

Optimized thickness of hot-mix asphalt mixture over a chemically stabilized full-depth
reclamation layer

by

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B.S., Kansas State University, 2013

M.S., Kansas State University, 2016

AN ABSTRACT OF A DISSERTATION

submitted in partial fulfillment of the requirements for the degree

DOCTOR OF PHILOSOPHY

Department of Civil Engineering
Carl R. Ice College of Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

2024

Abstract

Full-depth reclamation (FDR) is an in-place recycling process that utilizes the entire depth of deteriorated asphalt concrete (AC) pavement to produce a uniform stabilized base course and enhance new AC overlay performance. This study was conducted to determine the optimal stabilized FDR (SFDR) mixture design and thickness of an AC overlay over an SFDR base via laboratory and accelerated pavement testing (APT). SFDR mixtures were prepared using Portland Type I/II cement, Class C fly ash, and reclaimed asphalt pavement (RAP) materials. These mixtures were evaluated by the standard Proctor, unconfined compressive strength (UCS), vacuum saturation, and linear shrinkage tests. The AASHTOWare Pavement Mechanistic-Empirical Design (PMED) software (ver. 2.5) used the UCS test results as input parameters to determine the minimum thickness of the overlaying AC layers.

Four asphalt pavement test sections of the laboratory-designed SFDR mixtures and two AC overlay thicknesses (3 and 4.5 in.) were constructed at the Kansas State University (KSU) Civil Infrastructure System Laboratory (CISL) for APT tests. The test sections were instrumented to measure horizontal strain at the bottom of the AC layer, the temperature on top of the base layer, and stress on top of the subgrade. A single axle with dual tires was used to apply an 18-kip load on each test section for 500,000 repetitions. The data acquisition system intermittently recorded strain under the AC layer and stress beneath the SFDR layer. The temperature during testing was monitored. Rutting on the test sections was quantified at certain APT loading intervals. The test sections were also inspected for any signs of cracking. APT results and the PMED software (ver.2.6.2) reanalysis showed that predicted rut depths were lower than those observed in the APT test. APT testing did not show any cracking in the sections, which was in contrast with the PMED software prediction. All sections had a much lower predicted International Roughness Index (IRI) than the PMED IRI failure criterion. The APT results showed that for SFDR mixtures that met the minimum 250 psi 7-day stabilized base UCS required by the US Army Corps of Engineers, 5% cement was the most suitable stabilizer. For the SFDR mixture design, 5% cement by dry weight of FDR material would be an appropriate starting point to do an SFDR mixture design following

the US Army Corp of Engineers and Portland Cement Association guidelines. Moisture susceptibility evaluation using the vacuum saturation test is highly recommended.

A comparison of the AASHTOWare PMED results and 1993 AASHTO Pavement Design Guide computations showed that the latter can be applied to the SFDR flexible pavement design. Hence, this design method is recommended for the AC overlay design. The stabilized FDR base layer structural coefficients corresponding to the unconfined compressive strengths should be taken from the 1993 AASHTO Guide nomograph.

Finite element models (FEM) with moving and constant loading applications were developed to simulate the APT tests. The FEM outputs were compared with the APT test results. The FEM analysis with constant loading exhibited a similar trend to the APT results in terms of AC surface rutting and horizontal strain at the bottom of the AC layer but not for the subgrade stress. The pavement test sections with the cement-stabilized FDR layer showed lower rutting than those observed in the APT results. However, these sections had higher transverse and longitudinal strains measured at the bottom of the AC layer and stress on top of the subgrade. Better constitutive models for the SFDR mixtures are needed to be developed.

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Acknowledgments

I want to express my deepest gratitude to my advisor, Dr. Mustaque Hossain, for his unwavering guidance, expertise, patience, and encouragement throughout this research. I am incredibly fortunate to have had the opportunity to work under his mentorship.

I am also immensely grateful to the members of my dissertation committee, Dr. Christopher A. Jones, Dr. Jack Xin, and Dr. Eric J. Fitzsimmons, for their valuable suggestions for improving this dissertation report.

Special thanks to Mr. Randy Testa for his work on test section construction during the era of COVID-19 despite his busy schedule. Thanks to Ash Grove Cement for providing the Type I/II cement and KC Fly Ash for this project's Class C fly ash. Bayer Construction facilitated the in-situ mixing of the SFDR material, and Shilling Construction did the asphalt paving at CISL. Their cooperation was invaluable.

I also acknowledge KSU Civil Engineering Department Research Technologists Cody Delaney, Benjamin Thurlow, Andrew Schoeneck, and Clifford Temple, who assisted in this project whenever needed. Their dedication and professionalism have played a crucial role in ensuring the smooth progress of this project. Thanks to Grace (Ya) Gao, Binxiang (Gibson) Yu, Colyn Gugelman, and Jacob Manninen for their help in various phases of the laboratory work.

I want to express my deepest gratitude to my family for their unwavering support throughout this project: Qiuying Li and Louie Wu, Huazheng Wu and Aimei Li, and Feng Cai and Linshun Li. Without your support, this dissertation would not have been possible.

Finally, I want to thank the Kansas Department of Transportation for sponsoring this project, which made this research possible. Greg Schieber, P.E., was the project monitor and provided invaluable guidance, support, and encouragement throughout this project.

List of Abbreviations

5C	5% cement
5C&5FA	5% cement and 5% fly ash
AADT	average annual daily traffic
AADTT	annual average daily truck traffic
AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
AMPT	asphalt mixture performance tester
ANOVA	Analysis of Variance
APT	accelerated pavement testing
ARRA	Asphalt Recycling & Reclaiming Association
ASTM	American Society for Testing and Materials
C	Type I/II Portland cement
C3D8R	3D 8-noded linear brick element with reduced integration
C3D20R	3D 20-noded linear brick element with reduced integration
CBR	California bearing ratio
CISL	Civil Infrastructure System Laboratory
CIR	cold in-place recycling
CO ₂	carbon dioxide
COV	coefficient of variation
DAQ	data acquisition
DCP	dynamic cone penetrometer
DOT	Department of Transportation

E*	dynamic modulus
ESAL	18-kip equivalent single axle load
FA	Class C fly ash
FDR	full-depth reclamation
FE	finite element
FEM	finite-element model
FHWA	Federal Highway Administration
FPASS	Flexible Pavement Analysis Structural System
FWD	falling weight deflectometer
HMA	Hot-mix asphalt
HWTD	Hamburg wheel-tracking test device
IPC	industrial process control
IRI	International Roughness Index
ITS	indirect tensile strength
KSU	Kansas State University
KT	Kansas test standard
KDOT	Kansas Department of Transportation
LL	liquid limit
MDD	maximum dry density
MEPDG	mechanistic-empirical pavement design guide
NCHRP	National Cooperative Highway Research Program
NMAS	Nominal maximum aggregate size
OL	overlay

OMC	optimum moisture content
PCA	Portland Cement Association
PD	permanent deformation
PG	performance grade
PennDOT	Pennsylvania Department of Transportation
PL	plastic limit
PMED	pavement mechanics-empirical design
RAP	reclaimed asphalt pavement
SFDR	stabilized full-depth reclamation
SGC	Superpave Gyratory Compactor
TDR	time-domain reflectometer
TXDOT	Texas Department of Transportation
USACE	U.S. Army Corps of Engineers
USCS	Unified Soil Classification System
UCS	unconfined compressive strength
VA	air void
VFA	voids filled with asphalt
VMA	voids in the mineral aggregate
VS	vacuum saturation
VSR	vacuum saturation ratio
WP	wheel path

Chapter 1 - Introduction

1.1 Background

Cold recycling (CR) of asphalt concrete (AC) pavements consists of cold in-place recycling (CIR), cold central-plant recycling, and full-depth reclamation (FDR). In the CR processes, existing asphalt courses, base, subbase, or subgrade materials are recycled to produce a base course via pulverization and compaction at ambient temperature (Xiao et al., 2018). In general, the CR technique is typically applied to existing pavement in poor or mediocre condition to eliminate most pavement structural distresses, such as rutting, potholes, and raveling, while maintaining and improving the cross-slope and profile of the existing pavement (Jahren et al., 1999). Previous studies indicate that CR methods are sustainable alternatives for maintaining, rehabilitating, or reconstructing AC pavements due to their minor environmental impact and reduced transportation and construction costs (Alkins et al., 2008).

Kansas ranks fourth among the states in terms of public road system by miles, with 139,181 miles in 2020. Approximately 89% of its paved highway and local road network is surfaced with AC materials. About 110,000 miles of road are unpaved, and many of these roads have evolved from simple farm-to-market routes to significant highways through successive applications of chip seals and thin asphalt overlays. These pavements were not designed to handle the stress of modern traffic, according to the Kansas Department of Transportation (KDOT, 2022). The Transportation Advocacy Group (TRIP) reported that approximately 14% of Kansas's roadways were in poor or mediocre condition in 2024. Kansas drivers lose \$629 million a year (\$306 per driver) due to driving on roads in such conditions; the losses stem from increased tire wear, fuel consumption, and automotive repairs (TRIP, 2024). To help repair these deteriorated roads, conventional pavement rehabilitation techniques and improve pavement performance, such as thick overlays

and full reconstruction, can be costly. Consequently, there is a growing need for more cost-effective, environmentally sustainable, and efficient rehabilitation strategies to repair pavements without placing undue strain on public budgets. The FDR technique has emerged as a popular technique. FDR is a CIR asphalt pavement rehabilitation method that utilizes the entire depth of deteriorated AC pavement to produce a uniform stabilized base or subbase course to improve new AC overlay performance. Stabilized FDR (SFDR) material is formed when all existing AC layer materials are reused and mixed with stabilizing agents. According to the Asphalt Recycling and Reclaiming Association, the SFDR mixture achieves the desired design strength with adequate durability and resistance to moisture sensitivity (ARRA, 2015).

1.2 Problem Statement

Transportation agencies are interested in further developing new AC pavement rehabilitation processes utilizing a stabilized FDR base with a thin AC overlay to lower construction costs and improve pavement performance. However, the performance of FDR-based pavements depends on the new asphalt overlay thickness and the strength of the SFDR base layer. Inadequate AC overlay thickness could lead to premature pavement failures, such as rutting and cracking; on the other hand, weak base strength can result in AC overlay rutting under traffic loads. However, higher base strength can lead to reflective cracking in the overlaying asphalt layer, undermining the rehabilitation effort's longevity and cost-effectiveness.

Although the use of fully reclaimed asphalt pavements for road rehabilitation and reconstruction has increased in popularity over the past three decades, only a few state highway agencies and organizations have already developed their guidelines and specifications for SFDR mix design, particularly for FDR mixtures utilizing cement and fly ash as stabilizing agents. There is no nationally accepted design standard (Xiao et al., 2018). This lack of standardization could lead

to inconsistencies in project specifications across the states and bring a challenge in optimizing performance and ensuring the long-term durability of pavement. This study aimed to fill this gap by developing and validating a design methodology that optimizes cement and fly ash use in the FDR process and the AC overlay thickness. These designs would be evaluated through full-scale, accelerated load testing.

1.3 Objective

The primary objective of this study was to develop a practical and cost-effective design method for optimizing the thickness of asphalt overlays placed on the SFDR base. The aim was to develop design guidelines for SFDR mixtures with Portland cement and fly ash to achieve the desired strength and durability of the stabilized base. Another objective was to validate the performance of thin asphalt overlays placed on the SFDR base through full-scale accelerated pavement testing (APT) and numerical simulation using the ABAQUS software.

The SFDR materials were prepared by blending FDR materials with various quantities of Portland cement and fly ash. Then, the SFDR properties were evaluated by conducting laboratory tests for optimum moisture content (OMC), maximum dry density (MDD), unconfined compressive strength (UCS), moisture/freeze-thaw resistance, and shrinkage. The laboratory-based properties were used as inputs for the AASHTOWare Pavement ME Design (PMED) software to determine the optimum thickness of the AC overlay placed over the SFDR base. To validate the performance of the PMED-designed pavement structures, four test subsections were built at the Civil Infrastructure System Laboratory (CISL) of Kansas State University (KSU). All sections had the SFDR base layer with various cement and fly ash quantities or only cement. The sections were paved with multiple hot-mix asphalt (HMA) concrete thicknesses. An 18-kip single

axle load was applied repeatedly to the pavement sections. Finite element models were developed using ABAQUS to simulate the full-scale APT tests.

1.4 Synopsis of Research Plan

The study was conducted in two phases. In the first phase, laboratory experiments were conducted to determine the optimal mix designs for the SFDR base mixture. The AASHTOWare Pavement ME Design software was then used to determine the optimum thickness of the AC overlay on top of the SFDR base layer. Two of the four test sections constructed at CISL contained SFDR with 5% cement & 5% Class C fly ash, and the other two sections had 5% cement only. The test sections were paved with an AC overlay of two thicknesses. Additionally, each section was equipped with pressure cells, strain gauges, and thermocouples to monitor the mechanical and thermal response.

A full-scale APT was conducted on the test sections in the second phase. The performance of these test sections was monitored and evaluated to assess the effectiveness of the design method used in this study. A three-dimensional (3-D) finite element (FE) model (FEM) was created using the commercial FE software ABAQUS. The FE model predicted results were then compared with the APT results.

1.5 Report Outline

The dissertation report is organized into nine chapters:

Chapter 1: Introduction - provides the introduction to this report.

Chapter 2: Literature Review - provides a comprehensive overview of existing studies on FDR stabilization techniques, material properties, and performance evaluation of pavements with thin asphalt overlays.

Chapter 3: Research Methodology - outlines the experimental design, material mixing, laboratory testing procedures, and the PMED software.

Chapter 4: Laboratory Testing and Analysis- presents the results of the laboratory tests conducted on the stabilized FDR mixtures and discusses PMED software outputs for pavement design.

Chapter 5: Full-Scale Testing (Phase II) - describes the construction, instrumentation, and accelerated pavement testing (APT) of the test sections at the Civil Infrastructure Systems Laboratory (CISL).

Chapter 6: Full-Scale Test Results - analyzes the performance data obtained from APT, focusing on rutting, strain measurements, and pavement condition.

Chapter 7: Finite Element Modeling (FEM) Program – discusses the ABAQUS FEM for simulating the full-scale APT subsections.

Chapter 8: FEM Analysis Results - compares the FEM results with the APT test results based on the AC surface rutting, horizontal strain at the bottom of the AC layer, and vertical stress on top of the subgrade.

Chapter 9: Conclusions and Recommendations - summarizes the study's findings and provides recommendations based on the research outcomes.

Chapter 2 - Literature Review

This chapter introduces the FDR technique and includes a review of the properties, stabilization techniques, performance testing, and design approaches related to SFDR mixtures, providing a critical foundation for the study presented in this dissertation report. This section also reviews previous research on Full-Depth Reclamation (FDR), focusing on the stabilization techniques employed and the performance evaluation of flexible pavements.

2.1 Introduction

FDR is a cold-in-place recycling (CIR) technique that recycles the existing asphalt pavement layers and underlying base materials into a new base or subbase. The FDR process consists of pulverization, blending, reshaping, and compaction, as shown in Figure 2.1. Studies have indicated that the FDR technique is advantageous in terms of economic and environmental benefits and pavement performance improvement (ARRA, 2015).

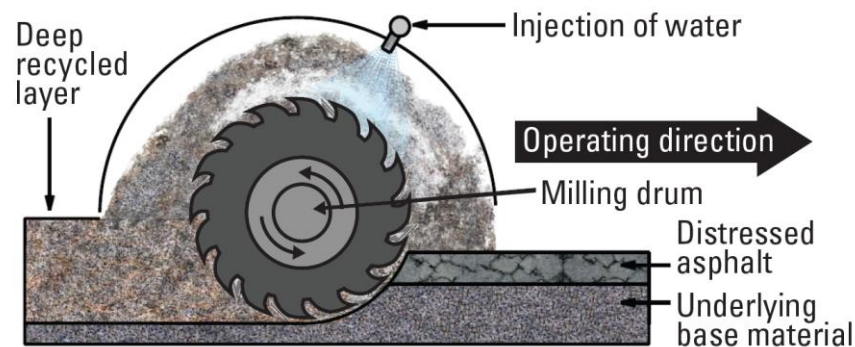


Figure 2.1 Schematic of FDR Process (Reede et al., 2017)

The earliest documented applications of FDR were reported in the 1910s when recycled pavement materials were primarily utilized for base-course construction (Morian et al., 2012). During this period, bituminous stabilization, typically using asphalt, emerged as the most prevalent method for enhancing the reclaimed materials' strength and stability. In the 1970s, the FDR

technique became popular due to economic considerations and a growing awareness of the need for energy conservation (Xiao et al., 2018). After the 1980s, the application of FDR gained wider acceptance, extending beyond its initial use on low-volume roads to encompass city streets, state highways, airport roadways, and parking lots. During this period, the engineers adopted cement, fly ash, and lime as stabilizers to blend with FDR material to enhance its stiffness (Lewis et al., 2006). Starting in the 2000s, with developments in materials science, pavement design methodologies, and construction equipment, FDR technology emphasized performance-based specifications, life cycle cost analysis, and integrating FDR with other pavement rehabilitation strategies. Transportation engineers focused on optimizing the FDR mix design and investigating the long-term performance of the FDR-based pavement structure (Xiao et al., 2018).

2.1.1 Economic and Environmental Benefits

FDR strategy significantly reduces energy consumption compared to conventional pavement methods, typically hauling virgin materials, and disposing of old materials. The Portland Cement Association (PCA) reported that using SFDR materials reduced the consumption of new materials by over 90% and diesel fuel by 80% compared to a standard reconstruction project (PCA, 2005). Nevada DOT reported saving over \$600 million over 20 years using CIR and FDR instead of complete reconstruction (Bemanian et al., 2016). Virginia DOT found potential cost savings of approximately \$10 million and \$30.5 million over 50 years using SFDR compared to the traditional primary and secondary networks approach, respectively (Diefenderfer & Apeagyei, 2011).

In addition, the FDR process consumes less energy and generates fewer environmental concerns because the process occurs at ambient temperature. Snizek et al. (2015) used a carbon footprint methodology to compare five paving methodologies' energy consumption and emissions. The results showed that CIR consumed approximately 20.4% of the energy required for

conventional HMA paving, which indicated that it is the most energy-efficient option among the methods analyzed. The reduced energy consumption also resulted in lower emissions of carbon dioxide (CO₂), nitric oxide (NO_x), hydrocarbon (HC), carbon monoxide (CO), and particulate matter.

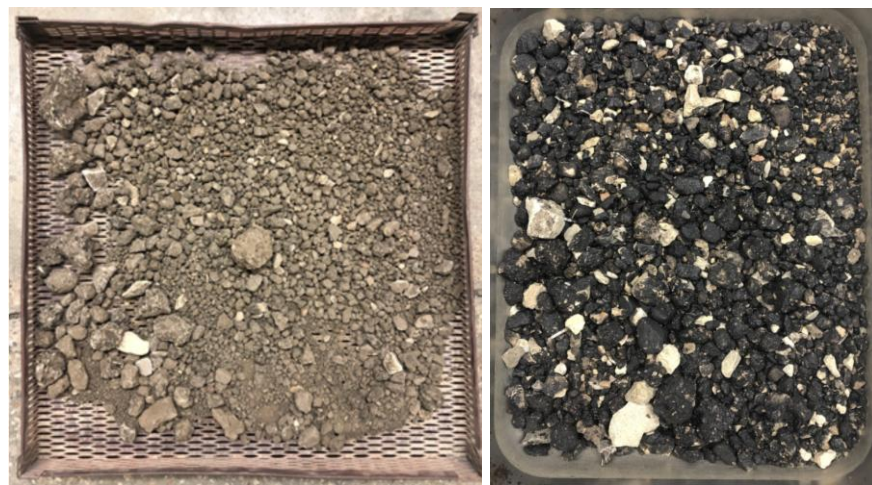
2.1.2 Pavement Performance Improvement

The FDR technique involves pulverizing the entire deteriorated asphalt layer with the underlying base layer and a partial/upper part of the subgrade. The blending process in FDR produces a more homogeneous material while eliminating inconsistencies and weak spots in the existing pavement structure (Thomas et al., 2007). The Pennsylvania Department of Transportation (PennDOT) reported that FDR directly mitigated surface defects, such as rutting, flushing, and slipperiness, by pulverizing the existing distressed asphalt layer and incorporating it into the recycled base. This process effectively removes the source of these defects and creates a more uniform surface for a new wearing course (Jahren, 2018). Garrett et al. (2017) demonstrated that FDR with chemical stabilization significantly reduces the occurrence of rutting and fatigue cracking, which are significant contributors to pavement failure. FDR is a comprehensive approach to pavement rehabilitation that removes existing pavement distresses and strengthens the entire pavement structure (ARRA, 2015).

2.2 Properties of FDR Material

FDR materials are typically produced by milling the deteriorated asphalt layer, the underlying base, and a portion of the subgrade, resulting in a uniform material, as shown in Figure 2.2. These materials include aggregates coated with aged binder, water, virgin aggregates, and fine materials. Figure 2.3 illustrates compressive strength (UCS) test specimens of unstabilized and stabilized FDR materials. Figure 2.3 (a) shows that the unstabilized FDR specimen showed poor

mechanical behavior, including brittleness and low load-bearing capacity. Figure 2.3 (b) shows some visible cracks on the SFDR specimen, but it maintained a more cohesive structure. This indicated significant improvements in load-bearing capacity. The reclaimed asphalt pavement (RAP) particle size from milling the deteriorated asphalt layer significantly impacts the SFDR effectiveness. Large RAP particles can hinder the reclaimer's ability to create a uniform and well-mixed SFDR layer (Reeder et al., 2019). It is noted that KDOT requires 100% of the RAP particles to pass 1-1/4" sieve for mixing with stabilizers (KDOT, 2016).



(a) Oven-Dried FDR Materials (b) Clean FDR Materials
Figure 2.2 FDR Material



(a) FDR Specimen (b) SFDR Specimen
Figure 2.3 Comparison of UCS Test Specimens

2.3 Stabilization Techniques for FDR Materials

ARRA (2015) has categorized FDR stabilization as mechanical, chemical, and bituminous. Because each stabilizer has unique characteristics, selecting a suitable stabilizer is primarily based on material properties, cost, long-term durability, and local policies.

2.3.1 Mechanical Stabilization

Mechanical stabilization requires the addition of granular aggregates (new or recycled) to existing subgrade materials to improve AC pavement's load-carrying capacity through enhanced soil-aggregate particle interlock. This technique primarily relies on manipulating existing pavement materials and their gradation to achieve the desired strength and stability. This method requires the existing pavement materials (aggregates) to reach a certain level of quality without any chemical stabilizer modification. PennDOT reported that mechanical stabilization is the most cost-effective FDR approach but provides less strength gain than chemical stabilization (Morian et al., 2012).

2.3.2 Chemical Stabilization

Chemical stabilization, also known as cementitious stabilization, blends cement, fly ash, or lime with FDR materials to increase the stabilized layer's strength via chemical reactions with water.

2.3.2.1 Cement Stabilization

Cement is a widely used stabilizer in FDR due to its easy availability and ability to significantly increase the recycled base's strength, stiffness, and durability. When blending the FDR materials with cement and water, the cement hydration creates a hardened matrix that binds the FDR aggregates together, increasing its strength and stability (Afrin, 2017). Garrett et al. (2017) stated that this water/cement hydration process is crucial for the effectiveness of cement

stabilization; it fills the voids between the particles, reducing permeability and increasing the material's load-bearing capacity. Cement-stabilized FDR bases distribute the wheel load uniformly across the pavement depth profile, the stress spread over a larger area in the base layer, and the load more evenly across the subgrade layers. This reduces the occurrence of rutting and extends the pavement's service life (Reeder et al., 2019).

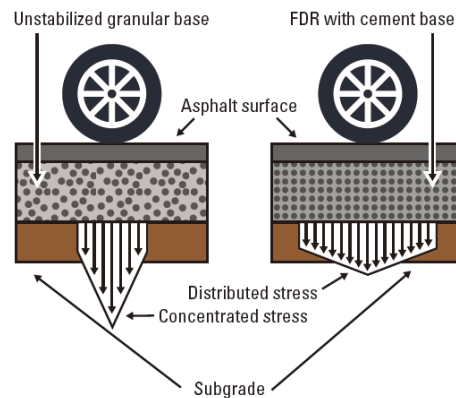


Figure 2.4 Stress Behavior between FDR and Cement-Treated FDR Layers (Reeder et al., 2017)

Owino et al. (2022) conducted a comprehensive nationwide survey on the design and construction practices in producing SFDR mixtures. The survey included responses from 361 participants across 41 states; the results revealed that over 80% of the respondents utilized cement as the primary stabilizer in FDR projects (Figure 2.5).

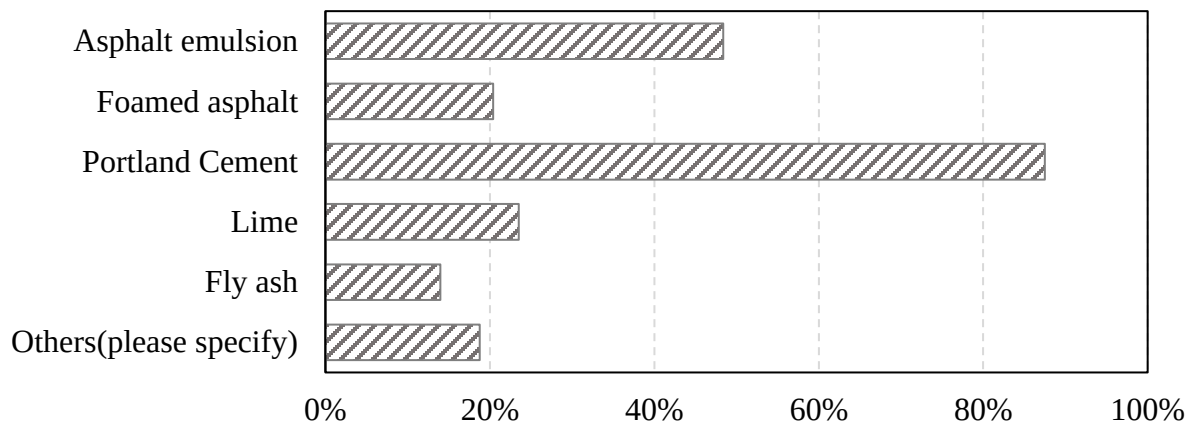


Figure 2.5 Survey Results of Stabilizer Selection for FDR Projects (Owino et al., 2022)

Studies have indicated that cement stabilization's effectiveness depends on the appropriate selection of cement content; there is concern about the range of cement content that maximizes strength without causing excessive shrinkage or environmental impact. PennDOT recommends that SFDR mixtures' cement content range from 3% to 8% by weight of the dry existing base course (Morian et al., 2012). Higher cement content improves the FDR base's strength and causes shrinkage cracking during the hydration and curing process. This shrinkage happens due to the cement-treated FDR mixture losing moisture, resulting in drying shrinkage. The progressive strain also develops with the mixture's drying shrinkage. These cracks can propagate through the asphalt overlay, leading to premature pavement failure due to reflection cracking. Jähren et al. (1999) stated that the SFDR mixtures' UCS, indirect tensile strength (ITS), rate of strength gain, long-term durability, and cracking characteristics must be considered when cement is mixed with FDR material. Euch et al. (2015) investigated the mechanical properties of cement-treated RAP mixtures that contained 6% cement by dry constituent weight. The laboratory results showed that mixture blends with high RAP content demonstrated higher compressive strength, flexural strength, indirect tensile strength (ITS), and elastic moduli. The results also showed that the RAP content did not statistically affect unrestrained shrinkage strain.

2.3.2.2 Fly Ash Stabilization

Fly ash is a byproduct of coal combustion for electricity generation that is a sustainable alternative to cement. Based on its chemical composition, it is categorized into two main classes: Class C and Class F. Class C fly ash consists of alumina, silicon, and iron; these elements exceed 70% in Class F. Compared to Class F, Class C fly ash has a higher calcium oxide content and a specific self-cementing property (Amran et al., 2021). Class C fly ash is widely used as a stabilizing agent in FDR applications in the Midwest, including Kansas, where it is the only class

of fly ash available. PennDOT recommends that fly ash content in SFDR mixtures range from 6% to 14% by weight of the dry existing base course materials (Morian et al., 2012). When fly ash-treated FDR material is mixed with water, the ash reacts with the calcium hydroxide released during the hydration of cementitious materials to form additional calcium silicate hydrates (C-S-H), thereby increasing the stabilized base layer's strength and durability (Amran et al., 2021). The pozzolanic reaction in fly ash stabilization binds the aggregates together and reduces the mixture's void pore ratio; it improves the stabilized base layer's permeability and decreases the moisture susceptibility (Wen et al., 2003). Previous studies in Kansas demonstrated that pavement structures with Class C fly ash treated base layer exhibited significantly increased structural capacity (Cross & Fager, 1995; Todd et al., 2000). Furthermore, Wen et al. (2003) found that the Class C fly ash-treated base course met the technical requirements for structural performance, proved cost-effective, and aligned with environmental conservation goals.

2.3.2.3 Lime Stabilization

Lime is a widely utilized stabilizer and is particularly effective when combined with fly ash to reduce soil plasticity and enhance stabilized layers' early strength, rapid reaction with water, and subsequent hydration process. Lime acts as an activator, triggering a reaction with the fine particles in soils to form cementitious compounds that significantly improve the FDR base layer's strength, stiffness, and overall performance (Xiao et al., 2018). PennDOT recommended that lime content in SFDR mixtures range from 2% to 6% (Morian et al., 2012). KDOT constructed two test sections using hot-lime slurry and Class C fly ash. Their results indicated that the hot-lime slurry could substitute for Class C fly ash due to its increased mixture tensile strength (Du & Cross, 2007), rutting resistance (Cross, 1999), and moisture damage resistance (Thomas et al., 2000). Niazi and Jalili (2009) investigated the effects of Portland cement and lime-treated CIR mixtures, revealing

that combining cement and lime enhances compressive strength and improves rutting resistance by increasing the CIR materials' moisture resilience. These results indicated that lime is effective on its own and in combination with other stabilizers in improving pavement base layers' overall performance and durability (Niazi & Jalili, 2009).

Xiao et al. (2018) concluded that lime stabilization's effectiveness could vary depending on the project site's soil type and environmental conditions. For example, lime is highly effective in clay-rich soil, where the pozzolanic reaction is more effective. In sandy or non-plastic soils, the benefits of lime stabilization may be limited, and alternative stabilizers may be more appropriate. Therefore, considering soil properties and site conditions is essential when selecting lime as a stabilizer for FDR projects.

2.3.3 Bituminous Stabilization

Bituminous stabilization involves using bituminous materials, such as asphalt cement, cutback asphalt, or emulsion, to enhance reclaimed pavement materials' mechanical properties and durability. The primary goal of bituminous stabilization is to increase the recycled materials' cohesion, thereby improving the base layer's load-bearing capacity, moisture resistance, and overall stability. The bituminous binder creates a waterproof layer that helps protect the underlying pavement structure from moisture infiltration or water capillarity to resist pavement distress, such as rutting and cracking. Studies have indicated that the asphalt-emulsion-stabilized pavement structure resulted in reduced susceptibility to moisture damage and improved resistance to freeze-thaw cycles compared to non-stabilized pavement (Morian et al., 2012). Snizek et al. (2015) concluded that the bituminous-stabilized FDR mixture had limitations in cold temperatures and high-moisture-content areas due to the weaker bonds between the FDR aggregates and asphalt. In addition, the cost of bituminous-stabilized materials is higher than other stabilizers.

2.4 SFDR Mix Design Guidelines

Due to a lack of standardized national-level SFDR mix design, many state agencies have developed their methods, which vary in sample preparation, specimen size, testing methodologies, and required physical properties (Jahren, 2018). Owino et al. (2022) conducted a nationwide survey report regarding SFDR mix design. Thirteen state agencies have specific FDR mix design guidelines, as shown in Figure 2.6. These design methods can typically be categorized by stabilizer type or stabilizer usage, with the primary objective of the mixture design being to select the most effective stabilizer. The SFDR mix designs consider local factors, such as climate, traffic conditions, available materials, and the project's specific requirements. Some current SFDR mix design methods have been derived from the hot-mix asphalt (HMA) mix design methods, such as Marshall, Hveem, and Superpave mix designs. The SFDR mix design approaches are meant to achieve performance-based goals for the pavement over its design life, such as high rutting and fatigue resistance (Du & Cross, 2007).

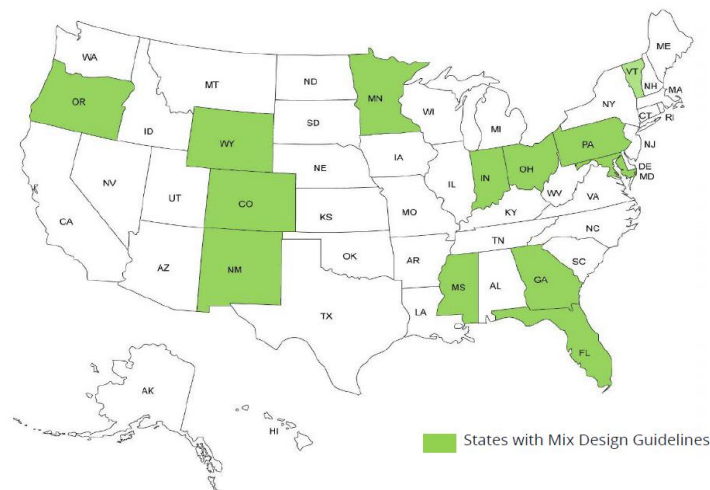


Figure 2.6 Distribution of State DOTs with FDR Mix Design Guidelines (Owino et al., 2022)

The current SFDR mix design practices focus on achieving the desired strength of the SFDR mixture, using the proper stabilized agents with appropriate dosage. PCA (2004) suggested

a 7-day target UCS value of 300 to 400 psi (average of 350 psi) for cement-treated base materials used in FDR with cement and 200 to 300 psi (average of 250 psi) for lime or fly ash-treated FDR materials. PCA also recommended a minimum compressive strength of 300 psi for the vacuum saturation test (ASTM C593) for cement-treated base materials (Reeder et al., 2019). The U.S. Army Corps of Engineers (USACE) recommended a minimum 7-day UCS of 750 psi for base courses for flexible pavement, and it required a minimum 7-day UCS of 250 psi for subbase layers in flexible pavements (USACE, 1994). Reeder et al. (2019) summarized some DOTs' 7-day UCS requirements for the cement-stabilized mx material, as shown in Table 2.1.

Table 2.1 Summary of DOTs' Design Standards for Cement-Stabilized Base Layer (Reeder et al., 2019)

Agency/DOT	Design 7-Day Compressive Strength, psi		Aggregate Properties			Comment
	Minimum	Maximum	Liquid Limit (LL)	Plasticity Index (PI)	% Passing No. 200 Sieve	
California DOT	750				3–15%	
Colorado (City of Thornton)	650	1000	<30		3–15%	Cement: >5%
Federal Aviation Administration	400	800	<25	<6		28-day strength < 1,000 psi
Georgia DOT	300	450				
Kansas DOT	650	1600				Pay based on percent within limit; reference SD: 260 psi.
Maryland DOT	750			<6	0–8%	Cement: 3.25–4.75%
Michigan DOT	750		<6	<6		0–25% passing No. 80 sieve; sliding pay scale with 98% to 95% compaction
Montana DOT	500	1500	<30	<7	4–12%	Minimum 95% compaction; sliding pay scale with 400–2,000-psi compressive strength
Oklahoma DOT	600	1200			1–15%	Target strength: 800–1,000 psi Cement: 3–5%
South Carolina DOT	600				0–20%	Cement: 2.5–5%
Tennessee DOT	500					

2.5 Performance Testing of SFDR Mixtures

Performance tests are commonly proposed before engineering applications to ensure the SFDR mixture meets specifications and performance standards. These tests help identify potential issues with the mixture composition and ensure that the stabilizers perform as expected under various environmental and load conditions. The SFDR mixture performance tests include laboratory tests and field tests.

2.5.1 Laboratory Tests

Laboratory performance tests are used to evaluate the SFDR mixture's design feasibility and effectiveness. These tests are typically modified from HMA test procedures that assess mixture strength, stiffness, and durability. Literature reviews have suggested that laboratory performance tests include UCS, shrinkage, and moisture susceptibility tests.

2.5.1.1 UCS Test

UCS tests measure the SFDR mixture's maximum uniaxial compressive strength. It is a critical parameter in SFDR mix design and helps determine the optimal binder content for a desired strength level. The UCS test involves compacting the SFDR mixture into cylindrical molds, typically with a height-to-diameter ratio of 2:1 or 1.15:1 (Jameson, 2019). It is recommended that an SFDR mixture containing rock or bound materials be tested using a height-to-diameter ratio of 2:1. SFDR specimens usually require a cure for 7 or 28 days; the PCA and the USACE adopted seven-day UCS values as the SFDR mixture strength standard (PCA, 2005; USACE, 1994). The SFDR specimen is then subjected to unconfined axial compression until failure. The peak load at failure defines the SFDR mixture's compressive strength.

2.5.1.2 Shrinkage Test

Shrinkage tests are essential for assessing SFDR mixtures' potential to shrink over time due to water loss, which can lead to shrinkage cracking. It was noted that mixtures with high UCS values typically contain a high cement content, leading to a stiff base layer. This may cause shrinkage cracking in the base, which may spread to the top AC layer due to repeated traffic load, thus reducing the pavement's service life. There is limited information on specific shrinkage standards for cement-treated FDR material, but Australian specifications set a maximum allowable shrinkage strain of 0.00025 in./in. after 21 days for cement-treated materials (Guthrie et al., 2002). George (2002) claimed that stabilized base materials with high cement content were more susceptible to shrinkage cracking. However, no shrinkage cracking occurred when 7-day UCS values of the cement-treated base mixtures were lower than 300 psi under wet curing conditions. Louw et al. (2016) suggested optimizing the cement-stabilized FDR design, which controlled the 7-day UCS in the range of 300–600 psi, or improving the in-place CTB construction via careful curing and microcrack repair before constructing the top/overlaying AC layer. Niazi and Jalili (2009) suggested combining cement with other stabilizers, such as fly ash or lime, can improve the FDR mixture's workability and reduce the shrinkage potential.

2.5.1.3 Moisture Susceptibility Test

The moisture susceptibility test measures the SFDR mixture's resistance to moisture damage, which is a significant concern for pavement durability. Some moisture susceptibility tests consist of indirect tensile strength ratio (TSR), Marshall stability ratio (MSR), freeze-thaw durability, and vacuum saturation (VS) tests.

TSR tests compare the ITS of conditioned specimens to that of unconditioned specimens; a higher TSR value indicates better resistance to moisture damage. MnDOT adopted this test method to test the durability of asphalt emulsion-stabilized FDR materials (Jahren, 2018).

The MSR test is like the TSR test. It compares the Marshall stability (MS) of conditioned and unconditioned specimens; a higher MSR value indicates greater resistance to moisture-induced damage. Niazi and Jalili (2009) used MS tests to compare CIR mixtures with cement and lime. Their results showed that CIR mixtures with cement or lime increased MS, and the mixture with higher cement content resulted in high strength. Kim and Lee (2006) also conducted MS and ITS tests to validate the effectiveness of the mix design method for CIR mixtures, demonstrating that the ITS test should be used in wet conditions to determine the optimal foamed asphalt content.

Freeze-thaw durability tests are typically used to evaluate SFDR mixtures' long-term durability; the tests subject specimens to repeat freezing and thawing cycles to simulate cold-weather conditions and assess the material's resistance to damage caused by freeze-thaw cycles. The specimen's durability is calculated by comparing its final mass after 12 freeze-thaw cycles and brushing with the original mass, with the measurement reported as a percentage lost (Reeder et al., 2017). MnDOT mentioned that SFDR mixture shows the variability of field conditions that can affect freeze-thaw durability test results; for instance, the laboratory tests may indicate a certain mixing ratio as optimal, but field tests often show different results, highlighting the need for comprehensive testing approaches (Jahren et al., 1999).

Vacuum saturation tests measure the SFDR mixture's UCS at both saturated and unsaturated conditions, with lower permeability and higher resistance to moisture for maintaining the SFDR material's strength under moisture. Dempsey and Thompson (1973) were the first to utilize vacuum saturation tests to predict chemically stabilized mixtures' freeze-thaw durability.

They tested soil cement, lime-fly ash, and lime-treated soil specimens following the vacuum saturation test and freeze-thaw test standards (6 cycles and 12 cycles). Linear regression analysis results indicated a significant correlation between vacuum saturation and cyclic freeze-thaw strength. The vacuum saturation test results showed a good correlation with traditional freeze-thaw durability tests, and this correlation helps validate the vacuum saturation test's effectiveness in predicting SFDR mixtures' long-term durability (Henrichs, 2015). Many other studies have shown that the vacuum saturation test is a quick, cost-effective method to predict chemically stabilized mixtures' freeze-thaw durability (Aldood et al., 2014; Zhang et al., 2016; Lake et al., 2017).

2.5.2 Field Tests

Field tests of the SFDR mixture are used for preconstruction evaluation, construction quality control, and pavement performance monitoring. Reports have shown that the field tests include a falling weight deflectometer (FWD), a dynamic cone penetrometer (DCP), and coring.

2.5.2.1 FWD Test

The FWD test is a nondestructive method for evaluating in-situ pavement structural properties. It involves measuring surface deflections by applying an impulse load to the pavement surface at various offsets from the load plate. The deflection data are used for back-calculation of pavement layer elastic moduli. The FWD test provides a reliable assessment of the in-situ conditions of pavement layers. Johanneck and Dai (2013) used FWD tests to compare the response and performance of three sections with different SFDR mixture combinations. The test results revealed that the section with the emulsion-stabilized mixture demonstrated improved strain levels.

Butalia et al. (2011) emphasized FWD tests' effectiveness in monitoring pavements' long-term performance, particularly in evaluating the benefits of using Class F fly ash in FDR projects. Romanoschi et al. (2004) conducted FWD tests to examine the structural contribution of

foamed asphalt-stabilized FDR materials; these SFDR mixtures were computed to have a structural layer coefficient of 0.18. This coefficient represents the SFDR layer's relative structural capacity contribution in the pavement cross-section. A higher coefficient indicates a more outstanding structural contribution.

2.5.2.2 DCP Test

The DCP test is an in-situ method for evaluating unbound pavement materials' strength and bearing capacity, particularly for subgrade materials. It involves driving a standard cone into the ground using a standard dropped weight (17.6 lbs.) from a specific height (39 in.) and measuring the penetration depth per drop (Scullion et al., 2008). The cone's penetration depth was typically measured in millimeters per blow (mm/blow), providing information about the material's resistance to penetration. The DCP test data were used to compute the California bearing ratio (CBR) of subgrade soils using Equation 2.1 (AASHTO, 2015). Jahren and Keneth (1999) stated that DCP results help to evaluate by checking whether the subgrade adequately supported the rehabilitated pavement structure. PennDOT conducted the DCP tests to verify compaction and material properties uniformity within unbound pavement layers (Morian et al., 2012).

$$\text{CBR (\%)} = \frac{292}{\text{DPI}^{1.12}} \quad \text{Equation 2.1}$$

Where:

CBR = California bearing ratio (%), and

DPI = dynamic penetration index (mm/blow).

2.5.2.3 Coring

In-situ coring is the most straightforward approach to evaluating compacted SFDR layers' strength and stiffness. Coring can verify the SFDR design parameters by comparing actual strength values obtained from the cored samples with the intended design values, directly assessing the

SFDR mix design's effectiveness. The core samples are subjected to UCS tests to determine their compressive strength and modulus of elasticity (Lewis et al., 2006).

2.6 Design Approaches

The philosophy of SFDR design is that optimizing HMA thickness over SFDR layers involves balancing the cost of the HMA layer and increasing the structural support the SFDR layer provides. The SFDR base layer's thickness may range from 8 to 12 in. The thickness is adjustable based on the existing pavement thickness, environmental conditions, and subgrade soil characteristics.

Low-traffic-volume roads were considered suitable candidates for FDR since they provided a cost-effective solution for rehabilitating pavements with structural deficiencies (Reeder et al., 2019). For these low-volume roads, the percentage of truck traffic is a significant pavement design factor, particularly in the agricultural states, such as Kansas, where over-sized and weighted tractors drive on these roads. These heavy agricultural vehicles exert higher stress on pavements than trucks, significantly impacting the pavement structure. Therefore, a flexible pavement structure consists of a thicker SFDR layer and a thinner AC overlaying layer, potentially resulting in a longer pavement service life at a lower construction cost.

