixture variations such as rubber and plastic.

- To further the findings of this study testing of the field cores are required for validation of the method and building calibration models. The recommended protocols predict well the long-term aging of asphalt mixtures as demonstrated by its correlation to the standard practices. However, to tune the proposed aging protocol for short- and intermediate-aging will require collection of field cores at twelve-month intervals.
- The current study can be used to optimize the standard, specifications, and acceptance limits for performance test methods and construction of asphalt pavements. Demonstration projects should be performed to develop, implement, and adopt the aging protocols as part of the highway agencies programs.
- For Protocol 2, work is still required to transform this method to an advanced stage in terms of implementation readiness, development of optimized laboratory specifications, and development of aging models based on kinetics.

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Appendix A: AC Mixture Performance Tests

The HWT test was used in this study to evaluate the rutting susceptibility and moisture damage of AC mixtures in accordance with TxDOT Test Procedure Tex-242-F. The AC specimens are preconditioned in a water bath at 122ºF (50ºC) and subjected to a steel wheel load of 158 ± 5 lb. (705 ± 22.2 N) at 50± 2 passes/min. Figure 4 shows a schematic of the rut depth points taken by the HWT test and the wheel path the device will induce onto the surface specimens.

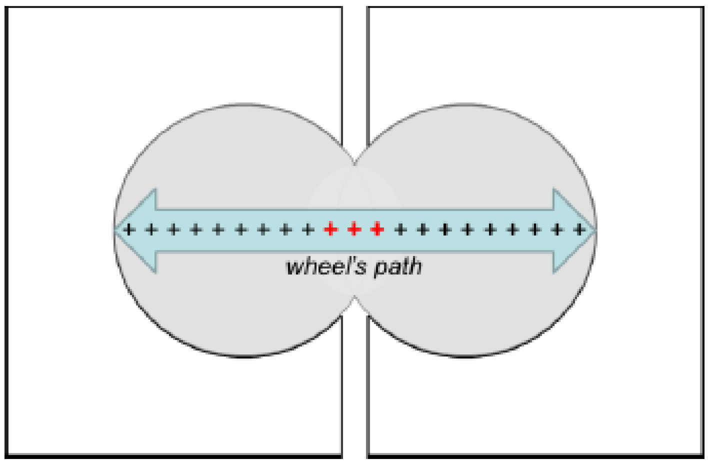

Figure A.1.Schematic of Wheel Path for HWT Test (*Tex-242-F*, *2019*)

The main output values from the HWT test are the rut depth and the number of passes the AC mixture endures. The rutting resistance index recommended by Wen et al. et al. (*2016*) is an alternative performance indicator to determine rutting susceptible mixtures, as expressed in Equation A.1:

$$RRI = N\ x\ (1 - RD) \qquad \textbf{(A.1)}$$

where *RRI* is rutting resistance index (in.), *N* is the number of passes and *RD* is the rut depth (in.).

The HWT test is run for 20,000 passes or until a rutting depth of 0.5 inches (12.5 mm) is observed. However, guidelines for the number of passes are dependent on the PG of the binder. Table A.1 shows the recommended number of passes and RRI values. In this study, only STOA specimens will be evaluated for the permanent deformation and moisture damage of AC mixtures.

Table A.1. Recommended Number of Passes and RRI for HWT Test

Binder Grade	Number of Passes	RRI Values
PG 64 or lower	10,000	5,100
PG 70	15,000	7,600
PG 76 or higher	20,000	10,100

The indirect tensile (IDT) test was performed in accordance with the TxDOT Test Procedure Tex-226-F to determine the tensile strength of the HMA mixtures. The compacted AC specimens will be preconditioned for 24 hrs at 77ºF ± 2ºF (25ºC ± 1ºC) in an environmental chamber prior to testing. The specimens were evaluated within three days from the molding period and subjected to a monotonic loading rate of 2 inches/min (50 mm/min). Equation A.2 is used to calculate the indirect tensile strength of the asphalt mixture specimens.

$$IDT = \frac{2\ x\ L}{3.14\ x\ (HxD)} \tag{A.2.}$$

where *IDT* is the indirect tensile strength (psi), *L* is the load at failure (lbs.), *H* is the height of the specimen (in.), and *D* is the diameter of the specimen (in.). All units must be kept consistent to effectively calculate the indirect tensile strength of specimens.

