Civil and Environmental Engineering Department

Laboratory and Field Investigation of US Highway 49 Portland Cement Stabilized Full Depth Reclamation

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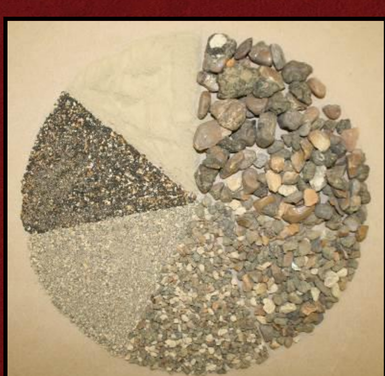

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16. Abstract

This report's primary objective was to study FDR performed on US Highway 49 (US 49) in Madison county, Mississippi for purposes of evaluating properties and performance. Several aspects of this report are effectively a case study of high traffic FDR, while other aspects are a controlled parametric laboratory investigation not necessarily intended to interface directly with US 49, rather were performed to shed light on specific issues associated with high traffic FDR. Field data is presented for FDR activities from construction through 53 months of service. Laboratory data is presented for material samples collected during US 49 construction and evaluated in the laboratory. Three aspects make US 49 appealing as a case study: 1) the highly variable and large amount of particles finer than 75 μm; 2) the relatively deep reclaimed depth of 16 in; and 3) the presence of numerous fine particles in a relatively deep reclaimed layer used for a high-traffic application. Laboratory and field testing suggest the US 49 FDR is performing well under high traffic. Specific findings are that: 1) US 49 field densities (as measured on cores) comfortably met 97% Proctor density requirements; 2) Density in the top 6 inches of the FDR layer was around 7 pcf higher than density in the lower six inches; 3) After 53 months of service, a representative unconfined compressive strength was around 400 psi, and a representative elastic modulus was around 200 ksi (this is a lower strength to modulus relationship than would be used in current Level 2 mechanistic-empirical pavement design); 4) a reasonable a_2 layer coefficient suitable for the 1993 AASHTO Pavement Design Guide was found to be 0.30.

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LIST OF SYMBOLS AND ACRONYMS

AADT	Average Annual Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt concrete
ADT	Average Daily Traffic
ANOVA	Analysis of Variance
APA	Asphalt Pavement Analyzer
ARRA	Asphalt Recycling and Reclaiming Association
BCD	Burns Cooley Dennis, Inc.
BIA	Bureau of Indian Affairs
C	FWD temperature correction factor
CD	Curing durability
C_I	Cement index: term previously used by MDOT to represent cement content
C_w	Cement content expressed as a percentage of dry reclaimed material mass
CIR	Cold in-place recycling
CP1	Curing protocol 1
CP2	Curing protocol 2
CP3	Curing protocol 3
CP4	Curing protocol 4
COV	Coefficient of Variation
CSRB	Cement-stabilized reclaimed base; a specific type of FDR
D	Specimen diameter
D_{Avg}	Average specimen diameter
D_p	Total pavement thickness for FWD analysis
D_1	Asphalt concrete thickness for FWD analysis
D_2	Base layer thickness for FWD analysis
DOT	Department of Transportation
E*	Dynamic modulus
E	Elastic Modulus
FDR	Full-depth reclamation
FHWA	Federal Highway Administration
FWD	Falling Weight Deflectometer
G	Gradation
GAB	Graded aggregate base
GDOT	Georgia DOT
G-V	Gradation variability
GV	Designation representing Holcim (US)'s St. Genevieve Plant
G_{sb}	Bulk specific gravity
H	Specimen height
H_{Avg}	Average specimen height
HMA	Hot mixed asphalt
IDT	Indirect tensile
LL	Liquid Limit
LOI	Loss on ignition
M	Total specimen mass (solid material and moisture)

MDOT	Mississippi Department of Transportation
MSU	Mississippi State University
MEPDG	Mechanistic Empirical Pavement Design Guide in Any Version or Form
MEPDG	2008 Interim Edition of the Mechanistic Empirical Pavement Design Guide
MS	Mississippi
MT	Mississippi Test Method
M_R	Resilient Modulus
MRI	Mean Roughness Index
N_G	Number of gyrations
$N_{G\text{-}Avg}$	Average number of gyrations
NMAS	Nominal maximum aggregate size
OMC	Optimum moisture content obtained from a Proctor compaction test
P_{200}	Percent passing 0.075 mm (No. 200) sieve
PCC	Portland cement concrete
PCI	Pavement Condition Index
PCR	Pavement Condition Rating based on MDOT protocols
PG	Performance Grade
PI	Plasticity Index
PL	Plastic Limit
RAP	Reclaimed asphalt pavement
SGC	Superpave Gyratory Compactor
SN	Structural number corresponding to the *1993 Guide*
SN_{eff}	Effective structural number corresponding to the *1993 Guide*
SS	State study
SSD	Saturated Surface Dry
S_t	Tensile strength measured by an IDT test
ST	Strength versus time
St. Dev.	Standard deviation
UC	Unconfined compression
UCS	Unconfined compressive strength
US	United States
US 49	United States Highway 49
VR1	Visual rating of coarse graded
VR2	Visual rating of average graded
VR3	Visual rating of fine graded
WT	Wheel tracking
a_i	Coefficient for AASHTO 1986 or 1993 guides for a given pavement layer
a_1	Coefficient for asphalt concrete layer in *1993 Guide*
a_2	Coefficient for base layer in *1993 Guide*
d_r	FWD measured deflections at a radial distance (*r*) from the center of loading
$d_{0\text{-}68}$	FWD measured deflections under the center of loading adjusted to 68 °F
mils	Unit of measure; 1 mil = 0.001 inch
n	Number of replicates
$w_{\%}$	Moisture content expressed as a percentage
$w_{\%\text{-}F}$	Moisture content of the FDR material immediately prior to compaction
$w_{\%\text{-}M}$	Moisture content measured for laboratory prepared specimens

1993 Guide	1993 AASHTO Guide for Design of Pavement Structures
γ	Target density for laboratory compaction: $\gamma_d[1 + OMC]$ with OMC as decimal
γ_d	Maximum dry density obtained from a Proctor compaction test
γ_{dry}	Dry density of any compacted specimen; does not necessarily correspond to γ_d
$\gamma_{dry\text{-}T331}$	Density measured on field cores by AASHTO T331 vacuum sealing
$\gamma_{dry\text{-}1\ pt}$	Dry density obtained from a 1 point field Proctor test
γ_T	Total density calculated from measured M, H_{Avg}, and D_{Avg} values
$\gamma_{T\text{-}Avg}$	Average total density calculated from measured values
$\gamma_{\%\text{-}Avg}$	γ_T divided by γ expressed as a percentage

CHAPTER 1-INTRODUCTION

1.1 General and Background Information

Full-depth reclamation (FDR) is a technique that has existed for around thirty years. FDR is one of a few in-place recycling techniques that have begun to gain more widespread attention as there has been an emphasis shift from construction of new pavements to rehabilitation of existing pavements. There is no evidence that this trend will not continue over the next several years with budgetary limitations and ever increasing sustainability interests being key factors in the paving community.

FDR's maturity (i.e. 30 years of use), however, should not be mistaken for a fully-developed concept. This is especially true since in recent years in-place recycling markets have expanded and can include high-traffic routes. It is not beyond reason to expect demands on techniques such as in-place recycling to continue to increase. While the concept of in-place recycling is well understood (and there have been several years of documented successful use of in-place recycling), case studies with longer-term documented performance in higher-traffic applications would be an addition to the in-place recycling body of knowledge.

1.2 Objectives and Scope

This report's primary objective was to study FDR performed on US Highway 49 (US 49) in Madison County, Mississippi (MS) for purposes of evaluating properties and performance. Several aspects of this report are effectively a case study of high-traffic FDR, while other aspects are a controlled parametric laboratory investigation not necessarily intended to interface directly with US 49, rather were performed to shed light on specific issues associated with high-traffic FDR. Field data is presented for FDR activities from construction through 53 months of service. Laboratory data is presented for material samples collected during US 49 construction and evaluated in the laboratory (sometimes after processing that could affect original properties). Three project specific aspects make US 49 appealing as a case study: 1) the highly variable and large amount of particles finer than 75 μm; 2) the relatively deep reclaimed depth of 16 inches; and 3) the presence of numerous fine particles in a relatively deep reclaimed layer used for a high-traffic application.

The information presented has the potential to be important for multiple reasons. First, more information is needed regarding how to characterize FDR, field data and associated performance where in-place properties are available. Second, use of FDR on US 49 was not originally planned; the original plan was to use cold in-place recycling (CIR) throughout the project. Challenges during construction were overcome by FDR (instead of CIR) use in half of the project. This paper documents how FDR targeting a depth of 16 inches was able to alleviate the challenges encountered. Third, information is provided in the context of the 1993 AASHTO Guide for Design of Pavement Structures (AASHTO 1993), and the Mechanistic-Empirical Pavement Design Guide-Interim Edition (MEPDG, 2008). These two pavement design approaches are referred to hereafter as the *1993 Guide* and the *MEPDG*, respectively. Note the Mississippi DOT (MDOT) was using layer coefficient based pavement design (i.e. philosophies such as those in the *1993 Guide*) at the time of US 49 construction, and the *MEPDG* is anticipated to be the prevailing pavement design guide within MDOT in the future. Overall, the potential for this report to have future importance to MDOT lies in the possibility of using FDR in high traffic applications in the future and before the efforts presented in this report, MDOT had not performed a comprehensive study where material properties were characterized in a manner to provide design, construction, and performance guidance.

This report was part of State Study 250 (SS 250), which was reported in three volumes. This report (Volume 1) focuses on in-place recycled material consisting of a wide variety of materials from asphalt concrete to fine grained soil; i.e. FDR. Volume 2 compliments Volume 1 in that it is also related to in-place recycling, but addressed cold-in-place recycling (CIR). Volume 3 is not related to in-place recycling, rather studies characteristics of thin lift asphalt concrete joints over time. Specific aspects of SS 250 addressed in this report are summarized in the remainder of this section. Note that the descriptions provided are for Modification 1 of the FDR component of SS 250's Scope of Work, which was reviewed and approved by MDOT in late 2014.

A literature review was performed for insight related to key facets of this research effort. Field evaluation of US 49 was performed that included Falling Weight Deflectometer (FWD), profiler, and coring operations. The data available was analyzed by Mississippi State University (MSU). Laboratory testing was performed by MSU on bulk samples of FDR material obtained during US 49 construction, and also on cores taken from US 49. Additionally, test data was provided by Burns Cooley Dennis, Inc. (BCD) that was investigated as part of this study. The overall investigation contained in this report included: 1) a field investigation of US 49 (in-place density, strength (tensile and compressive), elastic modulus, wheel tracking resistance, distresses, and structural capacity were all considered); 2) gradation variability effects on strength and wheel tracking resistance; 3) strength gain with time; 4) effects of curing on properties of different gradations; and 5) strength variability.

CHAPTER 2-LITERATURE REVIEW

2.1 Overview of Literature Review

Review of literature was performed with emphasis on FDR and similar activities. Parameters of key interest to this review of literature were FDR layer thicknesses, gradations of FDR materials, traffic levels where FDR was used, FDR performance, and pavement design inputs related to FDR. Findings are presented in the remainder of this chapter.

2.2 FDR Usage and Economics

Interest in FDR, as with many technologies, is often regional. For example, Maine, Connecticut, and Vermont have utilized FDR for over ten years (e.g. Mallick et al. 2002a, 2002b). Similarly, Lewis et al. (2006) reported that the Georgia DOT's (GDOT's) first FDR project was over ten years ago (CR 52 in 2004). Mississippi, on the other hand, is less experienced with FDR by comparison (e.g. Strickland 2010).