Studies have shown that the mechanistic-empirical pavement design approach is feasible for optimizing AC overlay thickness over an SFDR layer. This approach has become increasingly popular because it incorporates material properties, truck traffic distribution, and environmental factors into the design process.

The MEPDG is a comprehensive pavement design approach that predicts pavement distresses, including rutting, cracking, and roughness. It was developed by the American Association of State Highway and Transportation Officials (AASHTO). The AASHTOWare

Pavement Mechanistic-Empirical Design (PMED) software is a comprehensive application that helps transportation agencies analyze and design pavement structures using MEPDG (AASHTO, 2015). MEPDG design predicts pavement response under traffic load, structural material properties, and climate data. Scullion et al. (2008) used the PMED program to design HMA pavement with a CTB layer. Similarly, Le et al. (2016) used the PMED software to simulate the long-term rutting of pavements with SFDR mixtures and concluded that the computed rutting data matched in-place rut measurements.

2.7 Finite Element Modeling

The finite-element method (FEM) is a computational tool for investigating and predicting the behavior of flexible pavement structures under traffic and environmental loadings. In general, FEM divides a complex structure into smaller or simpler parts called finite elements, which are analyzed individually and then assembled to investigate the entire structure's behavior. FEM in pavement structure is used to analyze stress, strain, and displacement in the layers under various loading and environmental conditions (Saghebfar, 2014).

Duncan et al. (1968) carried out the first FEM application for analyzing flexible pavement. Later, another FEM-based computer program, ILLI-PAVE was developed to analyze pavement structure, which considered the pavement as an axisymmetric solid of revolution and divided it into finite elements. This program was limited in application due to its longer computational time and large data storage requirements (Lutfi & Figueroa, 1980). MICH-PAVE was another pavement analysis program based on FEM developed by Michigan State University. MICH-PAVE improved computational efficiency and defined the nonlinear properties of the base and subgrade materials. Like the ILLI-PAVE program, MICH-PAVE considered the pavement structures' linear elasticity (Tseng & Lytton, 1990). The Flexible Pavement Analysis Structural System (FPASS) was

developed based on the ILLI-PAVE program; it incorporated several enhancements, such as multiple-axle-load handling and prediction models for pavement distresses. FPASS was limited in material properties; the user could only run the program with default material properties (Wu et al., 2010). Sargand and Beegle (2006) developed a FEM called OUPAVE. This program defined the flexible pavement in linear-plastic and nonlinear-plastic behavior, and it was conducted to evaluate the flexible pavement responses under static and dynamic loading conditions. The related case study showed that the OUPAVE-computed results did not show good agreement with the filed measurements due to a lack of suitable material-property inputs for the program.

With the development of computer technology, more FEM-related programs and even commercial software have been used for pavement analysis. A commercial FEM program, ABAQUS, proved effective in predicting pavement structures' behavior (ABAQUS, 2016). ABAQUS provides a wide range of material models, such as linear elastic, viscoelastic, hypoplastic, and elastoplastic models; it also provides maximum flexibility in the simulation of various material behaviors under various loading/environmental conditions (Wu et al., 2010). Generally, creating an ABAQUS FEM to simulate the flexible pavement involves three considerations: geometry, traffic loading behavior, and material constitutive models.

2.7.1 Geometry

In ABAQUS, geometry is called dimensionality. Three types of dimensionalities are used to simulate pavement structures: three-dimensional (3D), plane-strain (2D), and axisymmetric. The 2D model simplifies the pavement structure into a 2D representation that extends infinitely in the longitudinal direction, making it easier to analyze long pavement sections where the geometry and loading conditions are relatively constant along the length (Ibrahim et al., 2014). Kim (2002) compared 2D and 3D models to investigate the effects of super-single tire loading on subgrades

and found that the 2D model stresses, strains, and displacements were higher than those in 3D modeling. 2D models in ABAQUS improved computational efficiency but were limited in accurately replicating real-world traffic loading since it modeled the wheel load as a line load. This prevented the accurate representation of the tire contact area (Wu et al., 2010). The axisymmetric (2D) model, which used a cylindrical coordinate system to assume the pavement structure, had uniform properties in the radial direction, and the load was applied symmetrically around a central axis. Although axisymmetric models had lower computational requirements than the 3-D models, the load distribution and pavement geometry were considered in the model (Wang, 2011). Many studies have indicated that the 3D ABAQUS FEM simulation achieves the most comprehensive and relatively accurate response of pavement structures as well as the full complexity of material behavior, loading conditions, and pavement geometry (Wu et al., 2010).

2.7.2 Traffic Loading Simulation

To simulate the near-real response of the pavement structure, the traffic load is considered a moving and repeated load on the pavement structure. The repeated loads are simulated by applying a static load for a specified number of cycles to simulate the cumulative effects of traffic loading. The moving load is simulated by applying the dynamic traffic load across the pavement. The pavement response, such as stresses, strains, and deflections, is computed during the traffic loadings.

In general, flexible pavement consists of multiple layers. Because traffic loads are applied to the pavement, a stress pulse transfers from the top pavement to the bottom layer. The pavement thickness, travel speed, and wheel load magnitude significantly affect stress (Wu et al., 2010). Zaghoul et al. (1994) simulated traffic loads by converting the tire-pavement contact pressure to a uniform loading area on the FEM mesh. Saghebfar (2014) and Bortz (2015) adopted a similar

approach to simulate the accelerated pavement testing (APT) wheel load on pavement structures. The accumulated traffic load passes were achieved by increasing the loading time on the pavement (Figure 2.7a). Al-Qadi and Hao (2009) simplified the complex tire-pavement interaction with a tire loading imprint gradually shifting over the loading area until a single-wheel path is traversed. The accumulated traffic load passes were achieved by accounting for the load cycles from one side to the other on the FEM mesh (Figure 2.7b). The FEM results from this simulation showed good agreement with the APT test results in predicting longitudinal strains.

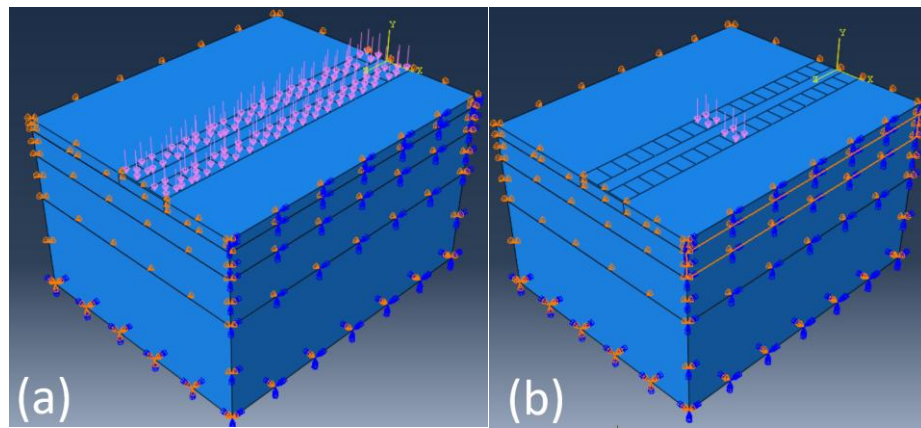


Figure 2.7 Traffic Loading Simulation: (a) Repeated Load; (b) Moving Load

2.7.3 Constitutive Material Models

Constitutive material models are essential to effectively simulate and analyze the behavior of pavement structures under various loading or environmental conditions. This model is mathematical and defines the stress-strain characteristics of materials in ABAQUS FEM (ABAQUS, 2016). Selecting the proper constitutive model is critical for defining the material characteristics accurately. As discussed earlier, flexible pavement is a multi-layered structure that consists of an AC wearing course, base, and subgrade layers. Technically, the layered systems have higher-quality materials at the top, where the stress is higher, and lower-quality materials at the bottom, where the stress is lower. Many studies have indicated that the constitutive material

models for all layers are quite different; even the same material can have various constitutive material models for different research objectives.

2.7.3.1 Asphalt Concrete (AC) Modeling

AC materials are considered viscoelastic or visco-elastic materials; they exhibit time-dependent and temperature-sensitive behavior. Traffic loading rate and creep influence time-dependent behavior rate dependency, which results in continuous deformation under a constant traffic load. This behavior is critical when the traffic loading is applied to the pavement. Temperature-sensitive behavior indicates that temperature significantly influences AC materials' properties, primarily due to the asphalt binder's sensitivity to temperature changes. The AC materials' tensile strength decreases when the temperature increases and increases strength when the temperature drops (Huang, 2004).

Zaghloul and White (1993) developed a 3D ABAQUS FEM to simulate the actual traffic load on the flexible pavement. A viscoelastic model was applied to the AC layer, and the results showed a higher load equivalency factor than the AASHTO design guide recommendations. Wu et al. (2011) developed a 3D FEM for predicting permanent deformation in pavement structures. A permanent deformation material model was used to simulate AC materials' behavior. The model was a modification of a conventional elastoplastic model with linear strain hardening. Saghebfar (2014) developed a 3D ABAQUS FEM to simulate the eight full-scale pavement test sections at CISL. Two constitutive material models were used for modeling the AC layer: the elastic model and the creep model. The creep model was found to be appropriate for showing permanent strain after unloading. Bortz (2015) developed a 3D ABAQUS FEM to simulate the eight full-scale test sections for evaluating the performance of geocell-reinforced HMA pavement; the linear elastic

characteristics were defined for the HMA layer. Table 2.2 provides a summarized comparison of elastic, viscoelastic, and visco-elastic plasticity of AC properties in ABAQUS.

Table 2.2 Comparison of AC Material Models in ABAQUS

AC Model Type	Description	Advantages	Disadvantages	Reference
Linear Elastic	A linear relationship between stress and strain; AC behaves linearly elastically and recovers its original shape after unloading.	Simple and easy Computing material inputs are included in the software. Useful for preliminary analyses or comparisons with other linear elastic pavement analysis methods (such as BIAR,	Does not accurately capture AC's viscoelastic or plastic behavior.	(Kim et al., 2009) (Bortz, 2015) (Huang, 2004)
Viscoelastic	Accounts for AC's creep behavior by considering its elastic and viscous properties. Able to simulate AC's delayed response to loading and its gradual recovery after unloading.	More accurately captures AC's response due to dynamic and environmental loading. Standard viscoelastic models used in ABAQUS include the generalized Maxwell model and the Prony series.	Requires accurate AC viscoelastic parameters through laboratory testing.	(Al-Qadi & Hao, 2009) (Zaghloul et al., 1994) (Wu et al., 2010) (Saghebfar, 2014)
Visco-Elasto Plastic	Captures AC materials' time-dependent and PD behavior.	Most accurately simulates AC's response under various loading and environmental conditions.	Requires specialized material models through user-defined subroutines (UMATs) in ABAQUS. Requires extensive material testing to calibrate model parameters.	(ABAQUS, 2016) (Wu et al., 2010)

2.7.3.2 Base Layer Modeling

The base course is beneath the AC layer in the pavement structure. It is an important layer that distributes wheel loads across the pavement structure while reducing stress on the subgrade. The base course consists of granular materials, stabilized granular materials, or Portland cement concrete. In FEM applications, the Mohr-Coulomb and Drucker-Prager plasticity models are commonly used to simulate the permanent strain in the base under a repeated traffic load.

The Mohr-Coulomb plasticity model simulates granular materials' shear strength behavior. This model is based on the traditional Mohr-Coulomb yield criterion, represented by a linear relationship between shear stress and normal stress on a failure plane, computed using Equation

2.2. The Mohr-Coulomb criterion assumes that failure occurs when the shear stress on a plane in the material exceeds the material's shear strength. Figure 2.8 shows the Mohr-Coulomb failure model. Sukumanran et al. (2004) applied the Mohr-Coulomb model (elastoplastic model) to define the base course with granular materials, and a 3D finite FEM was developed to compare the predicted pavement performance data with the failed pavement data.

$$\tau = c - \sigma \tan \phi, \quad \text{Equation 2.2}$$

where

τ = shear stress,

c = cohesion of soil,

σ = normal stress (negative in compression), and

ϕ = soil internal friction angle.

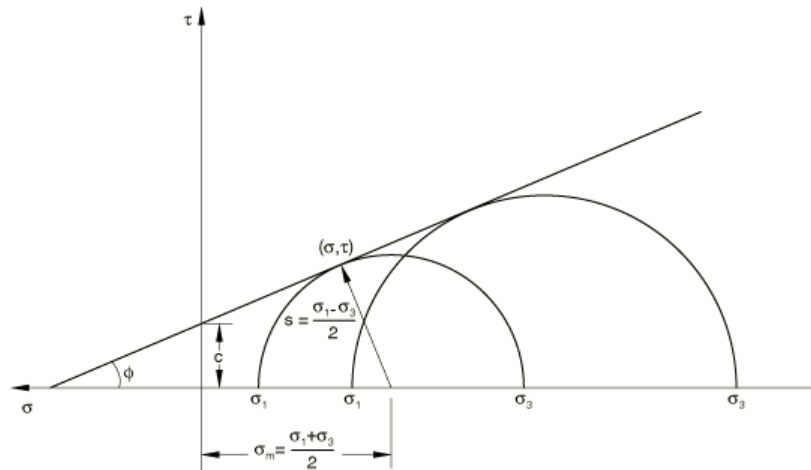


Figure 2.8 Mohr- Coulomb Failure Model (ABAQUS, 2016)

The Drucker-Prager model is considered a “smoothened” version of the Mohr-Coulomb criterion because it addresses the irregularity of the Mohr-Coulomb model in the principal stress space (Wang, 2011). The Drucker-Prager (DP) model accommodates isotropic hardening and softening behaviors and allows volumetric changes during inelastic deformation materials to harden or soften isotopically (ABAQUS, 2016). In ABAQUS, the yield criteria are based on yield

surface shape, and the yield surfaces are categorized as linear, hyperbolic, and exponent forms. Figure 2.9 (a) shows the linear DP model plot; the most straightforward form provides a linear yield surface in the meridional plane. The hyperbolic model (Figure 2.9 (b)) and general exponent forms (Figure 2.9 (c)) use a von Mises (circular) section in the deviatoric stress plane (ABAQUS, 2016). It provides a curved yield surface in the meridional plane that better resembles geomaterials' nonlinear behavior. The proper failure model selection depends on the material being modeled and the level of accuracy required. The general exponent form often best fits experimental data, but it is more computationally demanding, such as the UMAT program in ABAQUS. Zaghloul and White (1993) and Liu et al. (1998) used the DP model to simulate the base layer and investigate the effects of elasto-plasticity of the base material under dynamic loading conditions.

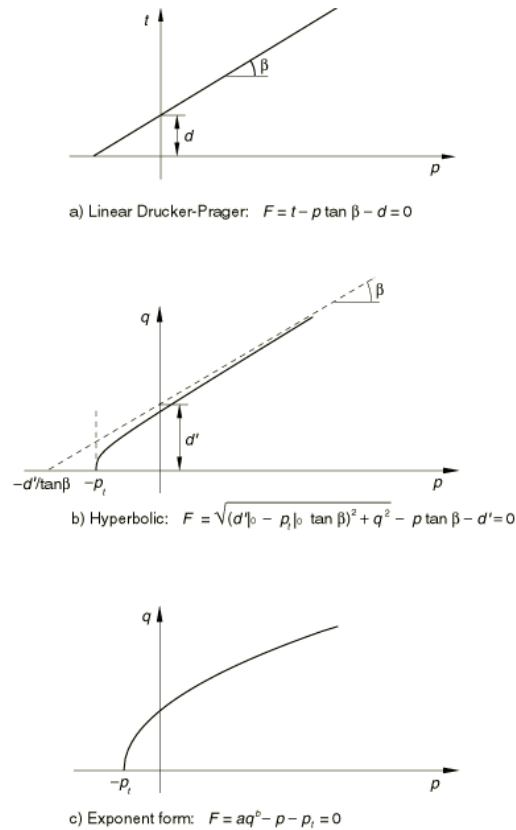


Figure 2.9 Drucker-Prager Yield Surfaces (ABAQUS, 2016)

Minimal literature addressing SFDR base modeling in flexible pavements is available. Valanis (1990) developed a damage model to investigate the behavior of materials under repeated loading. Heymsfield et al. (2006) applied a modified version of this model to simulate the behavior of cement-treated soil layers under unconfined compression tests with monotonic and cyclic loading. The results showed that Valanis's damage model accurately predicted the softening behavior of soil-cement mixtures under repeated loading without accounting for the mixture's plastic behavior (Heymsfield et al., 2006).

2.7.3.3 Subgrade Layer Modeling

The subgrade layer is the foundational layer of a pavement structure and provides essential support for all layers above it. It comprises natural materials such as clay, sand, and silt. In the ABAQUS FEM application, depending on the type of soil, the subgrade layer is defined as linear elastic, nonlinear elastic, or elastoplastic. The linear elastic soil behavior represents the relationship between stress and strain. It is characterized by a constant elastic modulus (E) and Poisson's ratio. The elastic model assumes the soil deformations are fully recoverable upon unloading (ABAQUS, 2016). Nonlinear elastic models assume that soil deformations are recoverable upon unloading. These models help analyze the resilient behavior of subgrades under repetitive traffic loads since they capture the soil's ability to rebound after a traffic load cycle. Elastoplastic soil behavior encompasses the recoverable (elastic) and irrecoverable (plastic) deformations of soil; it defines a yield surface in stress space. The soil behaves elastically when stresses remain on this yield surface. When the stresses exceed the yield surface, the soil undergoes plastic deformation, leading to permanent changes in its structure (Wu et al., 2010). Saghebfar (2014) and Bortz (2015) used Drucker-Prager and creep models to simulate the subgrade layer in the FEM for their full-scale APT test sections at CISL.

Chapter 3 - Phase I: Laboratory Test Program

3.1 Introduction

This chapter discusses a comprehensive laboratory test program to design and evaluate the SFDR mixtures. Figure 3.1 shows a flowchart of the laboratory test program (Phase I). The laboratory tests include Proctor compaction, unconfined compressive strength (UCS), linear shrinkage, and vacuum saturation tests. These tests yielded critical physical properties that influence the SFDR base layer's structural performance and long-term durability. The UCS test results were considered key input parameters in the AASHTOWare PMED software to determine the AC overlay's optimal thickness.

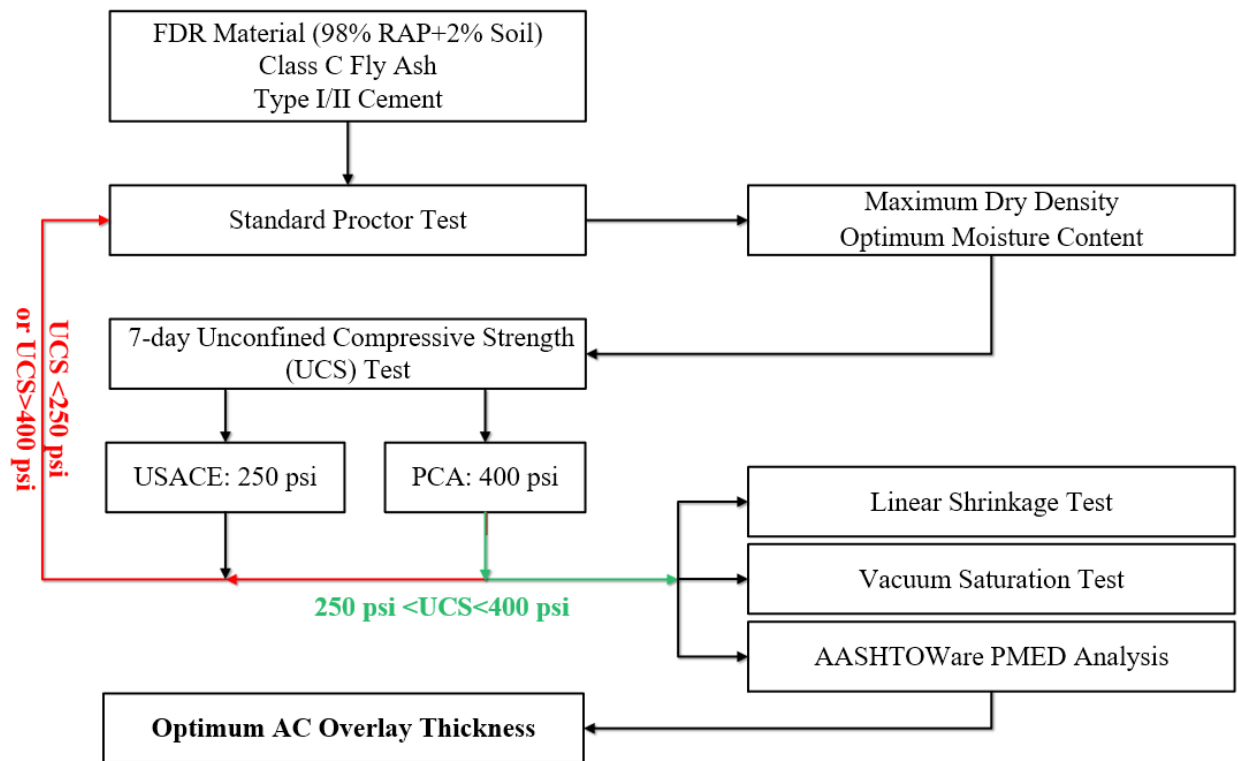


Figure 3.1 Laboratory Test Program Flowchart

3.2 Material Properties

3.2.1 Subgrade Soil

The clay soil in this study was obtained from Bayer Construction in Manhattan, Kansas. The Atterberg limit tests and standard Proctor tests were conducted following ASTM D4318 (2017a) to determine the soil's plasticity characteristics and moisture-density relationship. The Atterberg limit test results are summarized in Table 3.1. The results indicated a liquid limit (LL) of 45%, a plastic limit (PL) of 25%, and a plasticity index (PI) of 20%, classifying the soil as CL (lean clay) according to the Unified Soil Classification System (USCS) and an A-7-6 per the American Association of State Highway and Transportation Officials (AASHTO). The sieve analysis showed that 95.7% of the soil particles passed through a U.S. No. 200 sieve. The standard Proctor test showed that the soil's maximum dry density (MDD) was 98 pcf at an optimum moisture content OMC of 23%. Additionally, Tavakol (2019) reported a UCS of 27 psi for this soil.

Table 3.1 Atterberg Limit Test Results

Liquid Limit, LL (%)	45
Plastic Limit, PL (%)	25
Plasticity Index, PI (%)	20
Passing #200 Sieve (%)	95.7
USCS Classification	CL
AASHTO	A-7-6

3.2.2 FDR Materials

The FDR material utilized in this study consisted primarily of RAP and some A-7-6 soil. The RAP material was collected from a highway project site on K-177 in Marshall County, Kansas. To ensure representative field conditions, 2% soil by dry weight was premixed with RAP. The materials were then subjected to a series of Kansas Standard tests, including KT-34 (2016d), KT-

3 (2016c), and KT-57 (2016e) tests, which assess gradation, compaction, and moisture-density characteristics, respectively.

The sieve analysis results of the FDR material are depicted in Figure 3.2. Approximately 2% of fine material (passing #200) was present in the RAP materials. Proctor test results of the material are shown in Figure 3.3, the results showed that the FDR material had an MDD of 124 pcf and an OMC of 9.5%.

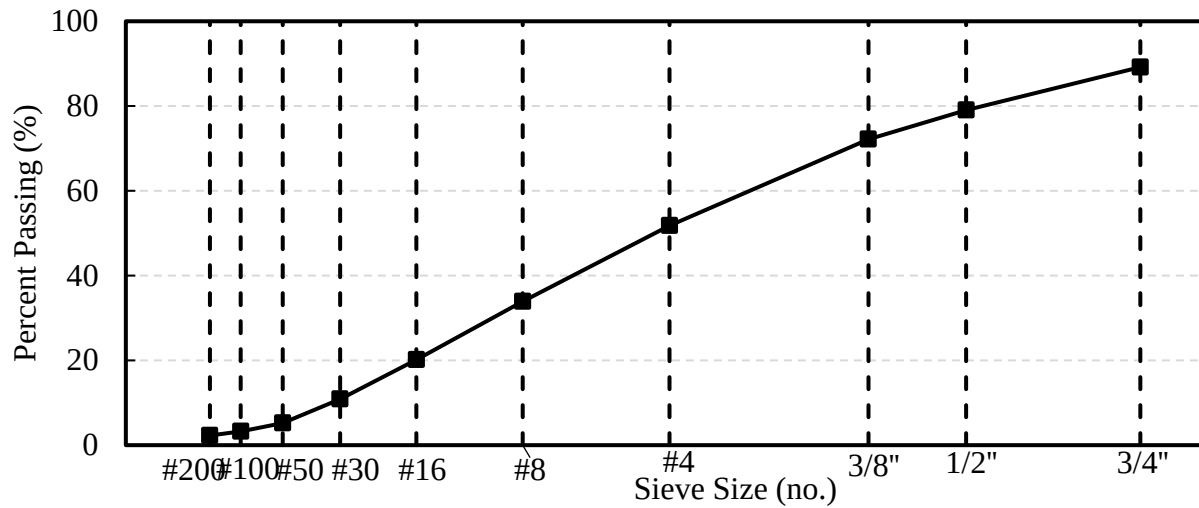


Figure 3.2 Sieve Analysis Results of FDR Material

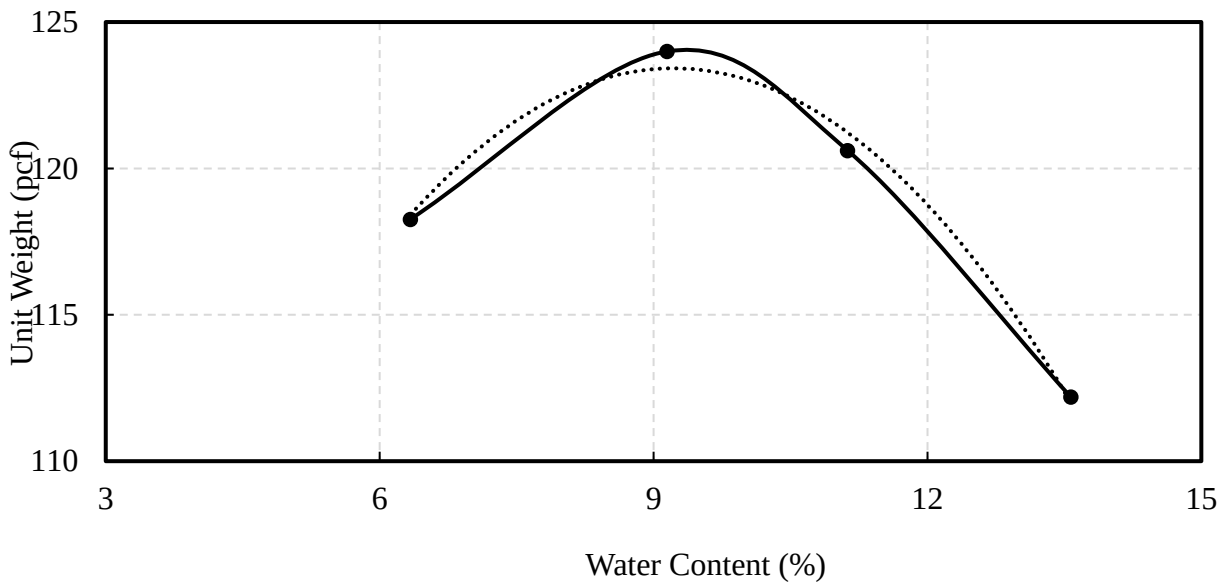


Figure 3.3 Proctor Test Results for FDR Material

3.2.3 Chemical Stabilizer

In this study, Type I/II Portland cement and Class C fly ash were used to stabilize the FDR material. These stabilizers were selected based on their widespread use in Kansas and availability from local sources. These materials satisfy KDOT's standard specification requirements. The physical and chemical properties of fly ash are summarized in Table 3.2. The fly ash was obtained from an electricity-generating power plant in Kansas City, Kansas. Detailed properties of the fly ash are provided in Appendix A.1. The fineness of this ash, measured by the amount retained on a No. 325 sieve, was 23.8%, well below the maximum limit of 34% set by ASTM C618 and AASHTO M295 standards. The specific gravity was recorded at 2.62. The moisture content was negligible, ensuring optimal reactivity during mixing.

Table 3.2 Chemical and Physical Properties of Class C Fly Ash

Source: Kansas City Performance Center			Class C	
Chemical Analysis	Units	Value	ASTM C618	AASHTO M295
SiO ₂	%	40.54		
Al ₂ O ₃	%	18.97		
Fe ₂ O ₃	%	6.23		
SiO ₂ +Al ₂ O ₃ +Fe ₂ O ₃	%	65.74	50 min.	50 min.
CaO	%	23.53		
MgO	%	4.73	5.0 max.	5.0 max.
SO ₃	%	1.16	3.0 max.	3.0 max.
Moisture content	%	0	6.0 max.	5.0 max.
Loss on ignition	%	0.35		
Na ₂ O	%	1.52		
K ₂ O	%	0.49		
Physical Analyses				
Fineness, the amount retained on #325 sieve	%	23.8	34 max.	34 max.
Variation points from the average.		0.5	Five max.	Five max.
Density	g/cm ³	2.62		
Variation points from the average	%	-0.4	Five max.	Five max.
Strength activity index with Portland cement at seven days, % of cement control		89		
Water requirement, % of cement control		96	105 max.	105 max.
Soundness, auto salvage expansion, or contraction	%	0.04	0.8 max.	0.8 max.

The physical properties of Type I/II Portland cement are summarized in Table 3.3. The cement was obtained from Ash Grove Cement Company. The detailed lab test report is attached in Appendix A.2. The compressive-strength tests of the cement showed results of 2,320 psi on day 1, 3,840 psi on day 3, and 4,710 psi on day 7, exceeding the minimum required strength in ASTM C109 standards. The air content of mortar mixed with the cement was 6%, and the setting times were recorded as 110 minutes for the initial set and 375 minutes for the final set.

Table 3.3 Physical Properties of Type I/II Portland Cement (Ash Grove)

Categories	Test method (ASTM)	Spec. limit	Test result
Air content of mortar (%)	C185	12.0 max	6
Blaine fineness (m ² /kg)	C204	2600 min	4020
Air permeability test			
Autoclave expansion (%)	C151	0.80 max	0.02
Compressive strengths, psi (MPa)	C109		
1 d		N/A	2320
3 d		1740 min	3840
7 d		2760 min	4710
Setting times (min)	C191		
Vicat initial		45	110
Vicat final		375	110
Heat of hydration (cal/g)-7 d	C186	A	N/A
False set (%)	C451	B	86

3.3 Laboratory Tests

Eight SFDR mixtures were prepared to evaluate the stabilized mixture performance for varying cement and fly ash contents. The proportions of stabilizers were determined as percentages of the oven-dried FDR material weight. The purpose was to identify the SFDR mixtures that would satisfy the required UCS thresholds—specifically, ensuring that the UCS values fall within the targeted range of 250 psi to 400 psi.

To ensure uniformity and consistency in the SFDR mixtures, the oven-dried FDR material was pre-sieved to remove RAP particles larger than 0.75-in. The SFDR mixtures were prepared with the following stabilizer combinations: 10% fly ash (10FA), 20% fly ash (20FA), 5% cement

(5C), 10% cement (10C), 5% cement & 5% fly ash (5C&5FA), 6% cement & 4% fly ash (6C&4FA), 3% cement & 7% fly ash (3C&7FA), and 2% cement & 8% fly ash (2C&8FA). A series of laboratory tests were conducted to evaluate moisture-density characteristics, UCS, linear shrinkage, and freeze-thaw resistance.

3.3.1 Moisture-Density Characteristics

The SFDR mixtures' moisture-density relationship was evaluated using the standard Proctor compaction test and following ASTM D698-12 (2012). The FDR material was oven-dried to constant weight. The oven-dried FDR material was then blended with the predetermined amount of the stabilizer. The dry SFDR mixture was then thoroughly mixed with predetermined water contents ranging from 3% to 15% by weight of the dry SFDR mixture in 3% increments, as depicted in Figure 3.4.

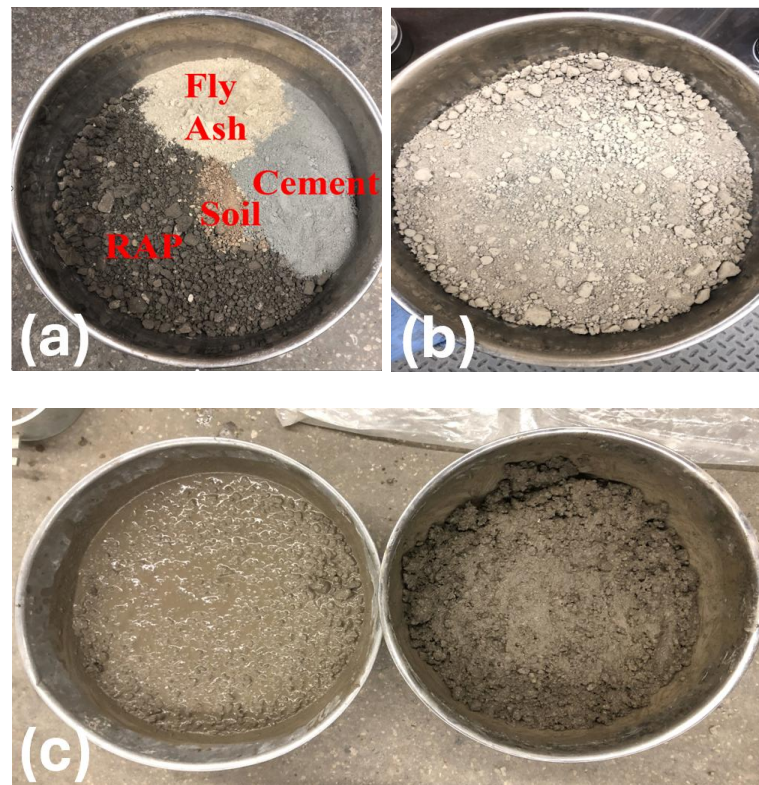


Figure 3.4 (a) SFDR Material; (b) Dry-Mixed SFDR; (c) Wet-Mixed SFDR

The specimens were prepared in five 6-in. diameter molds. Each specimen was compacted in three equal-height lifts, with each lift receiving 56 blows from a standard Proctor hammer (5.5 lb), resulting in a total compaction energy of 12,400 ft-lb/ft³. After compaction, a representative sample was taken from each lift to determine its water content using the oven-drying method following ASTM D2216 (2010). The MDD and OMC values were interpolated from the plotted moisture-density relationship curves obtained from the Proctor test results. The Proctor test setup and the compaction process for the SFDR mixture containing 10% fly ash are shown in Figure 3.5.



Figure 3.5 Proctor Test for SFDR Mixture: (a) SFDR Material Preparation; (b) SFDR Material Compaction

3.3.2 Unconfined Compressive Strength (UCS) Test

The UCS is a critical mechanical property used to characterize the strength of SFDR materials in pavement design. It is widely accepted as a single quantitative measure for designing SFDR base courses. This study determined the UCS values of the SFDR mixtures using the UCS test per ASTM D1633 (2017b).

The SFDR mixtures were prepared at their corresponding OMC and MDD values. Three replicate cylindrical specimens were compacted; each specimen was 8-in. height and 4-in. diameter (height-to-diameter ratio of 2:1). A heavy-duty loading frame was used to compact the

specimens at a prolonged loading rate. This was due to prevent water from squeezing out during compaction. Custom-made cylindrical steel molds were used to ensure uniform specimen dimensions. After compaction, the specimens were carefully de-molded and transferred to a moisture room for curing. The moisture room was maintained at 96% relative humidity (RH) and a temperature of 67°F. Additionally, the SFDR specimens containing fly ash were sealed in plastic bags to minimize moisture loss during curing. After the 7-day curing period, the specimens were tested for UCS. Each specimen was subjected to compressive loading at a constant loading rate of 0.05 in./min. The compressive strength was computed as the maximum stress that caused the specimen to fail. The entire UCS testing process, including specimen preparation, curing, and compression testing, is illustrated in Figure 3.6.

The UCS values were used to classify the mixtures into three performance categories: If the UCS value was lower than 250 psi (low strength), it would be considered insufficient to meet the minimum requirements for the base layer. Using such mixtures in base layers could cause premature pavement failure, such as rutting. If the UCS value was between 250 and 400 psi (moderate strength), mixtures achieving UCS values within this range would be considered proper material for the base layer. Their strength is sufficient to withstand typical traffic loads, reducing the risk of brittle failure. If the UCS was higher than 400 psi (high strength), it indicated that the SFDR mixtures were over-designed. An SFDR base material with an excessively high UCS can lead to increased stiffness and a higher risk of shrinkage and reflection cracking in the pavement over time.

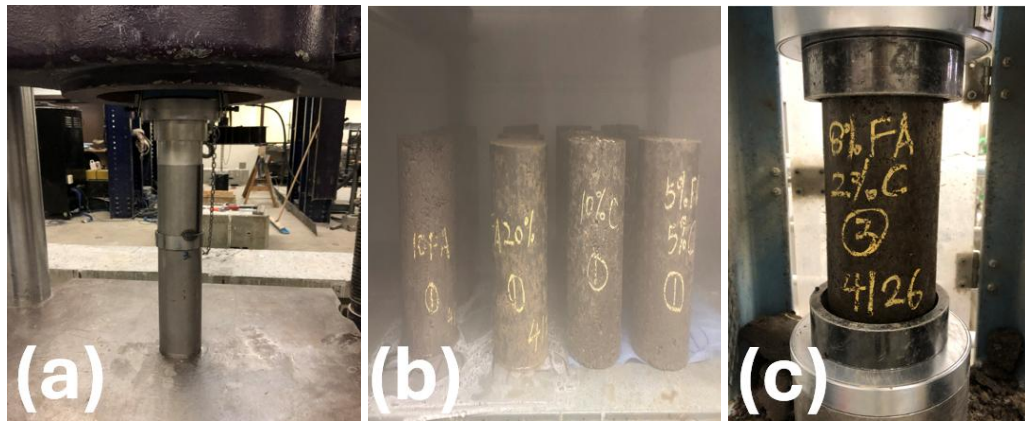


Figure 3.6 UCS Test: (a) Specimen Compaction; (b) Curing; (c) Compression

3.3.3 Vacuum Saturation Test

The vacuum saturation tests were conducted to investigate the SFDR mixtures' durability. This test simulates conditions where the pavement is exposed to water infiltration or capillary action, particularly in environments prone to freeze-thaw cycles (Dempsey & Thompson, 1973).

In this test, six specimens were prepared for each of the eight SFDR mixtures investigated, each with a height of 4.59 in. and a diameter of 4 in. (height-to-diameter ratio of 1.15:1). The specimens were compacted, de-molded, and then cured in the moisture room for seven days. After curing, three specimens were randomly selected from each batch for the vacuum saturation test. The remaining three specimens were set as a control group and underwent the UCS test, as discussed in subsection 3.3.2. The selected specimens were placed in a vacuum chamber and subjected to a constant vacuum of 24 in. of mercury for 30 minutes. This process ensured that any air voids in the specimens were removed, allowing for complete saturation when the specimens were soaked in water. After the vacuum phase, the specimens were soaked in an open water container at room temperature (77°F) for an additional hour for complete water penetration. Once removed from the water, the specimens were allowed to surface-dry until no visible water was observed before the UCS tests were conducted. The complete vacuum saturation test procedure,

including specimen preparation, curing, soaking, and compression testing stages, is shown in Figure 3.7.

To quantify moisture's impact on the SFDR mixtures' strength and durability, the results of the vacuum saturation test were compared to the UCS values obtained from the surface-dried specimens (control group). A vacuum saturation ratio (VSR) (or saturation coefficient), analogous to the tensile strength ratio (TSR) to assess moisture susceptibility in HMA materials, was computed using Equation 2.3. The VSR was used to compare the retained strength of the eight SFDR mixtures under moist conditions. VSR values close to 100% indicated better moisture resistance in terms of strength retention.

$$VSR = \frac{UCS_{wet}}{UCS_{dry}} \quad \text{Equation 2.3}$$

where:

UCS_{wet} = average compressive strength of saturated samples, and

UCS_{dry} = average compressive strength of unsaturated samples.

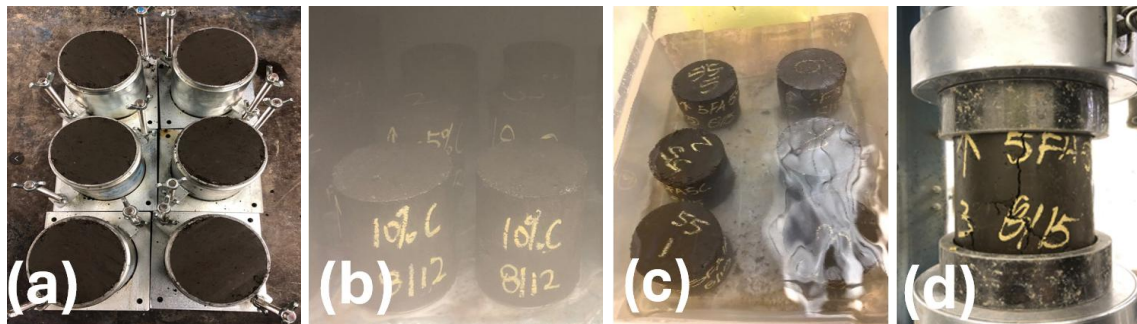


Figure 3.7 Vacuum Saturation Test (a) Compaction; (b) Curing; (c) Soaking; (d) Compression

3.3.4 Linear Shrinkage

Very few studies have addressed SFDR materials' shrinkage behavior in the mixture design phase. Shrinkage cracking in the SFDR base layer and its subsequent reflection in the AC overlay

has become a significant concern due to its contribution to increased pavement maintenance costs (Ghasemi et al., 2018).

To study the shrinkage behavior of SFDR mixtures with various chemical stabilizer content, the linear shrinkage test was conducted following ASTM C157 (2017c). Eight SFDR mixtures were prepared at their MDD and OMC. For each mixture, three identical specimens measuring 3 in. × 3 in. × 11.25 in. were compacted in two lifts using a standard Proctor rammer with 56 blows per lift. An FDR mixture without any stabilizer was also set as the control group. The control group sets a baseline for comparison, allowing for a quantitative assessment of chemical stabilizers' impact on the SFDR mixtures' shrinkage behavior.

After compaction, the specimens were sealed in plastic bags and placed in the moisture room with 96% relative humidity for 24 hours to cure. Once de-molded, the specimens were immediately transferred to a drying room maintained at 73 ± 3 °F. Initial length measurements were taken as soon as the specimens were placed in the drying room. Subsequent comparator readings were taken at specified intervals following ASTM C157, at 1, 2, 4, 7, 14, 21, and 28 days.

The comparator readings were used to calculate the shrinkage levels. Difficulties were experienced during the de-molding of the control group specimens, so its shrinkage measurements began at the 48-hour mark. The shrinkage test setup, including the specimens in the drying room and the comparator device, is shown in Figure 3.8. The linear shrinkage test results were compared across all mixtures, focusing on balancing strength and shrinkage resistance. Mixtures with lower shrinkage strains were considered more suitable for pavement applications because they reduce the likelihood of cracks forming in the base layer and spreading to the AC overlay.

The comparator readings were used to calculate the shrinkage strain. The length change of any specimen at any age after the initial comparator reading (in microstrain) was calculated using Equation 2.4.

$$\Delta L_x = \frac{CRD - \text{initial CRD}}{G} \times 10^6 \quad \text{Equation 2.4}$$

where:

ΔL_x = strain change of specimen at any age (microstrains),

CRD = difference between the comparator reading of the specimen and the reference rod at any age and

G = gage length (10 in).



Figure 3.8 Linear-Shrinkage Test: (a) Specimen Preparation; (b) Specimen Curing; (c) Drying; (d) Measurement Device; (e) Length-Reference Comparator

3.4 AASHTOWare PMED

To optimize the AC overlay thickness over the SFDR base layer, flexible pavements were designed using the AASHTOWare PMED software. AASHTOWare PMED is a mechanistic-empirical design tool that predicts pavement performance over time by considering the interplay of material properties, traffic loading, and environmental factors (Huang, 2004). Version 2.5 of the PMED software was used in this study. The software requires structured input across six areas:

general design information, pavement performance criteria, traffic data, climate station data, pavement layers/structure and material properties, and project-specific calibration factors.