In addition, TxDOT Test Procedure Tex-250-F was followed to determine the cracking tolerance index (CT_{index}) of AC mixture specimens. This specification follows similar procedures as Tex-226-F for IDT test method. AC specimens compacted at room temperature will be conditioned for 2 hrs at 77ºF ± 2ºF (25ºC ± 1ºC) in an environmental chamber before testing. The acquisition system captures the time, load, and displacement data at a minimum of 40 data points per second. Figure A.2 illustrates the typical results by plotting the recorded load data against displacement. The load versus displacement curve is used to determine the work of failure, failure energy, post-peak slope and the displacement at 75% after the peak load to calculate the CT_{index}.

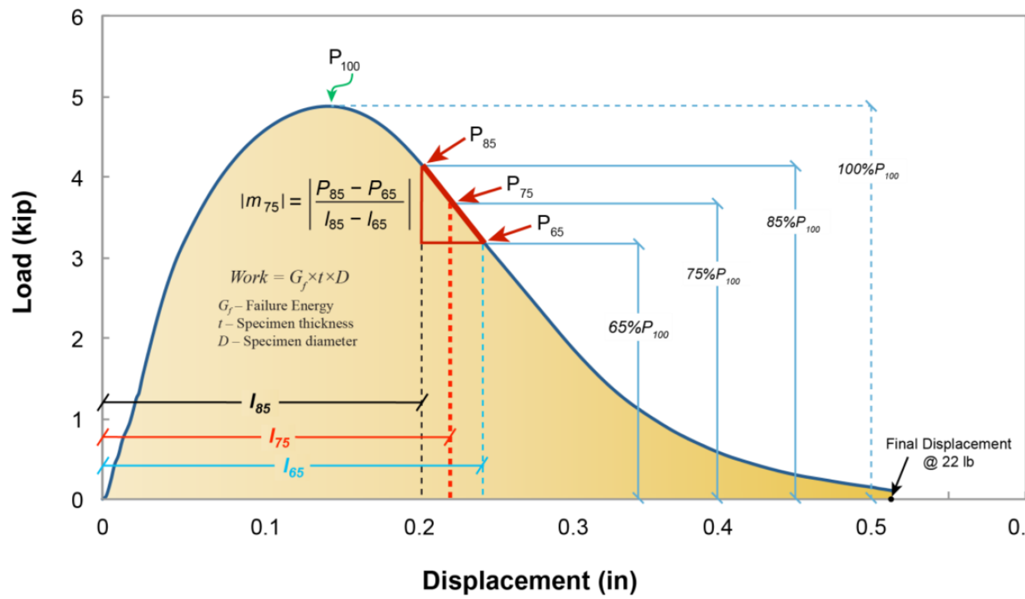

Figure A.2. Typical Results Recording Load Against Displacement Curve (*Tex-250-F, 2020)*

Equation A.3 is used to calculate the CT_{index} of an AC mixture.

$$CT_{index} = \frac{t}{2.4}\ x\ \frac{l_{75}}{D}\ x\ \frac{G_f}{|m_{75}|}\ x\ 10^6 \qquad \textbf{(A.3.)}$$

where t is the thickness of the specimen (in.), I_{75} is the displacement at 75% after the peak (in.), D is the diameter of the specimen (in.), G_f is the failure energy (lb/in.) and $|m_{75}|$ is the absolute value of the post-peak slope(lb/in.).

Work of failure is the area under the load against displacement curve using the quadrangle rule. The failure energy is calculated by dividing the work of failure by the area of the AC specimen. The $\frac{t}{2.4}$ factor is considered a unitless correction for the specimen thickness while the 10^6 is a scale factor. The proposed TxDOT test procedures to determine the cracking tolerance index (Tex-250-F) and the indirect tensile strength (Tex-226-F) were used to evaluate the performance of STOA and LTOA specimens.

The OT test was conducted in accordance with TxDOT Test Procedure Tex-248-F to assess AC mixture susceptibility to fatigue or reflective cracking. The OT applies repeated cyclic displacement in tension to the specimen. The OT machine has two plates in which the specimen is held in place with epoxy for 24 hrs. After, the specimen will be preconditioned for 1 hr at 77ºF ± 1ºF (25ºC ± 0.5ºC) in an environmental chamber prior to testing. The cyclic displacement-controlled load is applied following a triangular waveform with a maximum displacement of 0.025 in. (0.06 cm) at a loading rate of 10 sec/cycle. Figure A.3 shows a schematic of the typical layout and the set-up of the samples prepared for OT testing. Labeled as "*1"* in Figure A.3 is the gap between the two plates that is 0.16 in. (4 mm) and will be the region where the crack will propagate. Assuring the excess glue (marked as "*2*") is removed with a razor. These processes are crucial to assure the results from the specimens are accurate and there are no inaccuracies in the sample preparation process. The OT test was used in this study for STOA and LTOA conditions to assess the impact of premature cracking and in-service cracking of the asphalt mixtures.