Multiple sources report cost savings with FDR. The Asphalt Recycling and Reclaiming Association (ARRA) states that cost savings can range from 20 to 40% for FDR relative to conventional approaches, and energy savings are on the order of 40 to 50% (BARM 2001). Lewis et al. (2006) reported a 42% cost savings with FDR.

2.3 FDR Project Parameters

Cox and Howard (2013) is a white paper performed as part of State Study 250, where a total of 99 in place recycling references were summarized that were written between 1982 and 2013 (81 CIR references and 18 FDR references). The white paper was divided into five parts: traffic levels, layer thicknesses, moisture contents, binder types, and gradations. Each is summarized as follows. Nine FDR AADT levels were reported by 7 references and ranged from 600 to 4,260, with an average AADT of 2,248. Sixteen FDR layer thickness values were reported by 10 references and ranged from 6 to 12 in, with an average layer thickness of 8.5 in. The upper end of the range found by Cox and Howard (2013) matches that of the synthesis of highway practice summarized in Epps (1990) where typical recycling depths were 4 to 12 in.

Continuing with the findings of Cox and Howard (2013), dozens of FDR optimum moisture content (OMC) values were reported by 8 references and ranged from 4.5 to 10.3%, with an average OMC value of 7.2%. Numerous FDR binders and combinations were reported by 16 references. A variety of additives were used as binding agents in FDR, and in some cases, combinations of additives were used (e.g. asphalt emulsion and portland cement). When only portland cement was used, 3 to 7% by mass was the range of dosage rates reported. Ten FDR gradations were reported by 4 references and percent passing the 0.075 mm sieve (i.e. fines) ranged from 3.5 to 8.9%, with an average fines content of 7.1%.

2.4 FDR Field Performance

Several studies were identified that address various aspects of CIR performance. These studies were published between 1991 and 2009. The remainder of this section presents these studies in chronological order.

Shepard et al. (1991) performed FDR with calcium chloride, which in and of itself is beyond the scope of this effort, but the project did provide a few noteworthy items. First, it was indicated that FDR is for depths up to 8 in and made no mention of deeper FDR layers. Second, the FWD was utilized to document improvement from FDR of 40 to 50% increase in allowable standard traffic loadings based on analysis from the 1986 AASHTO Guide.

Mallick et al. (2002b) evaluated a pavement in western Maine (AADT = 1,920 with 23% trucks) with half mile long FDR sections: no additive, 2.2% emulsion, 3.4% emulsion, 3.4% emulsion with 2% lime, and 5% portland cement. The FDR material consisted of 4 in of existing asphalt concrete and 2 in of the underlying granular base (8.1% fines). Evaluations occurred over a one year period that included laboratory testing of materials sampled from the project, FWD testing, and coring. FWD testing and coring for resilient modulus measurement occurred 3 months after construction. DarWin 3.01 was used to determine subgrade modulus, effective pavement modulus, and structural number. Project objectives were to determine a suitable number of gyrations for designing FDR with the SGC, evaluate potential benefits of different stabilization strategies, and determine suitable structural numbers for pavements with different types of additives. Mallick et al. (2002b) recommended that FDR material be compacted to 50 gyrations during mix design and that a minimum of 98% of the density of in-place loose mix samples compacted to 50 gyrations should be achieved during construction. The following layer coefficients were reported: 0.28 for the 5% cement section, 0.37 for the 3.4% emulsion and 2% lime section, and 0.24 for the 2.2% emulsion section (data was not available for the other sections).

Johnson et al. (2006) documented efforts by the Rocky Mountain Region of the Bureau of Indian Affairs (BIA) regarding a demonstration project conducted in 2003. Non-destructive testing and visual preliminary performance of different test sections were reported, alongside construction and economic considerations. The authors noted that others have shown that typical FDR projects can be performed in less than half of the time of conventional maintenance.

The demonstration reported by Johnson et al. (2006) was on a route with an average daily traffic (ADT) of 1,100 with 5% trucks. Five test sections were constructed including one with 3% portland cement and another with 6% portland cement. FDR materials had 7 to 8% fines and layers were nominally 6 in thick. The demonstration was favorable to FDR and documented it can be a cost-effective alternative to conventional HMA overlays. The demonstration also showed FDR to cause only minimal disruption to the traveling public.

Lewis et al. (2006) studied cement-stabilized reclaimed base (CSRB) for the Georgia Department of Transportation (GDOT). GDOT defines CSRB as a portland cement stabilized sand-clay base underneath HMA and in their terminology, CSRB is a specific type of FDR. The FDR portion of the CR 52 project was a 1.1 mile section in Long county. CR 52 is a rural county road, but had an ADT of 3,375 with 15% trucks. A portland cement dosage of 6% was used. CSRB was being used by GDOT on non-state routes only. The article states that GDOT was the first agency to develop a specification for the use of CSRB in FDR. The article implies use of a 0.20 layer coefficient (a_2) for cement-stabilized base. Lewis et al.

(2006) noted that a 450 psi UCS, which was the CR 52 CSRB design strength, was required in the laboratory to achieve 300 psi in the field. The FDR section had 1.5 in asphalt concrete over 6 in CSRB over 1.25 in untreated sand-clay base. A comparative section in another Georgia county had 7 in of asphalt concrete over 6 in graded aggregate base (GAB), which was stated to be more conventional. Seven cores taken after 7 days of curing had an average UCS of 409 psi, a standard deviation (St. Dev.) of 60 psi, and a range of 348 to 532 psi. Actual thicknesses of the FWD cores were 6.0 to 6.25 in. In place density measured on cores was 98.3 to 104.5% (average of 102%) of T99 (specification in place was 98% or more).

Lewis et al. (2006) performed FWD testing soon after construction, and followed up approximately 9 months after construction with additional FWD testing. Temperature corrected data normalized to 9 kips was reported. Soon after construction, deflections were significantly less in the FDR section than they were originally. The FDR section was less variable than other portions of CR 52 that only got an overlay (St. Dev. of 5 versus 2 mils). Maximum deflections for the FDR section soon after construction were 8 to 12 mils, which was comparable to the comparer project (7 in asphalt over 6 in GAB) at around 10 to 14 mils.

The 9 month follow up evaluation presented in Lewis et al. (2006) showed no rutting, signs of cracking (isolated to 3 areas), and UCS values from 426 to 619 psi (average of 497 psi). FWD testing of the FDR section was slightly lower than just after construction at 8 to 11 mils. The FDR project was successful, and GDOT office of state aid endorsed CSRB as a viable option.

Smith et al. (2008) documented GDOT's first lime stabilized FDR at CR9 on Huckabee Road in 2006. Underlying failures were present in micaceous clay-silt soils. Reclamation was to a 14 in depth with lime applied at 6% by volume. The process resulted in substantial improvement in soil properties and structural strength. The original pavement had 1.5 in of asphalt over 6 in of unbound graded aggregate base. CR9 had an average clay content of 37%, an average liquid limit of 40, and an average percent fines of 52. The depth of treatment (14 in) was determined by equipment limitations of traditional reclamation equipment; deeper treatment would require more extensive rehabilitation and increase costs.

Initially, the contractor proposed a construction procedure with one pass of the rotary mixer and compaction of the entire section as one lift. This proposal exceeded GDOT's allowable lift (8 in), but the contractor was granted a test section. The test section did not have adequate mixing or compaction, and as such a two lift compaction procedure was used (7 in per lift). FWD testing occurred before and after reclamation. Deflections 60 in away from FWD load center were 0.5 to 1.5 mils. The authors concluded hydrated-lime stabilization to 14 in deep was successful. An asphalt overlay of 3.25 in was placed over the FDR layer.

Syed (2009) reported field performance of over 75 FDR projects in 8 states, with an average age of 9 years and an age range of 3 to 26 years. FDR layer thicknesses were 6 to 12 in for all projects. Annual daily traffic for candidate project sections was stated to vary from less than 1,000 to 25,000 with truck percentages as high as 16% (no supporting traffic data was provided). Performance evaluations consisted of interviewing agency/owners, performing visual surveys, taking cores (23 were successfully obtained), and performing strength measurements on these cores.

Pavement Condition Index (PCI) values ranged from 43 to 100, with an average of 89 (85 to 100 is an excellent rating) in the projects documented by Syed (2009). UCS varied from 260 to 2,110 psi, with an average of 914 psi. Typical designs were a 7-day UCS of 400-

600 psi, and traditional compaction requirements were 95 to 98% of standard Proctor (i.e. ASTM D558). Four cores experienced seismic modulus measurements and were then tested for UCS. UCS values ranged from 465 to 1,239 psi, and the resilient modulus values that corresponded to these extremes were 442 and 1,519 ksi. In consistent units, resilient modulus values exceeded UCS values by ratios of approximately 525, 925, 950, and 1225 (average ratio of around 900). Note that the MEPDG permits use of a ratio of 1200 to relate UCS to elastic modulus as a Level 2 input.

Durability was discussed subjectively by Syed (2009); no data was provided. It was noted that many agencies are trying to address durability during design since it is the key issue with cement FDR designs. One example is having pairs of UCS specimens, one subjected to standard curing and the other conditioned by soaking for a few hours or being frozen, and ensuring that strength loss due to conditioning is reasonable (e.g. 25% or less strength loss). It was also stated that the minimum cement content should be based on a mixture's ability to pass ASTM D559 and D560, or the Tube Suction Test.

Miller et al. (2011) evaluated an FDR project in New Hampshire during the winter and spring thaw periods that had been in service for around 2.5 years. The FDR was 8 in thick and consisted of 4 in asphalt concrete and 4 in of underlying base with 4% cement by mass. FWD testing during the spring thaw resulted in 2 to 16 mils of 9 kip normalized temperature adjusted deflection under the load plate and subgrade modulus values of 3 to 50 ksi. Results showed FDR as an economic alternative for roadway reconstruction and that FDR performed well during the spring thaw-weakened period.

2.5 FDR Pavement Design Inputs

Three studies were identified that address relevant FDR pavement design inputs. One of these studies made reference to several other studies that also contained relevant pavement design input information. The three studies identified were published between 2007 and 2011. The remainder of this section presents these studies in chronological order.

Thomas and May (2007) evaluated two laboratory-fabricated FDR materials: one with 25% RAP and 75% limestone and another with 75% RAP and 25% limestone. Four binder blends were tested with the 25% RAP blend: 5.5% emulsion and 5.5% emulsion plus 1% cement for two emulsions (A and B). The 75% RAP blend was tested with both emulsions at 3.7% (two blends) but was not tested with cement. Specimens were tested for dynamic modulus; mixtures with 1% cement had higher modulus at low frequencies (i.e. warmer temperatures).

An FDR layer coefficient of 0.25 was mentioned though there was no apparent supporting data. In general, Thomas and May (2007) focused on the lack of ability for the *1993 Guide* to perform comprehensively and, on the contrary, the promise of the *MEPDG*. It was stated that, at the time, the *MEPDG* considers FDR an unbound material (default modulus of 10 ksi) while dynamic modulus (E*) of mixtures tested ranged from 570 to 910 ksi at 21.1°C and 10 Hz, suggesting FDR should be given greater structural credit within the *MEPDG*.

Diefenderfer and Apeagyei (2010) evaluated the structural condition of three FDR demonstration projects on rural two-lane roadways with the following additives: 2.7% foamed asphalt plus 1% cement, 3.5% emulsion plus 1% cement, 5% cement, and 5% cement. AADT ranged from 1,500 to 3,700 with 10% trucks or less. The primary goals were to 1) evaluate structural condition time-dependency, and 2) develop layer coefficients for the *1993 Guide*. FDR reclaiming depths were 8 to 10 inches for all projects.