3.4.1 General Design Information

In this project, a new flexible pavement with a 10-year design life was selected for a two-lane, two-way road. Other software parameters, such as the base construction date and traffic opening date, were set to the default values defined by the software.

3.4.2 Pavement Performance Criteria

The design structure directly influences the pavement performance criteria. The flexible pavement was designed to meet specific PMED performance standards, including terminal International Roughness Index (IRI) (in./mile), Permanent Deformation (PD)—total pavement (in.), AC bottom-up fatigue cracking (% lane area), AC thermal cracking (ft./mile), AC top-down fatigue cracking (ft./mile), and PD—AC only (in.). Additionally, design reliability reflects the probability that the pavement will perform adequately over its intended service life, ensuring that distresses and roughness remain below critical levels (AASHTO, 2016).

Table 3.4 Pavement Performance Criteria and Reliability

Distress Type	Criteria/Default	
	Distress Target	Reliability Level (%)
Terminal IRI (in./mile)	180	75
Permanent deformation—total pavement (in.)	0.45	75
AC bottom-up fatigue cracking (% lane area)	20	85
AC thermal cracking (ft./mile)	750	75
AC top-down fatigue cracking (ft./mile)	2,000	85
Permanent deformation—AC only (in.)	0.45	85

3.4.3 Traffic Data

The traffic inputs included annual average daily truck traffic (AADTT) for the base year, growth factor, number of lanes in the design direction, percent of trucks in the design direction, percent of trucks in the design lane, and operational speed in mph. The detailed traffic inputs are summarized in Table 3.5. It was noted that some traffic inputs, such as traffic volume monthly adjustment factors (level 3), axle configuration, and monthly and hourly adjustment factors, were used as the default values provided by the PMED software. The traffic characteristics were kept consistent across all alternative pavement designs.

Table 3.5 PMED Traffic Inputs

Traffic Inputs	Flexible Pavement
AADTT	300
Growth rate, linear (%)	3
Number of lanes	2
Trucks in design lane (%)	50
Operational speed (mph)	60

3.4.4 Climate Station Data

Environmental factors, such as temperature, precipitation, and freeze-thaw cycles, can significantly impact pavement performance. PMED incorporates these factors using climate data from an existing weather station or a user-defined virtual station. For this project, a virtual climate station was created based on weather data from Topeka, Kansas. The input parameters include the mean annual air temperature, mean annual precipitation, freezing index, and average yearly freeze-thaw cycles. The detailed climate parameters are summarized in Table 3.6.

Table 3.6 Climate Inputs

Climate Station Information/Location	Virtual
Mean annual air temperature (°F)	55.17
Mean annual precipitation (in)	46.49
Freezing index (°F - days)	349.22
Average annual number of freeze/thaw cycles:	90.09

3.4.5 Pavement Structure and Material Properties

In Kansas, a typical asphalt pavement structure consists of three AC layers. This three-layer system includes a 1.5-in. surface layer, a 2.5-in. intermediate layer, and a variable-thickness AC base layer.

This study's designed pavement structure consists of a 1.5-in. AC overlay on top, with the intermediate and bottom AC layers combined into a single layer for design consideration. The base layer options include 10- and 12-in. SFDR base layers, followed by an 18-in. subgrade layer. The schematic of the designed flexible pavement structure is illustrated in Figure 3.9. The optimum AC thickness was the minimum required thickness of the total AC layer that satisfied all performance criteria. If one or more predicted performance criteria were not satisfied, the pavement structure was altered by changing the AC overlay layer's thickness.

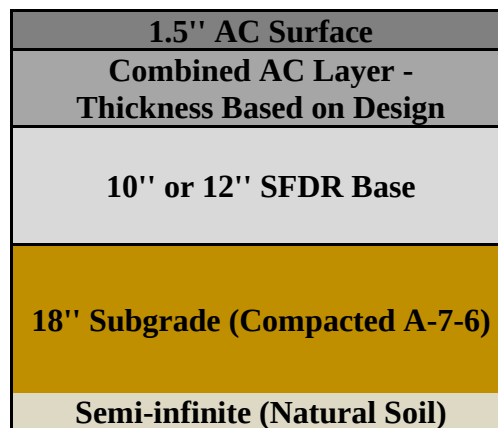


Figure 3.9 Trial Flexible Pavement Structure

3.4.5.1 Asphalt Layer

The PG binder for the AC layer was determined following KDOT requirements, which consider the total equivalent single axle loads (ESALs) and the percentage of RAP in the mixtures (Tavakol, 2019). This study used a PG 70-28 binder for the AC surface and the combined AC layer due to the thin AC overlay on the SFDR base layer. The AC properties used as inputs for the AASHTOWare software are summarized in Table 3.7. The combined AC layer thickness was adjusted to meet the specified performance criteria.

Table 3.7 AC Properties

AC Parameter	
Thickness (in)	1.5+Various
Binder grade	PG 70-28
NMAS (mm)	12.5
Unit weight (PCF)	140.0
Effective binder content (% volume)	11
Air voids (%)	7
Poisson's ratio	0.35
Reference temperature (°F)	70
Thermal conductivity (BTU/hr-ft-°F)	0.67
Heat capacity (BTU/lb-°F)	0.23

3.4.5.2 SFDR Base Layer

The SFDR layer's thickness is based on the load-carrying requirements of the rehabilitated roadway, subgrade soil type, stability, engineering properties, environmental conditions, presence of buried utilities, and existing pavement thickness (Reeder et al., 2019). The SFDR layer's strength significantly impacts the required thickness of the surface layer. Reeder et al. (2019) pointed out that a more substantial SFDR base allows for reduced AC layer thickness. This thickness varies from 6 to 12 in., depending on the specific project requirements.

This study considered two SFDR base layer thicknesses: 10 and 12 in. As discussed earlier, selecting layer thicknesses in pavement design is an iterative process that balances structural

capacity with cost considerations. The laboratory-measured properties of the SFDR mixtures were input into the software as the base layer properties for non-stabilized crushed stone since ver. 2.5 of PMED did not support chemically-treated base layers in flexible pavement design. The software selected a non-stabilized base with cold recycled (CR) asphalt, including milling.

The elastic modulus (E) of the SFDR layer was an essential input for the base layer. In PMED, the modulus of cement-treated soil is computed using Equation 3.1, and the modulus of lime-cement-fly ash–treated soil is computed by adding 500 psi to the laboratory-measured 7-day UCS values (psi) for the lime-cement-fly ash–stabilized soil, according to the PMED (AASHTO, 2015).

$$E = 1200 q_u \quad \text{Equation 3.1}$$

where:

E = elastic modulus (psi), and

q_u = 7-day UCS of cement-treated FDR measured according to ASTM D1633 (psi).

Other critical inputs for the SFDR base layer included gradation, specific gravity, MDD, and OMC, all based on corresponding laboratory measurements. The software mentioned that Poisson's values are between 0.15 and 0.35 and are typically used for chemically stabilized material (AASHTO, 2015). The Poisson's ratio for the SFDR base layer was assumed to be 0.35 for all pavement alternative designs.

3.4.5.3 Subgrade Layer

As mentioned earlier, a compacted subgrade layer (AASHTO A-7-6 soil) with an 18-in. thickness was used for trial designs. The subgrade properties used as inputs for the AASHTOWare software are summarized in Table 3.8.

Table 3.8 Subgrade Properties

Soil classification	A-7-6
Layer thickness (in)	18
Poisson's ratio	0.35
Modulus level input	2
Coefficient of lateral pressure, K_o	0.5
Modulus (input) (psi)	2,600
Maximum dry unit weight (pcf):	98.0
The specific gravity of solids, G_s :	2.700
Water content (%)	23

3.4.6 Project-Specific Calibration Factors

In the AASHTOWare PMED software, calibration factors are pavement distress-based coefficients that predict pavement performance. The default calibration factors are derived from a comprehensive dataset provided by the State DOT's agencies; it provides a general baseline for predicting pavement performance (AASHTO, 2015). For this project, the default (global) calibration factors for new flexible pavement designs were adopted for all alternative pavement designs.

3.4.7 Analysis Results

Once all the related parameters are inputted, the PMED software runs the analysis. It automatically generates a PDF report summarizing the input and output for the trial design. An example of PMED software analysis output is illustrated in Figure 3.10.

Design Inputs

Design Life: 10 years Base construction: June, 2019 Climate Data 37.5, -95
Design Type: FLEXIBLE Pavement construction: August, 2019 Sources (Lat/Lon)
Traffic opening: September, 2019

Design Structure

Layer type	Material Type	Thickness (in)
Flexible	Default asphalt concrete	1.5
Flexible	Default asphalt concrete	1.5
NonStabilized	Crushed stone	10.0
Subgrade	A-7-6	18.0
Subgrade	A-7-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	11.0
Air voids (%)	7.0

Traffic

Age (year)	Heavy Trucks (cumulative)
2019 (initial)	300
2024 (5 years)	276,331
2029 (10 years)	596,673

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in/mile)	180.00	111.69	85.00	99.99	Pass
Permanent deformation - total pavement (in)	0.55	0.29	85.00	100.00	Pass
AC bottom-up fatigue cracking (% lane area)	20.00	1.17	85.00	100.00	Pass
AC thermal cracking (ft/mile)	750.00	337.87	75.00	99.43	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	341.49	95.00	100.00	Pass
Permanent deformation - AC only (in)	0.45	0.05	85.00	100.00	Pass

Figure 3.10 PMED Software Analysis Results

Chapter 4 - Laboratory Test Results

This chapter discusses the laboratory test program results for SFDR mix design, durability, and shrinkage. The test results include moisture-density characteristics, unconfined compressive strength (UCS), vacuum saturation, and linear shrinkage. The optimum thickness of the AC layer was then determined based on the PMED software outputs.

4.1 Moisture-Density Characteristics

The moisture-density relationship of the SFDR mixture yields important physical parameters. In this test, eight SFDR mixtures were designed. Five samples were prepared for each SFDR mixture to conduct the Proctor test. The maximum dry density (MDD) and the related optimum moisture content (OMC) values were interpolated from the moisture content-density curve. An example of a moisture-density curve from the Proctor test results is shown in Figure 4.1. Detailed Proctor test results for each SFDR mixture are attached in Appendix B.

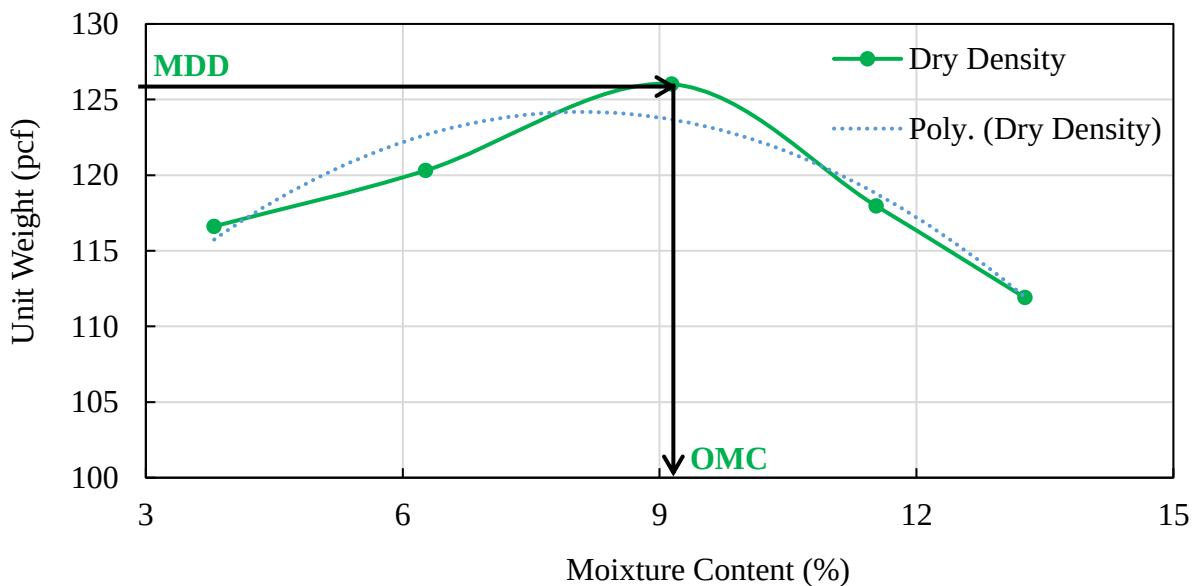


Figure 4.1 Moisture-Density Curve for SFDR with 5% Cement

The interpolated MDD and OMC results for all SFDR mixtures are summarized in Table 4.1, and the corresponding figure is plotted in Figure 4.2. The moisture-density characteristics of

SFDR mixtures are plotted in Figure 4.2. The plot shows that increasing fly ash content decreased MDD and OMC for the SFDR mixtures without cement. The control specimen (without any stabilizer) had an MDD of 128.7 pcf with an OMC of 9.5%, while the mixtures with 10% and 20% fly ash showed a decreased MDD of 126.5 and 125.5 pcf, respectively. The corresponding OMC values decreased from 8.03% to 7.4%. For mixtures without any fly ash but with cement, increasing cement content from 5% to 10% resulted in a slight increase in MDD and OMC. For the mixture group that contains both fly ash and cement, SFDR with 5% fly ash and 5% cement resulted in the highest MDD (129.3 pcf) and highest OMC (9.15%), while SFDR with 6% fly ash and 4% cement showed the lowest MDD (123.8 pcf) and OMC (8.50%).

Table 4.1 Proctor Test Results

SFDR	OMC (%)	MDD (pcf)
FDR (Control)	9.5	124.2
10FA	8.0	123.8
20FA	7.4	124.8
10C	8.9	123.8
5C	9.1	123.8
5C&5FA	9.2	126.0
4C&6FA	8.5	123.8
3C&7FA	8.4	124.5
2C&8FA	8.3	124.5

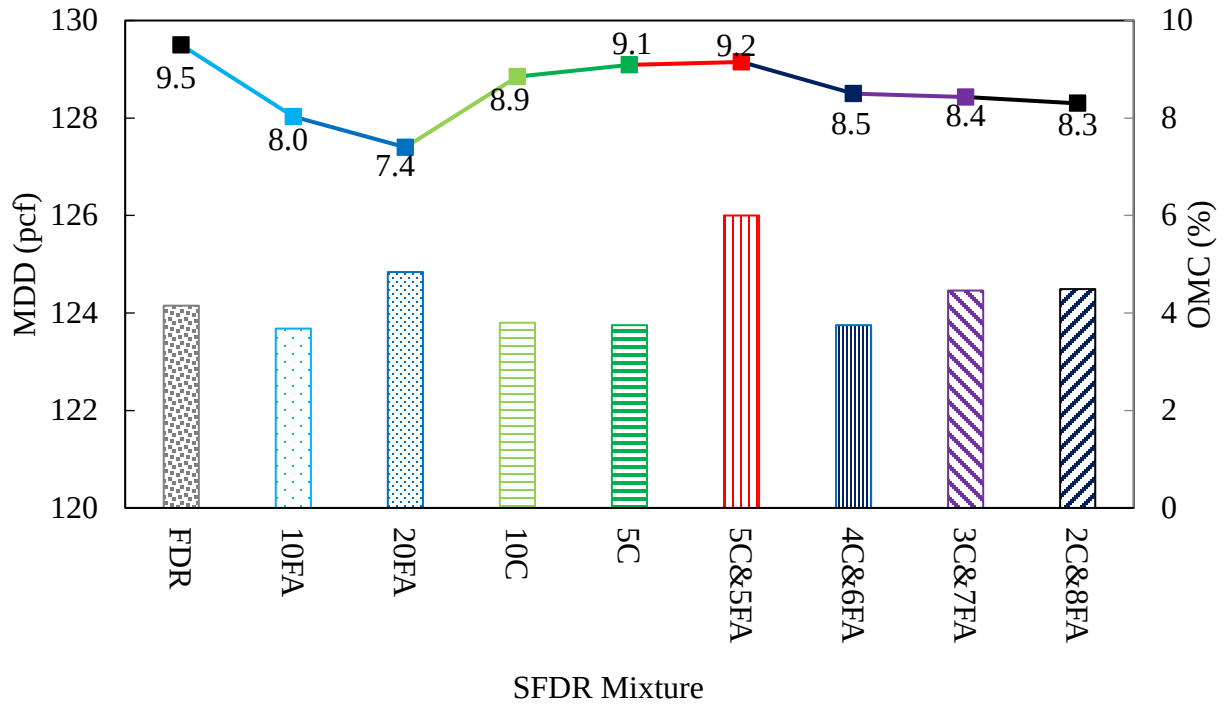


Figure 4.2 Moisture-Density Characteristics of SFDR Mixtures

4.2 Unconfined Compressive Strength (UCS)

Many state agencies emphasize the key design parameter of the SFDR mixture, the UCS value. As discussed in Section 3.3.2, this study targeted a UCS range between 250 and 400 psi.

The average 7-day UCS of each SFDR mixture is computed and summarized in Table 4.2. The results indicated that the SFDR mixture with 5% cement & 4% cement and 6% fly ash met the lower strength (250 psi) requirement. The SFDR mixture with 5% cement and 5% fly ash slightly exceeded the upper range of the strength requirement (400 psi). The SFDR mixture with 10% cement showed the highest UCS value of 610 psi, and the other SFDR mixtures showed unsatisfactory UCS values.

Table 4.2 UCS Test Results

SFDR	UCS (psi)	COV (%)	Remark
FDR	39.7	6	Low
10FA	59.8	11	Low
20FA	124.4	7	Low
10C	609.6	1	High
5C	244.3	1	Moderate
5C&5FA	473.1	4	High
4C&6FA	277.6	3	Moderate
3C&7FA	174.8	0	Low
2C&8FA	160.3	0	Low

Figure 4.3 illustrates the average UCS of the eight SFDR mixtures and the control specimen. For SFDR mixtures containing only fly ash (no cement), increased fly ash content resulted in higher UCS values, with 10% and 20% fly ash mixtures achieving 60 psi and 124 psi, respectively. A similar trend was observed for the SFDR mixture with only cement, where the UCS significantly increased from 244 psi (5% cement) to 610 psi (10% cement). Cement had a more significant effect on the UCS of SFDR mixtures than fly ash. For mixtures containing both cement and fly ash, UCS decreased as the fly ash content increased and the cement content decreased. The SFDR mixture with 5% cement & 5% fly ash achieved a UCS of 473 psi, while the SFDR mixture with 4% cement & 6% fly ash showed a UCS of 278 psi. The SFDR mixture with 3% cement & 7% fly ash had a UCS of 175 psi, and the SFDR mixture with 2% cement & 8% fly ash showed the lowest UCS at 160 psi. Thus, the SFDR mixture with 5% cement, 5% cement & 5% fly ash, and 6% cement & 4% fly ash met the unconfined compressive strength range (250-400 psi) selected in this study.

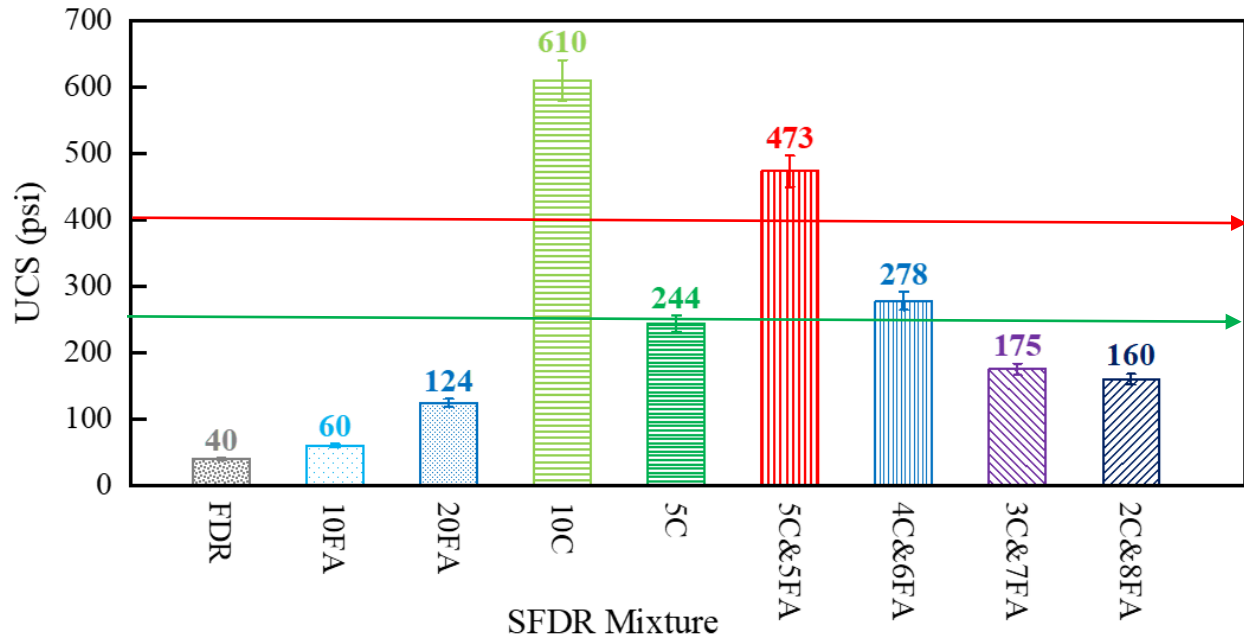


Figure 4.3 Average UCS of the SFDR Mixtures

4.3 Moisture Susceptibility (VS)

The vacuum saturation tests were conducted to evaluate the moisture susceptibility of eight SFDR mixtures. In this test, the SFDR specimens were subjected to conditions simulating saturation to assess their performance under moist conditions, particularly their resistance to strength loss when exposed to moisture. The vacuum saturation results are summarized in Table 4.2. The SFDR mixtures with 10% cement, 5% cement, and 5% cement & 5% fly ash emerged as the most promising mixtures in terms of UCS values, since their average retained UCS met the strength design criteria of 250–400 psi.

Table 4.3 VS Test Results

SFDR Mixture	Saturated		Unsaturated		VSR	Remark
	UCS (psi)	COV (%)	UCS (psi)	COV (%)		
FDR	-	-	-	-	-	-
10FA	38	22	48	3	80%	Low
20FA	115	4	86	3	134%	Low
10C	593	7	662	4	90%	High
5C	285	2	290	1	98%	Moderate
5C&5FA	419	2	347	10	121%	High
4C&6FA	403	4	369	8	109%	Moderate
3C&7FA	220	5	208	10	106%	Low
2C&8FA	142	8	143	10	99%	Low

Figure 4.4 illustrates the average UCS values for both saturated and unsaturated SFDR mixtures. The vacuum saturation test results exhibited trends similar to the UCS test results. Cementitious mixtures tend to gain strength when exposed to moisture due to the continued cement hydration process. Consequently, SFDR mixtures containing cement, fly ash, or both demonstrated higher strength upon vacuum saturation than the unsaturated (control) condition.

The eight SFDR mixtures showed increased UCS values when subjected to saturation, except for the 10% cement SFDR mixture. The SFDR mixture with 5% cement, and 2% cement & 8% fly ash performed slightly better in maintaining UCS under saturation conditions. For the mixtures containing both cement and fly ash, the decrease in vacuum saturation ratio (VSR) values was observed for decreasing fly ash and increasing cement content. Cement and fly ash helped control the UCS development under moist conditions. However, the SFDR mixture with fly ash content higher than 10% could result in a drastically decreased UCS under saturation conditions. This led to the conclusion that 10% fly ash by the FDR dry weight would be a threshold for maintaining UCS under saturation.

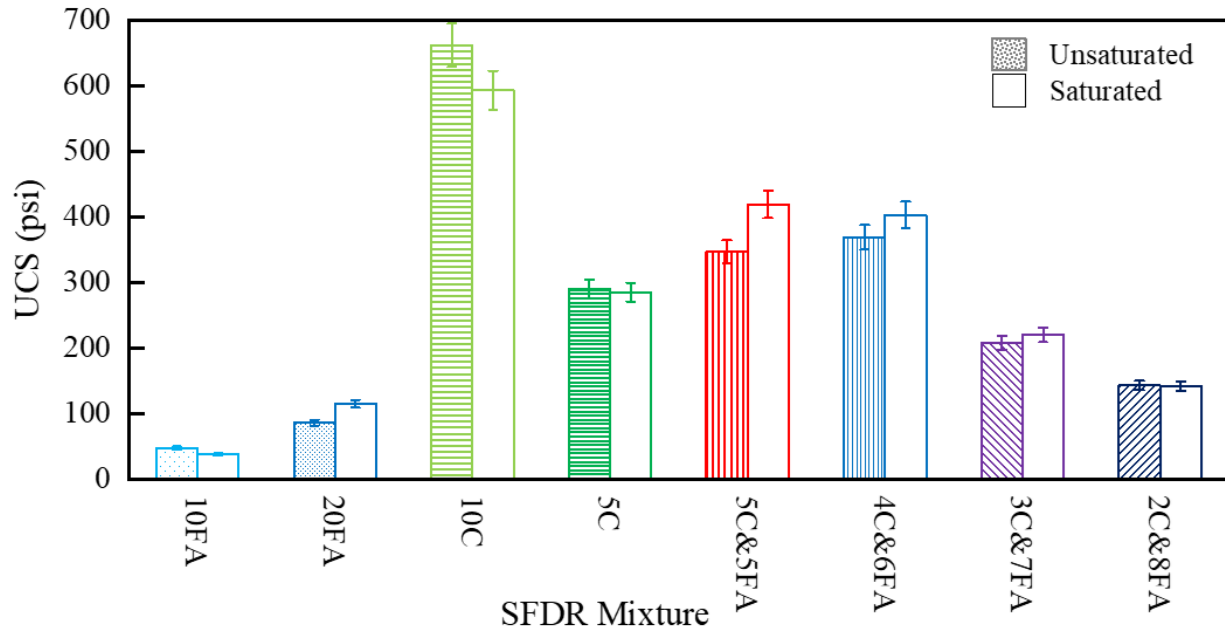


Figure 4.4 Vacuum Saturation Test Results

4.4 Linear Shrinkage

Linear shrinkage of cementitious mixtures, also known as drying shrinkage, is defined as the contraction of a cementitious material as it loses moisture. Higher linear shrinkage levels could cause cracking in the SFDR base layer that can further propagate to the pavement surface. Therefore, investigating the shrinkage characteristics of the SFDR mixtures with various stabilizers is necessary for pavement design consideration.

Figure 4.5 illustrates the average linear shrinkage strain (microstrains) for various SFDR mixtures over a 28-day monitoring period. The length change of specimens was at various time intervals, as detailed in Section 3.3.4. The results indicated that all SFDR mixtures exhibited higher shrinkage during the first seven days (earlier stage), with mixtures containing cement showing a rapid increase in shrinkage during the early stage. After seven days, the shrinkage levels remained stable for most SFDR mixtures. Guthrie et al. (2002) referenced an Australian standard while evaluating shrinkage of cement-aggregate mixtures in which the shrinkage strain should not

exceed 250 microstrains after 21 days. In this project, none of the SFDR mixtures met this Australian shrinkage level limit.

The shrinkage test results show that in the SFDR mixture with only cement, shrinkage levels increased with cement content. Conversely, for mixtures containing fly ash only, shrinkage levels decreased as the fly ash content increased. The SFDR mixture with 20% fly ash showed the lowest shrinkage (600 microstrains) among all SFDR mixtures. This observation agreed with the findings of Rashidi et al. (2020), who concluded that higher cement content in SFDR increased while higher fly ash content reduced the shrinkage level of the SFDR mixture. However, the SFDR mixture with both cement and fly ash showed a different shrinkage behavior. The SFDR with 2% cement & 8% fly ash showed the highest shrinkage levels (2,088 microstrains), while the SFDR mixture with 5% cement & 5% fly ash showed the lowest shrinkage strain of approximately 1,500 microstrains. These results indicated that in SFDR mixtures containing both cement and fly ash, lower cement content and higher fly ash amount tend to increase shrinkage.

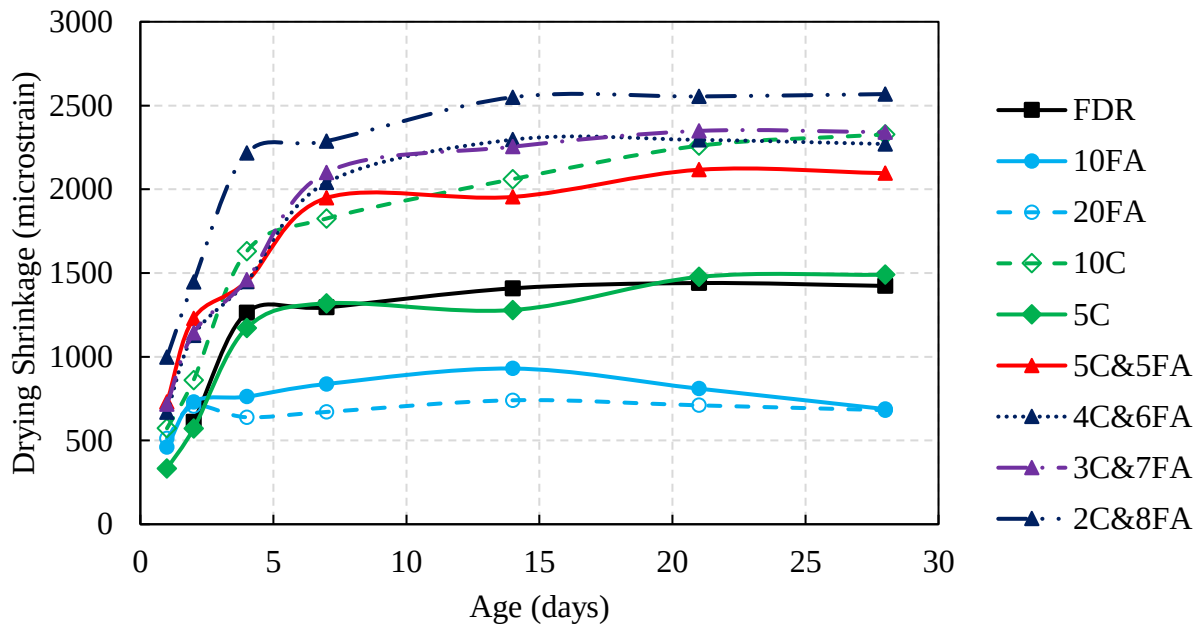


Figure 4.5 Linear Shrinkage Level of SFDR Mixtures

The overall drying shrinkage test results indicate that SFDR mixtures with higher cement content are more prone to shrinkage-related cracking, while fly ash serves as an effective stabilizer in reducing shrinkage when blended with the FDR materials. Additionally, when designing SFDR mixtures, the fly ash and cement quantities need to be balanced, which could help reduce the risks associated with excessive shrinkage in the SFDR base layer.

4.5 AASHTOWare PMED Analysis Results

This section presents the analysis results obtained from the AASHTOWare PMED software. For the PMED analysis, the SFDR base was treated as a separate layer, with the resilient modulus computed from the UCS test results. The computed resilient modulus values for various SFDR mixtures are summarized in Table 4.4. As discussed in Section 3.4, the optimum thickness of the AC layer was determined based on 10-in and 12-in SFDR base thicknesses. The AASHTO A-7-6 soil subgrade was subdivided into an 18-inch layer consisting of both compacted and natural soil.

Table 4.4 PMED Resilient Modulus of SFDR Mixtures

SFDR	7-day UCS (psi)	PMED Resilient Modulus (psi)
10FA	60	72,000
20FA	124	149,303
10C	610	731,549
5C	244	293,146
5C&5FA	473	567,703
4C&6FA	278	333,167
3C&7FA	175	209,756
2C&8FA	160	192,397

Figure 4.6 illustrates the optimal AC layer thicknesses for pavements with 10-in. and 12-in. SFDR bases, as determined by the software outputs. The red dashed line represents the 7-day

UCS values (in psi) for each SFDR mixture, while the bars represent the optimum AC thickness for the 10-in. and 12-in. SFDR base layers. The analysis indicated that SFDR mixtures with higher UCS values needed thinner AC layers for a desired pavement design, indicating that stronger base materials resulted in higher structural capacity. The SFDR mixtures with lower UCS values required a thicker AC layer.

The PMED analysis results showed that the SFDR mixture with 5% cement (244 psi UCS) needed an AC layer thickness of 4.5 in, while mixtures with 10% cement (610 psi UCS) and 5% cement & 5% fly ash (473 psi UCS) required a minimum of 3-in. AC layer. The SFDR mixtures with high strength resulted in AC layer thicknesses that were independent of the SFDR base layer thickness (10 or 12 in.). However, for mixtures with UCS values lower than 244 psi, such as SFDR mixtures with 10% fly ash, 20% fly ash, 3% cement & 7% fly ash, and 2% cement & and 8% fly ash, increasing the SFDR base layer thickness from 10 inches to 12 inches resulted in a noticeable reduction in the required AC layer thickness. As discussed earlier, the USACE requires a minimum UCS of 250 psi for a stabilized layer in the flexible pavement, irrespective of the stabilizer type used (USACE, 1994). This recommended minimum UCS value aligned closely with the observed threshold of 244 psi, below which the AC overlay thickness becomes sensitive to the SFDR base layer thickness.

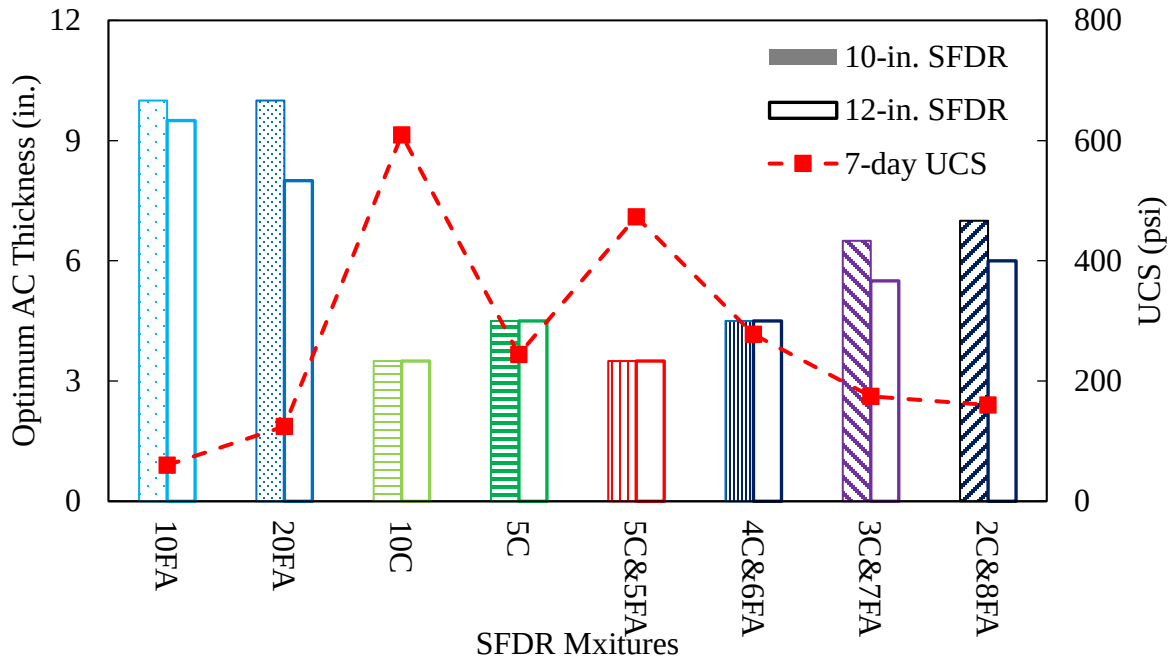


Figure 4.6 Optimum Thickness of AC Layers for 10-in. and 12-in. SFDR Layers

PMED design designed the AC layer to satisfy specific performance criteria at specified reliability. The performance criteria used for this study are summarized in Table 3.4 . The predicted pavement performance included rutting, top-down cracking, and IRI. Figure 4.7 shows the AC rutting for all SFDR mixtures. The results indicated that regardless of the SFDR layer thickness, the pavement section containing the SFDR mixture with the highest UCS (10% cement) exhibited the minimum rutting (0.08 in), followed by the pavement with 5% cement & 5% fly ash (0.09 in).

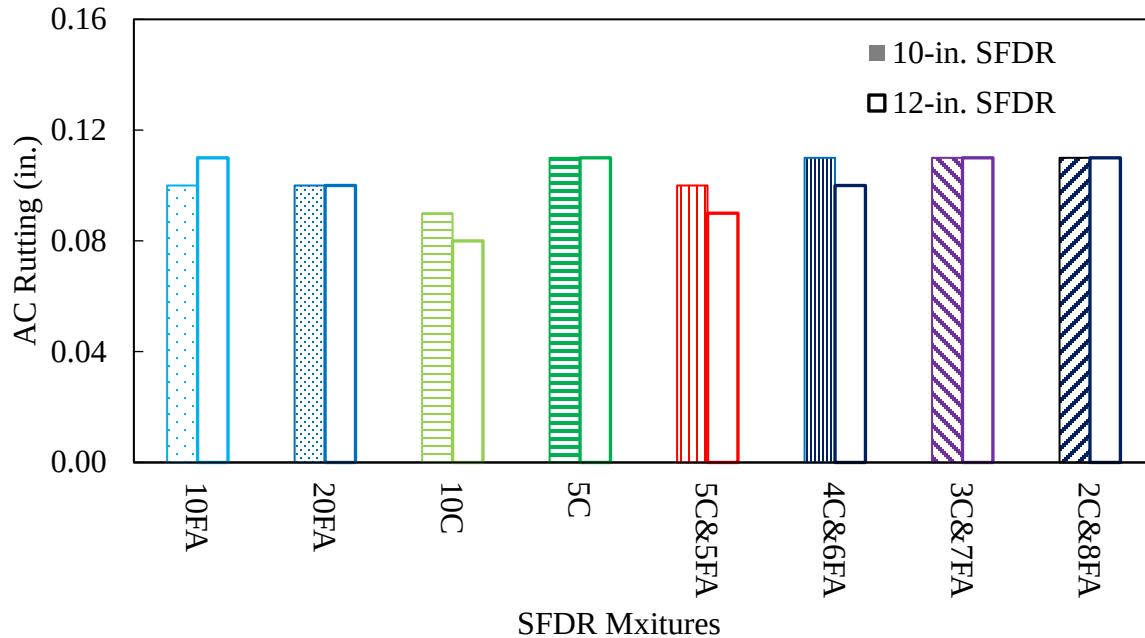


Figure 4.7 Rutting Performance of Designed Pavement Sections

Top-down cracking is a standard distress in flexible pavements, and it is characterized by cracks that initiate at the pavement surface and propagate downward. The PMED software-predicted top-down cracking results for all designed SFDR pavement sections are shown in Figure 4.8. For the 10-in. SFDR base, the mixtures with 10% cement and with 5% cement & 5% fly ash had the lowest top-down cracking with 542 and 533 ft/mile, respectively. For 12-in. SFDR base, the 5% cement and 4% cement & 6% fly ash mixtures had the lowest top-down cracking with 207 and 447 ft/mile, respectively. It appeared that significant differences in top-down cracking among all four SFDR mixtures were related to the stiffness of the SFDR layer.

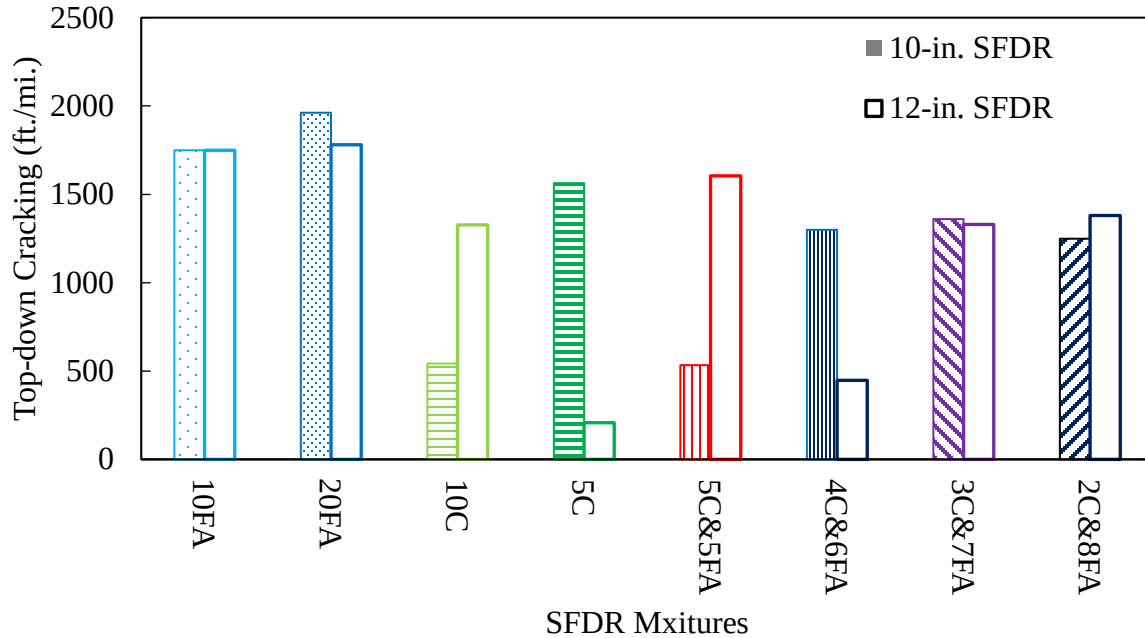


Figure 4.8 Top-Down Cracking for the SFDR Pavement Section

The IRI value quantifies the ride quality in pavement engineering. A higher IRI value indicates a rougher ride, while a lower IRI value indicates a smoother ride. The AASHTOWare PMED-predicted IRI results are summarized and plotted in Figure 4.9. For both 10-in. and 12-in. SFDR base layers, the mixtures with 10% cement, and 5% cement & 5% fly ash had the lowest IRI values, about 87 in./mile. Some other SFDR mixtures exhibited higher IRI for both 10-in. and 12-in. SFDR base layers.

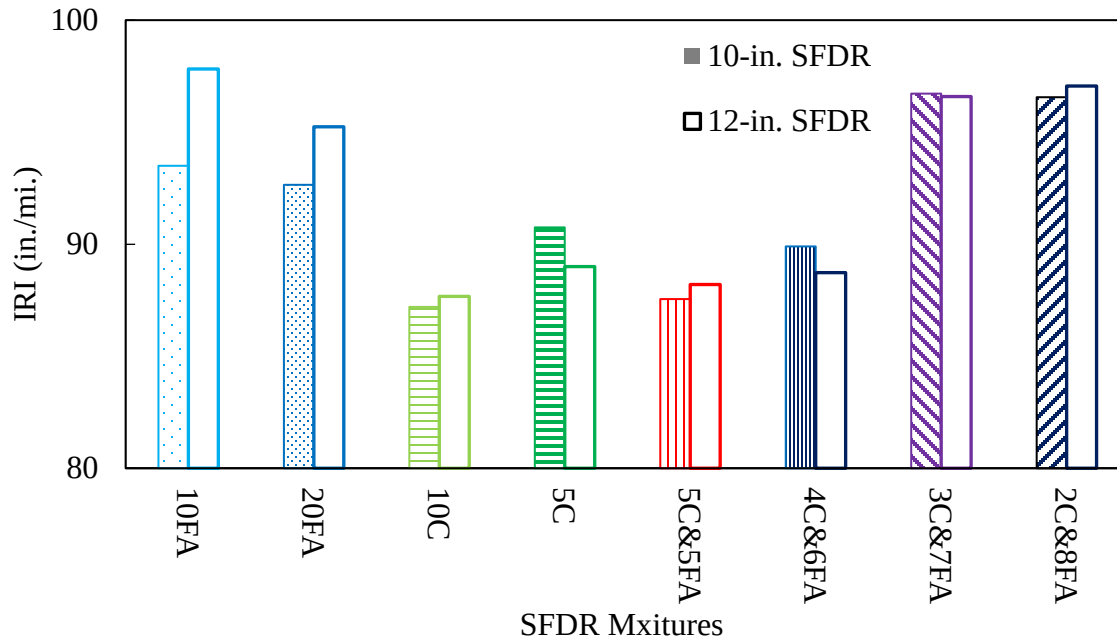


Figure 4.9 IRI Results for the SFDR Pavement Section

Based on the laboratory test results, three desirable SFDR mix designs were identified: SFDR with 5% cement (5C), 5% cement & 5% fly ash (5C & 5FA), and 4% cement & 6% fly ash (4C & 6FA). The PMED software outputs indicated that these mixtures' optimum AC layer thicknesses were 4.5 inches, 3.5 inches, and 4.5 inches, respectively. After consultation with KDOT, the final SFDR mix designs for the Phase II full-scale test were decided to select the SFDR mixtures for the base layer, SFDR with 5% cement and SFDR with 5% cement & 5% fly ash.