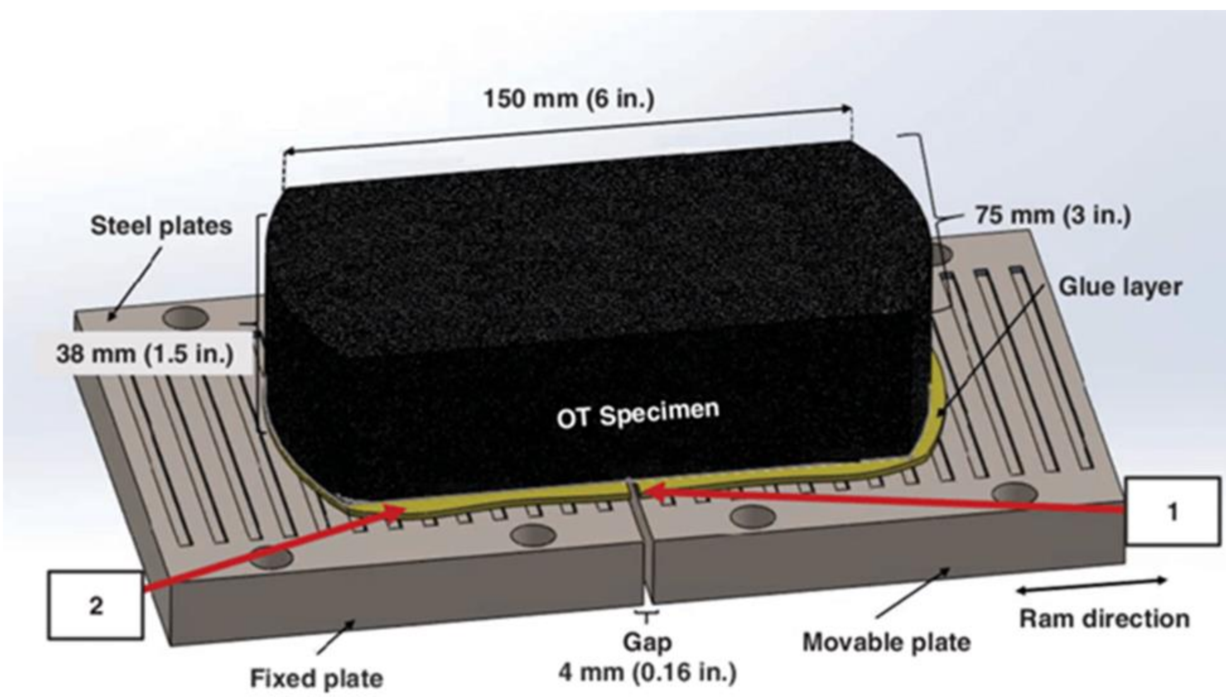

Figure A.3. Overlay Test Schematic Specimen Set-Up

Vita

Benjamin Arras is a native of El Paso, Texas; is from humble roots and raised in the border community. Benjamin holds a Bachelor's and Master of Science in Civil Engineering from the University of Texas at El Paso (UTEP). Benjamin worked as a researcher at the Center for Transportation Infrastructure Systems (CTIS), which is a nationally recognized university research laboratory that utilizes student skills by providing collaborative environment for undergraduate and graduate students to take on and solve real world problems. As an undergraduate assistant, he worked with the nondestructive testing and evaluation team to perform testing on structures to detect the presence of anomalies. As a graduate research assistant, he focused on relating the early-age concrete mechanical properties to its curing conditions. As a Ph.D. Research Associate, he worked on a research project for the Texas Department of Transportation (TxDOT) that simulated the short- and long-term aging of asphalt pavements and the distresses associated with them in the laboratory effectively through the implementation of optimized and practical aging protocols. Benjamin supported the research efforts of several other research projects funded by TxDOT. Benjamin's professional aspirations are to improve and contribute to innovation of our transportation infrastructure that will help bridge diversified comminutes. In his professional career, he plans to continue advancing as a civil engineer, researcher, and mentor.

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