Diefenderfer and Apeagyei (2010) presented a table of cited FDR layer coefficients, which are also referenced herein. For foamed asphalt, layer coefficients were cited to be 0.18 (Romanoschi et al., 2004), 0.18 (Bemanian et al., 2006), 0.22 to 0.35 (Marquis et al., 2003), and 0.20 to 0.42 (Dai et al., 2008). For asphalt emulsion, Dai et al. (2008) reported 0.17 to 0.41. For fly ash, Wen et al. (2004) reported 0.23.

FWD testing was conducted by Diefenderfer and Apeagyei (2010) over the first year of service and analyzed in accordance with the *1993 Guide*. Emulsion and foamed asphalt deflections under the center of loading adjusted to 68 °F ($d_{0\text{-}68}$'s) decreased with time from approximately 27 to 12 mils (emulsion) and 14 to 7 mils (foamed asphalt); effective structural number (SN_{eff}) values increased from approximately 2.5 to 3.4 (emulsion) and 3.5 to 4.6 (foamed asphalt). Before FDR, $d_{0\text{-}68}$ ranged 14 to 18 mils, and SN_{eff} ranged 3.1 to 3.7. For one cement FDR project, $d_{0\text{-}68}$ before FDR ranged 19 to 23 mils, and SN_{eff} was 2.1. After FDR, $d_{0\text{-}68}$ ranged 6 to 9 mils, and SN_{eff} ranged 4.1 to 4.8. For the second FDR project, $d_{0\text{-}68}$ ranged 8 to 11 mils, and SN_{eff} ranged 3.9 to 4.7 after FDR. Little change was observed over time for either cement FDR project.

Using data from the last round of FWD testing (FWD data had largely stabilized for both bituminous and cementitious FDR at that point in time), average SN_{eff} and layer coefficients were calculated by Diefenderfer and Apeagyei (2010) alongside coefficient of variation (COV) values. Overall, layer coefficients were approximately 0.30 (foamed asphalt), 0.18 (emulsion), and 0.31 (cement). COV of layer coefficients ranged from 10 to 29% and was 22% on average.

Nantung et al. (2011) documented an FDR project on rural two-lane SR1 (AADT was 600 with 9% trucks) in Randolph County, Indiana. The FDR layer was 8 inches thick and was stabilized with 3% cement and 1.3 gal/yd^2 of emulsion. An FWD structural analysis was conducted over the first four years of service. The suggested layer coefficient was 0.22.

2.6 Summary of Literature Review

A summary of findings from this review is presented. FDR layer thicknesses were typically 6 to 12 in; one study was identified with a 14 in thickness. The US 49 target thickness of 16 in seems relatively deep, even for FDR. Gradations in literature were usually much coarser than US 49. Projects identified in literature often had less than 10% fines, whereas US 49 had 6 to 36% (average of 15%) fines. Traffic levels in literature for FDR were typically less than US 49. FWD data in literature agrees from a general trend perspective with the data presented in this report. AASHTO layer coefficients (i.e. a_i terms) for cement stabilized FDR were identified for three projects in two references; 5% cement was used in each case. Layer coefficients were 0.28, 0.31 (with a coefficient of variation, COV, of 25 to 29%), and 0.29 to 0.32 (COV of 20 to 24%). In consistent units, Resilient Modulus (M_R) values exceeded unconfined compressive strength (UCS) values in one referenced work by an average of around 900. Table 11-7 of MEPDG (2008) permits level 2 input of a ratio of 1200 to relate UCS to elastic modulus.

CHAPTER 3 - EXPERIMENTAL PROGRAM

3.1 Overview of Experimental Program

US 49 FDR activities were performed, chronologically speaking, in four phases. They were: 1) activities during construction performed almost entirely by MDOT and Burns Cooley Dennis, Inc. (BCD) where data was provided to MSU for interpretation and analysis in the context of the entire FDR monitoring effort; 2) laboratory work performed at MSU on bulk samples obtained during construction; 3) Falling Weight Deflectometer (FWD) testing performed over time on the project; and 4) a field evaluation performed during the fifth year after construction where coring, FWD, and distress evaluations were performed. Field activities performed during phases 3 and 4 (i.e. Monitoring US 49 in service) were often combined for assessment purposes and as such this chapter has been broadly organized into: construction activities, laboratory testing of bulk samples, and field monitoring activities.

3.2 US 49 Construction Activities

US 49 informally refers to MDOT Project No. NH-0008-03(032), between Flora and Yazoo City, MS that contained FDR and CIR sections and occurred during the 2010 construction season. Figure 3.1 is a map showing relevant sampling and testing activities for US 49 FDR; additional activities occurred with CIR that are only briefly mentioned herein. Figure 3.1 is further explained later in this chapter. Strickland (2010) documents construction procedures used during US 49; information was obtained from Strickland (2010) and other parties (e.g. MDOT, engineers, consultants) and compiled herein into a summary of US 49 construction activities. Specifically, Sections 3.2.1 to 3.2.4 present details of the US 49 project and construction, Section 3.2.5 discusses materials sampled during construction, and Section 3.2.6 details testing associated with construction activities.

3.2.1 US 49 Project Details

The US 49 project was conducted on a 9.2 mile section of four-lane divided highway. The bid price was around $15 million; final project costs were around $16.5 million. Two pavement structures, composite (i.e. asphalt concrete (AC) over portland cement concrete (PCC)) and full-depth AC, were present on US 49 prior to rehabilitation. The original jointed concrete slabs and full-depth AC sections were built in 1959 and 1980, respectively. Immediately prior to rehabilitation, several types of pavement distresses were present. Distresses included longitudinal cracking, potholes, transverse cracking with spalling, and rutting. Several patches existed in heavily distressed sections. The quantity and severity of distresses present, in MDOT's assessment, made US 49 a viable in-place recycling candidate since milling and overlaying was a less suitable option.

3.2.2 US 49 Construction Stages

US 49 was constructed in 3 stages, primarily because the project included replacement of two northbound bridges. In stage 1, southbound lanes adjacent to the northbound bridges to be replaced were in-place recycled then overlaid with a nominal 3 inch base lift of 19 mm NMAS PG 76-22 AC (further denoted base mix with detailed properties provided in the Volume 2 State Study 250 report). This was necessary to route traffic onto southbound lanes near the bridges in a head-to-head fashion, while allowing construction traffic to use northbound lanes. In stage 2, the remaining in-place recycling was conducted, which was most of the in-place recycling, and all in-place recycled material was overlaid with a nominal 3 inch thick lift of base mix. The two bridges were also re-constructed in stage 2. In stage 3, a nominal 1.5 inch thick surface lift of 9.5 mm NMAS PG 76-22 AC (further denoted surface mix with detailed properties provided in the Volume 2 State Study 250 report) was placed over the entire project.

Extended period lane closures were frequently used to facilitate construction in one lane and allow traffic in the adjacent lane. A lane under construction remained closed until in-place recycling was completed and the base mix was placed, at which point it was reopened and the other lane was closed. The only exception to this practice would have been near the bridge replacements where traffic was routed head-to-head on southbound lanes.

3.2.3 Original and Modified US 49 Construction Plan

The original US 49 plan was to perform CIR with depths of 6 to 9 inches depending on underlying materials (i.e. full-depth AC or PCC). However, during stage 2 of construction, problems were encountered in some full-depth AC areas where the existing subgrade was unable to support in-place recycling equipment. It was decided that, in order to compensate for the insufficient subgrade strength, stabilization depths needed to increase and a supplemental agreement was developed. Strickland (2010) noted future in-place recycling efforts should conduct more extensive coring and materials testing prior to construction.

Figure 3.1 shows all FDR areas of US 49 as built. FDR was conducted only in full-depth AC sections, while CIR proceeded as planned in all composite pavement sections and some full-depth AC sections. Figure 3.1 truncates the north end of the project since this report focuses on FDR. As built, the project was approximately 36.8 total lane miles; 18.7 of these lane miles were FDR (i.e. half of the project).

3.2.4 Full-Depth Reclamation Order of Construction Operations

Hall Brothers Recycling & Reclamation, Inc. performed US 49 in-place recycling activities. FDR specific processes were as follows. Step one was to mill and remove the top 3 inches of existing AC and provide a uniform grade. Step two (Figure 3.2a) was to spread portland cement with an auger system onto the milled surface; note cement is often spread onto pulverized material instead.

A Caterpillar PR-1000 cold planing unit was used to pulverize the top 7 inches of existing pavement and mix this material with cement (Figure 3.2b). The cold planing unit deposited material into a windrow that was spread as shown in Figure 2c. After spreading, a Caterpillar RM 500 was used to mix the spread windrow and underlying materials to a total depth of 16 inches (Figures 3.2d, 3.2e). Note that in Figure 3.2e the material had been shaped up to a point, and the windrow is visible in front of the shaped material. The RM 500 was only working up to the edge of the windrow. Water was incorporated at some point in the reclamation train, though the exact location is unknown. Water was not, to the knowledge of the authors, introduced by a top-down method from a water truck.