Chapter 5 - Phase II: Full-Scale Testing

Phase II of this study focused on full-scale testing of pavement structures at the CISL. This chapter discussed the construction of the pavement structure in two test pits, where each pit was divided into two subsections. During construction, instruments, including strain gauges, pressure cells, and thermocouples, were embedded within the pavement structures at specific locations. An APT machine was programmed to apply up to 600,000-wheel load on the pavement sections. This chapter discusses the APT machine, test section construction, instrumentation, field tests, and full-scale testing program.

5.1 Civil Infrastructure Systems Laboratory (CISL)

The CISL at KSU is designed for large-scale testing of pavement sections under real-world traffic conditions. The facility includes three test pits, each measuring approximately 20 feet in length, 16 feet in width, and 6 feet in depth, and an APT machine. The APT machine is belt-driven by a 20-horsepower electric motor and can apply single or dual axle loads with air-bag suspension, simulating truck traffic. Depending on research needs, this system can be customized for various testing conditions, such as adjusting hydraulic pressure for loading, tire pressure, and test speed (Lewis, 2008).

The APT machine operates in an insulated aluminum chamber with two air conditioners on either side to maintain temperature during the test. Figure 5.1 depicts the APT machine with a test section. Additionally, the machine features a lateral wander capability, which simulates the varying wheel paths of heavy vehicles on highways. This is achieved by applying a ± 6 -in. In this study, lateral wander is in a truncated normal distribution, with each wander cycle consisting of 1352-wheel load repetitions.

The APT machine was configured with a single axle and dual tires, applying an 18-kip bidirectional load to the pavement section. The tire pressure was maintained at 90 psi, and the tests were conducted at a constant speed of 7 mph for the entire test. The wander distribution for the wheel load is shown in Figure 5.2.



Figure 5.1 (a) APT Tester; (b) APT Test Section

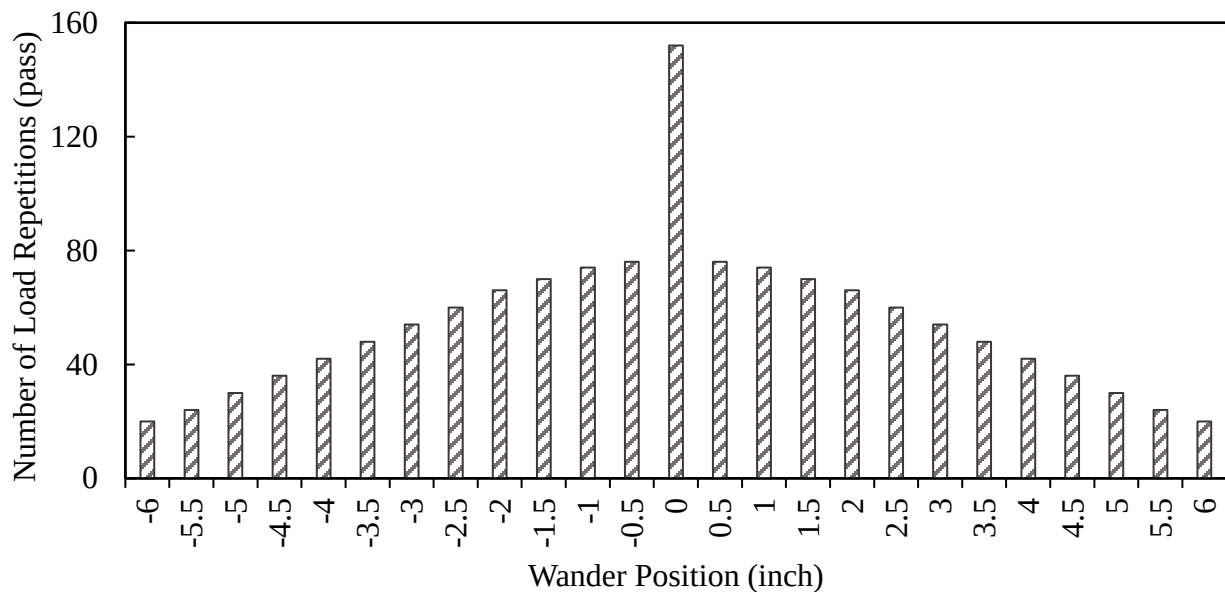


Figure 5.2 APT Tester Wander Distribution

5.2 Test Section Construction

This study used two test pits at CISL to construct the designed flexible pavement using the SFDR base. A picture of the middle and south test pits at CISL is shown in Figure 5.3. The two pits had identical pavement structures consisting of four layers: from bottom to top, a natural soil layer, a compacted subgrade layer, an SFDR base layer, and an HMA overlay. The middle pit was constructed with a 10-in. SFDR base and a 4.5-in. AC overlay, while the south pit consisted of a 12-in. SFDR base with a 3-in. AC overlay.

Each pit was divided into two subsections to accommodate the SFDR base mixtures. Each subsection was 10 ft. wide, 16 ft. long and 6 ft. deep. The SFDR mixture with higher strength containing 5% cement and 5% fly ash (5C&5FA) was used in the eastern subsections of both pits. In contrast, the western subsections of both pits were constructed with a lower-strength SFDR base layer stabilized with 5% cement (5C) only. The plan view of the SFDR base construction and subsectioning is shown in Figure 5.4. Four pavement structure combinations were designed and constructed as shown in Figure 5.5.



Figure 5.3 Pavement Construction Plan View

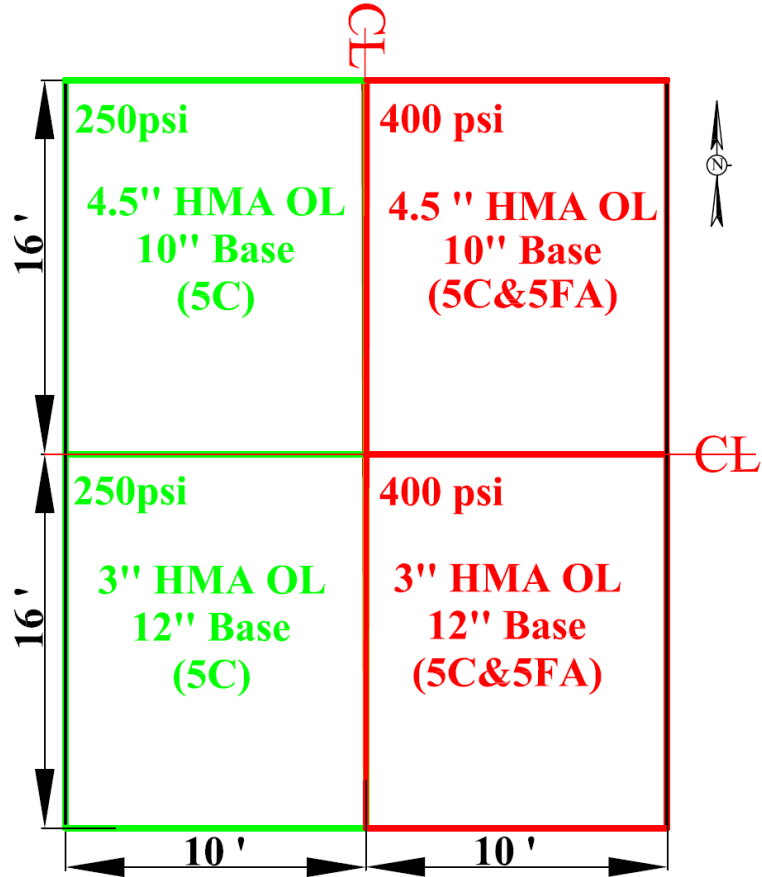


Figure 5.4 SFDR Base Construction and Subsectioning

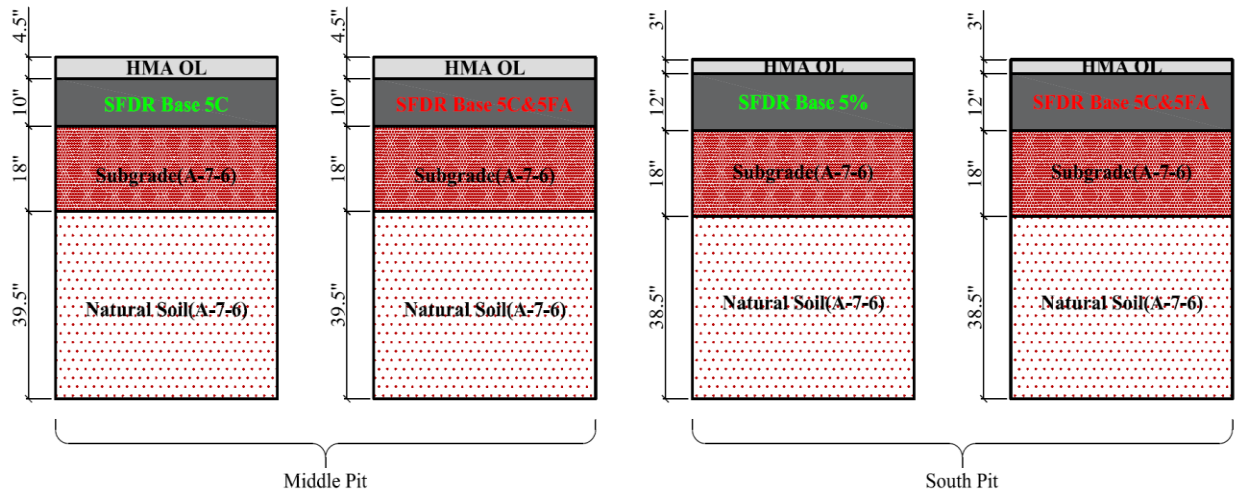


Figure 5.5 Pre-designed AC Pavement Profile

5.2.1 Subgrade

As shown in the designed asphalt pavement section in Figure 5.5, a 39.5-in. layer of natural soil (AASHTO A-7-6) was placed in the middle pit using seven lifts (6-in. loose lift), whereas a 38.5-in. layer of natural soil was constructed in the south pit. A vibratory sheepfoot roller BMP8500 was employed for soil compaction, which followed a specific pattern to ensure the compaction was applied to the pit uniformly. The compactor for soil compaction is shown in Figure 5.6.



Figure 5.6 Soil Compactor and Compacted Layer

A Troxler 3440 moisture-density gauge was used to monitor soil compaction and moisture content during pavement construction, as shown in Figure 5.7. A Speedy moisture tester was also used to verify soil moisture throughout construction. Each compacted lift had a moisture content ranging from 26% to 28% and a degree of compaction between 93% and 95%, meeting the expected subgrade layer construction standards by KDOT. KDOT standard specification states that the **“subgrade layer shall be compacted to a minimum of 95% of the MDD, and moisture content for subgrade compaction shall be within ± 3 % of the OMC”** (KDOT, 2015).



Figure 5.7 Nuclear Gauge Testing

The subgrade layer was constructed using the same soil (A-7-6) and construction procedures as the natural soil layer but with more compaction. The proctor test results indicated that the soil's maximum dry density was 98 pcf at an optimum moisture content of 23%. The 18-inch subgrade layer for all pits was compacted in three lifts, with the overall moisture content controlled at $25 \pm 1\%$. Each lift achieved an average compaction degree of 96% or higher.

A DCP test assessed the subgrade layer's uniformity, and CBR values were computed based on the DCP results. In each section, DCP tests were conducted at three randomly selected locations, resulting in 12 DCP tests across the pit. Figure 5.8 illustrates the CBR test results for the subsections. The results indicate consistent CBR values across the sections. Overall, the average CBR values for each section are summarized in Table 5.1. The overall pit was computed to have an average CBR of approximately 6.2% at 25% moisture content.

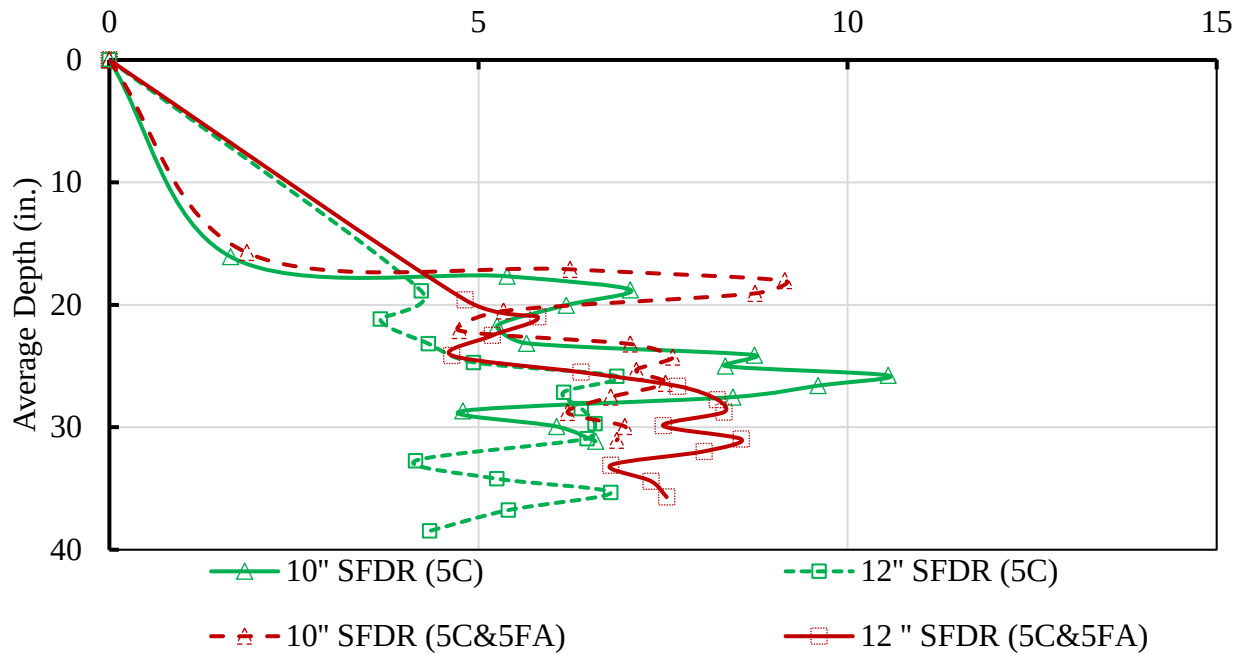


Figure 5.8 Average CBR Values of Subgrade Layer

Table 5.1 Average CBR of Subsection

Section	SFDR Thickness	5C	5C&5FA
	(in.)	(%)	(%)
Middle	10	6.5	6.0
South	12	6.2	6.3

5.2.2 SFDR Base

The construction of the SFDR base was completed over two days. The FDR mixture containing fly ash and cement for the east section was constructed on the first day. The FDR material was prepared and mixed with the stabilizer outside CISL, as shown in Figure 5.9 (a). The existing moisture content was determined before the stabilizer was added. Due to space constraints and working conditions limitations, a 5-gallon plastic bucket was used as a makeshift scale, referred to as the “bucket-scale” approach. Identical height marker lines were drawn on the inner walls of the buckets. The weight of the fly ash or cement was recorded when the bucket was filled

to each marked level. Calculating the number of buckets used to determine the total amount of fly ash or cement added to the mix. The premeasured and marked buckets of fly ash and cement are illustrated in Figure 5.9 (b) and (c). A water-flow meter attached to a hose was used to ensure the predetermined amount of water added to the SFDR material. The existing moisture content in the FDR mixture was considered when calculating the added water.

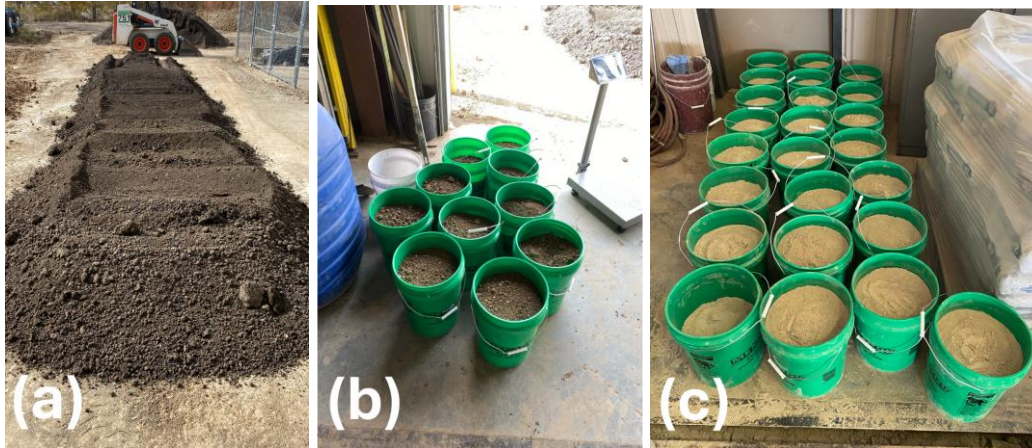


Figure 5.9 SFDR Mixing Material Preparation: (a) FDR Material; (b) “Bucket” Weight Scale (Soil); (c) “Bucket” Weight Scale (Fly Ash)

Two percent of soil by dry weight of FDR material was evenly spread over the surface of the prepared FDR material, as shown in Figure 5.10 (a). The fly ash and cement were then uniformly spread across the FDR material and leveled using a rake to ensure proper blending. The in-place mixed SFDR material, along with water application, is depicted in Figure 5.10 (b). An RS-500B reclaimer/stabilizer was utilized for on-site SFDR material mixing (Figure 5.11). A skid loader transferred the fresh SFDR mixtures to the pits. Ten Speedy moisture tests were conducted. The test samples were randomly collected along the SFDR mixture bed. The overall average moisture content of the fresh SFDR mixture with 5% cement & 5% fly ash was 9.7%.



Figure 5.10 FDR Materials: (a) with 2% Soil; (b) Cement

During SFDR mixture placement, four UCS test specimens (each measuring 4 in. in diameter by 4.59 in. in height) were compacted from each subsection. These specimens were sealed in a bucket and transferred to a moist room for a 7-day curing. After curing, the specimens were subjected to UCS tests to verify the design mixture strength. A steel-wheeled vibratory roller was used to compact the SFDR mixture, and a jumping jack-type compactor was employed to compact the edges of the pits (Figure 5.12). A 6-in. high timber board was installed to separate the 10-in. (middle) and 12-in. (south) SFDR base sections. The middle pit contained a 10-in. SFDR base with 5% cement & 5% fly ash whereas the south pit had a 12-in. SFDR base with 5% cement only. Both sections were compacted in two lifts, and a nuclear gauge was used to monitor the in-place SFDR density (Figure 5.12).

Two nuclear gauge tests were conducted for each subsection along the predefined wheel path zone, with nuclear gauge readings taken at three depth levels: 7 in., 6 in., and 4 in. The results of the nuclear gauge tests for the SFDR base with 5% cement & 5% fly ash (5C&5FA) are summarized in Table 5.2. The test results indicated that the overall SFDR compaction reached 98.2%, meeting KDOT's stabilized base standard requirements of minimum 95% (KDOT, 2015). To prevent premature cement hardening, the construction process was completed within an hour.



Figure 5.11 RS-500B Reclaimer



Figure 5.12 SFDR Base Construction

Table 5.2 Nuclear Tests Results (SFDR 5C&5FA)

Pit	Depth	4 in	6 in	7 in	Average
Middle	% Proctor	98.1	98.7	98.3	98.4
	Dry Density (pcf)	124.0	124.4	123.8	124.0
	Wet Density (pcf)	137.3	137.6	137.0	137.3
	% Moisture	10.8	10.7	10.7	10.7
South	% Proctor	97.8	99.2	97.1	98.0
	Dry Density (pcf)	123.2	124.9	122.4	123.5
	Wet Density (pcf)	136.2	138.0	135.4	136.5
	% Moisture	10.5	10.5	10.7	10.6

The west section was constructed following the same construction procedure as the east section on the following day. On-site mixing indicated that the SFDR mixtures with cement and fly ash had better workability than the SFDR mixtures with only cement. The Speedy moisture test results indicated that the overall average moisture content of the fresh SFDR mixture with 5% cement was 8.9%, which was lower than the SFDR mixture with 5% cement & 5% fly ash. The results of the nuclear gauge tests for the SFDR base with 5% cement (5C) are summarized in Table

5.3 The test results indicated that the overall SFDR compaction reached 97%. The finished SFDR base layer is shown in Figure 5.13. After the nuclear tests, all sections were covered with a plastic sheet to prevent moisture loss. A 7-day curing period was observed to allow the cement and the fly ash to react with water adequately. The surface was periodically moistened to prevent excessive drying and cracking. Figure 5.14 shows the compacted SFDR mixture covered by a plastic membrane during the 7-day curing process.

Table 5.3 Nuclear Tests Results (SFDR 5C)

Pit	Depth	4 in	6in	7 in	Average
Middle	% Proctor	96.8	96.5	97.1	96.8
	Dry Density (pcf)	122.2	121.6	122.4	122.1
	Wet Density (pcf)	137.4	137.1	137.7	137.4
	% Moisture	12.4	12.7	12.5	12.5
South	% Proctor	97.3	96.7	97.4	97.1
	Dry Density (pcf)	122.6	121.8	122.8	122.4
	Wet Density (pcf)	136.8	136.0	136.8	136.5
	% Moisture	11.6	11.7	11.4	11.5



Figure 5.13 Nuclear Gauge Testing on Compacted SFDR Base



Figure 5.14 SFDR Base Curing

5.2.3 AC Overlay

After seven days of curing, the AC overlay was placed on the SFDR base layer. A 3-in. AC layer was placed in one lift across the sections in the south pit, and a 4.5-inch AC layer was placed in two lifts in the middle pit. The HMA material consisted of a Superpave (SR-12.5A) mixture with a 12.5-mm nominal maximum aggregate size (NMAS), a PG 70-22 binder, and 15% RAP. The properties of the Superpave HMA mixture are summarized in Table 5.4. Shilling Construction, based in Manhattan, Kansas, completed the HMA paving and compaction. The paver and paved HMA mixture are shown in Figure 5.15 (a) and Figure 5.15 (b). A smooth steel vibratory compactor was used to compact the HMA mat (Figure 5.15 (c)). Furthermore, a nuclear gauge was used to check the density and degree of compaction for the HMA layer. The nuclear gauge tests were conducted at the predetermined wheel path zone, from the middle to the south pit. Four tests were conducted on the two pits. The test results are summarized in Table 5.5, the results indicate that the overall HMA compaction reached 92.6%.

Loose HMA materials were collected for laboratory performance tests, including Hamburg wheel tracking and dynamic modulus tests. Figure 5.15 shows the completed asphalt pavement

sections at the CISL. The entire construction process took 2.5 years due to delays caused by the COVID-19 pandemic and subsequent challenges in acquiring the necessary equipment and labor.

Table 5.4 HMA Mixture Properties

Mix Parameter	
% AC by Mass of Mix	6.0
% Aggregate by Mass of Mix	94.3
Sp. Gr. of AC	1.035
Bulk Sp. Gr. of Agg.	2.577
Theoretical Max. Sp. Gr.	2.433
Bulk Sp. Gr. of Mix	2.346
Eff. Sp. Gr. of Agg.	2.65
Absorbed % AC	1.106
Efferent Asphalt Content	4.960
% VMA	14.4
% Air Voids	3
% V.F.A.	78
Eff. Film Thickness	10.52
Dust/Binder Ratio	1.0



Figure 5.15 (a)HMA Mixture Paver; (b) Paving; (c) Compaction

Table 5.5 Nuclear Tests HMA Layer

Section	Middle		South		Overall
Nuclear Parameter	WP1	WP2	WP3	WP4	Avg.
% Compaction	93.2	92.0	92.5	92.8	92.6
Density (pcf)	139.4	137.6	138.4	138.8	138.5

5.3 Instrumentation

Permanent deformation and fatigue are the two primary failure mechanisms in flexible pavements and depend on critical responses (stresses or strains) in the pavements. Various sensors were installed in the pavement test sections in the test pits to investigate the pavement's structural response and performance. The typical instrumentation layout for the pavement structure is shown in Figure 5.16. Pressure cells were placed directly on top of the subgrade, and strain gauges and a thermocouple were installed at the surface of the SFDR base layer.

To ensure proper sensor placement, the wheel path and transverse profile zones were predetermined, as illustrated in Figure 5.17. The green zone on the left side of the figure represents the pavement section with a lower-strength (250 psi) SFDR base layer stabilized with 5% cement. The red zone on the right indicates a higher-strength 400-psi mixture, which the SFDR base layer stabilized with 5% cement & 5% fly ash.

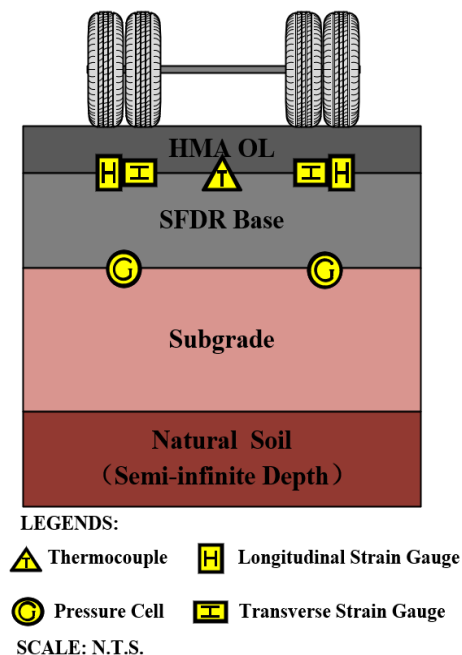


Figure 5.16 Schematic of Instrumentation Layout

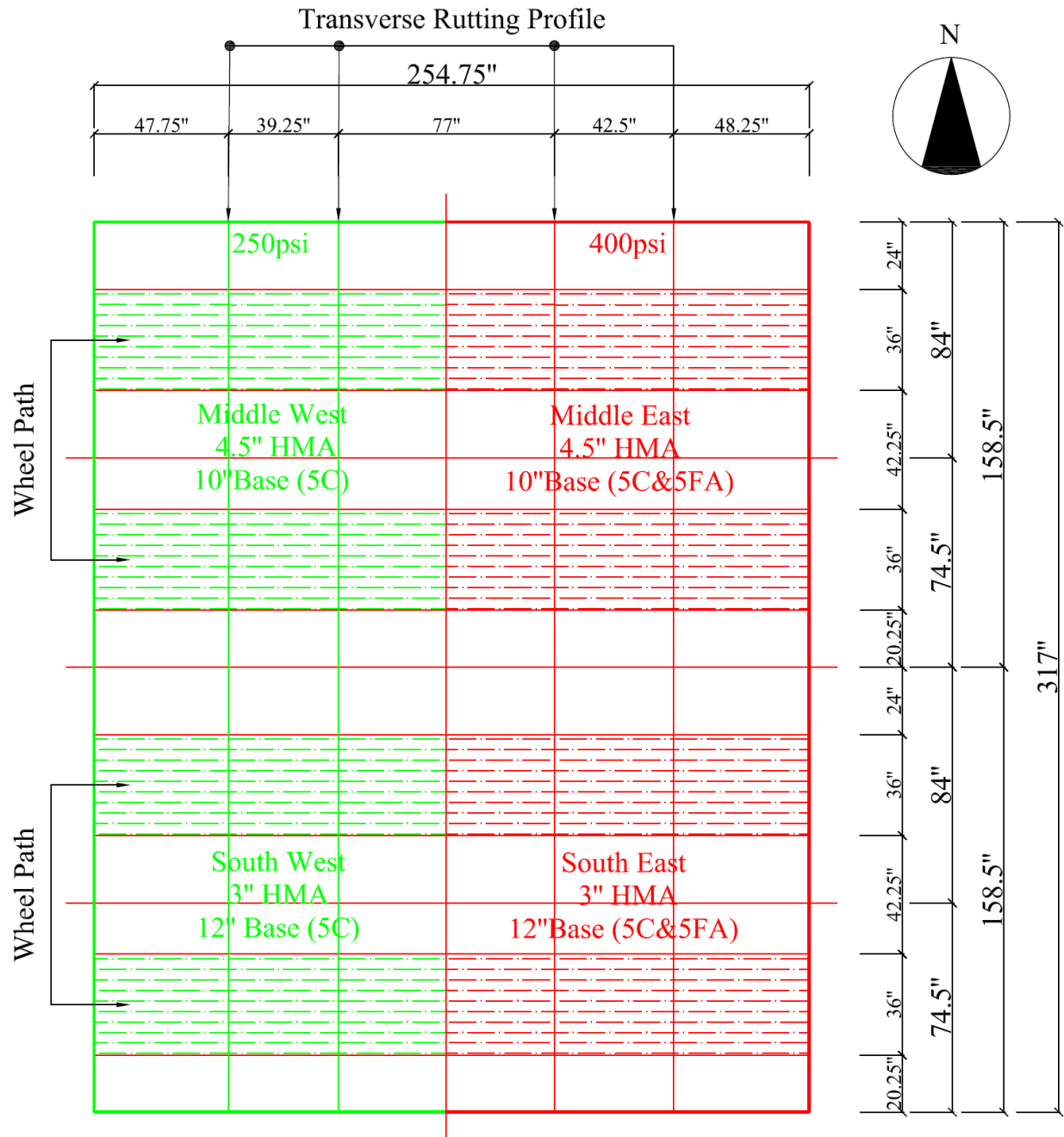


Figure 5.17 Schematic of Wheel Path and Rutting Profile

5.3.1 Pressure Cells

The hydraulic pressure cells were selected and installed on top of the subgrade layer to measure the vertical stress exerted on top of the subgrade layer. The cells function by measuring the hydraulic pressure of de-aired hydraulic oil confined between two steel plates, with the pressure

data converted into an electrical signal via either a semiconductor-type or vibrating wire transducer. Vibrating wire transducers are typically used for long-term static pressure measurements. The semiconductor-type transducers are better suited for capturing dynamic pressures from traffic loads (Saghebfar, 2014). These hydraulic pressure cells have been successfully used in previous projects at CISL.

This study selected Geokon 3500 hydraulic pressure cells with a pressure range of 0–58 psi due to their static and dynamic pressure measurement capability. These sensors function within a temperature range of 5°F–180°F. Additionally, each pressure cell was calibrated by Geokon and equipped with a 25-ft. wire for efficient data transmission before installation.

On each subsection, two pressure cells were installed along the centerline of the wheel path to measure vertical compressive stress on top of the subgrade accurately. A schematic diagram of the pressure cell installation is presented in Figure 5.18. The wheel path was geometrically predefined to ensure accurate vertical stress data collection during wheel load application. The total wheel path width included the 24-in. dual wheel path plus 12 in. of wander (± 6 in.). Small excavations were made in the compacted subgrade to ensure the precise placement of the cells, as shown in Figure 5.19 (a). Sand was used to fill any voids underneath the cells, and the cells were carefully leveled with additional sand to prevent stress concentration around the sensors (Figure 5.19 (c)). Eight Geokon pressure cells were installed across the four subsections.

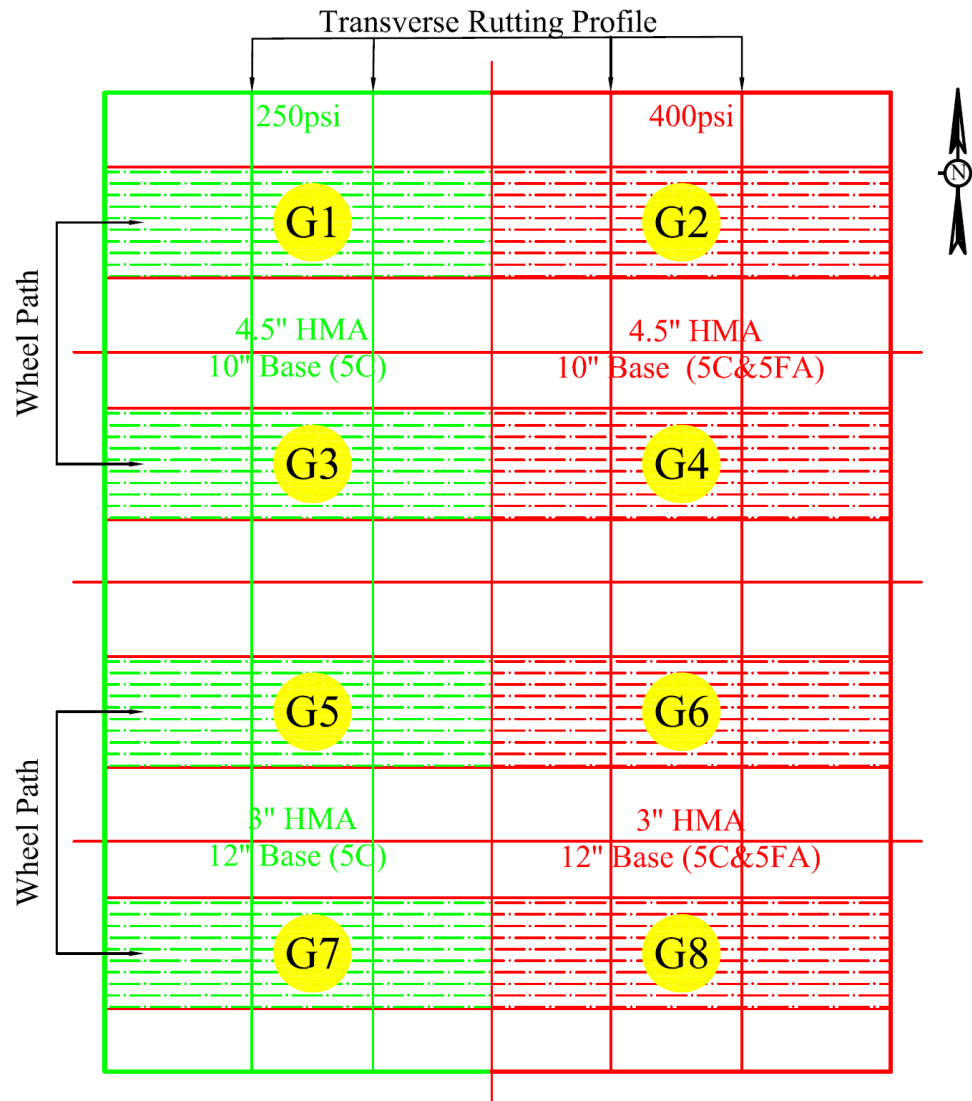


Figure 5.18 Schematic Plan of Pressure Cell Position

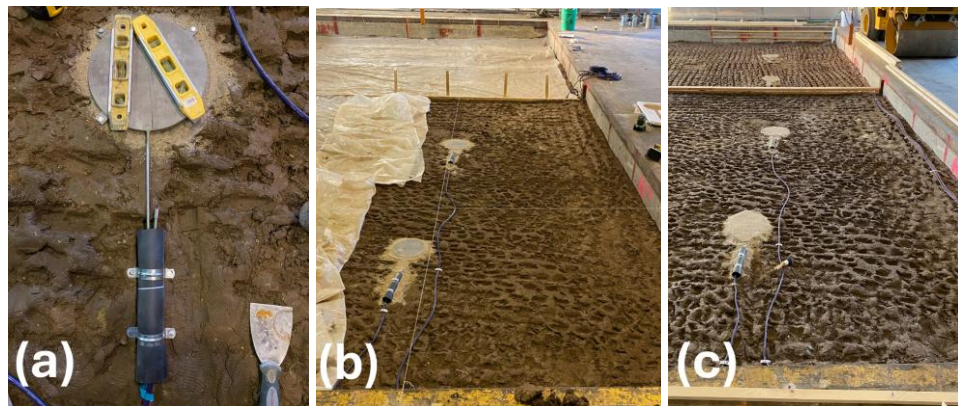


Figure 5.19 Pressure Cell Installation: (a) Pressure Cell Setup Leveling; (b) Pressure Cell installation; (c) Cable Management

5.3.2 H-Bar Strain Gauges and Thermocouple

According to the 1993 AASHTO Guide for Design of Pavement Structures, tensile strains at the interface between pavement layers are a primary factor contributing to fatigue cracking in asphalt pavements (AASHTO, 1993). H-bar strain gauges are known for their accuracy in measuring strain and have been successfully used in previous experiments at the CISL. For this study, PML-60-2L strain gauges, manufactured by Tokyo Sokki Kenkyujo Co. (TML), were used to measure strain at the bottom of the AC overlay.

Each gauge was attached by epoxying with two aluminum rods, forming the characteristic H-shape, as shown in Figure 5.20 (a). However, earlier studies have highlighted that the standard lead wires the manufacturer provided were not sufficiently resistant to the high temperatures encountered during HMA compaction (Saghebfar, 2014). To address this issue, the original lead wires were replaced with high-quality shielded wiring (22 AWG, 2 conductors) to enhance thermal resistance, as illustrated in Figure 5.20 (b). The connection points were soldered and further protected using a heat-shrinking sheath and a heavy-duty heat-shielding tube. To avoid the potential changes in gauge resistance during soldering and installation procedures, each H-bar gauge was calibrated twice: once after soldering and again after installation.

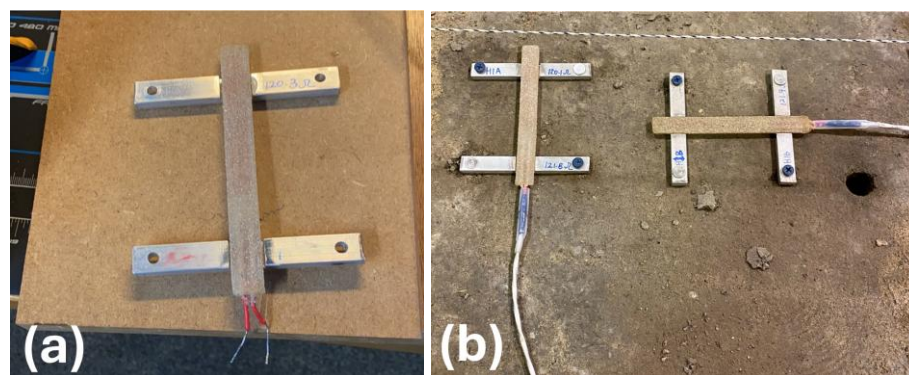


Figure 5.20 H-Bar Strain Gauge: (a) “H-Bar” Strain Gauge; (b) Strain Gauge Installation

As shown in Figure 5.21, two groups of strain gauges were installed within the predefined wheel path of each subsection, positioned directly above the locations of the pressure cells to capture critical strain response. Each group contained two H-bar strain gauges: one oriented longitudinally, parallel to the wheel path, and the other oriented transversely, perpendicular to the wheel path. A total of sixteen strain gauges were used for the project. The schematic layout of the H-bar strain gauges across the two test pits is illustrated in Figure 5.22. To protect the gauges during HMA construction, loose HMA mixture was spread on the gauges and lightly compacted. Two of the 16 strain gauges failed to register any electrical signal. These two transverse gauges located at the north wheel path in the middle pit lost their electrical signals during HMA construction.



Figure 5.21 H-Bar Strain Gauge Installation

In this study, four thermocouples were embedded in the SFDR layers at the center of each wheel path, as illustrated in Figure 5.22. Two thermocouples were connected to the data acquisition system for continuous monitoring, and the other two served as backups during wheel load applications. Including thermocouples was essential because Louw et al. (2016) demonstrated that the strain levels in SFDR base layers were susceptible to temperature variations during wheel loading. Accordingly, in this study, the temperature data collected by the thermocouples was

utilized to closely monitor and correlate temperature fluctuations with strain changes in the SFDR layer during testing.

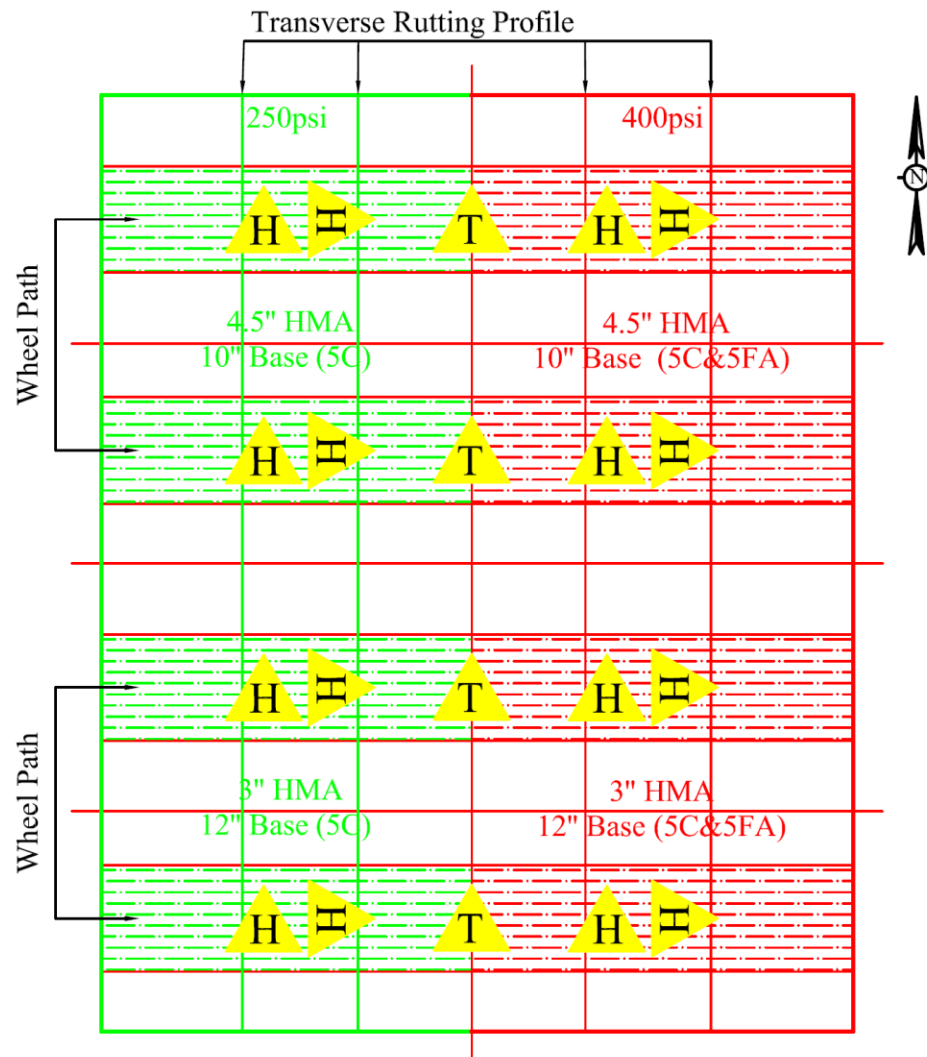


Figure 5.22 Plan View of H-Bar Strain Gauge Group

5.3.3 Data Collection

Stress and strain data were collected using a compact data acquisition (DAQ) system that National Instruments developed. The DAQ system consisted of a DAQ dock with attachable block modules interfacing with pressure cells, H-bar strain gauges, and thermocouples. Figure 5.23(a) shows the pressure cell (purple cable) and the H-bar gauges (white wire). Comprehensive signal diagnostic tests of the sensors were conducted to ensure sensor functionality before everything was

assembled in the terminal block modules. Resistance values for the pressure cells and H-bar strain gauges were carefully documented and used for final sensor calibration.

The DAQ system was connected to a desktop computer via a standard USB interface. The LabVIEW software was programmed to monitor and record data, which consisted of vertical stresses on the subgrade, longitudinal and transverse strains at the bottom of the AC (HMA) layer, and temperature in the SFDR layer. The LabVIEW program is shown in Figure 5.23 (b). LabVIEW facilitated the system to save all data in CSV format automatically, which could be imported into an Excel spreadsheet. The data collection was manually controlled. The data could be recorded for the entire time or terminated at the specific wheel load repetitions. In this test, the test data were collected for a wander cycle for each test round, which is 1,732-wheel load repetitions. The LabVIEW data collection program is shown in Figure 5.23 (b).