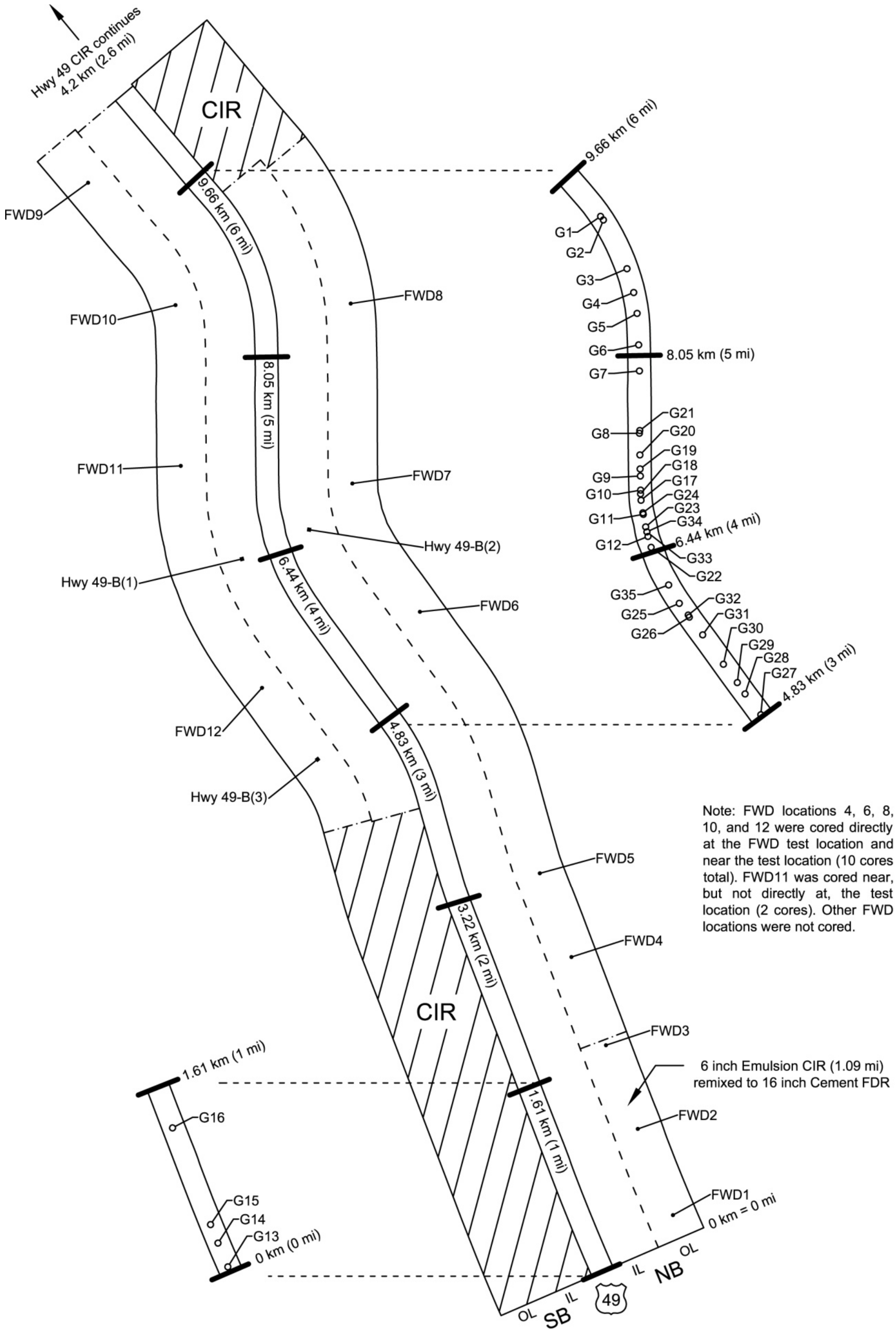


Figure 3.1. US 49 FDR Project Description

Figure 3.2. US 49 FDR Construction Photos

After full-depth mixing, material was smoothed then compacted in one lift using a Rex® 3-70A Compactor, which has steel wheels fitted with rectangular steel pads (Figure 3.2f). After this stage of compaction, a motor grader (e.g. Figure 3.2c) was used to smooth the surface left by the Rex® 3-70A. After smoothing, a Caterpillar CB-634D vibratory smooth drum steel wheel roller was used. Thereafter, a tack coat was applied to minimize moisture loss over the 7-day curing period. After curing and before overlaying, the FDR layer was milled, or trimmed, back to grade to counter material fluff. Full pay for in-place density required 97% of standard Proctor density; MDOT reported that density requirements were met across the entire project.

By November of 2010, all in-place recycling and AC base mix had been placed. Most of the FDR was placed from August to October of 2010. Public traffic began to use the entire route around November of 2010 with only the AC base mix in place. The AC surface mix was placed in July/August of 2011, and MDOT profiled the fully-completed project in September of 2011. Nominally, US 49 had 4.5 inches of AC over 16 inches of FDR.

3.2.5 Full-Depth Reclamation and Stabilization Materials

Three bulk samples were taken from FDR sections where ≈15.2 cm of AC and ≈25.4 cm of underlying materials were nominally reclaimed (total depth of ≈ 40.6 cm). Actual depths varied considerably throughout the project. This material was given the designation of Hwy 49B; "B" denotes FDR and "A" denotes CIR (all CIR efforts related to State Study 250 are reported in Volume 2). Locations of these bulk samples can be seen in Figure 3.1; Hwy 49 B(1), B(2), and B(3). A portion of each of these three bulk samples was taken by BCD, with the remainder going to MSU. The information in the next paragraph describes the material taken by MSU, and the paragraph thereafter describes material taken by BCD.

Individual FDR samples were labeled Hwy 49 B(1), (2), and (3). Individual sample sizes ranged from 280 to 330 kg each, which provided just over 900 kg of FDR material. Individual samples were stored in plastic drums prior to processing. Table 3.1 shows bulk (i.e. dry sieved with no water) gradations of each individual sample taken, and Figure 3.3 shows these materials visually.

Table 3.2 uses MSU terminology and summarizes test results collected at BCD. The materials exhibited a fairly low plasticity index (PI) at 4 to 7. Proctor compaction results presented in Table 3.2 were at a 6% cement content by mass of FDR material, denoted C_w. One item of note is the very large difference in the unwashed gradations performed by MSU and the water washed gradation performed by BCD. The FDR sample has a considerable amount of material adhered to itself well enough not to separate during dry sieving. This is not surprising, and is accounted for during batching activities discussed later in this chapter.

Table 3.1. Bulk (Unwashed) Gradations of FDR Bulk Samples from US 49 at MSU

Sieve Size (mm)	Hwy 49B(1)	Hwy 49B(2)	Hwy 49B(3)
12.5	91.4	94.9	91.6
9.5	84.8	88.9	84.9
4.75	64.8	68.0	67.4
2.36	49.7	51.4	56.4
0.075	5.1	6.3	15.9

Note: All material passed a 37.5 mm sieve and relative sizes between 12.5 and 37.5 mm were not evaluated.

Table 3.2. BCD Measured Properties for US 49 Bulk Samples

			Proctor Data[a]			Percent Passing (Water Washed)				
Hwy 49B	LL	PL	γ_d (pcf)	OMC (%)	C_w (%)	12.5 (mm)	9.5 (mm)	4.75 (mm)	2.36 (mm)	0.075 (mm)
(1)	24	19	116.5	11.1	6	---	---	---	---	---
(2)	25	18	119.8	9.8	6	---	---	---	---	---
(3)	25	21	116.8	9.0	6	100	90.5	77.1	68.9	38.1

a) Communication with BCD in July of 2015 indicated these values are likely adjusted for +12.5 mm material. Whether or not they are adjusted Proctor values is, however, not certain.

In addition to the three bulk samples, BCD sampled thirty five FDR locations during the course of the project where relatively small samples were taken to measure gradation, proctor density, and strength properties (See Figure 3.1 for locations). The primary initial interest in these samples was gradation (G), so these samples were labeled G1 to 35 and sample locations can be seen in Figure 3.1. G1 to G35 were sampled after cement addition meaning approximately 5% of the fines in the samples was cement. Note that distance along the project was reasonably well documented, but which of the four travel lanes was sampled was not documented.

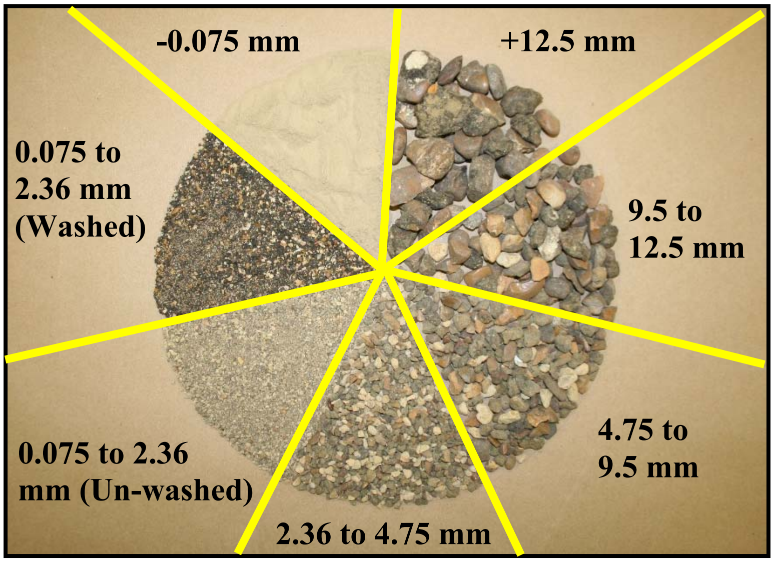


Figure 3.3. US 49 Full-Depth Reclamation Material Shown Visually

One ASTM C150 Type I-II portland cement supplied by Holcim (US), Inc. out of their St. Genevieve plant was used for all FDR activities. A cement index (C_I) of 6% was established by MDOT for the US 49 FDR project. A cement index of 6% translates to a cement content by mass (C_W) of approximately 4.5 to 4.8% for the gradations tested in the laboratory for this report. Table 3.3 provides properties of a cement sample from the St. Genevieve plant obtained by MSU and used for all testing. Note the Table 3.3 sample's time frame did not match US 49 construction activities. No records were available with respect to cement source used by MDOT or BCD for any testing performed by either group.

Table 3.3. Portland Cement Properties as Supplied by Holcim (US), Inc.

Cement	GV T I-II
Plant	St. Genevieve (Bloomsdale), MO
SiO_2 (%)	20.0
Al_2O_3 (%)	4.5
Fe_2O_3 (%)	3.1
CaO (%)	64.2
MgO (%)	2.3
C_3S (%)	62
C_2S (%)	9
C_3A (%)	6
C_4AF (%)	9
LOI (%)	2.7
Blaine (m^2/kg)	383
Vicat Initial (min)	90
1-day strength (MPa)	15.7
3-day strength (MPa)	27.5
7-day strength (MPa)	36.1

3.2.6 Testing Accompanying Construction Activities

During or shortly after construction, MDOT and BCD performed testing that was made available to MSU for use. Full experimental descriptions did not always accompany the data, though sufficient information was obtained to be useful. MDOT provided several compaction tests from August to October of 2010. Specific details regarding how the data was collected were not provided.

BCD performed 1-point proctor compaction (dry density referred to as $\gamma_{dry\text{-}1\ pt}$) with 6 in diameter by 6 in tall molds, and these specimens were subsequently tested in unconfined compression (UC) for UC strength (UCS) after moist curing. Indirect tensile (IDT) specimens were also prepared with nominal dimensions of 6 in diameter by 3 in tall and tested for IDT strength (S_t) after moist curing. Many of these specimens were prepared to a density other than the Proctor maximum dry density (γ_d). Dry density of a compacted specimen that does not necessarily correspond to γ_d was given a more generic label (γ_{dry}), which refers to any compacted dry density. During field operations, BCD screened material larger than a 12.5 mm sieve, and hold times prior to compaction were estimated to be on the order of 30 minutes. BCD compaction data collected during field operations was not adjusted for material larger than 12.5 mm. A few cores were also taken and tested within 14 days. BCD also tested gradation samples for field moisture content ($w_{\%\text{-}F}$) and performed washed gradations. BCD conducted a small amount of testing on bulk samples but provided most of the material to the authors for the testing described in the following section.

3.3 Laboratory Testing Performed at MSU on Laboratory Prepared Specimens

3.3.1 Terminology

An identification system was developed and used for laboratory activities and data organization. Terms relevant to this report that indicate a data category are: curing durability (CD), gradation variability (G-V), strength versus time (ST), and wheel tracking (WT). The most prevalent specimen type was a nominal 100 mm diameter by 114.6 mm tall Superpave Gyratory Compactor (SGC) prepared specimen, which is often referred to as a 100 mm SGC specimen. Cement index (C_I) and cement content by mass (C_w) are used frequently as well. The term cement index (C_I) is fully described in the MDOT State Study 206 report (Howard et al. 2013). As of the date of this report, C_I has been discontinued by MDOT, and as such C_w is the primary term to represent cement content, though C_I has been maintained in some locations as the laboratory experimental program was performed based on C_I. For purposes of this report, C_I values of 5, 6, and 7 equate to C_w values of 3.7 to 3.9, 4.5 to 4.8, and 5.3 to 5.6, respectively.

3.3.2 Specimen Preparation

3.3.2.1 Processing FDR Materials

All three bulk FDR samples (i.e. Hwy 49 B(1), B(2), and B(3)) were ultimately combined and are generically referred to as Hwy 49B. Each sample was kept separate until blending/combining initiated. Samples were mechanically dry sieved using 12.5 mm, 9.5 mm, 4.75 mm, 2.36 mm, and 0.075 mm screen sizes. Once sieved, material from each field sample and sieve size was initially kept separate in 19 liter (5 gallon) buckets. Thereafter, materials of the same size from different samples were proportionally mixed. A floor was swept, and a tarp was placed to contain material and prevent contamination (Figure 3.4a).

(a) Process Area (b) Material Mixing (c) Material Storage

Figure 3.4. Processing and Mixing FDR Material Prior to Producing Test Specimens

Since the samples were too large to mix at one time and since the three samples were of different sizes, proportions of a particular size material were combined. For example, the 12.5 mm material from Hwy 49B(1) was contained in four buckets; Hwy 49B(2) in two buckets; and Hwy 49B(3) in one bucket. To proportion the material for testing, one bucket of Hwy 49B(1), one half of a bucket of Hwy 49B(2), and one fourth of a bucket of Hwy 49B(3) were combined at one time. This kept the amount of material mixed to a manageable quantity. The materials were thoroughly mixed using hand tools (Figure 3.4b). All material that was spread and mixed on the tarp or on the concrete floor was carefully placed back in buckets and sealed to prevent moisture loss (Figure 3.4c). Buckets each held around 20 to 25 kg of mixed Hwy 49B material.

At the end of this material processing stage, there were six categories of material as shown on Figure 3.3 (note the seventh category shown in Figure 3.3, -0.075 to 2.36 mm (Washed), was produced as described later in this section). This material was uniform within each category, and was used to represent the Hwy 49B FDR activities. The main rationale for combining all three samples in a proportional way was to provide sufficient uniform material to conduct a variety of tests. A negative aspect of this approach is the laboratory specimens produced weren't replicates of any location within US 49. Once washed material was added to the gradations as described later in this section, the laboratory specimens became even more separated from the actual project FDR, which is noted because the laboratory testing was to look for general trends, not to replicate in situ conditions at any location(s).

After blending the Hwy 49B material as described in Figure 3.4, moisture content ($w_{\%}$) and fines content (P_{200}) data was collected on a sample from each size fraction (or category) to compensate for these values during batching. Table 3.4 shows the moisture and fines contents per size fraction used for batching purposes. Note that the P_{200} values were obtained with water washing; i.e. no solvents were used. Note the high amount of fines left in several sieve sizes, which indicates dry sieving did not fully separate the FDR materials.

To be able to produce a variety of gradations, a portion of the 0.075 to 2.36 mm category material was washed to remove essentially all fines. Figure 3.5 illustrates the process followed for fines removal. A 378 L tub drained onto a 0.075 mm sieve in order to wash the material (Figure 3.5a). Approximately 22 kg of material, or one bucket full, was combined with water in the tub and allowed to soak for 24 hours before the washing process began (Figure 3.5b). During the washing process, water was consistently flowing at a rate of around 9.5 L per minute. To keep the fines suspended in water, hand tools were periodically used for agitation (Figure 3.5c).

Table 3.4. Hwy 49B Moisture and Fines Contents for Batching

Size Fraction	$w_{\%}$ (%)	P_{200} (%)
+12.5 mm	0.3	1.9
9.5 to 12.5 mm	0.3	1.6
4.75 to 9.5 mm	0.4	7.9
2.36 to 4.75 mm	0.7	26.2
0.075 to 2.36 mm (Un-washed)	0.9	43.8

(a) Washing Apparatus

(b) Fill Tub/Soak Material

(c) Agitate to suspend fines

(d) Continuous flow through sieve

(e) Wash all remaining material

(f) Wash from sieve

(g) Oven drying at 49 °C

(h) 0.075 to 2.36 mm (Washed)

Figure 3.5. 0.075 to 2.36 mm (Washed) Material Processing

Figure 3.5d shows the water and material mixture pouring onto a baffle created by aluminum foil to disperse the mixture on the 0.075 mm sieve. After continuously washing for 24 hours, all material left in the tub was washed by hand to remove remaining fines (Figure 3.5e). Washed material on the sieve was removed by water over a clean pan, frequently pouring excess water back over the sieve (Figure 3.5f). Once washed, material was dried for 3 days in a 49 °C oven before being thoroughly remixed (Figure 3.5h) and returned to 19 liter buckets. This material is the seventh material category, is designated 0.075 to 2.36 mm (Washed), and is shown visually in Figure 3.3. It is clear that this material category was handled in a manner that does not represent practices on any FDR project. It is possible these handling practices affected compactibility and strength relative to US 49.

3.3.2.2 Batching FDR Materials

The seven material categories depicted in Figure 3.3 were used to create three gradations for producing test specimens (Figure 3.6). Figure 3.6 plots G1 to G35 gradations provided by BCD (black lines). Recall these gradations included cement; therefore, P_{200} would be approximately 5% lower than indicated. In retrospect, MSU should have reduced the Figure 3.6 gradations from BCD by 5% fines prior to developing the laboratory gradation bands. As can be seen, there are major gradation variations throughout the US 49 project. Table 3.5 presents summary information regarding G1 to G35 to provide numerical values to the visual assessments of Figure 3.6.

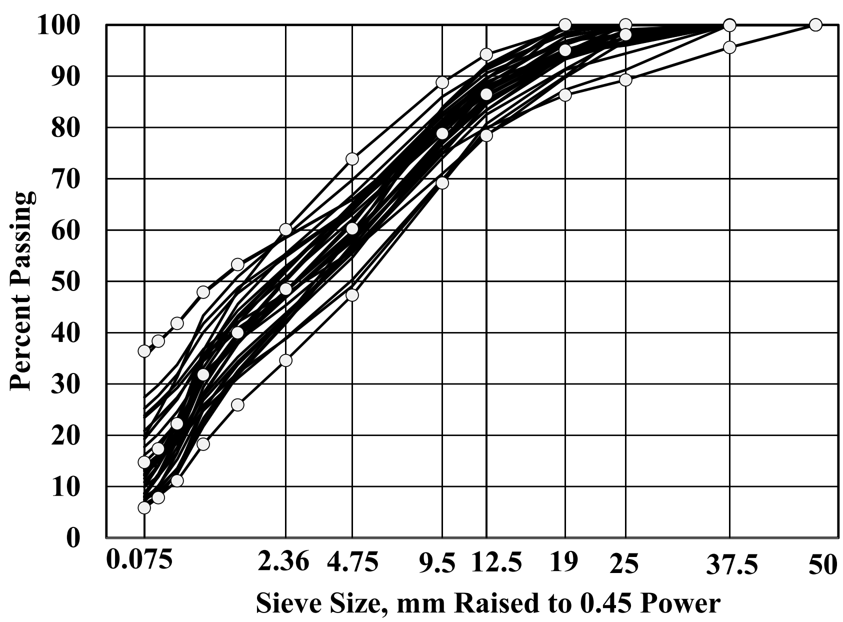


Figure 3.6. Hwy 49B Gradations Collected By BCD (Solid Lines) and Gradations Tested in the Laboratory by MSU (Circles)

For each sieve size (35 values each), the average, minimum, and maximum percent passing was determined. These values are shown in three columns in Table 3.5. The column of minimum values represents the coarse (or low percent passing) side of the Figure 3.6 gradation band, and likewise the column of maximum values represents the fine (or high percent passing) side of the Figure 3.6 gradation band. The coarse, fine, and average gradations are shown on Figure 3.6 as circles and were the gradations produced to bracket the overall behavior of US 49 FDR. Note that it was not learned until after all MSU laboratory testing was completed that BCD gradations included cement. Thus, MSU-constructed gradations (coarse, fine, and average) were batched with approximately 5% excess fines. While this prevented direct comparisons between laboratory and field data, an understanding of general trends was of more importance within this report. General trends were not believed to be meaningfully affected by this discrepancy given the highly variable amount of fines (i.e. gradation extremes were still largely represented by MSU constructed gradations.

To develop each of the three gradations required superimposing two or more of the measured gradations (G1 to G35) together. For the coarse and fine gradations, two samples (G1 to G35) made up most of either gradation. For example, the fine gradation was largely a combination of G21 and G33, with most of the portion larger than 2.36 mm representing G33 and most of the portion smaller than 2.36 mm representing G21. This is another distinction in this report's laboratory activities in that the gradations batched and tested in the laboratory were not directly observed at any one sample location.

To produce the gradations developed, Figure 3.3 proportions were batched as shown in Table 3.6. Recall that, for example, batching 22% of the +12.5 mm size fraction brought with it the moisture and fines represented in Table 3.4. Of primary importance related to batching was the amount of 0.075 to 2.36 mm (Washed) material that was utilized. The fine gradation did not utilize any washed material, and as such is more representative of US 49 raw materials than the average or coarse gradations.

Table 3.5. Summary of US 49 FDR Gradations Collected by BCD

Sieve Size (mm)	Coarsest (or Minimum)	Average	Finest (or Maximum)	St. Dev.	COV
50	100.0	100.0	100.0	0.0	0%
37.5	95.6	99.9	100.0	0.7	1%
25	89.2	98.1	100.0	2.5	3%
19	86.3	95.1	100.0	3.4	4%
12.5	78.4	86.5	94.2	3.9	5%
9.5	69.2	78.8	88.7	4.6	6%
4.75	47.3	60.3	73.8	5.6	9%
2.36	34.6	48.5	60.1	6.2	13%
1.18	25.9	40.0	53.3	6.5	16%
0.60	18.2	31.8	47.9	6.9	22%
0.30	11.1	22.2	41.8	7.5	34%
0.15	7.8	17.4	38.3	7.8	45%
0.075	5.9	14.7	36.4	7.7	52%

Note: 35 gradations which included cement are represented in this table (G1 to G35 shown in Figure 3.1).

Producing the three laboratory gradations in this manner is important when interpreting laboratory results in the context of field results. The coarse and average gradations were fabricated in a manner that would not have occurred during construction of US 49, and it is possible that 21 to 28% of the total mixture being produced with washed particles could have affected compactibility relative to what would occur if the particles were not washed. Washed particles would be completely dislodged from one another and, all other factors being equal, would be expected to be more compactable than the same particles loosely adhered to one another as would be the case in an actual FDR mixture.

Table 3.6. Batching Quantities Used for Specimen Preparation Referencing Figure 3.3

Sieve	Coarse Gradation	Average Gradation	Fine Gradation
+12.5 mm	22	14	6
9.5 to 12.5 mm	9	8	5
4.75 to 9.5 mm	24	20	16
2.36 to 4.75 mm	17	16	19
0.075 to 2.36 mm (Un-washed)	0	21	40
0.075 to 2.36 mm (Washed)	28	21	0
-0.075 mm	0	0	14

Note: values are accurate to within a few tenths of a percent and changed slightly with moisture.

A water content adjustment was made to achieve the desired value in specimens. After experimentation, it was determined that batching 0.7% more water by mass than was required in the specimens was suitable for all batching. It was found that a tolerance of $\pm$ 0.5% of optimum moisture could be achieved. Hwy 49B moisture content measurements were greatly affected by the material size and the small allowable size of the moisture content sample. Because of limited material quantities, only a small moisture content sample was taken. A small experiment was performed to ensure that the correct moisture was present in FDR specimens, and the as tested moisture contents reported in Chapter 4 should be interpreted with small sample sizes in mind.

3.3.2.3 Mixing FDR Materials

Materials were mixed using a 19 L, table-mounted bucket mixer. Mixing began with the addition of water to material. Water was added at approximately 90 grams per second in order to combat material clumping. Once water was added, the material and water was mixed for two minutes with a paddle and hand trowel. Cement was then added and was mixed for two additional minutes, resulting in approximately four minutes of mixing per batch. Upon complete material mixing, a sample was taken from the bucket to measure moisture content.