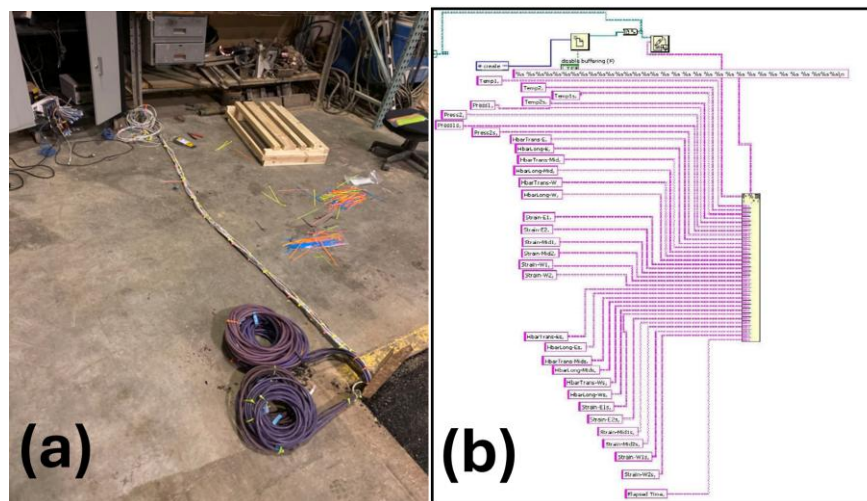


Figure 5.23 (a) Sensor Connections; (b) LabVIEW Program for Data Collection

5.4 Rutting Measurement

Flexible pavement rutting refers to the permanent deformation of the AC pavement surface, primarily occurring in the wheel paths due to repeated traffic loads, as shown in Figure 5.24. Rutting is a critical factor in flexible pavement performance evaluation. To measure the pavement

rutting, transverse profiles at two locations on each subsection were measured after a predetermined number of wheel load repetitions. The deformation measurement locations were predetermined geometrically, as shown in Figure 5.17. A Chicago dial indicator was equipped with a digital linear variable differential transformer (LVDT) and a roller attached to a leveled aluminum beam, as illustrated in Figure 5.25. The LVDT was connected to a laptop computer to facilitate data collection.



Figure 5.24 Rutting on Pavement Test Section Surface

Elevation readings were taken at 0.5-in. intervals across the lane. Each transverse profile consists of 288 data points. The elevation data were referenced against two predefined elevation control points at the ends of each transverse profile to provide a consistent baseline for rutting computation. The transverse-profile measurements were imported into an Excel spreadsheet to calculate rut depths and analyze pavement deformation.



Figure 5.25 Transverse Profile Measurement

5.5 Falling Weight Deflectometer (FWD) Test

The FWD test is a nondestructive testing method used to evaluate pavement's structural capacity. The FWD simulates a wheel load's impact on the pavement structure by applying a controlled impulse load to the pavement surface and measuring the pavement surface deflections. This data is then used to back-calculate the moduli of each pavement layer to evaluate the pavement's stiffness.

The FWD trailer used in the study was equipped with a JILS FWD system featuring an 11.8-in.-diameter load plate and seven transducer sensors. The sensors were positioned at specific distances from the center of the load plate (0, 8, 12, 24, 36, 48, and 60 in.) to measure the pavement surface deflection. The picture of the JILS FWD trailer is shown in Figure 5.26. The target load for the test was set to 9 kips to simulate the impact of an APT axle load. Due to space constraints at the test site, the FWD test was conducted twice on each wheel path, with the trailer reversing direction for the second pass. This resulted in four FWD tests on each subsection, as shown in Figure 5.26.

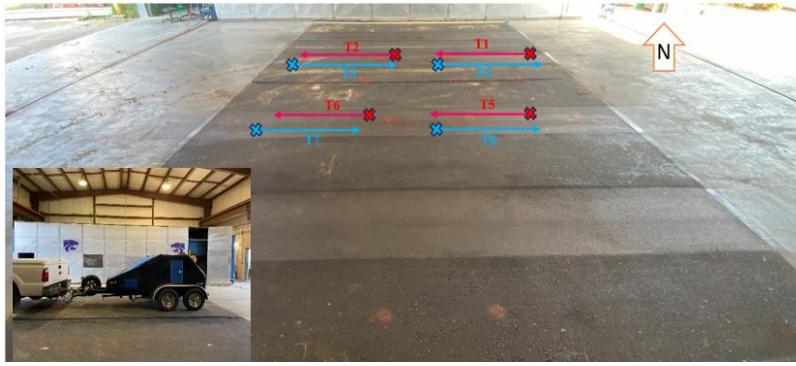


Figure 5.26 FWD Testing

5.6 Full-Scale Test Program

As mentioned, the full-scale APT validated the SFDR mix and pavement design using the AASHTOWare PMED software. The goal was to investigate the feasibility of using a thin AC overlay over the SRDR base layers of varying strength. The full-scale test program was planned to apply 500,000-wheel load repetitions of 18-kip single axle (0.5 million ESALs) to the test sections. The loading was also planned to terminate whenever the pavement reached a rut depth of 0.5-in. The detailed APT wheel load sequence is summarized in Table 5.6. An extra 100,000 wheel-load repetitions were applied to observe any cracking that might ensue after 500,000 repetitions. The complete full-scale test was finished in 1.5 years due to equipment issues.

Table 5.6 APT Wheel Load Repetition Sequence

Test No.	Number of Load Reps.	Remark
0	0	FWD Test
1	1,372	
2	10,000	
3	15,000	
4	25,000	
5	50,000	
6	100,000	
7	250,000	FWD Test
8	300,000	
9	400,000	
10	500,000	FWD Test
11	600,000	

Chapter 6 - Full-Scale Test Results

6.1 Introduction

This chapter discusses the full-scale test results at CISL. In parallel to the APT test, laboratory tests were conducted to find the physical properties of an in-place SFDR mixture with 5% cement and a combination of 5% cement & 5% fly ash. The test results consist of moisture content, unconfined compressive strength (UCS), and elastic modulus. The properties of HMA material were also evaluated via Hamburg Wheel-Tracking and Dynamic Modulus tests. Full-scale tests were conducted to investigate the fatigue and rutting performance of the pavement structure. The test results included back-calculated layer moduli using the FWD test data, the rutting on the AC pavement surface, the horizontal tensile strains on the bottom of the AC layer, and vertical compressive stress on top of the subgrade. The new version of PMED software was also used to verify the full-scale APT test results.

6.2 In-Place SFDR and HMA Mixture Properties

6.2.1 SFDR Base Properties

6.2.1.1 UCS

As discussed in the literature review, moisture content and strength are the primary quality assurance factors for the base construction with SFDR mixture (ARRA, 2015; PCA, 2017). This study conducted 20 Speedy moisture tests, and five SFDR mixture samples were collected from each subsection before compaction. The comparison between fresh and designed moisture contents for the SFDR mixtures is summarized in Table 6.1. The results indicated that approximately 0.56% more water was applied to the SFDR mixture with 5% cement and 5% fly ash, and approximately 0.46% more water was used for the SFDR mixture with 5% cement. Thus, the speedy moisture

test results were lower than the nuclear gauge readings due to capillaries drawing up more water during compaction.

Table 6.1 Comparison of Design and Fresh Mixture Moisture Contents

Test Section (AC Thickness)	SFDR Mixture	Average Moisture %	COV %	Designed OMC %	Difference %
Middle (4.5 in.)	5C&5FA	9.3	10	9.2	0.2
	5C	10.3	11	9.1	1.2
South (3 in.)	5C&5FA	10.1	6	9.2	1.0
	5C	8.8	10	9.1	-0.3

As discussed in Section 5.2.2, to verify the SFDR mix design, four UCS specimens, each with a height of 4.56 in. and a diameter of 4 in., were compacted from each subsection. After a 7-day curing period, three specimens were randomly selected for the UCS tests. The UCS results of the fresh SFDR compacted specimens have been summarized in Table 6.2.

For the middle test section with a 4.5-inch AC layer, the average UCS for the SFDR mixtures with 5% cement and a combination of 5% cement & 5% fly ash was 539 psi and 260 psi, respectively, which were 14% and 6% higher than the design UCS values. In the south section with a 3-in. AC layer, the average UCS for the SFDR with 5% cement & 5% fly ash was 427 psi, which was 10% lower than the design UCS. The SFDR with 5% cement was 283 psi, 17% higher than the design UCS.

Table 6.2 UCS Test Results for In-Place SFDR Mixtures

Test Section (AC Thickness)	SFDR Mixture	Average UCS (psi)	COV (%)	Design UCS (psi)	%Difference (%)
Middle (4.5 in.)	5C&5FA	539	7	473	14
	5C	260	19	244	6
South (3 in.)	5C&5FA	427	9	473	-10
	5C	285	7	244	17

6.2.1.2 In-Situ UCS and Elastic Modulus of SFDR Layer

The in-situ SFDR specimens were cored and subjected to UCS tests, as shown in Figure 6.1. The UCS test results are summarized in Table 6.3. The results indicate that the in-situ SFDR mixtures with 5% cement & 5% fly ash had UCS values approximately 20% less than the design value. The SFDR mixture UCS with 5% cement in the middle section was 20% higher than the design value, while the south section was 3% less than the design UCS.

As shown in the Speedy tester results in Table 6.1. The SFDR mixture with 5% cement in the middle section had a 13% higher water content than its corresponding OMC, and in the south section, it showed 3% less moisture than its OMC. However, the SFDR mixture with cement and fly ash did not show a higher UCS value due to its higher water content than its OMC.

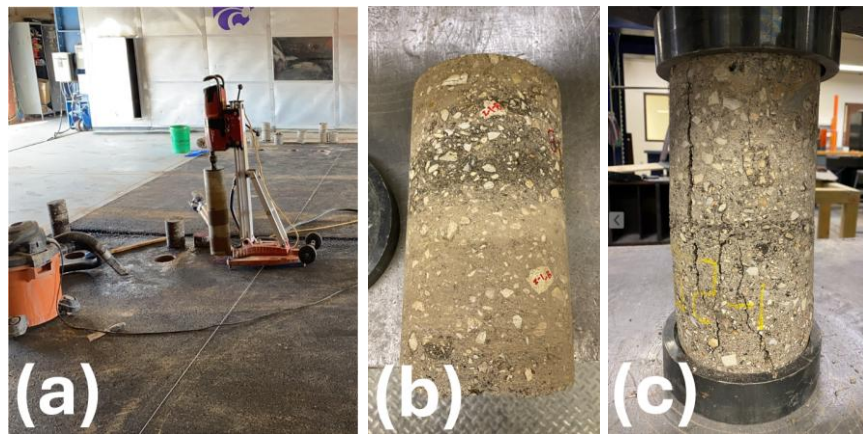


Figure 6.1 (a) In-Situ SFDR Specimen Extraction; (b) Cored SFDR Specimen; (c) UCS Test

Table 6.3 UCS Test Results of Cored SFDR Mixtures

Test Section (AC Thickness)	SFDR Mixture	Average (psi)	COV (%)	Design UCS (psi)	Difference (%)
Middle (4.5 in .)	5C&5FA	377	2	473	-20
	5C	294	10	244	20
South (3 in.)	5C&5FA	360	5	473	-24
	5C	236	22	244	-3

The load and displacement data were recorded during the UCS tests to determine the elastic modulus of the SFDR base layer. The elastic modulus was interpolated from the stress versus strain

curve. Figure 6.2 shows an example of the stress-strain plot for an SFDR specimen. The average computed elastic modulus values of in-situ SFDR mixtures are summarized in Table 6.4. The results showed that the mixtures with higher UCS values also contribute to higher elastic modulus. Specifically, the SFDR mixture with 5% cement & 5% fly ash had an elastic modulus of 28.7 ksi for the middle section and 32.1 ksi for the south section. The SFDR mixtures with 5% cement had an elastic modulus of 19.7 ksi for the middle section and 21.9 ksi for the south section.

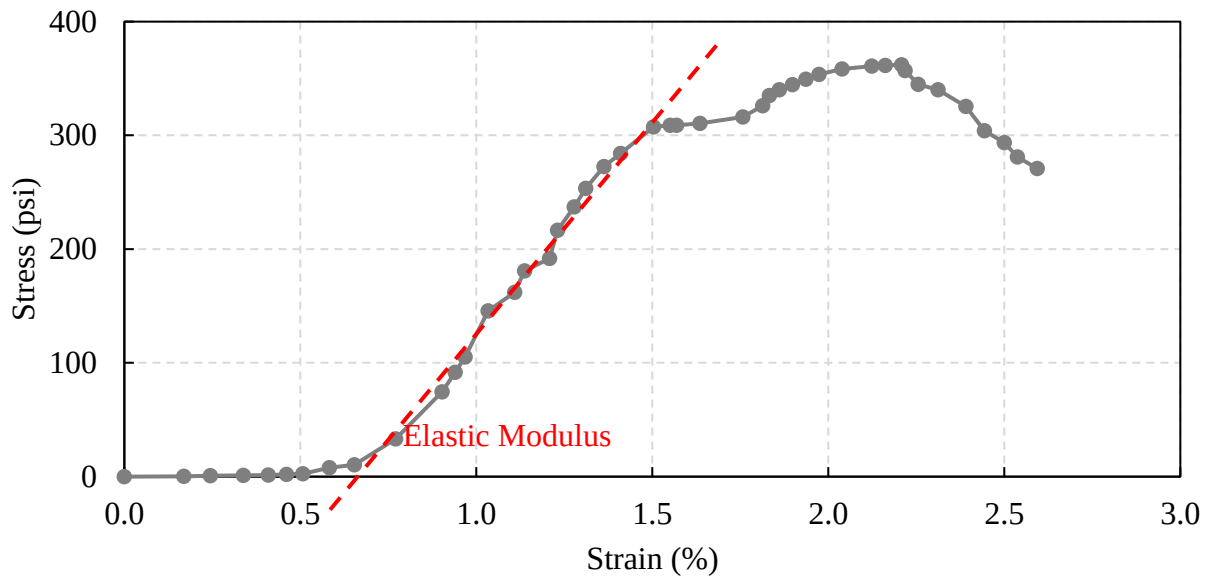


Figure 6.2 Stress-Strain Curve of SFDR Material

Table 6.4 Elastic Modulus of the SFDR Base

Test Section (AC Thickness)	SFDR Mixture	Elastic Modulus (ksi)	COV (%)
Middle	5C&5FA	28.7	11
(4.5 in.)	5C	19.7	9
South	5C&5FA	32.1	9
(3 in.)	5C	21.9	32

6.2.2 HMA Mixture Properties

To evaluate the performance characteristics of the HMA overlay mixture, loose HMA materials were collected during AC layer compaction. These materials were used to prepare samples for the Hamburg wheel-tracking and dynamic modulus tests.

6.2.2.1 Hamburg Wheel-Tracking Test

The Hamburg wheel-tracking test was used to evaluate the rutting and moisture susceptibility of the HMA mixtures. The test was conducted following AASHTO T324. The asphalt specimens were subjected to moving loaded wheels in a temperature-controlled water bath in the test. The specimens' rut depths and the corresponding number of wheel passes were recorded during the test. The test would be terminated upon reaching either a pre-defined maximum number of passes (20,000 in AASHTO T324) or a pre-defined maximum rut depth (0.5 in. for Texas Department of Transportation (TxDOT) with PG 70 binder). Figure 6.3 illustrates the HWTD test output for the in-situ cored HMA specimens. In this study, failure criteria were set following TXDOT requirements. TXDOT sets the maximum rut depth for HMA mixtures with a high-temperature binder grade of 70°C (PG 70) at 0.5 in. (12.5 mm) for 15,000-wheel passes (Button et al., 2004).

A Superpave gyratory compactor (SGC) was used to prepare the HWTD test specimens. Each specimen was compacted with a diameter of 5.9 in. and a height of 2.44 in. The HWTD test program is shown in Figure 6.4. Twenty-seven specimens were compacted and subjected to bulk-specific gravity tests to evaluate air voids, with the HWTD test requiring $7 \pm 1\%$ air voids (KT-15, 2016b). The in-situ cored HMA specimens were also used in the test. The HWTD test results are summarized in Table 6.5. The test results indicated that all samples narrowly failed to meet these requirements.

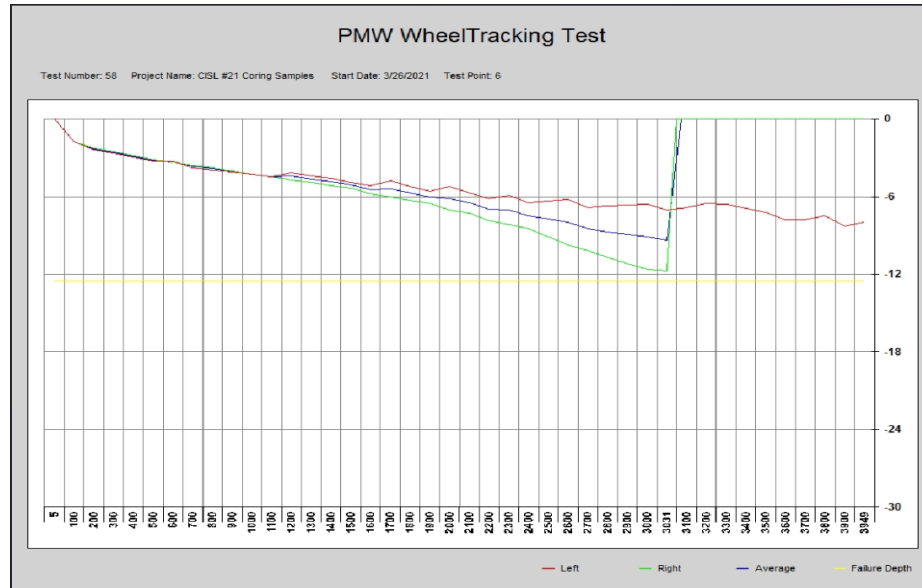


Figure 6.3 HWTD Test Output for In-Situ Cored Specimen

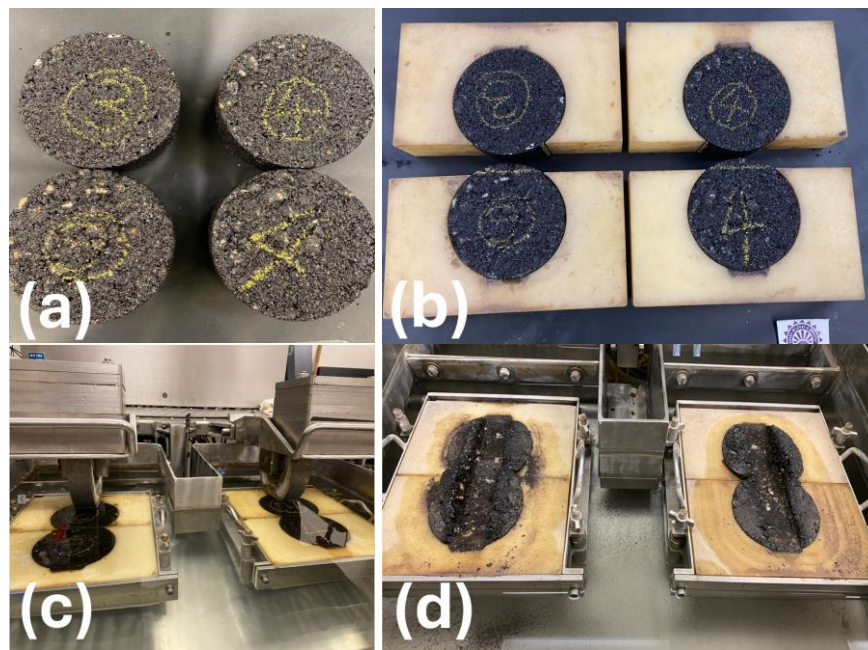


Figure 6.4 (a) SFDR Cored Specimens; (b) HWTD SFDR Specimens; (c) HWTD Tester; (d) HWTD Tested SFDR Specimens

Table 6.5 HWTD Test Results

Sample ID	Test Number	Wheel Passes	Average
Lab Compacted	1	12,671	10,999
	2	10,140	
	3	11,588	
	4	9,598	
In-situ Cores	5		3,949

6.2.2.2 Dynamic Modulus Test

The dynamic modulus test is a standard method for evaluating the stiffness of HMA mixtures. The dynamic modulus ($|E^*|$) value determined from the test is typically used to analyze the permanent deformation and fatigue cracking of HMA pavement. It is also an essential input for the mechanistic-empirical pavement design software PMED (AASHTO, 2015). Higher $|E^*|$ values indicate greater resistance to pavement rutting.

In this study, three SGC-compact samples (5.9-in. diameter and 6.69-in. height) were compacted and subjected to bulk-specific gravity tests to evaluate air voids. The desired samples were then cored and trimmed to the dynamic modulus test specimens, each with a 3.9-in. diameter and height of 5.9 in. (Figure 6.5(a)). An Industrial Process Control (IPC) Asphalt Mixture Performance Tester (AMPT) tester was used to conduct the test at the reference temperature of 68°F according to AASHTO TP62-07 (Figure 6.5(b)).

The specimens were tested in an environmentally controlled chamber at three temperatures (40 °F, 70 °F, and 100 °F) and six frequencies (25, 10, 5, 1, 0.5, and 0.1 Hz) (Figure 6.5(c)). An E^* master curve plot was interpolated by shifting data horizontally using the time-temperature superposition principle (reduced frequency). Figure 6.6 illustrates the computed E^* master curve, with the solid line representing the average of the three HMA mixture specimens and the dashed line showing each specimen. The coefficient of variation (COV) ranged from 5% to 9%.



Figure 6.5 Dynamic Modulus of HMA Mixtures

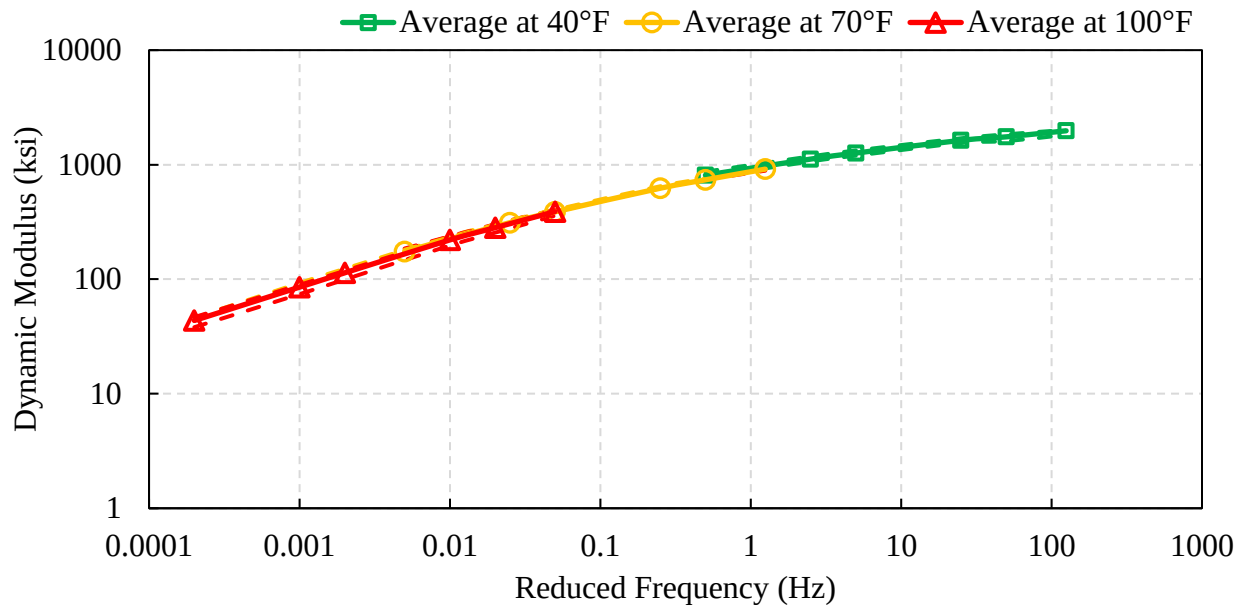


Figure 6.6 Dynamic Modulus Master Curve for HMA Material

6.3 Full-Scale APT Test Results

As discussed earlier, the full-scale test was conducted in 11 cycles, each cycle consisting of various wheel load repetitions. A total of 600,000-wheel load repetitions were applied to the sections. This section discusses the full-scale test results, FWD test results, pavement surface rutting, strain at the bottom of the AC layer, stress on top of the subgrade layer, and the effects of temperature and moisture on stress and strain levels.

6.3.1 FWD Test Results

The FWD tests were conducted by KDOT and repeated three times at various test stages of the APT testing program: after construction but before any traffic loading, midway through the APT at 250,000-wheel load repetitions, and finally at the end of the APT cycle at 500,000 repetitions. The measured FWD deflection data were imported into the EVERCALC back-calculation program for the back-calculation of layer moduli. The EVERCALC program was developed by the Washington State Department of Transportation (WSDOT). The average

calculated moduli for the test sections from the FWD tests in two directions are summarized in Table 6.6. The results indicated that the moduli of the HMA and SFDR base layers increased with wheel-load repetitions. The HMA layer moduli increased due to pavement densification, while the base moduli decreased, possibly due to microcracking. It was noted that the back-calculated SFDR base modulus was much higher than the stress-strain interpolated modulus (Table 6.4). As expected, the subgrade moduli remained unchanged.

Table 6.6 Back-Calculated Layer Moduli (ksi)

Test Section (AC Thickness)	Wheel-Load Repetition	0			250k			600k		
	SFDR	HMA	SFDR Base	Subgrade	HMA	SFDR Base	Subgrade	HMA	SFDR Base	Subgrade
Middle (4.5 in.)	5C&5FA	672	1813	5	850	1114	6	850	1500	7
	5C	699	1861	6	850	1565	6	850	1446	6
South (3 in.)	5C&5FA	857	941	5	725	1166	5	850	1373	5
	5C	809	977	7	850	1532	6	850	1383	5

6.3.2 Rutting on the AC Pavement Surface

Figure 6.7 illustrates the final HMA deformation in the four subsections after 600,000 APT load repetitions. In these plots, the black line represents the initial rut depth measurement, the green line corresponds to the SFDR section with 5% cement (250 psi) section, while the red line represents the SFDR section with higher strength of SFDR (5% cement & 5% fly ash). The latter section had higher rutting than the lower strength SFDR (5% cement). A “W” shape area under the initial transverse profile line was defined as the rutting area after 600,000 APT load repetitions; the rutting area was determined using AutoCAD at a 1:1 scale from the transverse rutting profile. The summarized rut depths and rutting area after 600,000 load repetitions are tabulated in Table 6.7. The results indicated that the pavement section with a base layer stabilized by cement and fly ash had relatively higher rut depths and rutting areas.

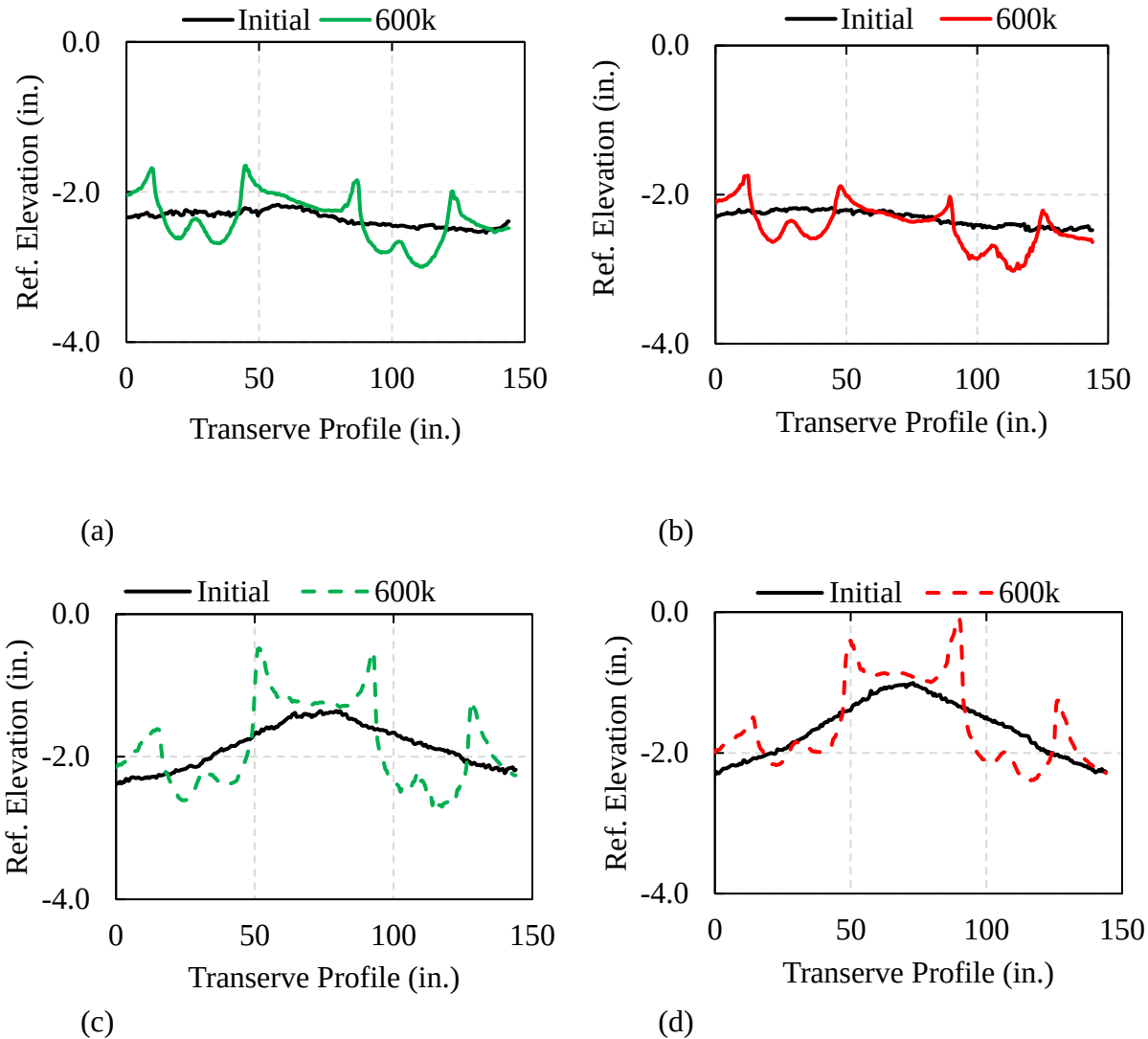


Figure 6.7 HMA Deformation after 600,000-wheel load repetitions: (a) 4.5-in. HMA (5C); (b) 4.5-in. HMA (5C&5FA); (c) 3-in. HMA (5C); (d) 3-in. HMA (5C&5FA)

Table 6.7 Average rut depth and rutting bowl area after 600,000 load repetitions

Test Section (AC Thickness)	SFDR Mixture	Average Rut Depth (in.)	Rutting Bowl Area (in. ²)
Middle (4.5 in.)	5C&5FA	0.47	21.73
	5C	0.41	19.03
South (3 in.)	5C&5FA	0.44	20.18
	5C	0.33	18.89

Figure 6.8 summarizes the periodic rut measurements from the test sections. The sections with higher strength SFDR (5% cement & 5% fly ash) showed higher rutting than those with lower strength SFDR (5% cement) throughout the test. The section with 4.5-in. AC layer with a 10-in SFDR base (5C&5FA) had higher rutting, and the rut depth reached approximately 0.45-in. rut depth after 600,000 load repetitions. The section with 3-in. AC layer and 12-in. SFDR (5% cement) experienced the least rutting - approximately 0.25 in. rut depth after 600,000 load repetitions. The section has a 3-in AC layer, 10-in. SFDR base, and 5% cement exhibited the best rutting performance.

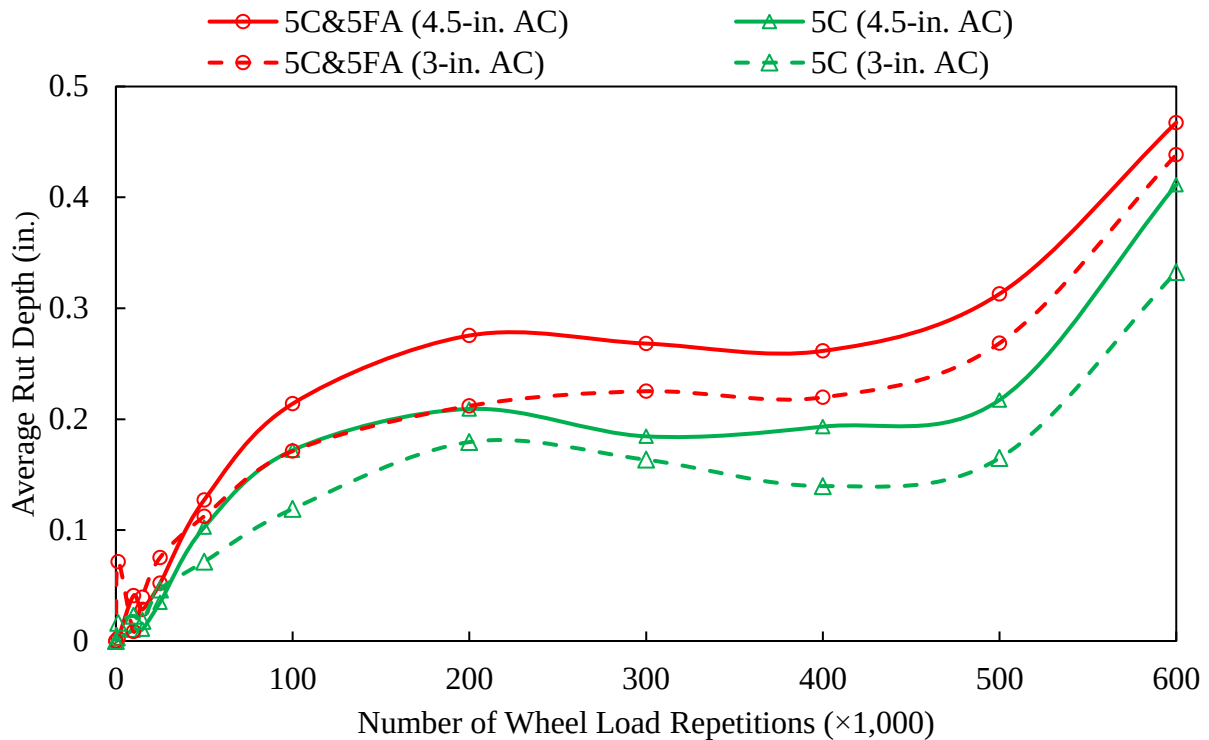


Figure 6.8 Rutting Comparison

A one-way Analysis of Variance (ANOVA) analysis was done to investigate whether AC overlay thickness and SFDR base strength significantly affect the measured average rut depth during APT testing. The results are summarized in Table 6.8, with p-values of 0.000 for AC overlay thickness and 0.002 for base strength. Both factors have statistically significant effects on

the measured rut depth. The low p-values suggest that both AC thickness and SFDR base strength contributed to variations in rut depth, with the AC overlay thickness showing a slightly more substantial effect due to its lower p-value. Similar statistical analysis results were found for the rutting bowl area, as listed in Table 6.9. Conclusions were the same as the rut depths.

Table 6.8 ANOVA Analysis of Rutting

Test Section	SFDR Mixture	HMA Thickness (in.)	Rutting (in.)	ANOVA PR(>F)	SFDR Strength (psi)	Rutting (in.)	ANOVA PR(>F)
Middle	5C&5FA	4.5	0.47	0.000	473	0.47	0.002
	5C	4.5	0.41		244	0.41	
South	5C&5FA	3	0.44		473	0.44	
	5C	3	0.33		244	0.33	

Table 6.9 ANOVA Analysis of Rutting Bowl Area

Test Section	SFDR Mixture	HMA Thickness (in.)	Rutting Bowl Area (in. ²)	ANOVA PR(>F)	SFDR Strength (psi)	Rutting Bowl Area (in. ²)	ANOVA PR(>F)
Middle	5C&5FA	4.5	21.73	0.000	473	21.73	0.002
	5C	4.5	19.03		244	19.03	
South	5C&5FA	3	20.18		473	20.18	
	5C	3	18.89		244	18.89	

According to the MEPDG Guide (AASHTO, 2015), rutting on asphalt pavements is caused by plastic or permanent vertical deformation in the HMA, unbound layers, and foundation soil. In the PMED software, plastic deformation is estimated for each subsection at the mid-depth of each sublayer within the pavement structure and then summed, as shown by:

$$\Delta_{p(HMA)} = \epsilon_{p(HMA)} h_{HMA} = \beta_{1r} k_z \epsilon_{r(HMA)} 10^{k_{1r} n^{k_{2r}} \beta_{2r} T^{k_{3r}} \beta_{3r}} \quad \text{Equation 6.1}$$

where:

$\Delta_{p(HMA)}$ = accumulated permanent or plastic vertical deformation in the HMA layer (in.),

$\epsilon_{p(HMA)}$ = accumulated permanent or plastic strain in the HMA layer/sublayer (in./in.),

ϵ_r (HMA) = resilient or elastic strain calculated by the structural response model at the mid-depth of each HMA layer (in.),

n = number of axle-load repetitions,

T = mix or pavement temperature ($^{\circ}\text{F}$),

k_z = depth confinement factor,

k_{1r} , k_{2r} , k_{3r} = global calibration parameters, and

β_{1r} , β_{2r} , β_{3r} = local calibration coefficients (default value: 1.0).

In this study, the back-calculated layer moduli (as shown in Table 6.6) were used to compute the resilient (or elastic strain) in the mid-depth of the HMA layer for all test sections at CISL. These calculations were performed using a 9,000-lb load and a tire pressure of 90 psi. The computed results are summarized in Table 6.10. The results show that both sections with 3-in. AC layers had much lower elastic strain than the 4.5-in. AC sections, which led to lower rutting on these sections.

Table 6.10 Resilient (or Elastic) Strain at Various Locations

4.5-in. AC (5C)			
Wheel Passes	0	250K	600K
Depth (in.)	ϵ_r^*	ϵ_r^*	ϵ_r^*
2.25	75.3	57.8	57.1
4.499	61.6	51.5	52.0
14.501	92.4	97.6	101.7
3-in. AC (5C)			
Wheel Passes	0	250K	600K
Depth (in.)	ϵ_r^*	ϵ_r^*	ϵ_r^*
1.50	50.7	51.7	50.0
2.999	61.9	55.6	55.6
15.001	113.2	89.9	100.5

4.5-in. AC (5C&5FA)			
Wheel Passes	0	250K	600K
Depth (in.)	ϵ_r^*	ϵ_r^*	ϵ_r^*
2.25	78.0	54.8	58
4.499	63.7	54.5	52.1
14.501	99.5	116.7	95.6
3-in. AC (5C&5FA)			
Wheel Passes	0	250K	600K
Depth (in.)	ϵ_r^*	ϵ_r^*	ϵ_r^*
1.50	45.4	49.1	49.9
2.999	58.4	55.7	55.6
15.001	125.5	103.4	100.9

*microstrain

6.3.3 AC Layer Strain

Four H-bar strain gauges were installed at each subsection to measure the longitudinal and transverse strains at the bottom of the AC/HMA layer. The average strain values were mean values from two gauge data sets in the wheel paths. However, two transverse strain gauges lost signal during paving in the middle of the test subsections. As a result, the transverse strain for these two middle subsections was based on individual transverse strain gauge data rather than an average value.

Lower strain levels at the bottom of the HMA layer indicate that the HMA pavement section is less prone to fatigue. Huang (1994) stated that the fatigue cracking in flexible pavements typically initiates at the bottom of the asphalt concrete surface layer and propagates upward (bottom-up cracking). The tensile strain at the bottom of the HMA layer significantly influences this process. As discussed earlier, the APT full-scale test programmed a lateral wander during the test, which was used to simulate the varying wheel paths of actual vehicles on roadways. In this study, the APT was set to a wander of ± 6 in. and applied in a truncated normal distribution with 1,352-wheel load repetitions to finish one complete wander cycle. The detailed wander distribution is shown in Figure 5.2.

During the APT test, approximately 13,500 strain/stress data points were recorded from each sensor for one wander cycle. Two transverse H-bar strain gauges failed during the AC layer construction, so 14 H-bars were used to collect strain data from the sections. Romanoschi et al. (2004) summarized the mathematical computation of the strain level using six types of strain signal shapes, as shown in Figure 6.9. Type 1 through 3 were typically applied to the single axle with the single wheel, while type 4 and type 6 were applied to the single axle with the dual wheel.

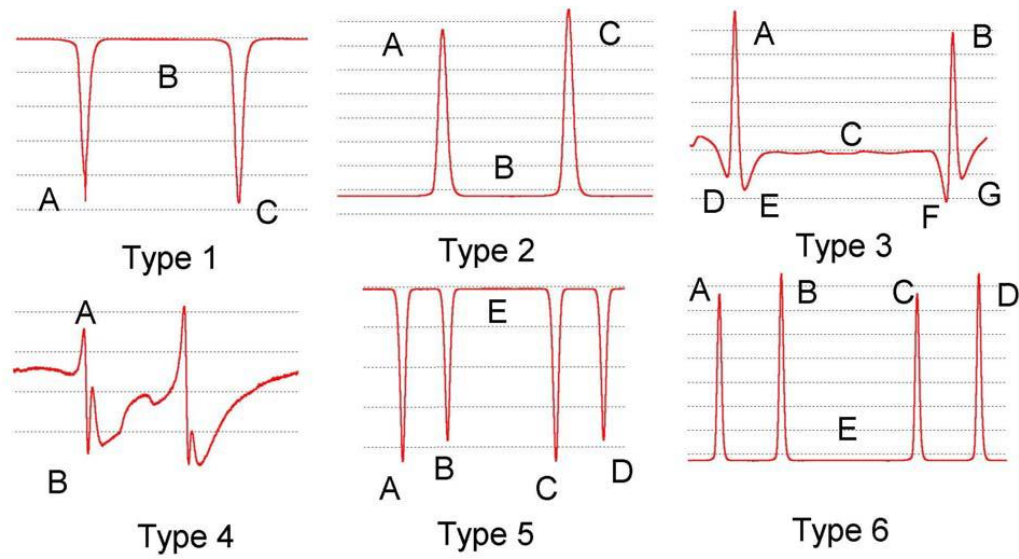


Figure 6.9 Strain Gauge Signal Shapes (Romanoschiet al., 2004)

Figure 6.10 shows the test sections' transverse and longitudinal strain signal shapes. This study utilized types 4, 5, and 6 to compute the strain. Signal type 4 was calculated by averaging the maximum and minimum strains in one reading cycle. Types 5 and 6 matched the longitudinal strain signals, and the strain values were computed using Equation 6.2.

$$S = \frac{A+B+C+D}{4} - E \quad \text{Equation 6.2}$$

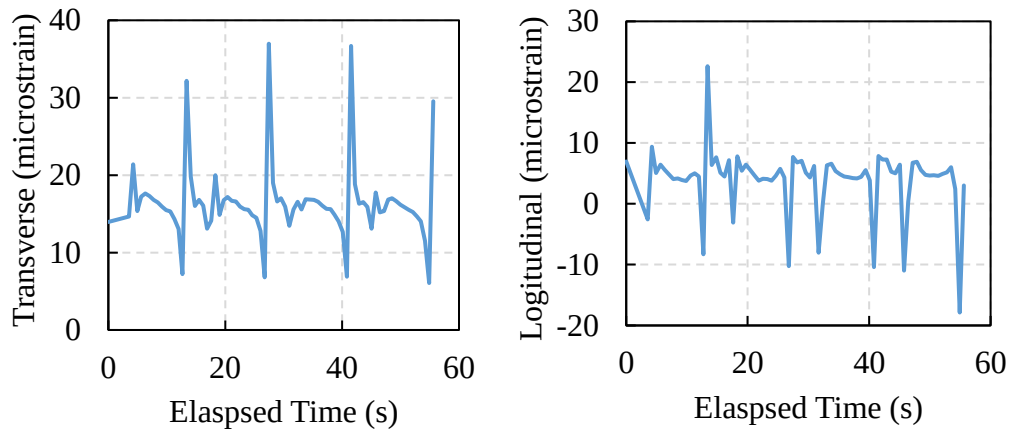


Figure 6.10 Measured Strain Signal Shapes

As mentioned in the APT program test, 600,000 load repetitions were applied to each test section, and a total of 10 strain-measured cycles were conducted at 1,352, 10,000, 15,000, 25,000,

50,000, 100,000, 200,000, 200,000, 300,000, 400,000, and 500,000 load repetitions. After the initial data collection session, the sensor data acquisition was reprogrammed and reinitialized before applying a new APT loading round. The collected data were filtered to remove any outliers, and the average of the top 300 strain data points was computed for each measurement cycle to ensure accuracy.

The summarized results of the strain levels at the bottom of the HMA layer are presented in Figure 6.11 and Figure 6.12. The average transverse strains (Figure 6.11) were higher than the longitudinal strains (Figure 6.12). The sections with 3-in. HMA overlay and 12-in. SFDR layer with 5% cement & 5% fly ash demonstrated stable transverse strain behavior, performing consistently at lower strain levels after 200,000 wheel-load repetitions. Additionally, the section with 4.5-in. HMA overlay and 10-in. SFDR layer with 5% cement & 5% fly ash exhibited a lower average strain level throughout the entire test. The initial strain values were comparable to the computed values in Table 6.10. Overall, the sections containing 5% cement & 5% fly ash stabilized base had low tensile strain throughout the test. This indicates that these two sections showed favorable long-term pavement performance in terms of HMA fatigue.

SFDR pavements are expected to last longer or function as a de facto perpetual pavement for low-volume roads. These pavements consist of a thinner structure compared to thick granular base courses, significantly reducing the potential for fatigue cracking by minimizing tensile strains at the bottom of the asphalt layer. A previous study by Brown & Cross (1992) suggested that cost-effective mill-and-fill rehabilitation is appropriate when rutting in thicker pavements is confined to the top two inches of the pavement structure. Monismith and Long (1999) also recommended that the tensile strain should not exceed 60 microstrains at the bottom of the asphalt layer, and the

vertical strain should be limited to 200 microstrains at the top of the subgrade for any perpetual pavements.

In this study, as shown in Figure 6.11, the transverse strains at the bottom of the SFDR pavements with HMA overlays and SFDR bases with 5% cement & 5% fly ash were below the 60-micron threshold, which indicates that these pavement sections had better resistance to fatigue cracking and would have longer service life.

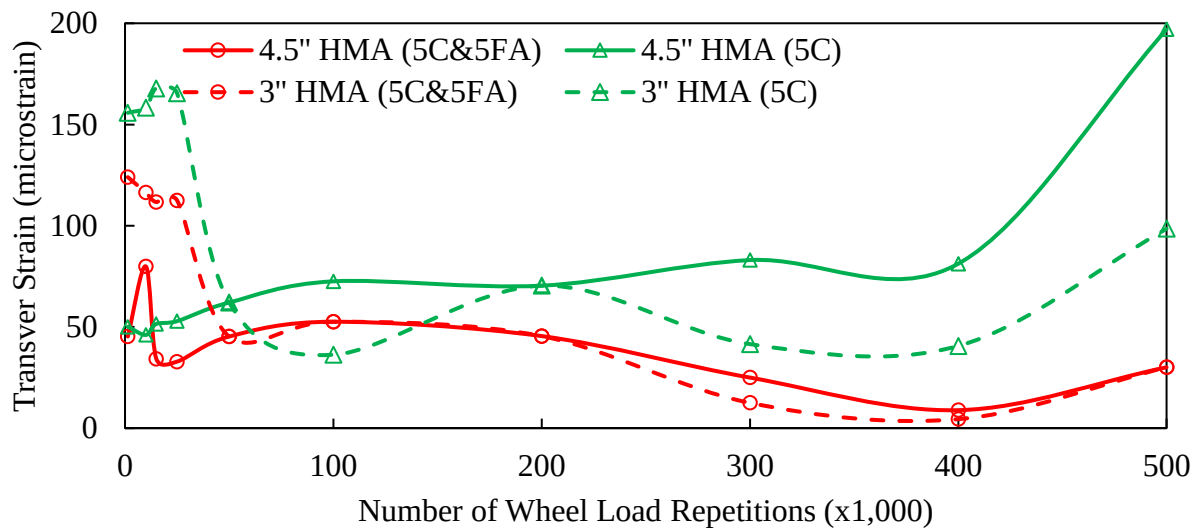


Figure 6.11 Average Transverse Strain Levels

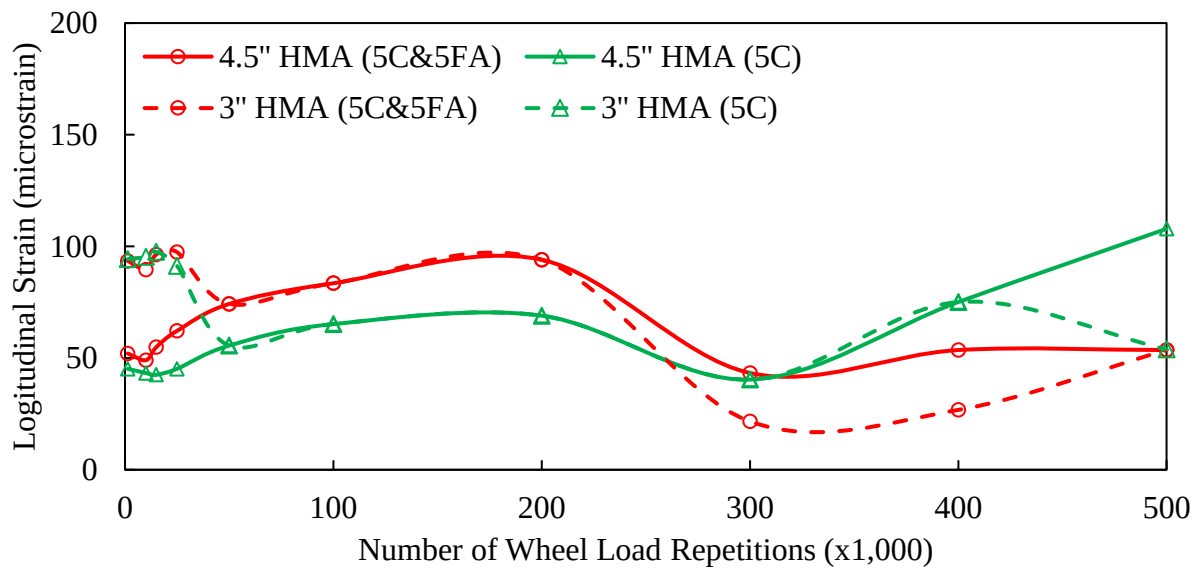


Figure 6.12 Average Longitudinal Strain Levels

A one-way ANOVA analysis was done to investigate whether HMA overlay thickness and SFDR base strength significantly affect the measured average transverse and longitudinal strain during APT testing. The ANOVA analysis results of the average transverse are summarized in Table 6.11. The results show that SFDR strength is a significant factor influencing transverse strain. The HMA thickness has an insignificant effect on the transverse strain since the p-value for the HMA thickness is 0.071 (>0.05).

Table 6.11 ANOVA Analysis of Transverse Strain

Test Section	SFDR Mixture	HMA Thickness (in.)	Transverse Strain (microstrains)	ANOVA PR(>F)	SFDR Strength (psi)	Transverse Strain (microstrains)	ANOVA PR(>F)
Middle	5C&5F A	4.5	30.06	0.071	473	30.06	0.013
	5C	4.5	196.96		244	196.96	
South	5C&5F A	3	33.00		473	33.00	
	5C	3	98.48		244	98.48	

The ANOVA analysis results of average longitudinal strains are summarized in Table 6.12. The p-value for the HMA thickness is 0.003, which indicates that HMA thickness has a statistically significant effect on longitudinal strain. The p-value for the SFDR base strength is 0.005, which indicates that SFDR strength also has a statistically significant effect on the longitudinal strain.

Table 6.12 ANOVA Analysis of Longitudinal Strain

Test Section	SFDR Mixture	HMA Thickness (in.)	Longitudinal Strain (microstrains)	ANOVA PR(>F)	SFDR Strength (psi)	Longitudinal Strain (microstrains)	ANOVA PR(>F)
Middle	5C&5FA	4.5	53.60	0.003	473	53.60	0.005
	5C	4.5	107.89		244	107.89	
South	5C&5FA	3	55.60		473	55.60	
	5C	3	53.94		244	53.94	

6.3.4 Subgrade Vertical Stress

In this study, two Geokon pressure cells were installed on top of the subgrade at each subsection and positioned within each wheel path lane to monitor the distribution of the vertical compressive stress. The top 300 pressure readings were collected from the first complete wander cycle (1,352 repetitions) for each test cycle data collection round. The average stress values were computed, and the results are plotted in Figure 6.13.

The results indicate that the compressive stress within the subgrade layer increased as the number of APT wheel load repetitions increased. The section containing the SFDR with 5% cement had a higher compressive stress increase than the section with SFDR stabilized with 5% cement & 5% fly ash.

A rapid increase in stress was observed at the beginning of the 200,000-wheel load repetition cycle due to the densification of the HMA overlay and the possible weakening of the SFDR base layer. The sections with 5% cement & 5% fly ash consistently showed a lower vertical compressive stress on the subgrade for both 3-in. and 4.5-in. HMA overlays. These two sections were better able to distribute wheel loads and reduce stress concentrations, resulting in better rutting resistance, as lower stress levels on the subgrade indicate less pavement deformation under repeated loading. As shown in Table 6.10, the vertical compressive strains on the top of the subgrade for these sections were well below 200 microns, indicating that they can be classified as perpetual pavement sections.

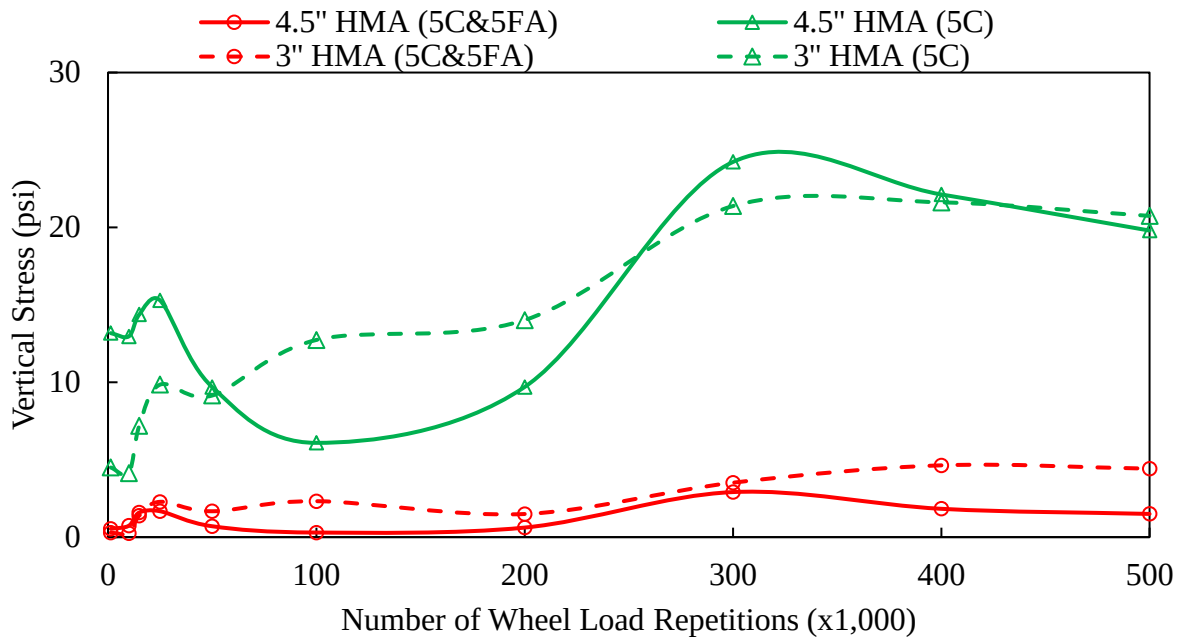


Figure 6.13 Average Vertical Stress at the Top of the Subgrade

A one-way ANOVA analysis investigated whether HMA overlay thickness and SFDR base strength significantly affect the measured average vertical stress on top of the subgrade. The ANOVA analysis results of the average subgrade stresses are summarized in Table 6.13. The p-value for HMA thickness is 0.171 (>0.05). Thus, the HMA thickness does not have a statistically significant effect on subgrade stress. For the SFDR base strength, the p-value is 0.002 (<0.05). Thus, only SFDR base strength has a statistically significant effect on subgrade stress.

Table 6.13 ANOVA Analysis of Subgrade Vertical Stress

Test Section	SFDR Mixture	HMA Thickness (in.)	Stress (psi)	ANOVA PR(>F)	SFDR Strength (psi)	Stress (psi)	ANOVA PR(>F)
Middle	5C&5FA	4.5	1.50	0.171	473	1.50	0.002
	5C	4.5	19.80		244	19.80	
South	5C&5FA	3	4.42		473	4.42	
	5C	3	20.74		244	20.74	

6.3.5 Temperature and Moisture Effects on Stress and Strain Level

Four thermocouples were installed in the center of each test section to monitor temperature on top of the SFDR base layer (Figure 5.22). One thermocouple was connected to a portable thermometer for data verification during APT testing, while the remaining thermocouples recorded temperature on top of the SFDR base layer. After filtering outliers, the average of the top 300 temperature data points from the first complete wander cycle (1,352 repetitions) of each test round was collected, and the average temperature values were computed and plotted in Figure 6.14.

The results indicate a similar trend for the average strain levels and temperature. However, the strain level increased rapidly after 300,000 wheel-load repetitions, while the associated SFDR surface temperature showed only a slight change. This suggests that temperature had a minimal effect on the strain levels at the SFDR base surface.

As confirmed in previous CISL studies, moisture in the subgrade layer remained constant during the entire full-scale test. There were no significant changes during the testing period because the test sections were paved wall-to-wall; the whole testing process occurred indoor. Supporting evidence with a similar study's time domain reflectometry (TDR) testing confirmed this observation (Saghebfar, 2014).

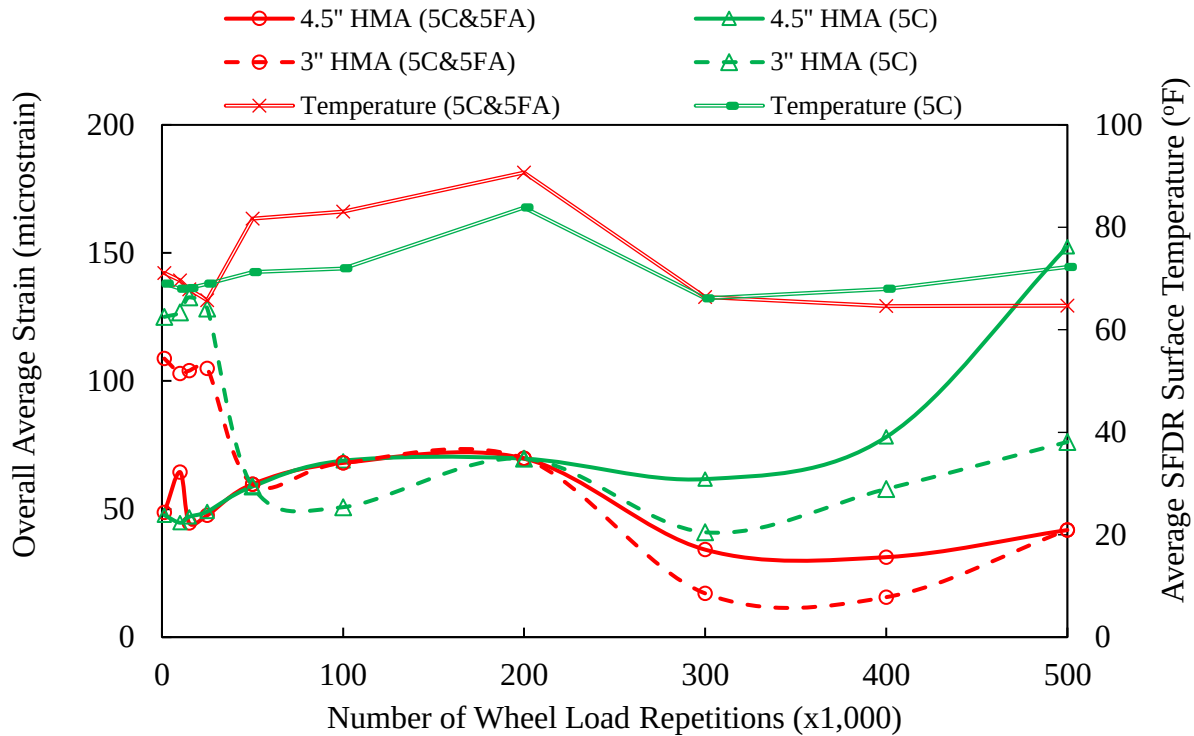


Figure 6.14 Average Strain and Related Temperature

6.4 SFDR Pavement Performance Modeling and Verification

As discussed in Section 3.4, the thicknesses of the overlaying AC and SFDR base layers were selected in the AASHTOWare PMED analysis and used in the APT experiment. During PMED software analysis, the optimum thickness of the new AC layer was the minimum required thickness that would satisfy all performance criteria with defined reliability. If one or more performance criteria were not satisfied, the pavement structure was altered by changing the thickness of the AC overlay layer. The PMED software version 2.5 was used when the APT pavement sections were designed. Later, a new version (2.6.2) of this software was released in 2022 and was used to verify the full-scale APT test results.

The AASHTO MEPDG manual defines FDR as in-place pulverizing conventional flexible pavements and a cold recycling method (AASHTO 2015). The design of such a pavement was not included in the AASHTOWare Pavement Mechanistic-Empirical Design (PMED) software before

2022 (ver. 6.2.2). Cold in-place recycling as a rehabilitation strategy is now considered reconstruction under the AASHTOWare PMED design analysis process and defined as a new flexible pavement. However, FDR was not included in the global calibration of PMED (AASHTO 2020).

All four pavement structures were used for pavement response verification using version 2.6.2 of the PMED software. As discussed in Section 5.2, four designed pavement structures were used - 4.5 in. AC and 10-in SFDR with 5% cement, 4.5-in. AC and 12-in SFDR with 5% cement & 5% fly ash, 3-in. AC and 12-in SFDR with 5% cement, and 3-in. AC and 12-in SFDR with 5% cement & 5% fly ash. Both versions of the PEMD software have the same program steps and similar property parameter inputs. The analysis procedure with version 2.5.0 was described in Section 3.4. The updated performance criteria and reliability for version 2.6.2 verification are summarized in Table 6.14. An initial IRI of 30 in./mile was used, and reliability levels were selected based on minor arterial functional classification, as suggested by KDOT.

Table 6.14 Pavement Performance Criteria and Reliability

No.	Distress Type	Criteria/Default	
		Distress Target	Reliability Level (%)
1	Terminal IRI (in./mile)	200	65
2	AC bottom-up fatigue cracking (% lane area)	30*	75
3	AC top-down fatigue cracking (% lane area)	25*	75
4	AC thermal cracking (ft/mile)	750	65
5	Permanent deformation – total pavement (in.)	0.65	75
6	Permanent deformation - AC only (in.)	0.55	75

***PMED Default**

Figure 6.15 shows the difference between the PMED-predicted rutting and measured rut depths after 600,000 wheel-load repetitions. The results showed that the PMED predictions were consistently lower than the rut depths measured from the full-scale APT testing. However, except

for two sections with cement and fly ash, predicted rut depths were below the projected failure rut depth of 0.55 in. at approximately 648,000 ESALs.

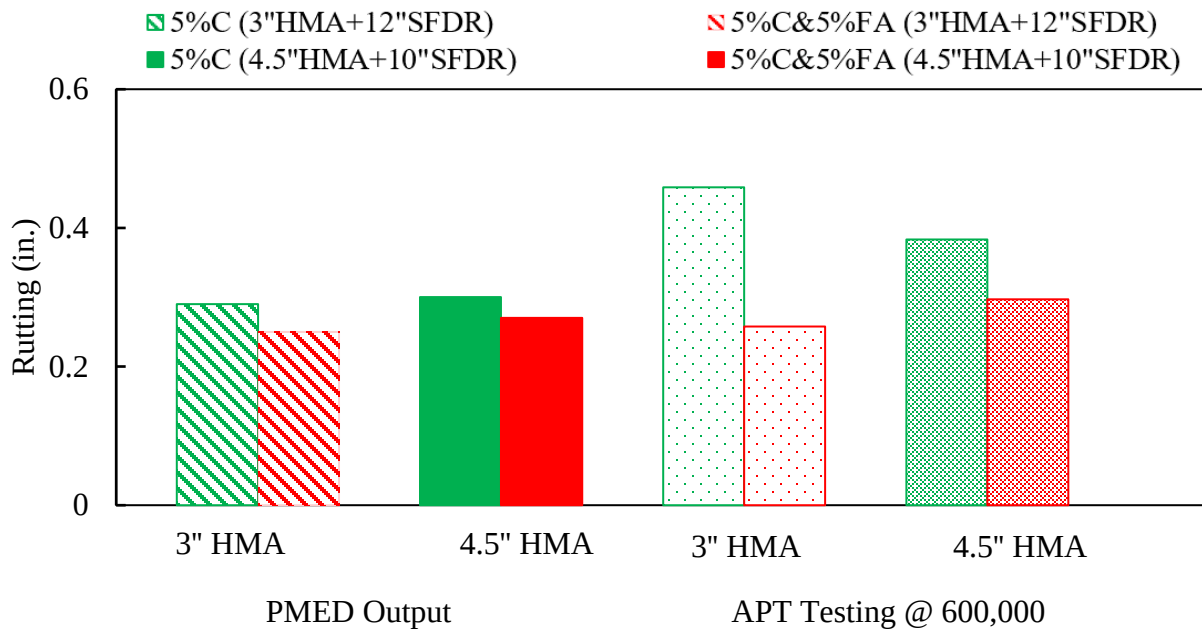


Figure 6.15 Rutting on the Test Sections

In the AASHTOWare PMED software, a standard deviation of 10 in. is set as the default value for the traffic wander for lateral offset from the lane center where the wheel loads are applied. The meaning of the outer edge of the wheel load is located 18 inches from the pavement lane marking, as shown in Figure 6.16. (AASHTO, 2015). Theoretically, applying the traffic wander spreads the traffic load over a wider area on the pavement, which can reduce the stress concentration in the pavement structure, increasing pavement life by reducing the rutting and fatigue cracking in the PMED analysis. The higher rutting measurements AT CISL sections happened due to more load channelization from the APT wander of ± 6 in.

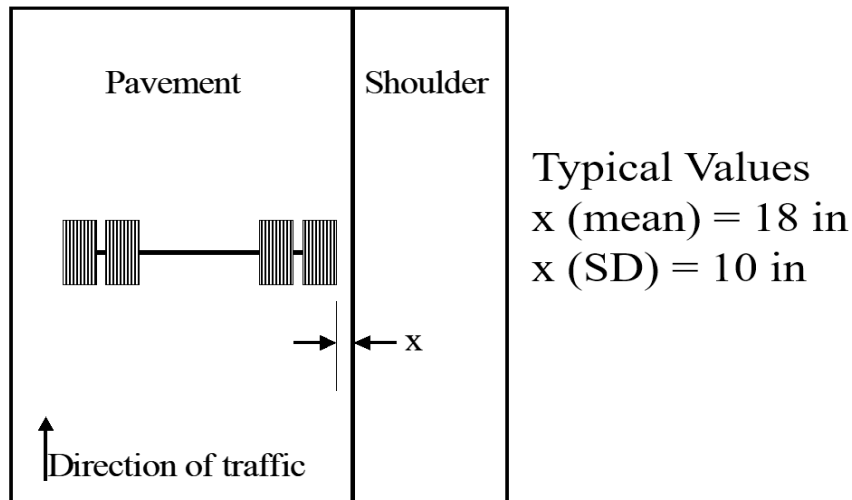


Figure 6.16 PMED Default Traffic Wander

Figure 6.17 summarizes the top-down cracking for all SFDR pavement sections. The test results indicated that the 4.5-in. AC overlay sections had less top-down cracking than the 3-in. AC sections, regardless of stabilizer type in the SFDR base. However, all sections exceeded the failure criteria of 3.2% of the lane area (converted from PMED version 2.5 criterion of 2,000 ft/mile).

In the PMED software (after version 2.5), a fracture mechanics-based cracking model was developed under NCHRP project 1-52 and added (AASHTO, 2021). The top-down cracking model from NCHRP 1-52 replaced the older bending beam-based model in the earlier version of the PMED software. In addition, longitudinal and alligator cracks in the wheel paths were confirmed using cores to initiate at the surface and propagate down through the asphalt layers. The top-down cracking transfer function was modified to include longitudinal and alligator cracks in the wheel paths regarding the total lane area cracked percentage. In version PMED 2.6.2, the default criterion for top-down cracking is 25%. This study observed no cracking on sections during APT up to 600,000 ESALs. Thus, the default criterion for top-down cracking is recommended for the design of new AC pavements with FDR.

Results from the AASHTOWare PMED software show that the sections with stabilized FDR mixtures with cement had lower IRI than those with cement and fly ash-stabilized bases (Figure 6.18). However, all sections had much lower IRI than the failure criteria (200 in./mile).

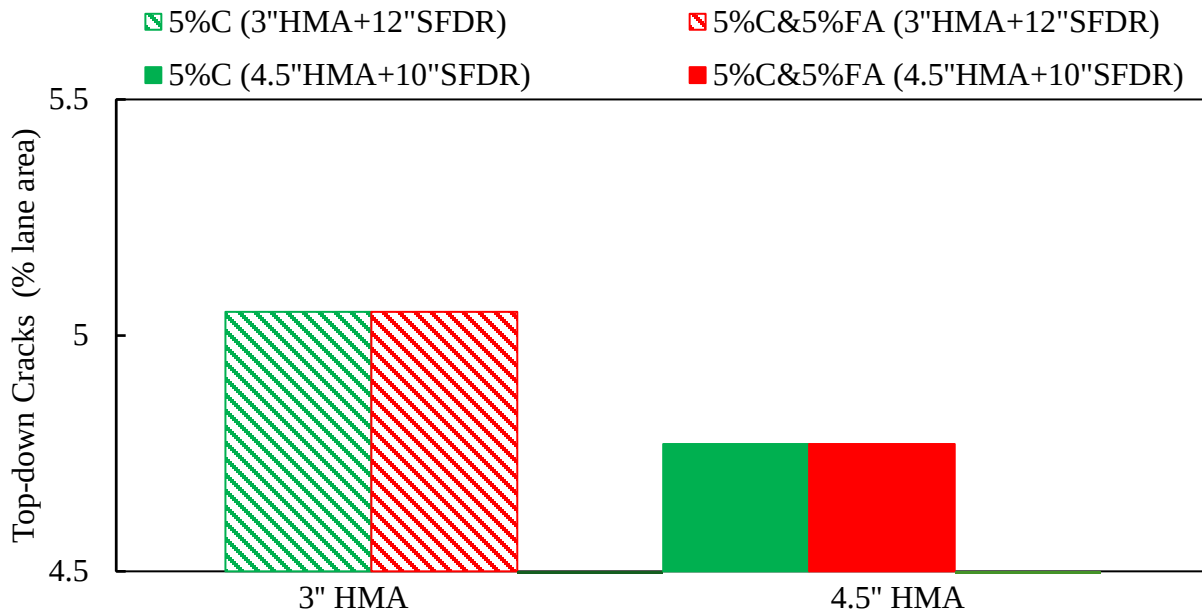


Figure 6.17 Predicted Top-Down Cracking on Test Sections

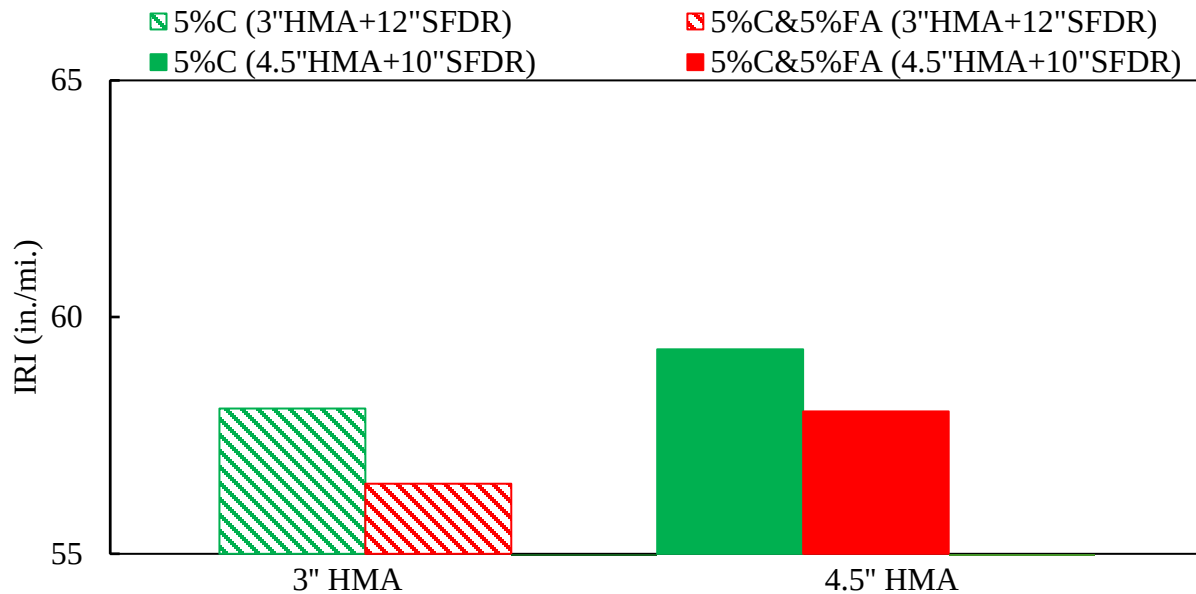


Figure 6.18 Predicted IRI on the Test Sections

6.5 AC Overlay Design

The 1993 AASHTO Design Guide is commonly used to determine the required thickness of AC overlays (AASHTO, 1993). This guide uses a structural number (SN) to represent the overall pavement structure necessary to support the anticipated traffic load throughout the designated pavement service life. The required SN for the designed pavement was computed using Equation 6.3.

$$\log(W_{18}) = Z_R \times S_o + 9.36 \times \log(SN + 1) - 0.2 + \frac{\log\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log(M_R) - 8.07$$

Equation 6.3

where:

W_{18} = predicted number of 18-kip Equivalent Single Axle Loads (ESAL) applications,

Z_R = standard normal deviate,

S_o = combined standard error of the traffic prediction and performance prediction,

SN = structure number,

ΔPSI = difference between the initial design serviceability index(p_i), and the design terminal serviceability index (p_t), and

M_R = subgrade resilient modulus.

In this study, the PMED software predicted 600,000 (ESALs) for the SFDR pavement sections at CISL. This traffic was used to compute the SN of the designed flexible pavement. A reliability (R) of 50% was assumed for low-volume road classification. The initial serviceability index and the terminal serviceability index were set at 4.2 and 2.0, respectively. The M_R was computed using Equation 6.4. The provided SN was calculated using Equation 6.5.

$$M_R = 2555 \times CBR^{0.64}$$

Equation 6.4

where:

CBR = California bearing ratio (%).

$$SN = a_1 D_1 + a_2 D_2 m_2 \quad \text{Equation 6.5}$$

where:

a_1, a_2 = structural layer coefficients of the surface and base layers,

D_1 = thickness of HMA surface course (inch),

D_2 = thickness of base course (inch), and

m_2 = drainage coefficient of base course.

The structural layer coefficient of the HMA layer (a_1) was set to 0.42 per KDOT practices. The structural layer coefficients of the SFDR base layer were obtained from the nomograph, Figure 6.19, of the 1993 AASHTO Pavement Design Guide. The UCS values of the SFDR bases were used. The drainage coefficient of the base course (m_2) was assumed to be 1.0. Table 6.15 tabulates the base layer coefficient and subgrade resilient modulus of the for each subsection.

Table 6.15 Summary of Base Layer Coefficients and Subgrade Resilient Modulus

Test Section (AC Thickness)	SFDR Mixture	UCS (psi)	a_2	M_R (psi)
Middle (4.5 in.)	5C&5FA 5C	473 244	0.166 0.132	8,077 8,432
South (3 in.)	5C&5FA 5C	473 244	0.166 0.132	8,289 8,171

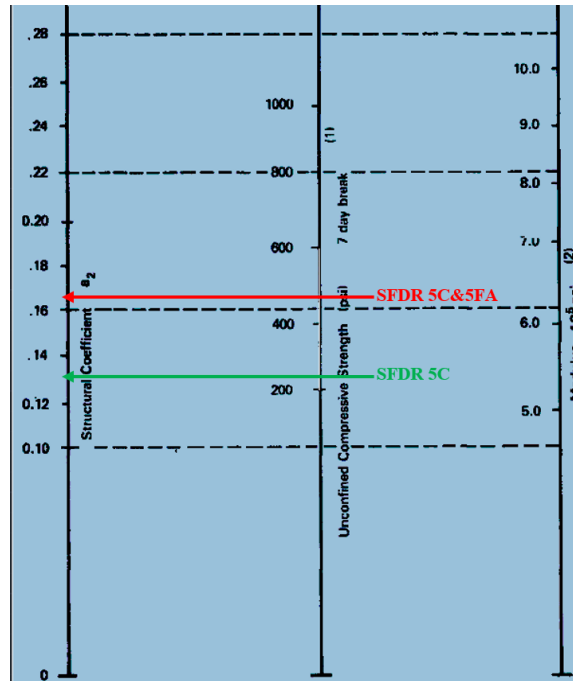


Figure 6.19 Cement-Treated Base Layer Coefficient Interpolation (after AASHTO,1993)

The required SN and provided SN were computed and summarized in Table 6.16. The pavement thickness is considered adequate when the required SN is less than the provided SN. All designed pavements at CISL were adequate for the predicted 600,000 ESALs. This shows that the 1993 AASHTO Design Guide can be used for SFDR flexible pavements. The SFDR base structural layer coefficient, in that case, should be obtained from the nomograph in the 1993 AASHTO Pavement Design Guide based on the unconfined compressive strength. This value should not be taken as 0.2, as ARRA (2015) suggested.

Table 6.16 Summary of Required and Provided SN

Test Section	SFDR Mixture	AC Thickness (in.)	SFDR Base Thickness (in.)	Required SN	Provided SN	Remark
Middle	5C&5FA	4.5	10	2.44	3.55	Adequate
	5C	4.5	10	2.40	3.21	Adequate
South	5C&5FA	3	12	2.42	3.25	Adequate
	5C	3	12	2.43	2.84	Adequate

Chapter 7 - Finite Element Modeling (FEM)

7.1 Introduction

As discussed earlier in the research objective, a FEM simulation was done using a commercial 3-Dimensional Finite Element (FE) software, ABAQUS software, to simulate APT and analyze the full-scale test subsections. The simulation results investigated the pavement responses (deformation, strains, and stresses) under repeated or constant wheel load. The FE model was developed to check the feasibility of a thin AC overlay design over an SFDR base layer. The APT test results were used to check those from the FEM analysis. This chapter discussed the process of developing the FEM using ABAQUS, including model geometry, material characterization, meshing, boundary and loading conditions, and analysis. The ABAQUS software had no pre-defined standard unit system before creating any model. Table 7.1 lists the conventional US units used in ABAQUS for this study.

Table 7.1 Unit System in ABAQUS

Quantity	US Unit
Length	In
Force	lbf
Mass	$\text{lbf s}^2/\text{in}$
Time	S
Stress	$\text{psi (lbf/in}^2\text{)}$
Energy	in lbf
Density	$\text{lbf s}^2/\text{in}^2$

7.2 Geometry Modeling

In this study, two test pits were divided into four subsections, each subsection was 16 ft. long, 10 ft. wide, and 6 ft. deep. Since these subsections had symmetrical geometry and the dual wheel load application, only one-half was modeled, as shown in Figure 7.1. Each model was 80 in. long by 120 in. wide and 72 in. deep.

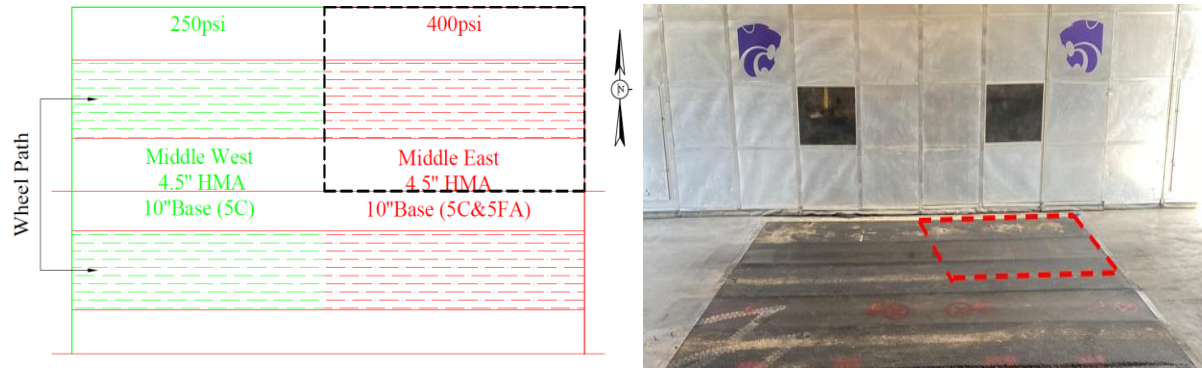


Figure 7.1 Geometry of Subsection

A three-layer system with a top HMA layer, SFDR base layer, and subgrade layer was assumed to model the pavement structure. Each layer used a 3-D solid shape with deformable properties, which was used for pavement structure simulation (Wu et al., 2010). Figure 7.2 shows the part module in the ABAQUS.

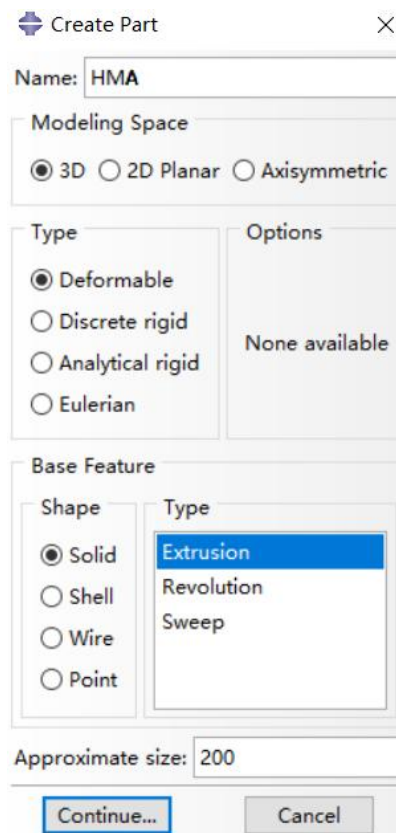


Figure 7.2 ABAQUS Create Part Setup

7.3 Material Characterization

In ABAQUS, defining material characteristics is a critical step for the FEM analysis. Material characterization involves selecting appropriate constitutive models and designating material property parameters within ABAQUS. This study's model consisted of three layers: HMA/AC, SFDR base, and clay subgrade.

7.3.1 HMA Layer Characterization

As discussed in the literature review, the HMA layer has been characterized using simplified, elastoplastic, and viscoelastic models.

7.3.1.1 Simplified Model

The simplified model refers to the linear elastic model - the asphalt layer deforms under loading and fully recovers on unloading. No permanent strain is produced in the AC material. This model cannot accurately simulate AC deformation under repeated wheel loads. Thus, the simplified model for this project was limited to investigating the AC layer's long-term rutting performance.

7.3.1.2 Elastoplastic Model

The elastoplastic model considers both the elastic and plastic behavior of the AC mixture. In this model, the AC material shows an elastic behavior with a yield stress, but it starts to have a plastic/permanent deformation when the stress exceeds the yield stress. This permanent deformation does not recover even when the load is removed. The model can simulate the permanent deformation behavior in the HMA layer under unloading conditions. Previous studies mentioned that the elastoplastic mode could simulate the permanent deformation of aggregate base and subgrade layers, with assumed isotropic hardening characteristics like the Drucker-Prager and Mohr-Coulomb models.

7.3.1.3 Viscoelastic Model

AC material exhibits a viscoelastic behavior, meaning its mechanical response to applied loads is time-dependent, and the stress-strain relationship changes over time (Wu et al., 2010). Under constant load, the AC layer deforms gradually and then recovers slowly when unloaded; the process is known as "creep". In general, the creep behavior of the AC mixture is characterized by three stages, as shown in Figure 7.3.

In the primary stage, the creep rate gradually decreases over time due to strain hardening. At the secondary stage, also known as the steady-state stage, the creep rate becomes relatively stable and uniform over a longer time; the AC materials exhibit a consistent rate of deformation. At the tertiary stage, there is an exponential increase in the creep rate quickly. At this stage, the AC material showed an accelerated deformation due to the dominance of the necking phenomenon. This causes the AC materials to fail or rupture eventually. In this study, a secondary stage was used for HMA rutting.

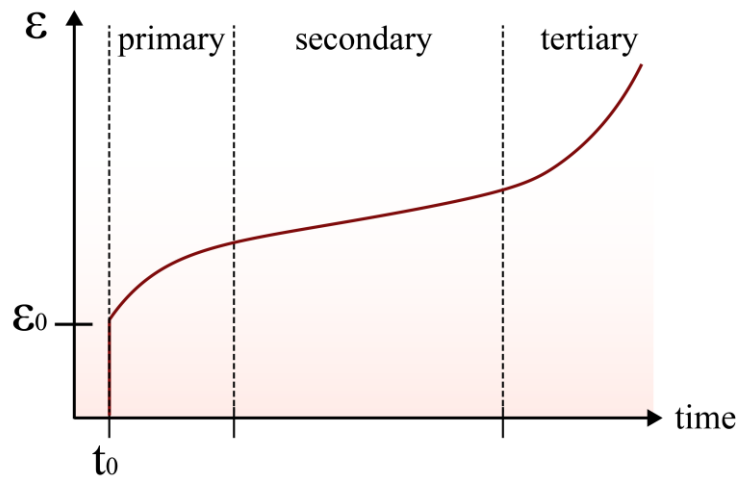


Figure 7.3 Creep Behavior of AC Mixture

In ABAQUS, a power-law creep model is defined as time-hardening or strain-hardening. For time hardening, the creep strain rate depends on the loading time. This model is used when

considering the constant stress applied to the object throughout the analysis. The time-hardening form is expressed as in Equation 7.1. The creep strain rate depends on the accumulated strain over time for the strain-hardening model. This model is selected when considering different stresses applied to the object throughout the analysis. The strain-hardening form is expressed as in Equation 7.2.

$$\dot{\bar{\epsilon}}^{cr} = A\tilde{q}^n t^m \quad \text{Equation 7.1}$$

where:

$\dot{\bar{\epsilon}}^{cr}$ = uniaxial equivalent creep strain rate,

$\tilde{q}$ = uniaxial equivalent deviatoric stress,

t = total loading time, and

A, n, and m=constant parameters defined by the user as functions of temperature and material, A and n are positive, $-1 < m \leq 0$ (ABAQUS, 2021).

$$\dot{\bar{\epsilon}}^{cr} = (A\tilde{q}^n [(m + 1)\bar{\epsilon}^{cr}]^m)^{\frac{1}{m+1}} \quad \text{Equation 7.2}$$

where:

$\bar{\epsilon}^{cr}$ = equivalent creep strain.

The viscoelastic properties of the AC material were determined by conducting a creep test. In the creep test, three Superpave gyratory compacted HMA samples were cored and trimmed to have a 4-in. diameter and 6-in. height. The specimen was mounted into the AMPT machine to apply a repeated haversine loading of 0.1 seconds and unloading of 0.9 seconds. The test was terminated by either finishing 10,000 load cycles or reaching a maximum plastic strain of 5,000 microstrains (Saghebfar, 2014). The AMPT machine with creep test setup is shown in Figure 7.4. The creep test was conducted at room temperature (77°F) and a deviatoric stress of 30 psi (Saghebfar, 2014).



Figure 7.4 Creep Test Setup (Saghebfar, 2014)

Figure 7.5 shows a creep test result plot for the AC/HMA material. In this study, the secondary stage was only considered for modeling the creep behavior of the AC materials. The creep parameters of AC for inputs in ABAQUS were determined using a polynomial function with a stress power law (Equation 7.1) via regression analysis. As mentioned in Section 5.1, the pavement subsections were insulated in a chamber room at a constant temperature (77°F), and a constant wheel load of 18 kips was applied for the entire test. Thus, the time and strain-hardening models theoretically output the same or similar analysis results for this project.