3.3.2.4 Compacting FDR Materials-Proctor Hammer

Unless otherwise noted, proctors prepared by MSU were performed as follows. In a few instances, parameters were varied for small subsets of data to investigate a specific issue, which are noted when discussed. Specimen compaction used a 6 in mold, 4.58 in height, 4 lifts, 56 blows per lift, and a standard proctor hammer (5.5. lb mass falling 12 in). Note that 6 in tall molds were used by BCD for some of their Proctors and that MT-9 describes Proctor testing with 6 in tall molds. Also note that MDOT's central materials laboratory performed Proctors in the same manner as MSU during the time frame of this research. MSU re-used raw proctor material, but did not re-use cement proctor material (each cement proctor specimen was batched and produced separately). Compaction was performed on specimens that were mechanically sieved, mechanically mixed, and with individual data points compacted within 20 minutes of cement being introduced to water. All +12.5 mm material was removed, and Proctors were performed on -12.5 mm material. The procedures in MT-8 and MT-9 were followed to adjust maximum dry density (γ_d) and Optimum Moisture Content (OMC) from the Proctor test as a function of the amount of +12.5 mm material and its bulk specific gravity (G_{sb}).

3.3.2.5 Compacting FDR Materials-Superpave Gyratory Compactor

A Pine AFGC 125X Superpave gyratory compactor (SGC) was used to compact everything in the MSU laboratory except specimens compacted for measurement of Proctor density. A specified mass of mixed material was placed in the appropriate SGC mold (100 mm or 150 mm diameter) and compacted to target 100% of the wet density (γ) that would occur from γ_d and OMC (i.e. $\gamma = \gamma_d[1+OMC]$ with OMC expressed as a decimal). Wet density (γ) was based off Proctor testing at C_I of 6% due to material quantity limitations, which is noted since specimens were compacted at other C_I values in several cases throughout this report. The difference in γ as a result of this approach was neglected in the report, which should not have a pronounced effect on the laboratory compacted specimen findings as trends were of most importance.

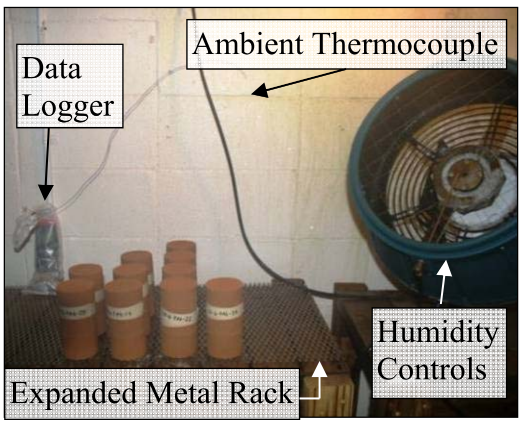
The number of gyrations (N_G) required for compaction was usually recorded. For replicate specimens, the average number of gyrations ($N_{G\text{-}Avg}$) required to compact all replicates was typically used during analysis. Batch moisture contents were adjusted OMC values ultimately intended to achieve OMC in the specimens during compaction (i.e. a slight amount of excess water was added to account for losses during mixing and handling). The as measured moisture content ($w_{\%\text{-}M}$) was reported for the majority of specimens to compare to the target value (OMC), though sample sizes were often smaller than standard test methods would allow. In most cases, a small amount above the target mixture mass was also added to counter any lost mass during mixing and handling. Spacer papers and a thin piece of aluminum foil were placed between the material and plates to assist in removing the top and bottom compaction plates. Specimens were compacted to a target height (114.6 or 75 mm). Specimens were then extruded from the mold; and the top plate, foil, and spacer paper were removed. Carefully, the specimen was loosened from the bottom plate with a slight shearing action, followed by removal of the bottom spacer paper and foil. After extrusion, specimens were labeled and placed under damp towels for 2 $\pm$ 0.5 hr.

3.3.2.6 Density Measurements

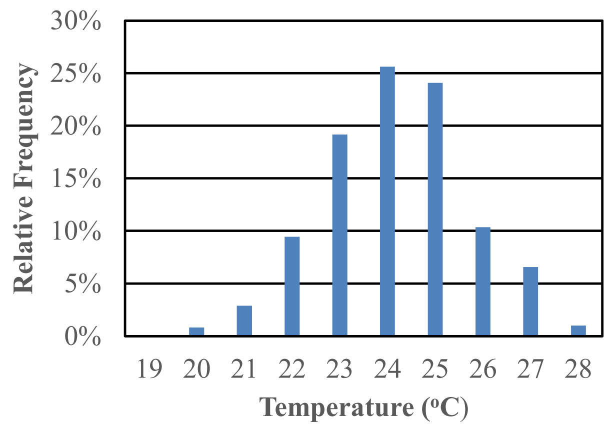
Immediately after being taken from under the damp towel post SGC compaction, density was measured prior to a curing protocol being initiated. Four diameter (D) values were measured, two 90˚ from each other at the top and bottom of the specimen. The average of these was taken as the diameter (D_{Avg}). Height (H) was measured at four equally spaced locations on each specimen. The average of these heights (H_{Avg}) was taken as the specimen height. The total mass (M), which included specimen moisture, and calculated volume using D_{Avg} and H_{Avg} was used to calculate a specimen's density (γ_T). In cases where replicate specimens were produced, the average γ_T value ($\gamma_{T\text{-}Avg}$) was often calculated and reported. Acceptable compaction was defined as γ_T / γ of 0.98 to 1.01 (i.e. achieving 98 to 101% of the target wet density). This target value was often reported as an average value on a percentage basis ($\gamma_{\%\text{-}Avg}$). Recall that γ was always determined with C_I value of 6%, which is not the case for several specimens, but was necessary due to limited material and deemed reasonable for the laboratory testing portion of this project to achieve the desired goal of trend identification.

3.3.3 Specimen Curing

Four curing protocols were utilized, and each is described in individual sub-sections. The curing room shown in Figure 3.7a was used in some way in each of these protocols. Humidity was 99.5 and 100% inside the curing room, and to prevent specimens from resting in standing water, shelves were covered with stainless steel expanded metal (12.7 mm, number 18 style) mounted on 6.5 mm diameter wooden dowels. Temperature was monitored every 60 minutes by a SPER Scientific Model 800024 data logger. Figure 3.7b is a relative frequency histogram of the ambient temperature distribution observed from June 2011 to February 2015, which is a time period longer than testing occurred. A total of 26,262 readings are shown in Figure 3.7b, and the average value is 24 °C.



(a) Moisture Curing Room



(b) Ambient Temperature Distribution

Figure 3.7. Curing Room Utilized for Multiple Protocols

3.3.3.1 Curing Protocol 1

Curing protocol 1 (CP1) was the default or control protocol and was utilized unless otherwise noted. Specimens subject to CP1 were placed under a damp towel for 2 $\pm$ 0.5 hr after compaction, and density was measured thereafter. This allowed to specimens to set up enough in order to prevent damage during measuring and handling (some specimens could be handled immediately without damage). Thereafter, specimens were immediately placed in the Figure 3.7 curing room exposed to the environment (i.e. they were not in a plastic bag or other container) for a prescribed amount of time before testing.

3.3.3.2 Curing Protocol 2

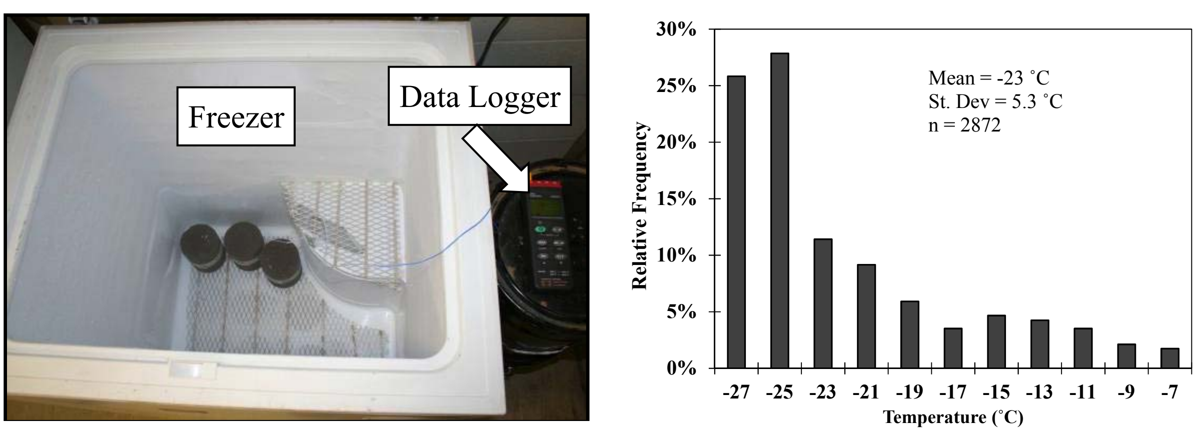
A wetting and drying protocol with heat was referred to as curing protocol 2 (CP2). After specimens were compacted, they were placed under a damp towel for two hours (± 0.5 hours). Density measurements were taken at the end of the two hour period. Unconfined compression tests were conducted at 7, 56, and 120 days in conjunction with CP2. The following curing sequence took place over the seven days of curing before testing (i.e. days 1 to 7, 50 to 56, or 114 to 120); otherwise specimens remained exposed in the moist curing room (i.e. Figure 3.7). A curing cycle consisted of two stages: 1) submerging the specimen in room temperature water (around 24 °C) for 24 hours (Figure 3.8a), then 2) placing the specimen in an oven at 71°C for 24 hours (Figure 3.8c). After each stage, the specimen mass was recorded to monitor ingress and egress of water. If specimens were wet, a paper towel was used to blot the specimens to approximately a saturated surface dry (SSD) state before a mass was recorded (Figure 3.8b). After three curing cycles were complete, specimens were submerged under water for 20 hours, then removed and placed on shelves in the moist curing room 4 hours before being tested. Figure 3.8d shows that the drying in the oven had a pronounced visual effect on specimens.

(a) Submerged in Water (b) SSD Weight (c) Oven Drying (71°C) (d) Oven Close Up

Figure 3.8. Wetting and Drying Protocol (CP2)

3.3.3.3 Curing Protocol 3

A wetting and freezing protocol was referred to as curing protocol 3 (CP3). This protocol resembles the overall framework of CP2; however, the second stage of a curing cycle used a chest freezer (Figure 3.9a) to subject the specimens to freezing temperatures. Replacing an oven with a freezer is the only difference between CP2 and CP3. A SPER Scientific Model 800024 data logger was used to record freezer temperatures. Over three periods of a few days around 7, 56, and 120 days, freezer temperatures were logged to develop the histogram shown in Figure 3.9b. Temperatures were measured at 5 minute intervals. An average temperature of -23 °C occurred within the freezer, with warmer temperatures (as warm as -7 °C) occurring in some instances but with the distribution skewed toward temperatures between -21 and -27 °C.



(a) Freezer and Data Logger (b) Histogram of All Temperatures

Figure 3.9. CP3 Freezer and Temperature Histogram

3.3.3.4 Curing Protocol 4

Cylindrical wheel tracking specimens (150 mm diameter by 75 mm tall) were gyratory compacted and placed under a damp towel for two hours before being moved to the moist curing room. Specimens remained in the moisture curing room (i.e. CP1) for 56 days; wheel tracking was performed less than seven days after removal from the curing room. This protocol is referred to as curing protocol 4 (CP4) and identical to CP1 except specimens were placed onto a laboratory bench for up to 7 days after exiting the curing room before testing.

3.3.4 Specimen Testing

3.3.4.1 Unconfined Compression Testing

Unconfined compression (UC) tests were conducted in accordance with ASTM D1633 and MT-26 to produce an unconfined compressive strength (UCS). Specimens were not soaked before being tested as prescribed in the aforementioned specifications; rather, the aforementioned curing protocols were followed. Procedures for conducting the unconfined compression tests were the same as given in the specifications. In this report, one height to diameter (H/D) ratio for specimens were used; 1.15:1. According to ASTM D1633, compressive strengths of the 2:1 ratio specimens can be adjusted to 1.15:1 ratio strengths by a factor of 1.10. Specimens were tested after the appropriate curing protocol was followed. Smoothness requirements were met; therefore, capping was not required for any specimen. Testing took place on a load frame fitted with a proving ring and spherically seated swiveling load head. Specimens were tested at a constant rate of 1.27 mm/min (0.05 in/min).

3.3.4.2 Wheel Tracking

Asphalt Pavement Analyzer (APA) testing was for 8,000 cycles at 64 °C with 100 lb load and 100 psi hose pressure. Some APA testing was performed in the customary dry state, while other testing was performed with the aforementioned conditions but submerged in 64 °C water. If not stated, the test was conducted dry.

3.3.4.3 Test Matrices

Four categories of laboratory testing on laboratory prepared specimens were performed. They are: 1) strength gain with time (ST), 2) gradation variability (G-V), 3) curing durability (CD), and 4) wheel tracking (WT). A common reference for test matrices was a design cement index (C_I) value determined via MT-27. Test matrix details of each category are described in the remainder of this section. The default test replication for UCS was three.

Strength gain with time specimens were compacted with the SGC to nominal dimensions of 100 by 114.6 mm. UC specimens were tested at the following eleven cure times: 1, 3, 7, 14, 21, 28, 56, 120, 240, 360, and 540 days. Curing followed CP1. Specimens were produced at the design C_I of 6% using the average gradation.

Gradation variability testing was performed on 110 specimens that were SGC compacted to nominal dimensions of 100 by 114.6 mm. Three gradations (coarse, average, and fine) were each tested at three cement indices (design, +1% of design, and -1% of design) after being cured as per CP1 protocols. Ten replicates were performed for each gradation-cement index combination, and in addition twenty additional replicates were conducted at the average gradation and design cement index (i.e. 30 total replicates were tested at this set of conditions).

Curing durability testing was performed on 81 specimens that were SGC compacted to nominal dimensions of 100 by 114.6 mm at a design cement index. Three gradations (coarse, average, and fine) were tested at three test times (7, 56, and 120 days) in the presence of three curing methods (CP1, CP2, CP3).

Wheel tracking was performed on 150 by 75 mm specimens that were SGC compacted. Three gradations (coarse, average, and fine) were tested at the design cement index. In each case two specimens (i.e. one APA track) were tested dry, and then the same specimens were re-tested submerged in water. Specimens were cured following CP4.

3.4 Field Testing Performed Over a Period of Time After Construction

Field testing over time can generally be divided into three activities: FWD testing, coring with subsequent material characterization, and automated profiler distress surveys. MDOT periodically monitored US 49 via FWD tests leading up to a more comprehensive evaluation during the 2015 construction season (i.e. 5th construction season since the FDR project). FWD locations related to US 49 FDR evaluation are shown labeled FWD1 to FWD12 in Figure 3.1. Also shown in Figure 3.1 are locations were cores were taken.

The culminating US 49 evaluation occurred in the May to June 2015 timeframe (after 53 months of service). During this evaluation, a total of 68 cores were taken from US 49, 12 of them (half 4-inch diameter, half 6-inch diameter) from the FDR section (Figure 3.10). Cores were obtained spatially to represent the FDR section as a whole. Five of the 12 cores were taken at the exact FWD drop location to supplement FWD analysis (See Figure 3.1).

A frame was designed by MSU and fabricated by a local machine shop that allowed for coring depths up to 26 inches (Figure 3.10a). Cores up to 24 inches long were obtained, as were shorter cores, as shown in Figures 3.10b and 3.10c, by making use of coring bit extensions. These capabilities allowed the full depth of the FDR layer to be cored in a single piece. Many cores separated cleanly from the subgrade such that striations were still visible from FDR mixing operations (Figure 3.10d). The cores obtained were characterized as described in Section 3.5.

Subgrade samples were taken for 6 of the 12 cores and were visually grouped into two categories which were combined to form two composite samples: 1) brown fine-grained soil and 2) grayish-brown fine-grained soil. Subgrade soils were tested for basic index properties, washed gradation, and Atterberg limits for soil classification and potential use in the FWD analysis.

MDOT conducted a pavement distress survey on April 23, 2015 (i.e. a 53 month survey) using their Pathrunner™ profiler, which is equipped with multiple computers for distress measurement. Data was collected in 500 ft long units which were eventually merged to produce results by test section. Parameters considered were MDOT's pavement condition rating (PCR), mean roughness index (MRI), rutting, fatigue cracking, block cracking, longitudinal cracking, and transverse cracking. Each distress was quantified by severity level based on the Federal Highway Administration (FHWA) publication RD-03-031 (Miller and

Bellinger 2003). MDOT's profiler was capable of measuring other distresses (e.g. edge cracking), but these were not reported since they were not observed. Note that PCR values are reported on a 0 to 100 scale where the thresholds for various condition ratings vary depending on route type. PCR is a composite index which combines roughness and distress into a single index and is calculated using an algorithm defined by MDOT.

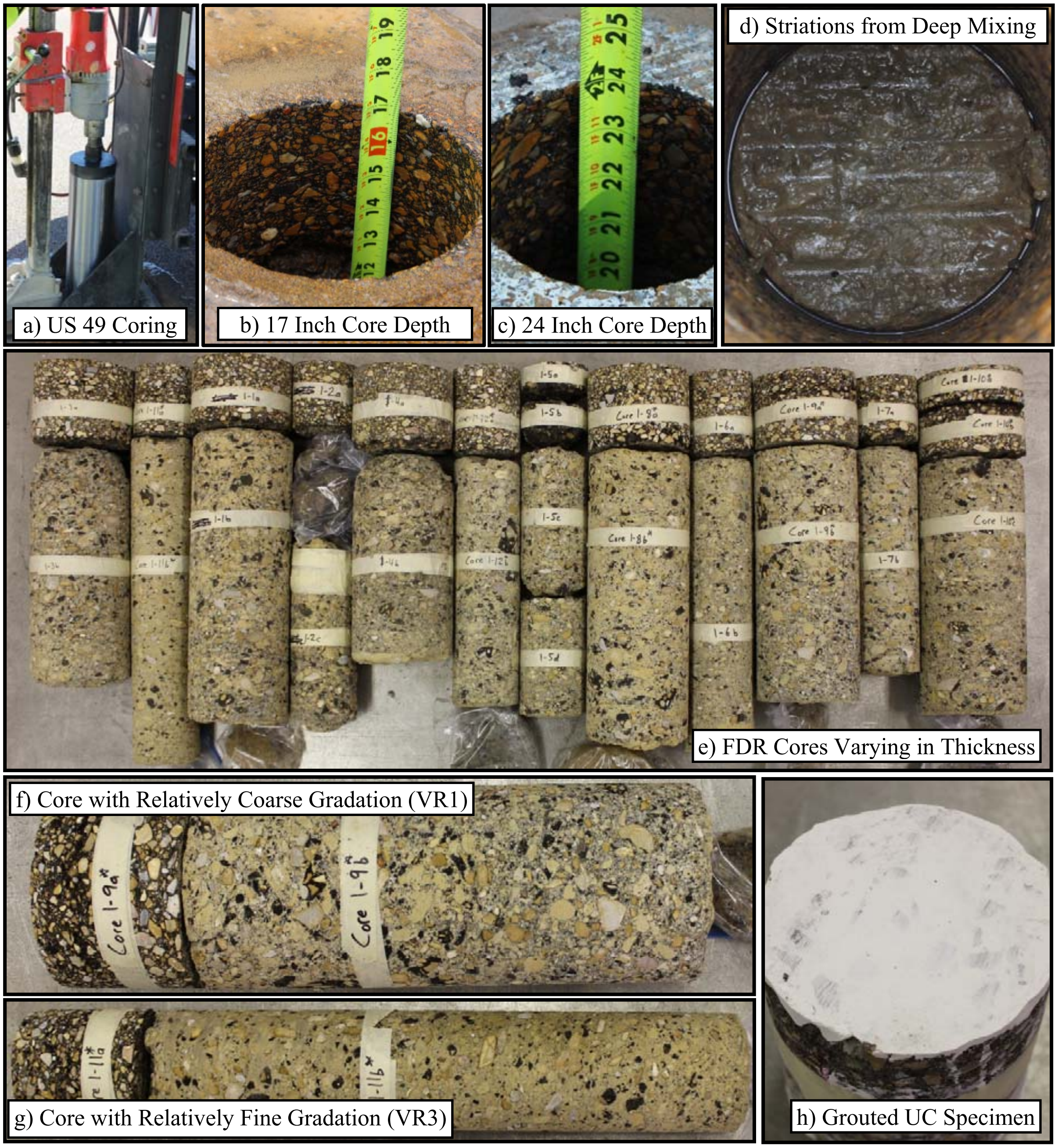


Figure 3.10. Photographs of Field Testing Performed After Construction

3.5 Laboratory Testing Performed on Field Cores

The cores shown in Figure 3.10e to 3.10h were visually examined and logged. Thicknesses varied (Figure 3.10e) and some cores were marred near the ends. Therefore, with the cores available, a test plan was constructed in a hierarchal manner prioritizing UCS, elastic modulus (*E*), and dry density (γ_{dry}). Asphalt Pavement Analyzer (APA) rutting and S_t values were of secondary priority. With this approach, test replication varied slightly as shown in Chapter 4. Generally speaking, UCS and γ_{dry} testing was performed on the topmost 12 inches of FDR cores.

Prior to sawing cores into test specimens, cores were visually rated (VR1 to VR3) with respect to the perceived amount of fines present since gradation varied noticeably from coarse (Figure 3.10f) to fine (Figure 3.10g), 1 being coarse-graded and 3 being fine-graded. Fine-graded cores generally corresponded to thicker cores, and vice versa (i.e. more underlying layers were incorporated and stabilized).

Cores were sliced using a wet-cut masonry saw. Materials sliced below 12 inches were visually in worse condition than that above 12 inches; marginal compaction during construction and damage during coring are possible explanations. This material was used for APA testing (AASHTO T340) as it was generally not suitable for other purposes.

Large aggregate pop-outs and broken edges were not uncommon during slicing, making reliable density measurements difficult. Therefore, density was measured only on UCS specimens for consistency and since they were larger, thus minimizing surface texture effects from sawing. As an extra measure against unreliable densities and to obtain full cross-section area for UC tests, a blend of Plaster of Paris and portland cement was mixed with water, applied to specimen ends, struck off via putty knife, and then sanded smooth (Figure 3.10h). The plaster-cement grout was proportioned so that its density was approximately that of the cores based on several preliminary density measurements. Errors between grout and core density would be less than errors associated with poor density measurement due to rough surface texture. Density was measured via AASHTO T331 CoreLok vacuum sealing to obtain bulk densities (semi-wet) which were then adjusted to dry densities ($\gamma_{dry\text{-}T331}$) once moisture contents were measured (moisture content was measured on all possible specimens except APA specimens).

IDT specimens were nominally 2 inches thick and 4 inches diameter. These were generally taken at whatever depths within a core were available after allocating all other test specimens. Specimens were tested at room temperature and a 2 in/min load rate similar to loading procedures in AASHTO T283.

UCS specimen dimensions followed previous MDOT practices on US 49. Four-inch diameter specimens were nominally 4.5 inches tall, and 6-inch diameter specimens were nominally 5.75 inches tall. After grout application and density measurement, specimens were UC tested at a 0.05 or 0.20 in/min load rate (4-inch or 6-inch diameter, respectively).