Previous studies have used a similar AC constitutive model to investigate the rutting resistance of pavements under repeated wheel loads (Al-Qadi et al., 2003; Wu et al., 2010; Wang, 2011; Saghebfar, 2014). The related creep model parameters and elastic properties for their projects are summarized in Table 7.2. Creep model parameters for a 12.5 mm NMAH mixture with the PG 70-22 binder in this study were obtained from earlier work at CISL (Williamson, 2015). The binder grade and NMAH were the same in the current CISL project.

Moreover, the AC creep behavior is sensitive to the temperature. All full-scale APT projects at CISL were conducted at room temperature (77°F); herein, the creep behavior at similar temperature was considered to have a minor effect on the pavement rutting in the FEM analysis, Poisson's ratio was assumed to be 0.30 for the AC layers.

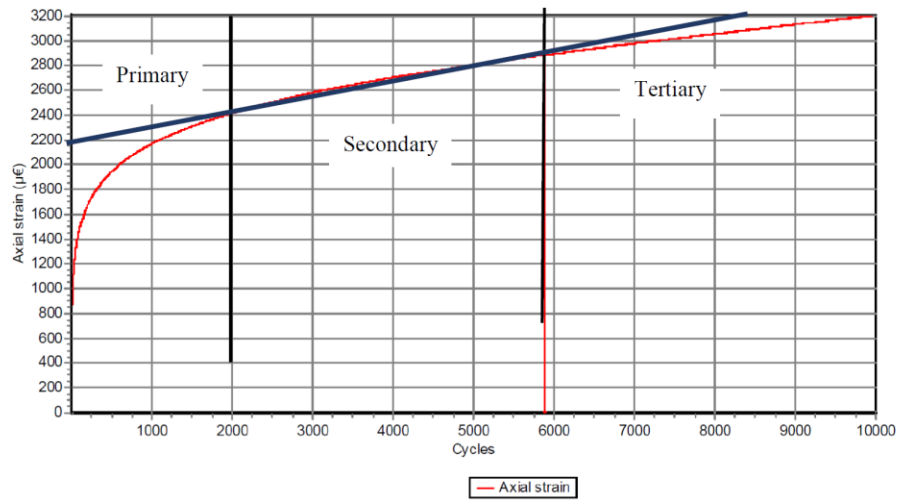


Figure 7.5 HMA Creep Test Result (Saghebfar, 2014)

Table 7.2 Previous AC Creep Compliance Test Results

AC Binder PG Grade	RAP content (%)	Creep Parameter			Elastic Modulus (ksi)	Poisson's Ratio	Reference
A	m	n					
PG 58-28	25	4E-08	-0.8	1	300	0.35	(Saghebfar, 2014)
PG 70-28	0	5E-08	-0.6	0.6	218	0.30	(Bortz, 2015)
PG 58-28	0	5E-09	-0.5	0.5	383	0.35	(Williamson, 2015)
PG 70-22*	0	5E-09	-0.5	0.6	320	0.35	
PG 64-22	0	1E-09	-0.5	7	452	0.35	(Onyango, 2009)
PG 64-28	0	5E-09	-0.5	6	235	0.35	
PG 70-22	0	5E-09	-0.5	0.5	350	0.35	
PG 64-22	0	4E-08	-0.5	0.8	271	0.35	

***Used in this study**

7.3.2 SFDR Base Layer

As discussed earlier, the SFDR mixture used cement or cement and fly ash as stabilizers. As shown in Figure 7.6, the SFDR base material was considered to have elastic and plastic characteristics. Poisson's ratio was assumed to be 0.35 for both SFDR base layers. During modulus testing, the SFDR base material was observed to have elastic and plastic characteristics. Thus, an elastoplastic model was selected for the SFDR base layer in FEM. Base model parameters are summarized in Table 7.3.

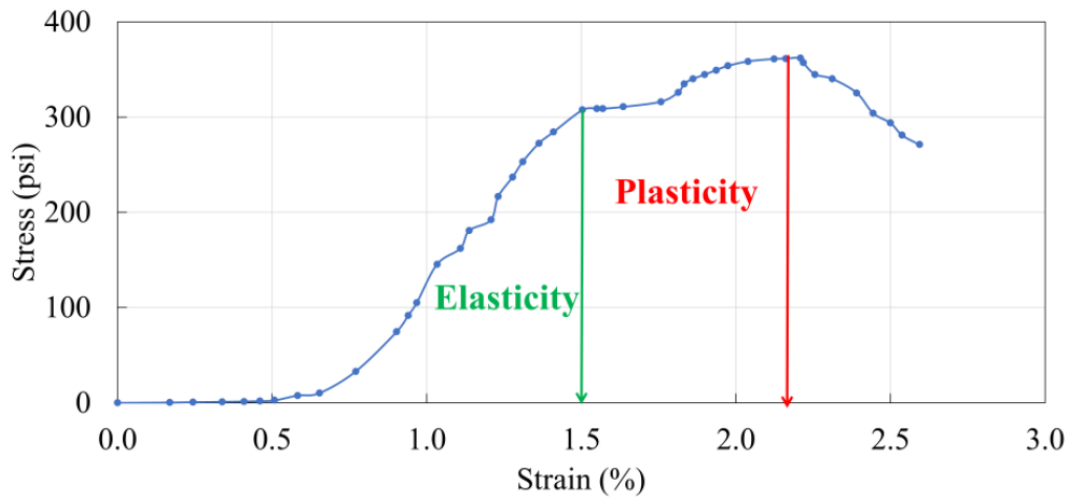


Figure 7.6 Stress-Strain Curve of SFDR Mixture

Table 7.3 SFDR Base Layer Modeling Parameters

Test Section (AC Thickness)	SFDR Mixture	Density ($\text{lbf s}^2/\text{in}^4$)	Elastic		Plastic	
			Young's Modulus (psi)	Poisson's Ratio	Yield/Ult imate Strength (psi)	Plastic Strain (%)
Middle (4.5 in.)	5C&5FA	0.000205121	28,716	0.35	307	2.66
	5C	0.000200574	19,720	0.35	237	3.26
South (3 in.)	5C&5FA	0.000205121	32,118	0.35	320	2.55
	5C	0.000200574	21,286	0.35	254	3.93

7.3.3 Clay Subgrade Layer

As mentioned earlier in Section 5.2, four layers were constructed for each subsection. It was noted that the third and fourth layers had the same soil, A-7-6 clay with varying degrees of compaction. To simplify the geometry of the ABAQUS model, these two layers were combined into a single layer as the subgrade layer in the model. In previous studies at CISL, Saghebfar (2014) and Bortz (2015) used a creep model to simulate the plastic characteristics of clay soil. In this study, creep model parameters were obtained from previous studies due to similar soil (AASHTO A-7-6) used in the test sections of the current research and summarized in Table 7.4, Poisson's ratio was assumed to be 0.45 for the subgrade layer.

Table 7.4 Subgrade Model Parameter

Subgrade	
Density (lbf s ² /in ⁴)	1.81E-04
E (psi)	10,000*
Poisson's Ratio	0.45
A	5.26E-08
m	0.6
n	-0.58

* AASHTO (2021)

7.4 Meshing

In ABAQUS, meshing involves dividing a complex model's geometry into smaller, manageable elements. This discretization helps to model and analyze complex irregularly shaped structures at these finer elements (ABAQUS, 2021). In this study, the pavement structure had a rectangular shape, and the literature mentioned that two types of elements were used for modeling pavement structure - 8-noded linear brick element with reduced integration (C3D8R) and 20-noded linear brick element with reduced integration (C3D20R). These elements are shown in Figure 7.7.

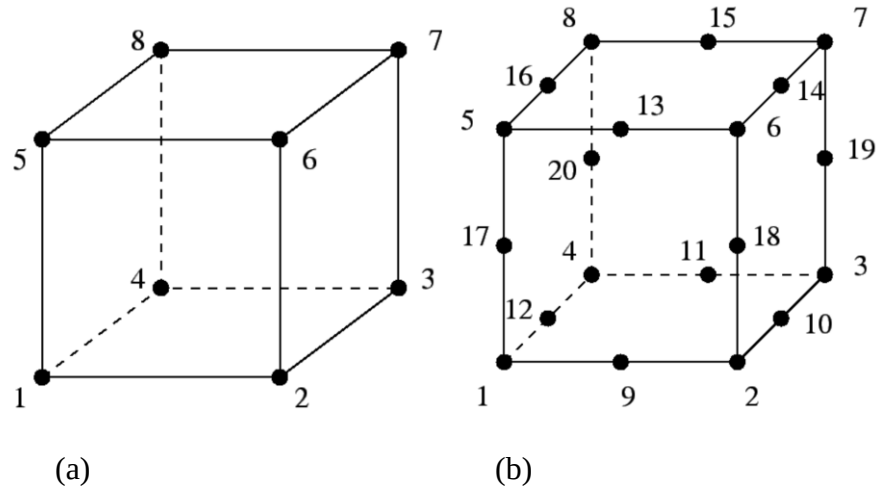


Figure 7.7 Element Type: (a) C3D8R; (b) C3D20R

The C3D8R is commonly applied to simple geometrical structures such as road pavement structures. Previous studies found that the C3D8R was an appropriate element type for simulating the APT test section (Sagherbafar, 2014; Bortz, 2015). The C3D20R is a higher-order element used to capture curved surfaces or complex geometrical models with high-stress gradients more accurately, but the model requires longer computational time.

In ABAQUS, the finer elements produce more accurate results in FEM simulation. Figure 7.8 illustrates the FEM mesh with different dimensions. To select the appropriate mesh size for the APT pavement structures, 15,000 load repetitions were applied to the FEM with various combinations of mesh sizes. The rutting in the HMA layer and vertical stress on the top of the subgrade were computed and compared. The summary comparison results are shown in Table 7.5. The results indicated that pavement structures with both mesh types exhibit similar rutting and stresses on top of the subgrade. Thus, Model 5 was selected for modeling all the subsections since it significantly reduced the computational time while maintaining data extraction efficiency. The lengths of mesh units for the HMA overlay, SFDR base, and subgrade, respectively, are 6, 6, and 20 units

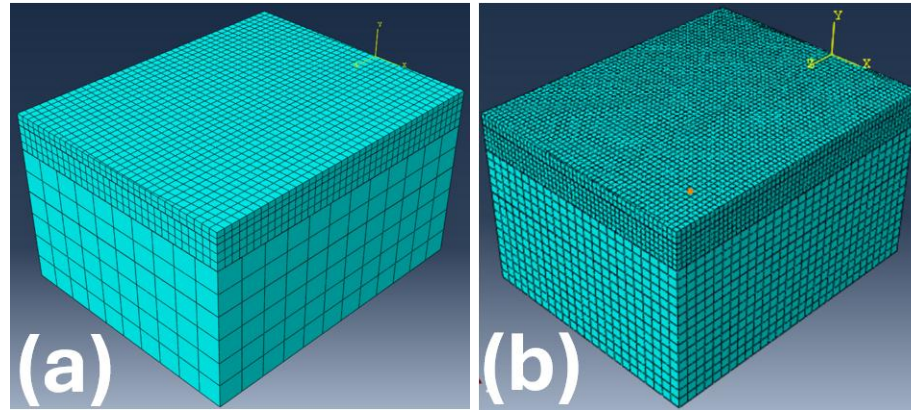


Figure 7.8 Various Types of ABAQUS Mesh Sizes

Table 7.5 Various Mesh Types for Pavement FEM

Mesh Model	Length of Mesh Unit			Total Number of Elements	Rutting (in)		% Difference	Stress (psi)		% Difference
	AC- OL	SFDR Base	Subgr ade		C3D20R	C3D8R		C3D20R	C3D8R	
1	3	3	6	9,680	-0.04	-0.04	-11.1%	11.21	10.15	-9.5%
2	3	3	10	7,200	-0.06	-0.06	-0.3%	16.31	17.57	7.7%
3	4	4	10	3,744	-0.06	-0.06	0.8%	17.43	17.76	1.9%
4	5	5	10	2,072	-0.06	-0.06	1.6%	17.38	17.71	1.9%
5*	6	6	20	1,087	-0.06	-0.06	0.8%	17.35	17.59	1.4%

*Used in this study

7.5 Boundary and Loading Conditions

7.5.1 Define Boundary Conditions

In CISL, the HMA layer was paved wall-to-wall, and half of each APT subsection was modeled. Figure 7.9 illustrates modeled subsections. The overall model was set to be fixed, which means no displacement or rotation was allowed; the bottom of the model was defined as "ENCASTRE" with fully built-in ($U1 = U2 = U3 = UR1 = UR2 = UR3 = 0$), as shown in Figure 7.10. In the X-direction (east/west), the boundary was set to $U1 = UR2 = UR3 = 0$, which means no displacement in the X-direction and no rotations on the Y and Z axes. Similarly, in the Y-direction, the boundary was set to $U3 = UR1 = UR2 = 0$.

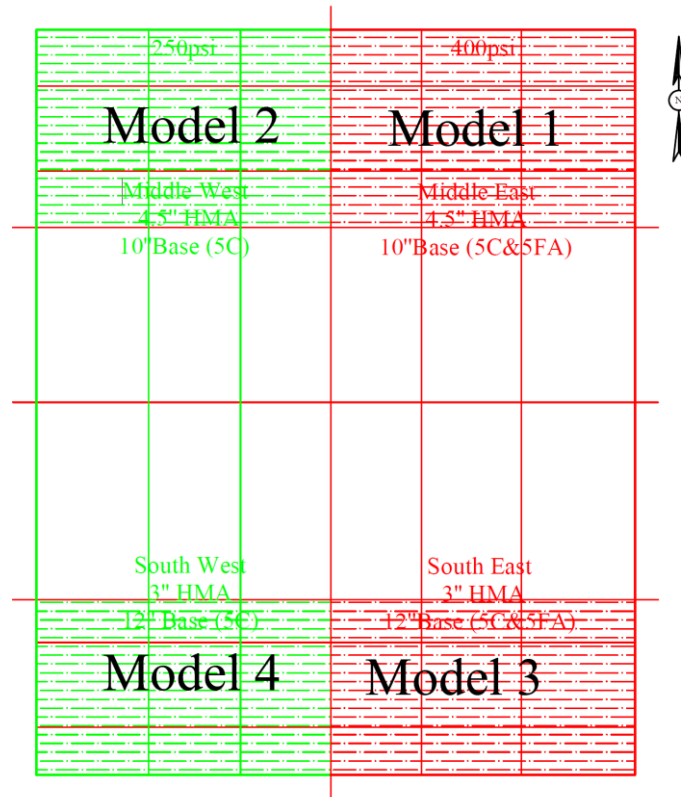


Figure 7.9 FEM Section

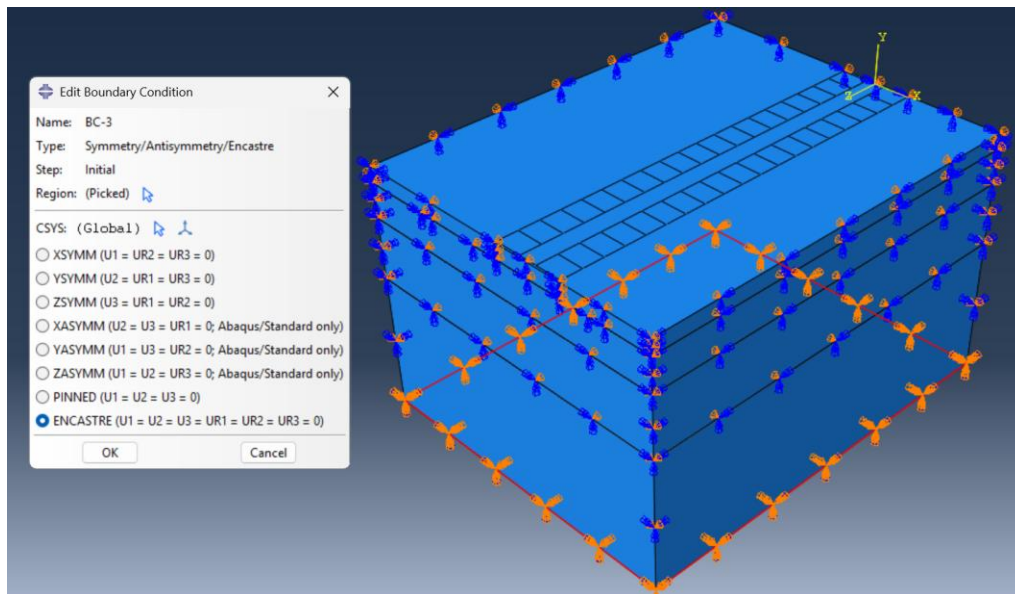


Figure 7.10 FEM Boundary Conditions

7.5.2 Define Interaction

In ABAQUS, the interactions in pavement structure refer to the contact behavior between different layers. There are two defined contact types: full contact and contact with friction. Full contact was defined as the perfect bonding at the interface. No relative movement or sliding is allowed. In contrast, contact with friction allows relative movement and slip at the interface while considering frictional resistance. In this study, the interaction settings between the layers were defined as:

1. HMA bottom/SFDR Base top (Tie constraint), and
2. SFDR Base top/Subgrade layer top (Tie constraint)

7.5.3 Define Rutting Loading Behavior

According to the literature review, three types of loading are commonly used for FEM analysis of pavements. They are pulse wheel load on a single tire contact area, equivalent (constant) wheel load on the whole wheel path, and moving wheel load along the entire wheel path.

The pulse wheel loading simulates a single short repetitive load application, as illustrated in (Figure 7.11 (a)). The number of wheel load passes is achieved by accounting for load pulse times. This method is computationally efficient for analyzing the quick response of the pavement to a single load but may not accurately capture the cumulative effects of repeated traffic loading. The constant wheel loading (Figure 7.11 (b)) simulates a static load applied to the whole wheel path. This method simplifies the analysis by assuming a constant load presence, but it cannot capture the dynamic effects of a moving load on the flexible pavement. The moving wheel load (Figure 7.11 (c)) simulates a uniform, rectangular/circular stress block applied along the whole wheel path. The number of wheel load passages is simulated by accounting for the cycles of the stress block as it travels from one end to the other end of the model. This method represents the

most realistic representation of wheel loading at CISL, capturing the dynamic effects of loading and unloading, variations in stress over time, and the influence of vehicle speed.

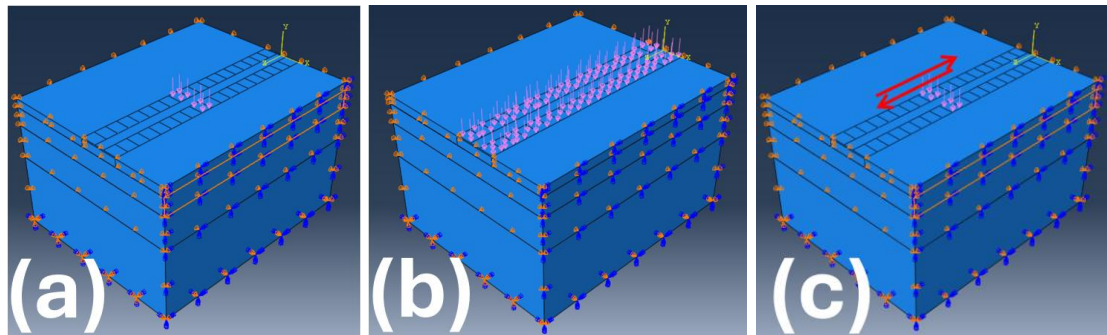


Figure 7.11 FEM with Loading Behavior: (a) Pulse Load; (b) Constant Load; (c) Moving Load

In this study, the constant and moving wheel loading was used in the FEM simulation. For the constant loading simulation, the wheel load was uniformly applied across the entire wheel path, and one load cycle was calculated for a loading duration of 0.05 seconds, following the methodology outlined by Bortz (2015). For example, simulating 10,000 load cycles in this setup would result in a total loading time of 500 seconds. The applied stress was consistently set at 90 psi throughout the simulation to reflect the APT device tire pressure. Table 7.6 provides a summary of the constant loading program parameters used in the FEM analysis, including the details of the six predefined measurement points after a certain number of load repetitions.

Table 7.6 Constand Loading Program in FEM

Step	Duration (s)	Loading (psi)	Remark
Initial	0	0	Number of Loading Cycles
1	68	90	1,352
2	1	0	Record Data
3	432	90	10,000
4	1	0	Record Data
5	2,068	90	50,000
6	1	0	Record Data
7	2,932	90	100,000
8	1	0	Record Data
9	9,568	90	250,000
10	1	0	Record Data
11	10,432	90	500,000
12	1	0	Record Data

A FORTRAN subroutine, DLODE, was developed to program the moving load behavior. The wheel pass in the APT was simulated by moving a stress block along a coordinate system for a specific time. As discussed earlier, the APT device was equipped with an 18-kip load on a single axle with a dual wheel with 90 psi tire pressure on each side. The actual tire contact area of a single tire consists of two semi-circles and one rectangle, as shown in Figure 7.12 (a). PCA (1984) converted this actual tire contact area into a rectangular area of $0.5227 L^2$ for finite element analysis of concrete pavements (Figure 7.12 (b)), where the length is $0.8712L$, and the width is $0.6L$ (Huang, 2004). This FEM-based contact area was computed by dividing the single wheel load by the tire pressure. In this study, for FEM simulation, the APT tire contact area (50 in^2) was determined to be a rectangle of 8 in. by 6.25 in, as shown in Figure 7.12 (c).

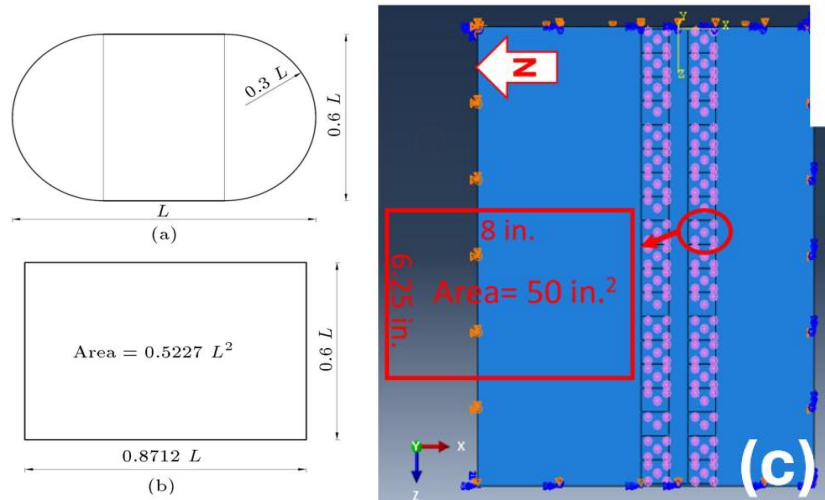


Figure 7.12 Tire Contact Area (Huang, 2004)

7.6 Analysis and Results

The ABAQUS model analyzed the pavement structure in steps, providing a detailed and incremental assessment of the pavement's response to applied loads and subjected to the boundary conditions. Figure 7.13 shows a typical example of FEM analysis results. The analysis time was variable depending on the model, load, and complexity of the model (ABAQUS, 2021).

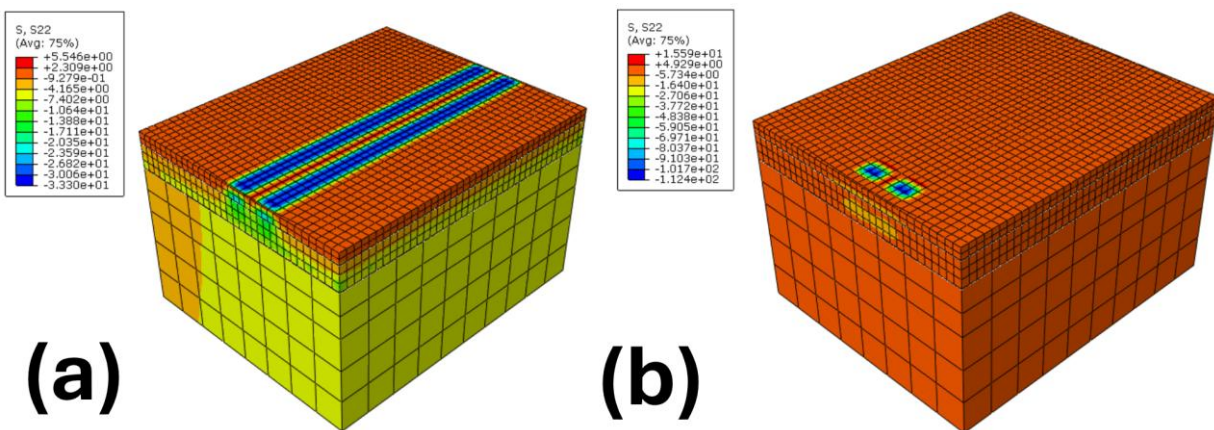


Figure 7.13 FEM Analyzed Results: (a) Constand Load; (b) Moving Load

Chapter 8 - FEM Analysis Results

8.1 Introduction

Four FEM models were developed with material properties and pavement layer structures identical to the four test subsections at CISL. The objective of the FEM analysis was to compare the full-scale APT test results with those from the FEM model and use it for future research and development in FDR pavement design. The simulated results consisted of rutting on the AC/HMA pavement surface, horizontal/transverse strain at the bottom of the HMA/AC layer, and vertical stress on top of the subgrade layer.

8.2 Rutting on the AC Pavement Surface

Analyzing the 3-D FEM model proposed with moving load was computationally intensive. It was estimated to take approximately 815 hours for one model on a regular desktop computer with a 5.3 GHz CPU and 128GB DRAM memory to simulate 500,000 load repetitions. Due to this limitation, 5,000 load cycles were applied in the moving load analysis, which took about five hours of analysis time for one model. Additionally, the FEM models with constant load application were also analyzed. This analysis took about eight hours for one model with 500,000 load repetitions. The constant loading model results were compared with the APT results.

Figure 8.1 and Figure 8.2 illustrate the rutting results in the FEM models with moving and constant loading, respectively. The results show that the pavement section with a 4.5-inch HMA overlay and an SFDR base stabilized with 5% cement & 5% fly ash exhibited the highest rutting, followed by the section containing 3-in. HMA and SFDR with 5% cement and 5% fly ash.

In general, three types of rutting form in the flexible pavement: instability rutting, structural rutting, and densification. Instability rutting results from the shear and displacements in the wheel

path area. The HMA layer moves downward, laterally, and upward, creating a transverse profile shaped like a “w.” The instability rutting, shown in Figure 8.3 (a), happens when high tire pressure and truck traffic volume are present. Structural rutting results from weaker support in the pavement foundation layers, such as the base or subgrade. All layers in the pavement structure deform due to repeated traffic loads, as shown in Figure 8.3 (b). Densification refers to further compaction of the HMA layer due to traffic, as shown in Figure 8.3 (c). In this study, FEM-predicted and APT results showed a rapid increase in rutting at an early stage (within 10,000 load repetitions) for some sections, likely due to the HMA layer densification.

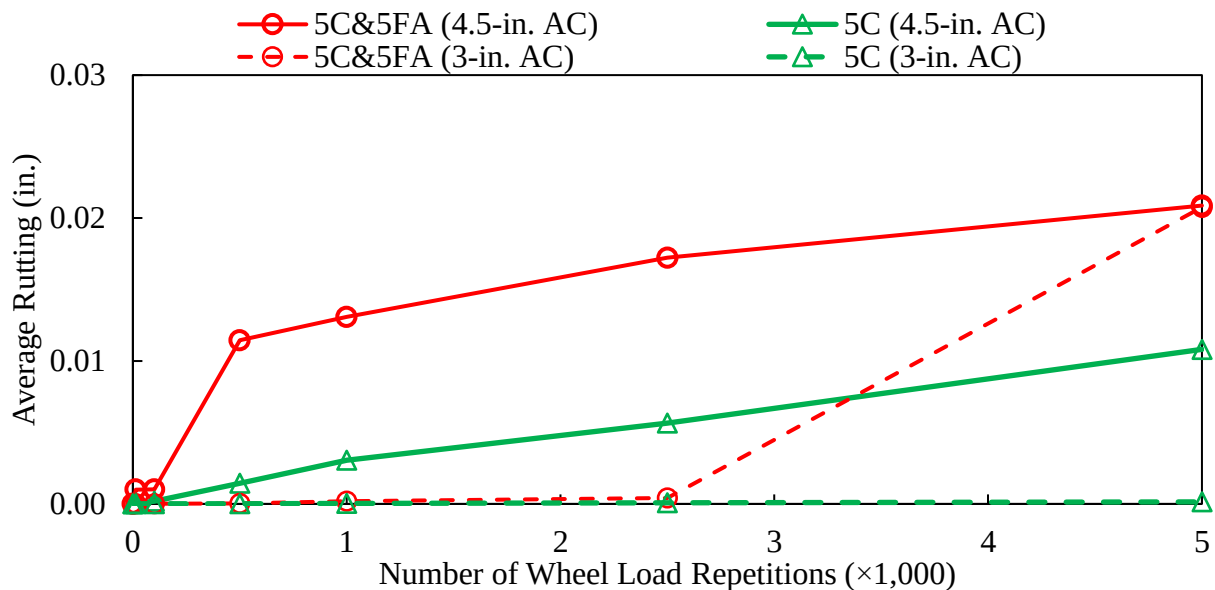


Figure 8.1 Rutting Results from FEM with Moving Loading

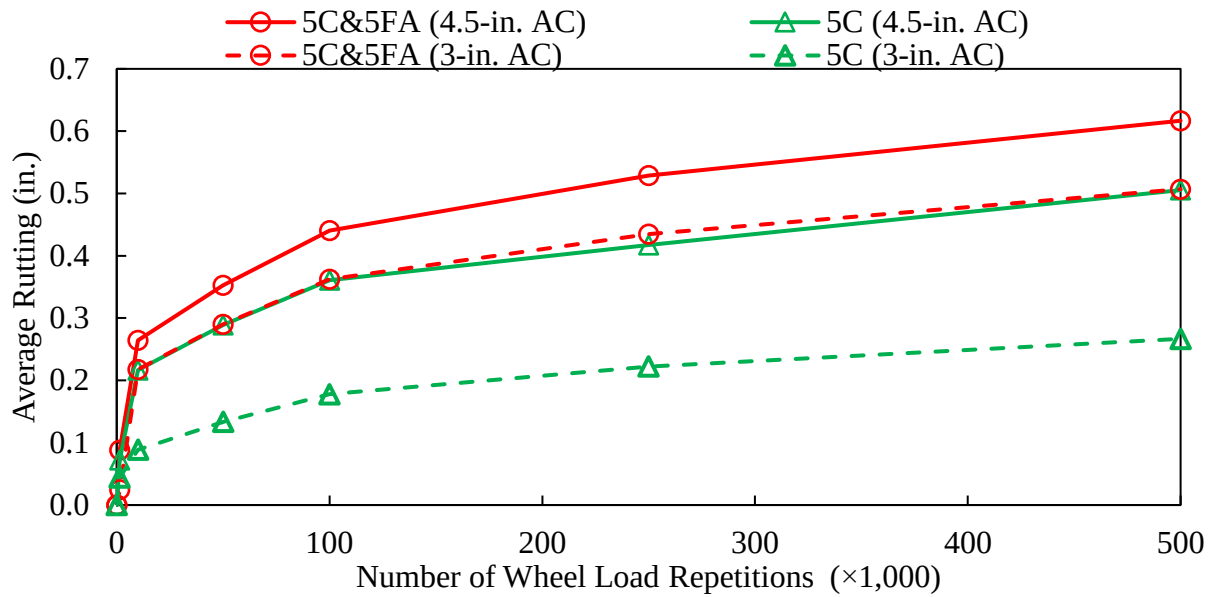


Figure 8.2 Rutting Results from FEM with Constant Loading

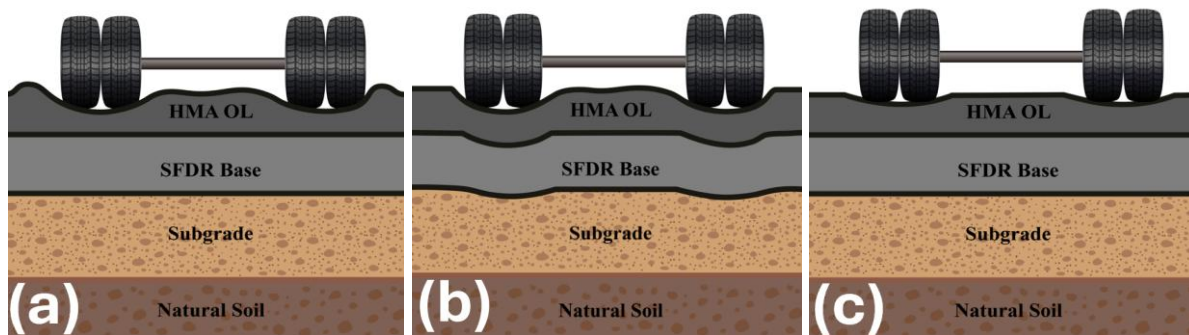


Figure 8.3 Flexible Pavement Rutting: (a) Instability Rutting (b) Structural Rutting (c) Density Rutting (after Onyango, 2009)

Figure 8.4 illustrates the comparison of rut depths from the FEM outputs and APT test results. The results indicated that the FEM analysis significantly overpredicted the rutting performance for all subsections. However, the results show that FEM captured the long-term rutting trends observed in the APT test results. The section with a 3-inch HMA layer and SFDR base stabilized with 5% cement (Figure 8.4(c)) showed the lowest rutting over the entire test. This trend was also observed in the AASHTOWare PMED analysis that featured a multi-layer linear-

elastic response program (Wu et al., 2024). The test section with 4.5-in. HMA overlay over SFDR base stabilized with 5% cement, and the section with 3-in. HMA overlay over SFDR base stabilized with 5% cement & 5% fly ash showed similar average rut depths. Rutting on these sections was structural (Figure 8.3 (c)). De Beer (1990) found that in a thick pavement structure consisting of a thin AC wearing course over a relatively thick SFDR base layer, the surface deformation can be attributed to the compression of the lightly stabilized base layer. This manifestation of compression failure in the cementitious base layer is shown in Figure 8.5.

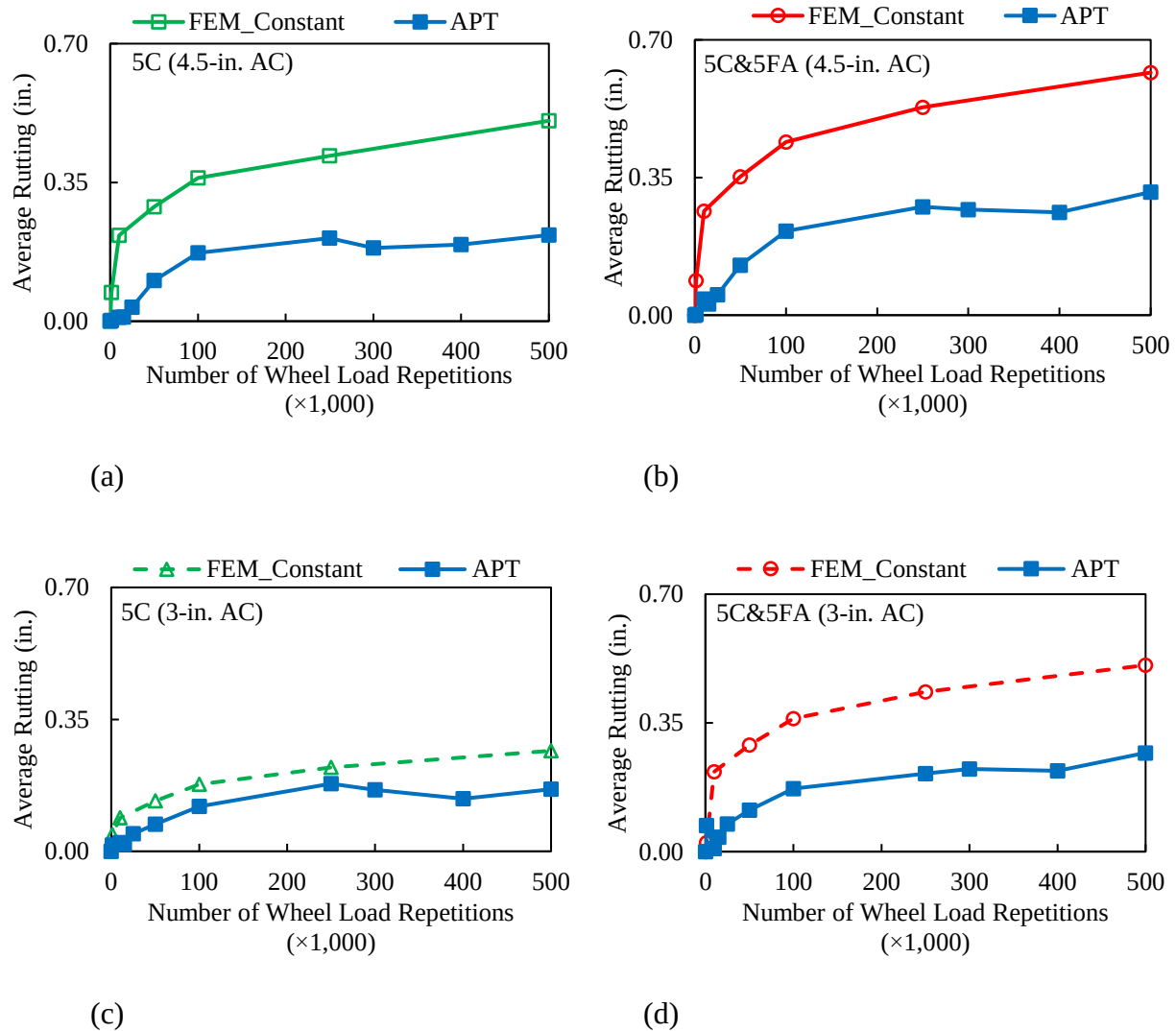


Figure 8.4 Rutting Results Comparison: (a) 4.5-in. HMA (5C); (b) 4.5-in. HMA (5C&5FA); (c) 3-in. HMA (5C); (d) 3-in. HMA (5C&5FA)

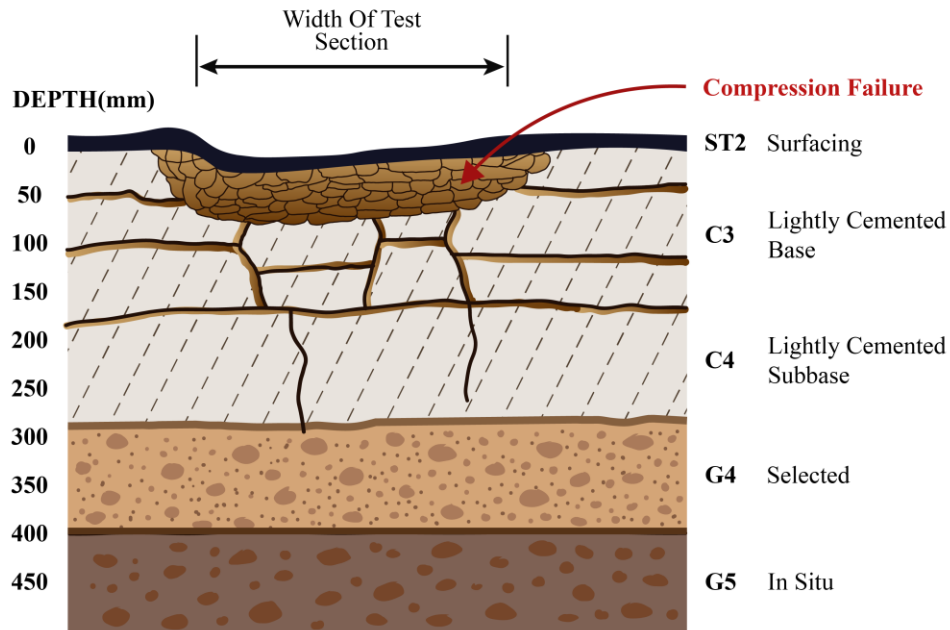


Figure 8.5 Manifestation of Compression Failure (after De Beer, 1990)

8.3 AC/HMA Layer Strain

As discussed in Chapter 6, strain at the bottom of the HMA layer is a critical factor affecting the fatigue performance of flexible pavements (Huang, 2004). Figure 8.6 and Figure 8.7 illustrate the FEM analysis results for the horizontal tensile strain at the bottom of the HMA layer. The results show that the subsection with a 3-inch AC overlay exhibited lower tensile strain when compared to the 4.5-inch AC sections in moving load analysis. The behavior in the constant load analysis was the opposite. The rapid increase in strain observed at the early loading stages could be attributed to the densification of the AC layer.

Figure 8.8 shows the comparison of the FEM with constant loading outputs and APT results. A nonlinear increase in strain was observed with increased wheel load repetitions. The FEM analysis indicated these strain values were based on the thickness of AC layer, where thicker AC layers caused a higher tensile strain. The measured tensile strains were a function of the SFDR base layer stabilizer. The SFDR layer with 5% cement had higher tensile strains. This might

indicate that there is potentially micro-crack damage. On the other hand, differences between the FEM and APT results could be due to the assumed HMA and base material parameters used in the FEM model as well as loading differences.

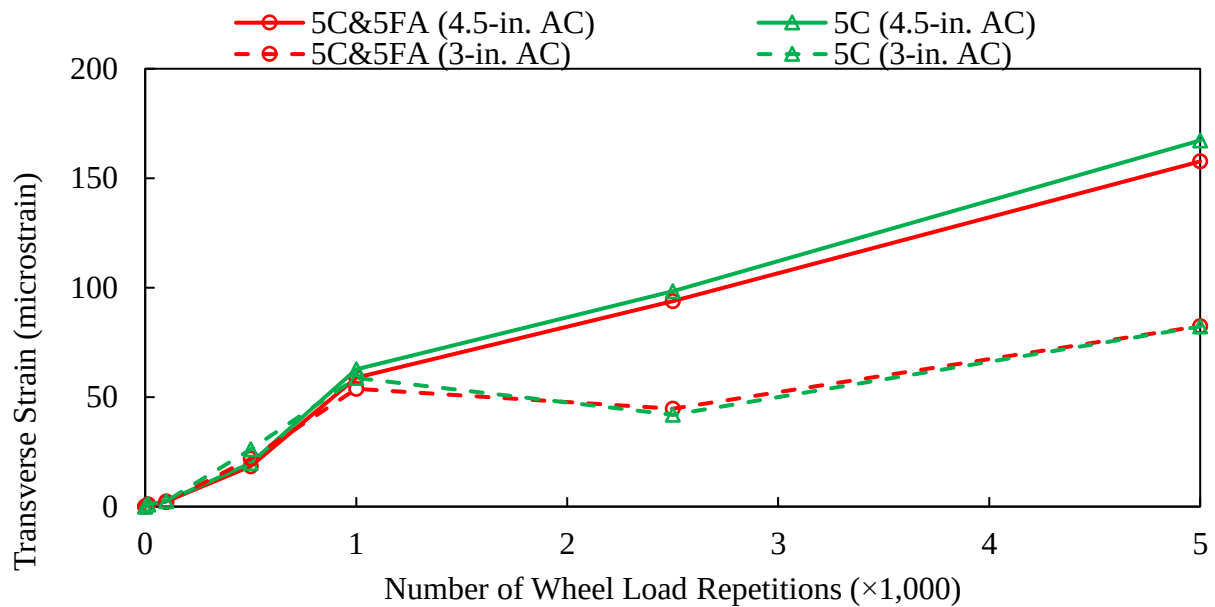


Figure 8.6 FEM (Moving Load) Tensile Strain at the Bottom of HMA Layer

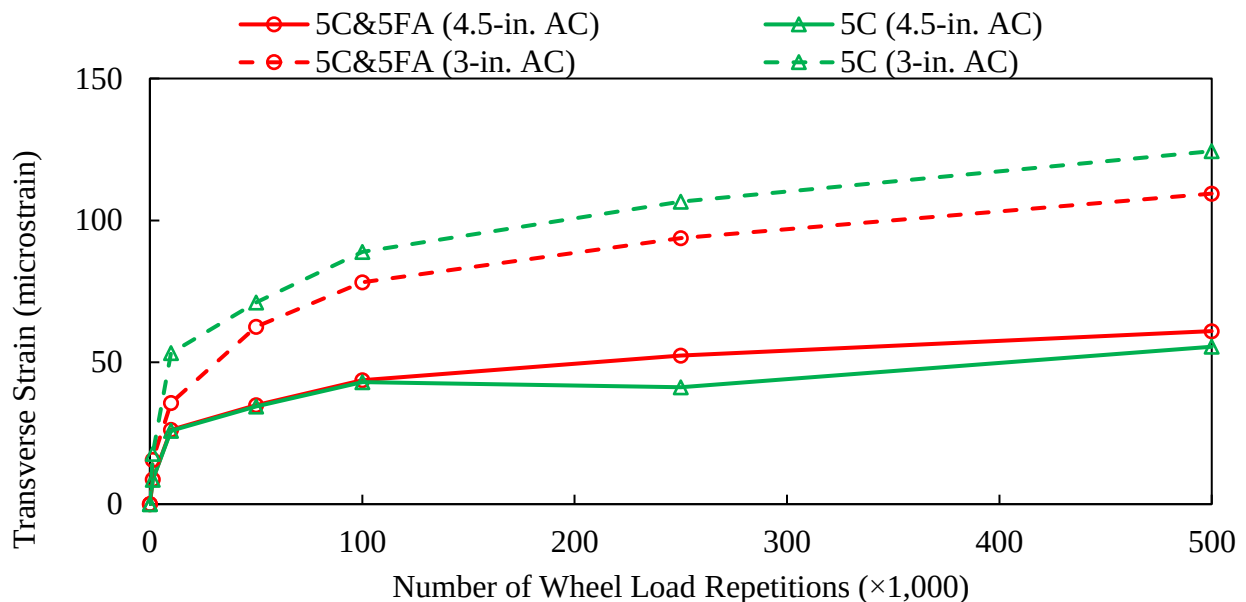
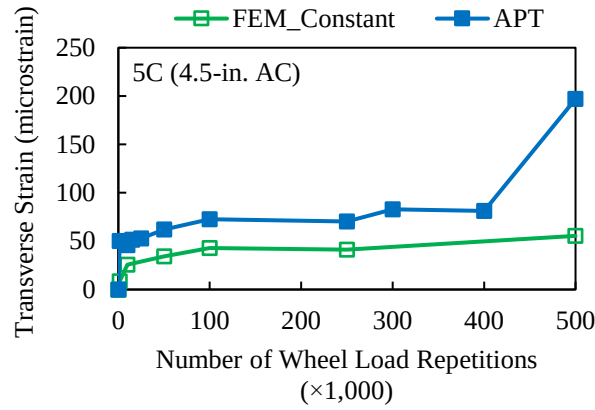
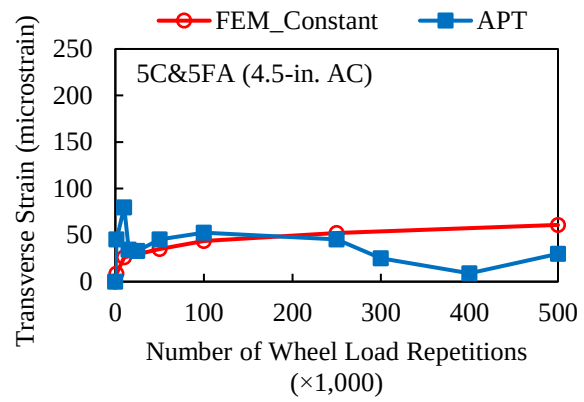


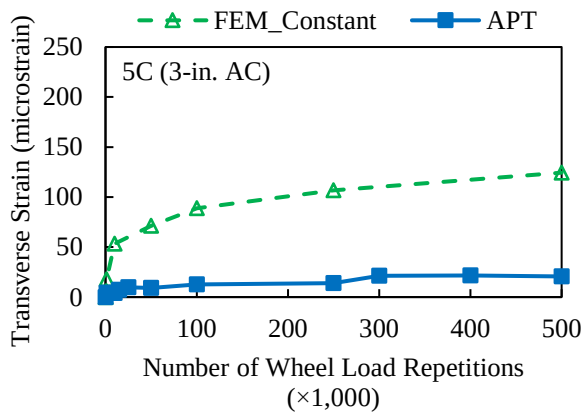
Figure 8.7 FEM (Constant Load) Tensile Strain at the Bottom of HMA Layer



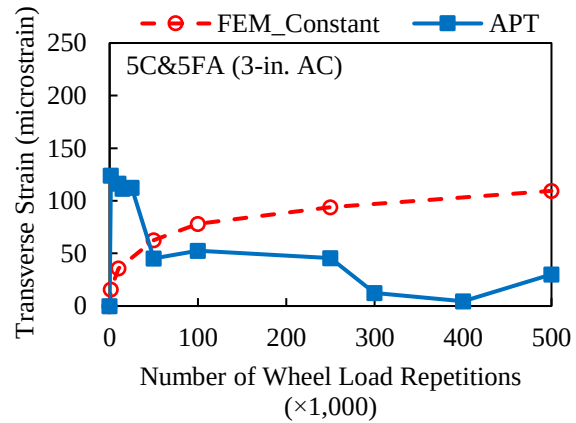
(a)



(b)



(c)



(d)

Figure 8.8 AC Transverse Strain Results Comparison: (a) 4.5-in. HMA (5C); (b) 4.5-in. HMA (5C&5FA); (c) 3-in. HMA (5C); (d) 3-in. HMA (5C&5FA)

8.4 Subgrade Vertical Stress

In the FEM model output, the vertical stress on the top of the subgrade was determined by averaging eight stress data points from the top C3D8R mesh element of the subgrade layer. The FEM results for the moving and constant loading are summarized and plotted in Figure 8.9 and Figure 8.10, respectively. The FEM with moving load results showed that the section with a 3-inch AC overlay containing SFDR stabilized with 5% cement demonstrated lower vertical stress on the subgrade for the entire simulation. The section with a 4.5-inch AC overlay containing SFDR stabilized with 5% cement demonstrated the highest vertical stress on the subgrade. Both 3-inch

and 4.5-inch AC sections stabilized with 5% cement & 5% fly ash had consistently lower vertical stress on top of the subgrade, which indicated that the SFDR base with higher strength resulted in better load distribution across the pavement structure.

Figure 8.11 presents the comparison between the FEM outputs and stresses measured during APT. It showed an increasing trend in stress with increasing wheel load repetitions. In addition, the subsection with 5% cement & 5% fly ash SFDR base experienced higher vertical stress, which deviated from the APT results. One explanation for this deviation is that the high-strength SFDR base layers may have partially fractured under repeated wheel loads. This compression failure in the SFDR base layer contributed to structural rutting along the wheel path. In the FEM simulation, each layered material was assumed to be uniform and homogeneous, which differs from the conditions during testing at CISL. This discrepancy indicates that the constitutive model for the SFDR layer stabilized with cement and fly ash needs further improvement in the FEM analysis.

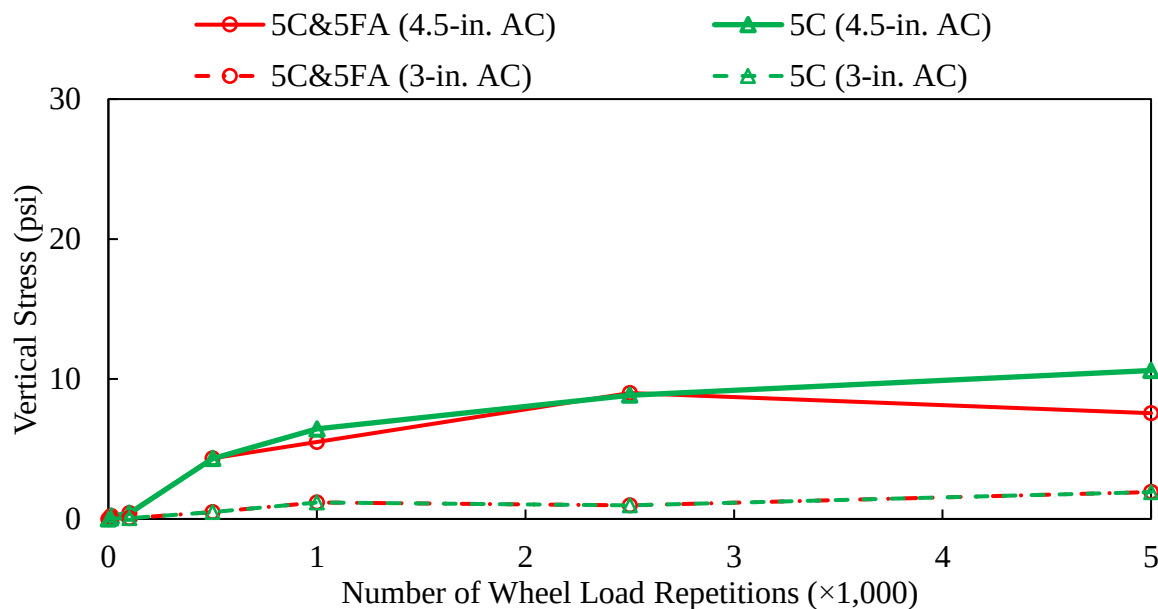


Figure 8.9 FEM (Moving Load) Vertical Stress on Top of Subgrade Layer

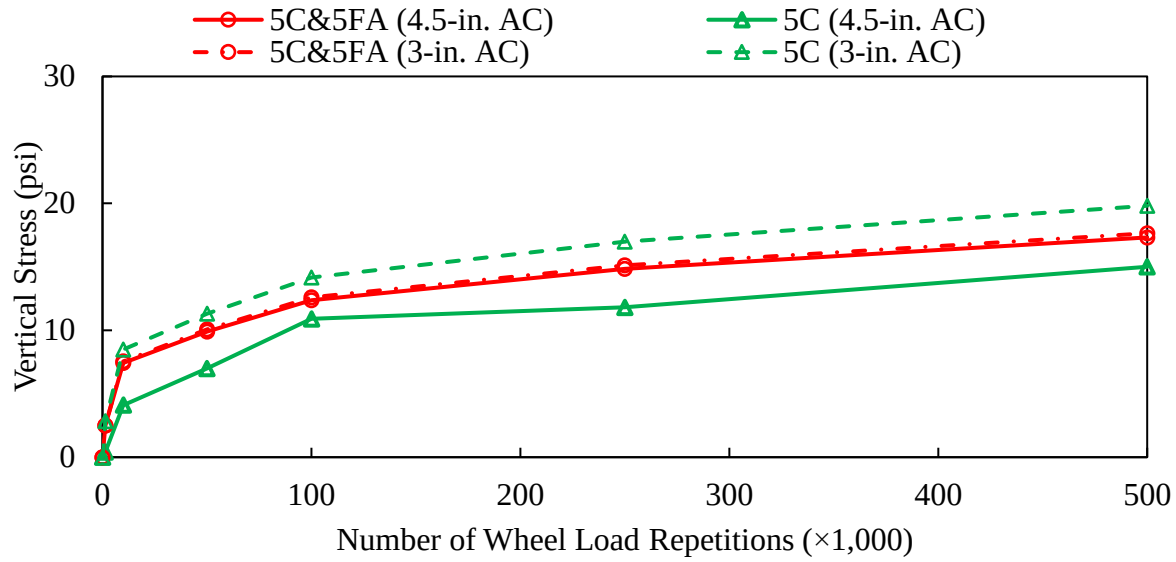


Figure 8.10 FEM (Constant Load) Vertical Stress on Top of Subgrade Layer

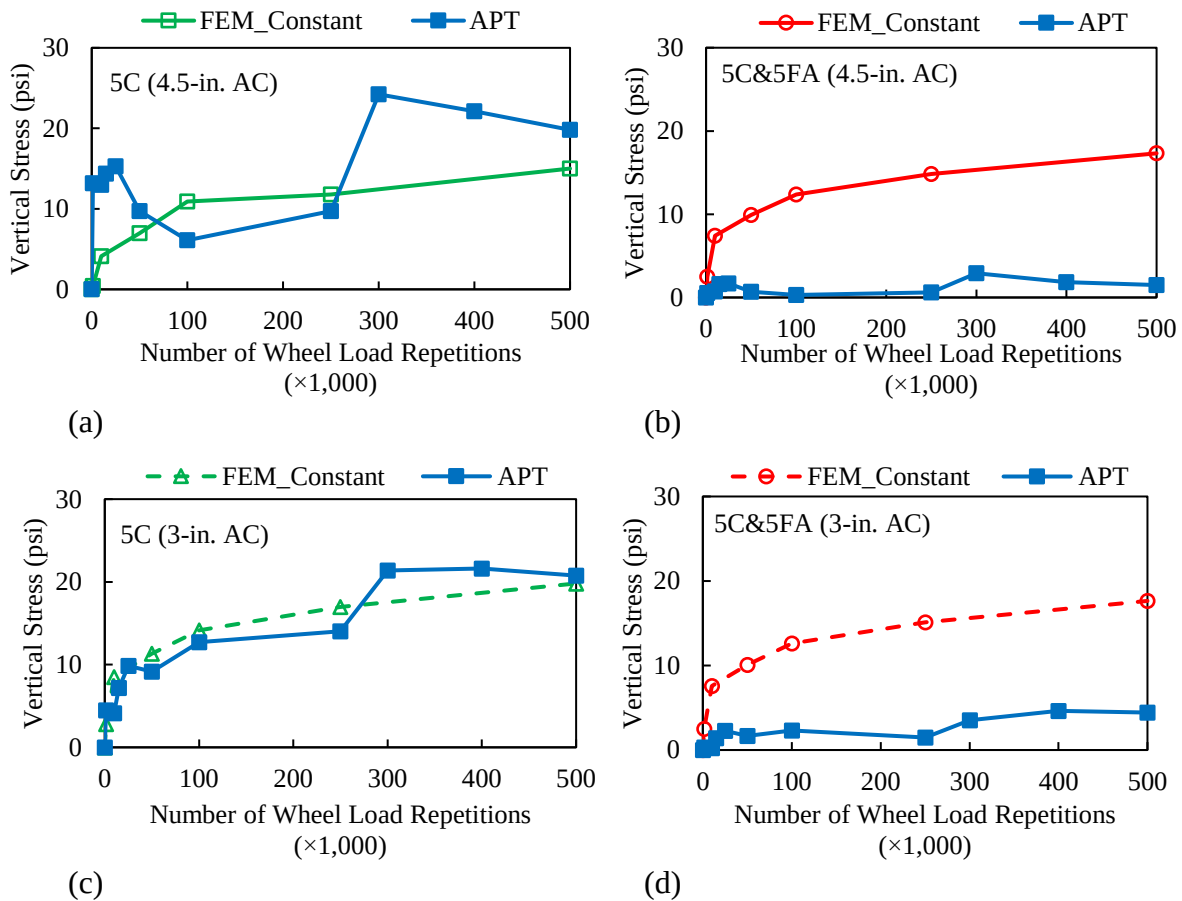


Figure 8.11 Subgrade Vertical Stress Results Comparison: (a) 4.5-in. HMA (5C); (b) 4.5-in. HMA (5C&5FA); (c) 3-in. HMA (5C); (d) 3-in. HMA (5C&5FA)

8.5 FEM Results Discussion

Overall, the FEM with moving load predicted expected results for investigating the behavior of the flexible pavement structure with the SFDR material. The FEM captured plot trends like those observed in the APT test. However, the actual outputs deviated from those measured during APT results. Both FEM models (moving and constant load) could not capture the non-linear strain response at the bottom of the HMA layer as the APT loading repetitions increased. This may indicate that such a simulation can be difficult to accomplish. As mentioned earlier, the discrepancy between multi-layer linear-elastic analysis and APT responses was also observed during AASHTOWare PMED analysis (Wu et al., 2024).

Chapter 9 - Conclusions and Recommendations

9.1 Conclusions

9.1.1 Phase I Laboratory Investigation

This part of the study evaluated eight stabilized FDR (SFDR) mixtures with various fly ash and cement contents using standard Proctor, unconfined compressive strength (UCS), vacuum saturation, and linear shrinkage tests. The optimum thickness of the AC layer with the SFDR mixtures in the base layer was determined using the AASHTOWare Pavement ME Design (PMED) software, a state-of-the-art flexible pavement design procedure. Results from this study led to the following conclusions:

- Fly ash content in the SFDR mixture significantly affects optimum moisture content (OMC). High fly ash content results in decreased OMC. However, the cement and fly ash & cement contents only minimally impact the OMC and maximum dry density, except for the SFDR mixture with 5% cement and 5% fly ash.
- The cement and fly ash contents in the SFDR mixture significantly impact its strength and shrinkage. Higher cement content results in higher UCS, while the higher fly ash content decreases shrinkage.
- For SFDR mixtures that met the minimum 250 psi 7-day UCS required by the US Army Corps of Engineers, 5% cement was the most suitable stabilizer.
- An SFDR mixture with a fly ash content higher than 10% could significantly decrease UCS under moist conditions. Thus, a moisture susceptibility test can identify unsuitable chemical stabilizers in the SFDR mixture design process.

- For the SFDR mixture design, a starting point of 5% cement by dry weight of FDR material would be appropriate.
- The AASHTOWare PMED software (version 2.5) predicted that the SFDR mixture with a 274 psi UCS would have an optimum AC overlay layer thickness of 3 in. In comparison, 473 and 610 psi UCS values were expected to result in a 4.5-in. AC layer. These thicknesses remained unchanged regardless of the SFDR base layer thickness (10 or 12 in.).

9.1.2 Phase II Accelerated Pavement Testing (APT)

Based on the condition and response-monitoring data from the APT full-scale tests and performance analysis using the AASHTOWare PMED software, the following conclusions can be made:

- No cracking was observed on all test sections after applying 600,000-wheel load repetitions.
- The SFDR mixture with 5% cement exhibited lower HMA rutting than the SFDR mixture with 5% cement and 5% fly ash.
- For HMA pavement with an SFDR base layer, the average transverse strain was higher than the longitudinal strain.
- The SFDR mixture with 5% cement and 5% fly ash resulted in lower transverse and longitudinal strain levels than the 5% cement in the pavement.
- The SFDR mixture with 5% cement and 5% fly ash resulted in a lower vertical stress on top of the subgrade than the SFDR mixture with 5% cement in the pavement.

- Temperature change had a relatively minor effect on the strain at the bottom of the HMA overlay and the stress at the top of the subgrade.
- The PMED software (version 6.2.2) predicted lower rut depths than those observed in the APT test. However, for sections with FDR bases stabilized with cement, predicted rut depths were below the projected failure rut depth (0.5 in.).
- The PMED software (version 6.2.2) predicted lower amounts of top-down cracking for the thicker AC overlay sections regardless of the stabilizer type in the mixture. APT testing did not show any cracking on these sections. The default PMED criterion of 25% of the lane area should be used for top-down cracking to design new pavements with SFDR bases.
- Results from the PMED software (version 6.2.2) showed that sections with cement-stabilized FDR mixtures had lower IRI than cement and fly ash-stabilized sections. However, all sections had much lower predicted IRI than the IRI failure criterion.
- The 1993 AASHTO Pavement Design Guide can be used for SFDR flexible pavement design. The SFDR base layer coefficients need to be obtained from the nomographs in the Guide based on the unconfined compressive strengths.

9.1.3 FEM

Based on the comparison between the APT tests and FEM (constant loading) analysis, the following conclusions can be made:

- The FEM using a creep model for the AC layer was able to capture rutting, and it predicted rutting trends like those from the APT test.

- The FEM using the elastoplastic constitutive model for the SFDR layer stabilized with cement showed some good correlations with the APT test results, except for the SFDR layer stabilized with cement and fly ash.
- The FEM analysis results showed that the section with cement and fly ash stabilized base exhibited lower HMA rutting than the section with only cement.
- The FEM analysis results indicated that the horizontal strain was higher in the section with 3-in. AC than the section with the 4.5-in. AC.
- Both FEM and PMED outputs confirm that in a thick pavement structure consisting of a thin AC wearing course over a thick SFDR base layer, some of the surface deformations can be attributed to the compression of the lightly stabilized base layer.

9.2 Recommendations

The following are the recommendations regarding SFDR mixture design, AC overlay design, and future work:

- The SFDR mixture design is recommended to be done following the guidelines from the US Army Corp of Engineers and PCA. Moisture susceptibility evaluation using the vacuum saturation test is highly recommended.
- The 1993 AASHTO Pavement Design Guide for SFDR flexible pavement design is recommended. The SFDR base layer coefficients should be obtained from the nomographs in the Guide based on the unconfined compressive strengths.
- While the APT and FEM analysis provided detailed data for investigating the SFDR-based pavements, long-term field monitoring is recommended to validate these findings over the pavement's life cycle. This monitoring will help capture the effects of

environmental factors, providing data that can further refine design models and guidelines.

- Future studies should consider refining the constitutive model for the SFDR mixture in the FEM analysis.
- A comprehensive life-cycle cost analysis would provide a more realistic understanding of the economic benefits of using stabilized FDR materials over conventional rehabilitation techniques, further supporting the case for broader adoption of SFDR in pavement rehabilitation projects.

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Appendix A - Fly Ash and Portland Cement Lab Reports

 KCPC MATERIALS LAB Kansas City Performance Center 15100 E. Courtney Atherton Rd. Sugar Creek, MO 64056 Phone: (816)257-4093 Fax: (816)257-5930	<table border="0" style="width: 100%;"> <tr> <td style="width: 50%;">REPORT DATE</td> <td>09/24/2018</td> </tr> <tr> <td>FLY ASH SOURCE</td> <td>Iatan</td> </tr> <tr> <td>COMPOSITE START DATE</td> <td>8/16/2018 10:09:00 AM</td> </tr> <tr> <td>COMPOSITE END DATE</td> <td>8/31/2018 10:09:00 AM</td> </tr> <tr> <td>SAMPLE IDENTIFICATION</td> <td>IA1180816-0831</td> </tr> </table>	REPORT DATE	09/24/2018	FLY ASH SOURCE	Iatan	COMPOSITE START DATE	8/16/2018 10:09:00 AM	COMPOSITE END DATE	8/31/2018 10:09:00 AM	SAMPLE IDENTIFICATION	IA1180816-0831
REPORT DATE	09/24/2018										
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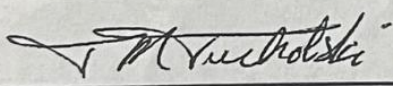
CHEMICAL ANALYSES

Analysis	Value	ASTM C 618 CLASS C	AASHTO M295 CLASS C
SiO ₂ (silicon dioxide), %	40.54		
Al ₂ O ₃ (aluminum oxide), %	18.97		
Fe ₂ O ₃ (iron oxide), %	6.23		
SiO ₂ +Al ₂ O ₃ +Fe ₂ O ₃ , %	65.74	50 min	50 min
CaO (calcium oxide), %	23.53		
MgO (magnesium oxide), %	4.73		
SO ₃ (sulfur trioxide), %	1.16	5.0 max	5.0 max
Moisture content, %	0.00	3.0 max	3.0 max
Loss On Ignition, %	0.35	6.0 max	5.0 max
Na ₂ O (sodium oxide), %	1.52		
K ₂ O (potassium oxide), %	0.49		

PHYSICAL ANALYSES

Analysis	Value	ASTM C 618 CLASS C	AASHTO M295 CLASS C
Fineness, amount retained on #325 sieve, %	23.8	34 max	34 max
variation, points from average	0.5	5 max	5 max
Density, g/cm ³	2.62		
variation from average, %	-0.4	5 max	5 max
Strength Activity Index with Portland Cement at 7 days, % of cement control	89		
Water Requirement, % of cement control	96	105 max	105 max
Soundness, autoclave expansion or contraction, %	0.04	0.8 max	0.8 max

We hereby certify that the fly ash represented by the above chemical and physical analysis meets the requirements of ASTM C 618 and AASHTO M 295.



09/24/2018

Tina M. Tucholski
KCPC Laboratory Manager

ASTM C 618 Note 1 - Finely divided materials may tend to reduce the entrained air content of concrete. Hence, if a mineral admixture is added to any concrete for which entrainment of air is specified, provision should be made to ensure that the specified air content is maintained by air content tests and by use of additional air-entraining admixture or use of an air-entraining admixture in combination with air-entraining hydraulic cement.

Figure A.1 Class C Fly Ash KCPC Report

ASH GROVE CEMENT COMPANY



1801 North Santa Fe
P.O. Box 519
Chanute, KS 66720
Phone: 620-433-3500
Fax: 620-431-3282

LOT # 2757

Type I/II (Low Alkali)

Production Period: July 1 thru July 31, 2019

Date: 8/12/2019

The following information is based on average test data during the production period. The data is typical of cement shipped from the Chanute, Kansas plant. Individual shipments may vary.

STANDARD REQUIREMENTS ASTM C150/C150M-16, Tables 1 and 3

CHEMICAL				PHYSICAL			
Item	A.S.T.M. Test Method	Spec. Limit	Test Result	Item	A.S.T.M. Test Method	Spec. Limit	Test Result
SiO ₂ (%)	C114	A	21.02	Air content of mortar (volume %)	C185	12 max	6
Al ₂ O ₃ (%)	C114	6.0 max	4.71	Fineness (cm ² /g):			
Fe ₂ O ₃ (%)	C114	6.0 max	3.15	Air permeability	C204	2600 min	4020
CaO (%)	C114	A	65.00	Autoclave expansion (%)	C151	0.80 max	0.02
MgO (%)	C114	6.0 max	2.55	Compressive strength (psi)			
SO ₃ (%)	C114	3.0 max	3.23	1 Day	C109	A	2320
Loss on ignition (%)	C114	3.5 max	2.58	3 Days	C109	1740 min	3840
Na ₂ O (%)	C114	A	0.19	7 Days	C109	2760 min	4710
K ₂ O (%)	C114	A	0.58	Time of setting (minutes)			
Insoluble Residue (%)	C114	1.5 max	0.40	(Vicat)			
CO ₂ (%)	C114	A	1.85	Initial: Not less than	C191	45	110
Limestone (%)	C114	5.0 max	4.7	Not more than		375	110
CaCO ₃ in limestone (%)	C114	70 min	90.32	Mortar Bar Expansion	C1038	E, B	0.005
Potential compounds (%) ^D				Specific Gravity	C188	A	3.15
C ₃ S	C114	A	52				
C ₂ S	C114	A	21				
C ₃ A	C114	8.0 max	7				
C ₄ AF	C114	A	9				
C ₃ S + 4.75 C ₃ A	C114	100 max	85				

OPTIONAL REQUIREMENTS ASTM C150/C150M-16, Tables 2 and 4

CHEMICAL				PHYSICAL			
Item	A.S.T.M. Test Method	Spec. Limit	Test Result	Item	A.S.T.M. Test Method	Spec. Limit	Test Result
C ₃ S + C ₃ A (%)	C114	A		False set (%)	C451	B	86
Equivalent alkalis (%)	C114	0.60	0.57	Heat of hydration (kJ/kg)			
				7 days	C186	A	

A = Not applicable.

B = Test result provided for information only.

C = Test results for this period not available.

D = Adjusted per Annex A1.6 M85

E = Required only if the SO₃ exceeds 3.0, in which case the expansion shall not exceed 0.020% at 14 days.

We certify that the above described cement, at the time of shipment, meets the chemical and physical requirements of ASTM C150/C150M-17 (Types I/II) and AASHTO M85-17 (Type I/II), or (other) _____ specification.

Signature: Marc D. Melton
Marc D. Melton
Title: Chief Chemist

Figure A.2 Type I/II Portland Cement Ash Grove Report

Appendix B - Standard Proctor Test Results

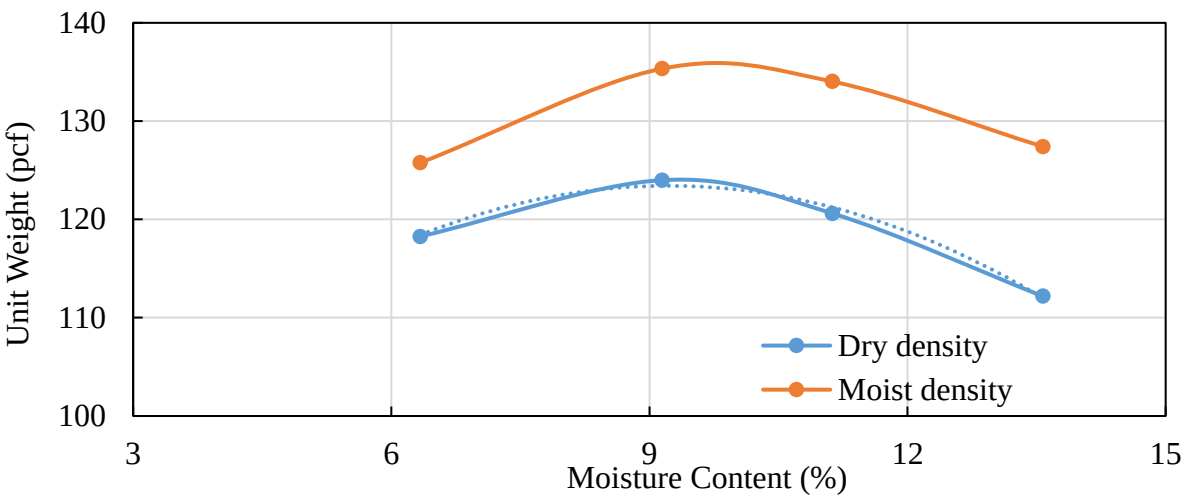


Figure B. 1 FDR Material with 2% A-7-6 Soil

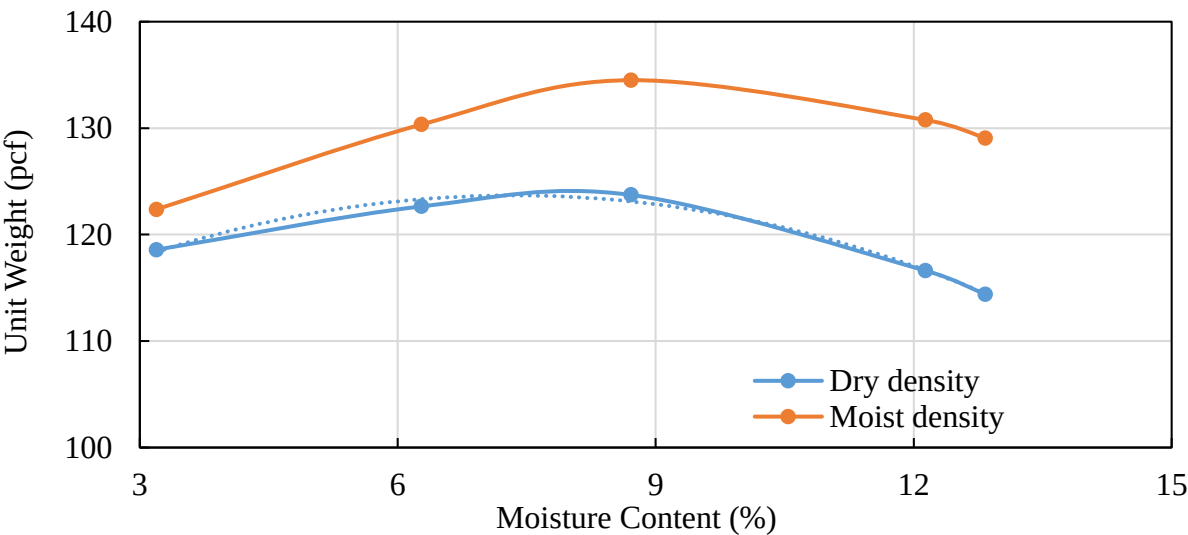


Figure B. 2 SFDR Mixture with 10% Fly Ash

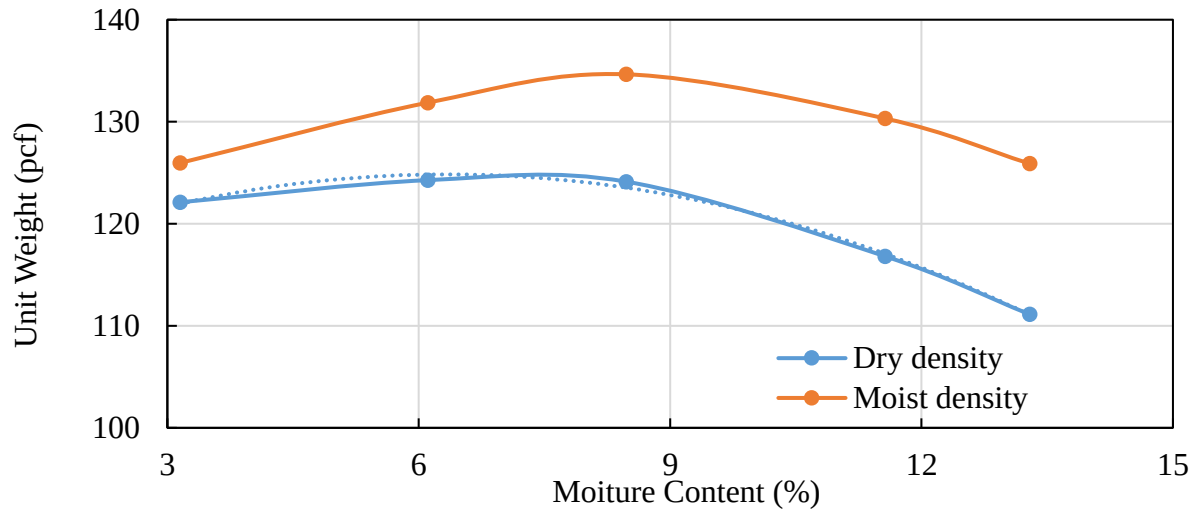


Figure B. 3 SFDR Mixture with 20% Fly Ash

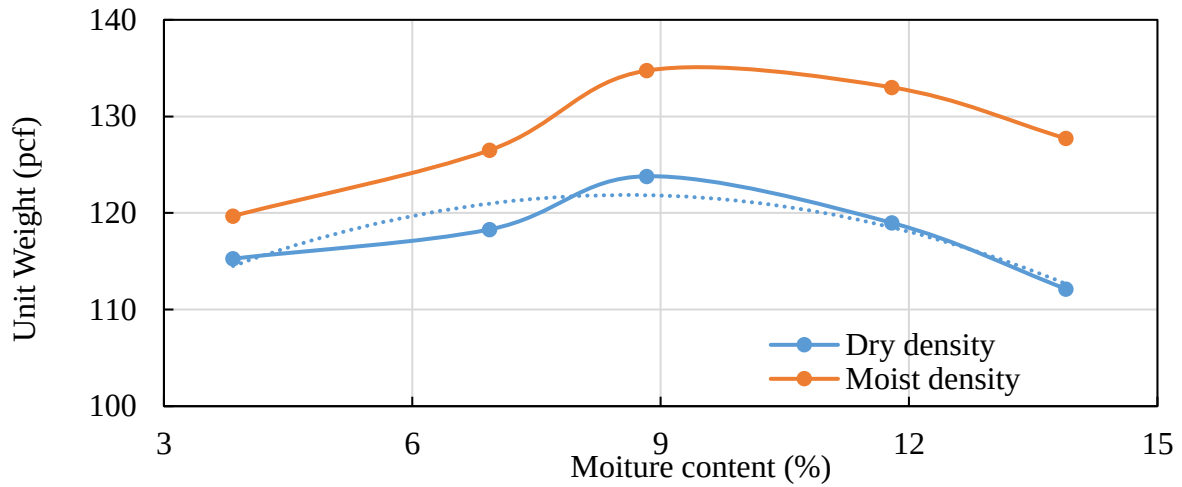


Figure B. 4 SFDR Mixture with 10% Cement

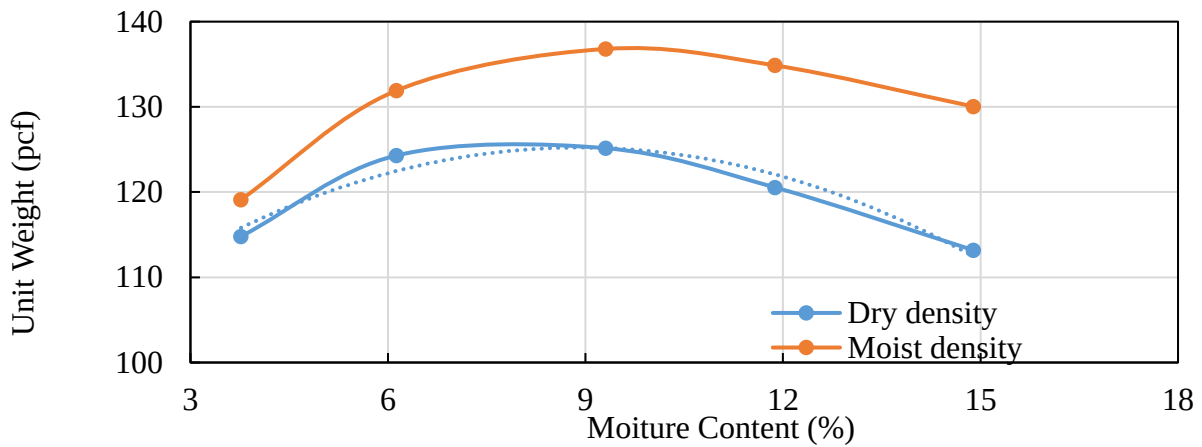


Figure B. 5 SFDR Mixture with 5% Cement

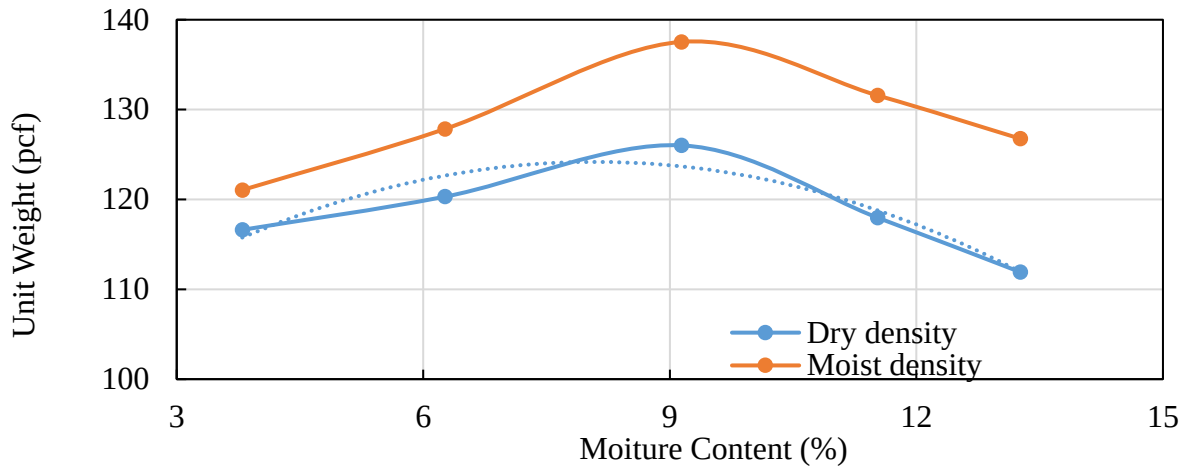


Figure B. 6 SFDR Mixture with 5% Cement & 5% Fly Ash

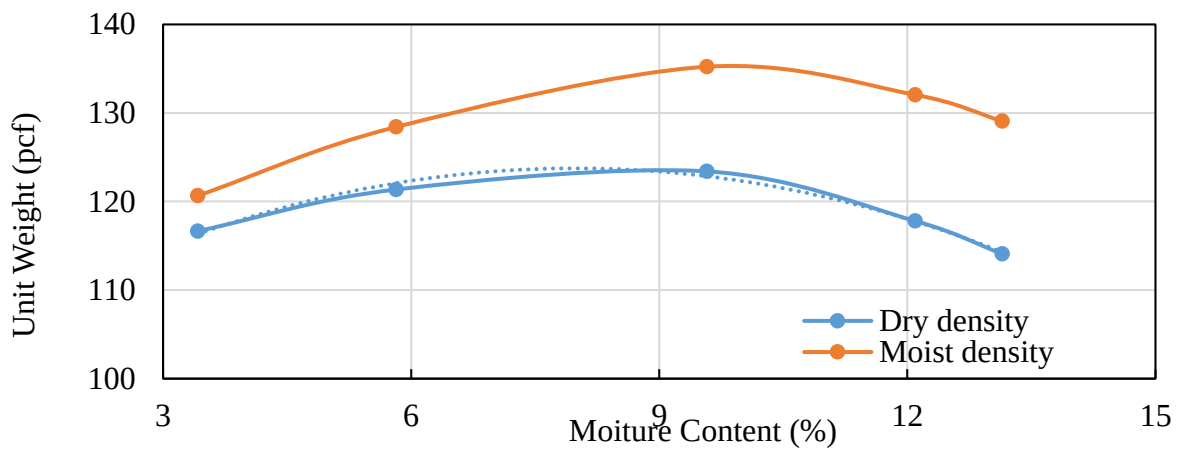


Figure B. 7 SFDR Mixture with 4% Cement & 6% Fly Ash

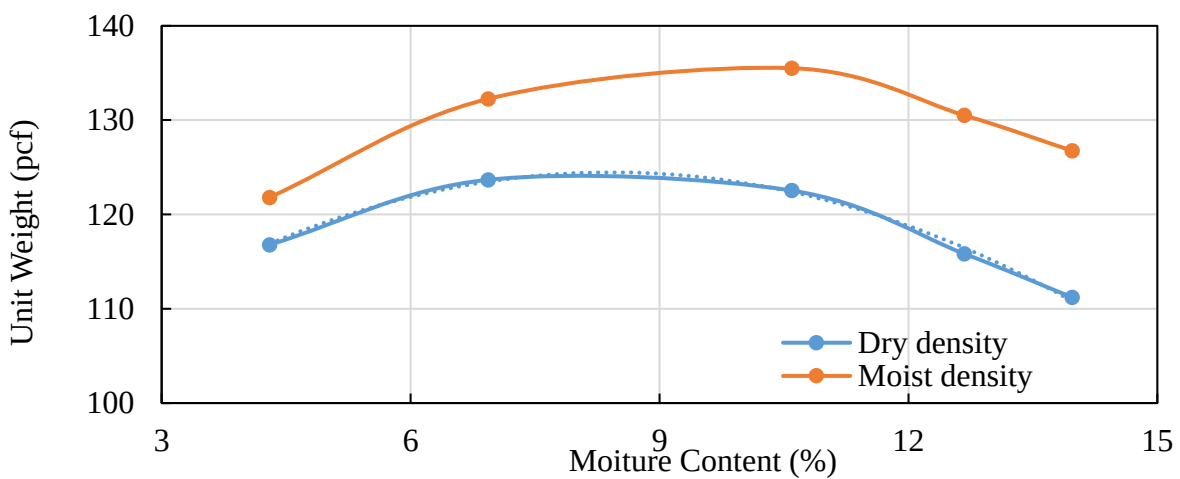


Figure B. 8 SFDR Mixture with 3% Cement & 7% Fly Ash

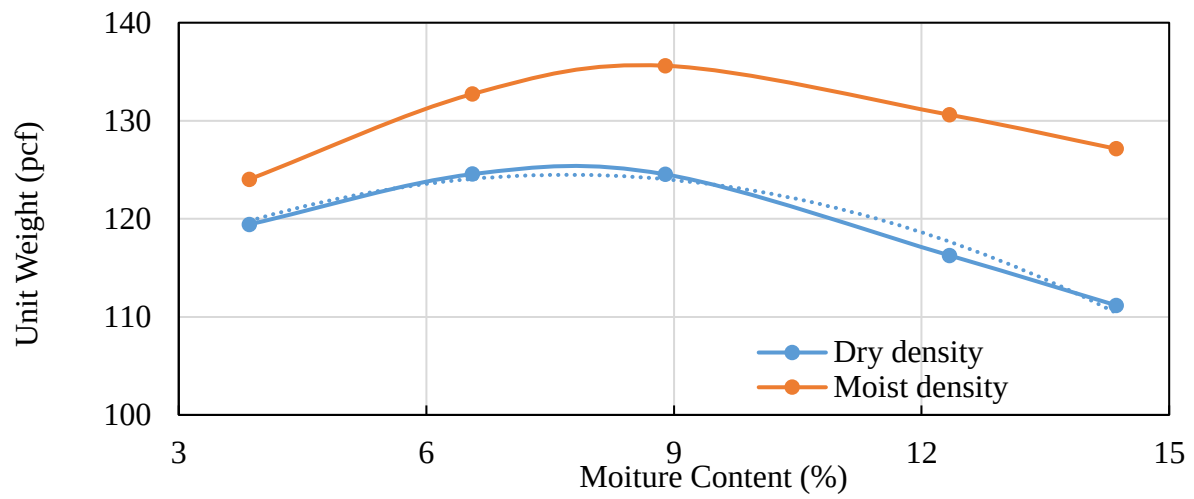


Figure B. 9 SFDR Mixture with 2% Cement & 8% Fly Ash