E specimens were trimmed to a 2:1 height-to-diameter (H/D) ratio and tested largely in accordance with ASTM C469. Specimen ends were grouted as with UCS specimens. An 0.05 in/min load rate was used for both 4- and 6-inch diameter specimens. After testing, all *E* specimens were sliced again into UCS or IDT specimens since *E* tests are nondestructive.

CHAPTER 4 – TEST RESULTS AND DISCUSSION

4.1 Overview of Results and Discussion

All test results are presented in this chapter. Results are first presented individually for activities during construction, laboratory testing activities at MSU, and field monitoring activities. Thereafter, discussion is provided for all results. Refer to Chapter for 3 for constituent material properties and test methods. Refer to the List of Symbols for definitions of several of the terms presented.

4.2 Results Collected During Construction of US 49

MDOT provided FDR compaction data on random sample locations at US 49 from August to October of 2010 that is summarized in Table 4.1. Tables 4.2 and 4.3 contain data collected by BCD on the bulk FDR samples described in Chapter 3. Proctor compaction and subsequent strength testing was performed in two manners. Table 4.2 summarizes UC testing performed on nominal 6 in diameter by 6 in tall specimens at 6% cement by mass (i.e. C_w of 6%) to evaluate strength as a function of density. Table 4.3 provides test results as a function of cement content and contains UC testing of nominal 6 in diameter by 6 in tall specimens and indirect tensile (IDT) testing of nominal 6 in diameter by 3 in tall specimens. Table 4.4 summarizes the 35 Figure 3.5 gradations alongside corresponding Proctor compaction and subsequent strength properties that were measured.

Table 4.1. Moisture and Compaction Data Collected During Construction by MDOT

Parameter	$w_{\%\text{-}F}$ (%)	γ_d (pcf)[a]	OMC (%)[a]
Average	9.9	116.9	11.0
St. Dev	1.7	2.3	1.2
COV (%)	17.2	2.0	10.9
Max	13.2	121.4	14.1
Min	6.5	111.4	8.6

Note: 23 tests were performed (i.e. n = 23).
a: it is unknown if these values were corrected.

Table 4.4 UCS results were not correlated to γ_{dry}. An initial reaction would be this is not logical, but if one considers the likely G_{sb} variability of an FDR project where recycled depths varied from 12.0 to 19.5 in and also considers the standard compactive effort provided to each specimen, the variability observed in Table 4.4 could be largely attributed to G_{sb} variability. There are no obvious correlations (e.g. percent passing 0.075 mm sieve and γ_{dry}), but the likelihood of G_{sb} variability in a highway corridor that has existed for decades is likely. If Gsb varied $\pm$ 0.1 from its mean value (i.e. range of 0.2) and the mean G_{sb} occurred at the mean γ_{dry} (115.1 pcf), this would more than explain the maximum range of γ_{dry} values observed (109.9 to 121.1 pcf).

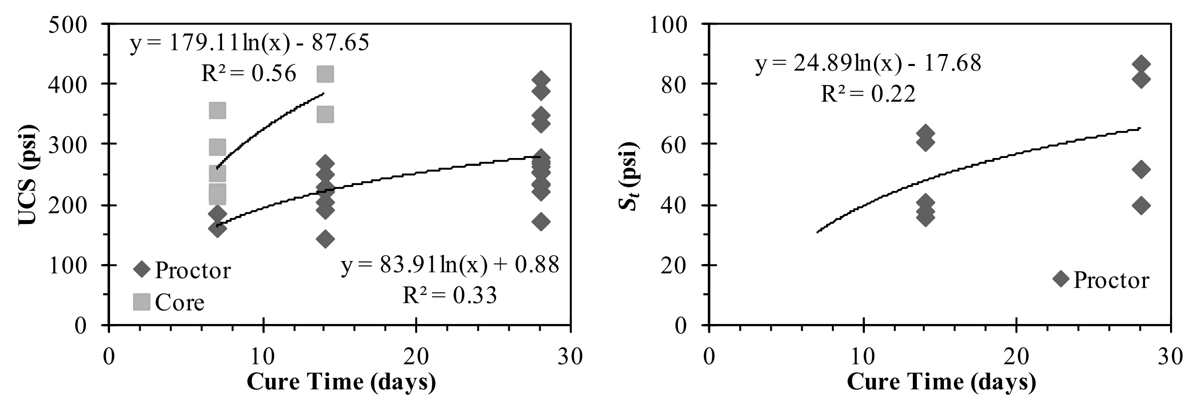
Figure 4.1 plots UCS and S_t after 7, 14, and 28 of curing using Table 4.4 data. UCS values were higher for cores than for Proctor specimens. For core UCS values, a UCS of 300 psi (typical MDOT criteria applied to 1.15:1 aspect ratio specimens) was always exceeded at

14 days and sometimes exceeded at 7 days. UCS was on the order of five times greater than S_t. Tensile strengths were, on average, around or above 45 psi, which is a typical minimum S_t value (State Study 250 Volume 2 provides additional information); however, a wide range of values were measured. Based on Figure 4.1, variability appears visually evident for both UCS and S_t.

Table 4.2. BCD Proctor and Strength Results: Varying Density and C_w of 6%

Sample	Test Day	γ_{dry} (pcf)	$w_{\%}$	UCS (psi)
Hwy 49B(1)	7	115.2	8.7	275
	7	116.3	10.9	361
	7	114.7	12.5	285
	7	111.6	14.6	222
	14	115.1	8.8	331
	14	116.6	11.4	406
	14	115.1	12.1	315
	14	110.6	14.9	271
Hwy 49B(2)	7	118.0	7.4	339
	7	120.4	9.6	355
	7	118.6	11.2	323
	7	115.1	13.6	255
	14	117.5	7.6	355
	14	119.1	9.8	414
	14	118.3	11.3	323
	14	113.2	13.4	220
Hwy 49B(3)	7	115.0	7.9	262
	7	116.7	9.0	294
	7	114.5	12.3	231
	7	110.8	13.8	169
	14	116.2	7.8	288
	14	116.3	9.4	300
	14	114.4	12.3	251
	14	111.1	13.5	220

Note: γ_{dry} is the dry density as tested, not the maximum proctor compaction dry density (γ_d).



a) UCS versus Time *b) S_t versus Time*

Figure 4.1. Strength with Time Using Table 4.4 Data

Table 4.5 provides an overall assessment of strength variability for Table 4.4 data that was plotted in Figure 4.1. Results are shown by cure time. All data is presented for completeness; however, some cases exist (e.g. 7-day Proctor UCS) where as few as two replicates were tested. Variability results are more reliable for cases where replication is greater and should be interpreted accordingly.

Table 4.3. BCD Proctor and Strength Results: Varying Cement Content

Sample	Test Day	C_w (%)	γ_{dry} (pcf)	$w_{\%}$	UCS (psi)	S_t (psi)
Hwy 49B(1)	7	5.5	118.0	9.3	326	---
	14	5.5	119.0	9.2	355	---
	7	5.5	118.8	9.6	---	123
	14	5.5	119.6	9.0	---	137
	7	6.0	115.3	7.0	289	---
	14	6.0	115.4	6.9	288	---
	7	6.0	119.6	9.1	---	112
	14	6.0	118.4	9.6	---	130
Hwy 49B(2)	7	5.5	121.3	9.7	201	---
	7	5.5	120.1	10.0	---	63
	14	5.5	120.0	10.3	245	---
	14	5.5	120.0	10.4	---	69
	14	6.0	122.0	9.9	248	---
	14	6.0	122.1	9.5	---	87
	7	6.5	120.9	10.5	220	---
	7	6.5	119.5	10.5	---	67
	14	6.5	119.4	10.9	258	---
	14	6.5	122.4	10.8	---	84
Hwy 49B(3)	7	5.5	118.1	9.9	149	---
	7	5.5	117.6	10.4	---	36
	14	5.5	118.1	10.2	159	---
	14	5.5	117.4	10.5	---	47
	14	6.0	116.7	10.5	161	---
	14	6.0	117.1	10.7	---	46
	7	6.5	116.8	10.8	156	---
	7	6.5	117.8	9.9	---	43
	14	6.5	117.7	10.2	156	---
	14	6.5	117.4	10.3	---	50

Note: γ_{dry} is the dry density as tested, not the maximum proctor compaction dry density (γ_d).

On average, Table 4.5 UCS variability is relatively similar between Proctor specimens and cores with an average COV of 17%. COV where only two replicates were tested ranges from 10 to 12%, which, while manageable, may not be as representative of overall UCS variability as when replication is greater. Where replication is between 6 and 14, COV ranges from 18 to 23%, which is likely a more representative range. This amount of variability is reasonable for FDR materials when all factors are taken into consideration; for example, Table 4.4 data includes other second-order factors such as variability in gradation, density, and cement content. COV is considerably higher for S_t, ranging from 28 to 35%. While higher, S_t variability being higher than UCS variability for cement-stabilized FDR materials is not beyond reason.

Table 4.4. Results of BCD Field Samples G1 to G35

	MC	$\gamma_{dry\text{-}1\ pt}$	$\%\gamma_{dry\text{-}max}$	(UCS) or [S_t] - Proctor (psi)			UCS - Core (psi)		% Passing			
Sample	**(%)**	**(pcf)**	**(%)**	**7 day**	**14 day**	**28 day**	**7 day**	**14 day**	**0.075 mm**	**2.36 mm**	**4.75 mm**	**9.5 mm**
G1	6.0	118.4	101.3	---	(144)	---	---	---	7.9	43.0	59.0	83.9
G2	7.9	120.3	102.9	---	---	(223)	---	---	7.5	41.8	57.2	83.6
G3	10.7	117.3	100.3	---	(270)	---	---	---	7.1	50.1	63.8	81.5
G4	12.3	111.6	95.5	(162)	---	---	---	---	8.1	48.2	59.8	80.7
G5	13.6	109.9	94.0	---	---	(254)	---	---	13.4	51.9	65.2	82.8
G6	13.1	114.1	97.6	---	[64]	---	---	---	13.5	48.1	61.5	81.1
G7	12.5	116.0	99.2	---	---	[82]	---	---	8.0	42.1	56.7	77.7
G8	13.3	112.5	96.2	---	---	(273)	---	---	14.7	55.4	66.8	83.6
G9	13.7	111.0	95.0	---	(205)	---	---	---	10.8	52.4	64.5	81.5
G10	12.6	115.7	99.0	---	---	(390)	---	---	27.4	55.4	64.2	79.9
G11	11.2	114.3	97.8	---	---	[87]	---	---	8.8	48.1	61.6	80.3
G12	14.5	113.9	97.4	---	---	(256)	---	---	25.2	54.8	64.1	79.7
G13	11.0	112.6	96.3	---	[41]	---	---	---	12.1	43.8	55.6	75.7
G14	10.2	113.2	96.8	---	(231)	---	---	---	6.6	34.6	47.3	69.2
G15	11.8	111.6	95.5	---	---	(279)	---	---	13.0	38.8	49.1	69.7
G16	10.8	116.5	99.7	---	---	(336)	---	---	19.3	52.9	63.1	78.7
G17	10.2	114.6	98.0	---	[36]	---	---	---	23.8	47.6	55.9	71.0
G18	7.6	117.7	100.7	---	(192)	---	---	---	16.1	46.9	58.1	75.0
G19	8.7	113.8	97.3	---	---	---	---	---	13.2	39.0	50.3	70.0
G20	9.9	115.5	98.8	---	---	(234)	---	---	36.4	58.5	65.9	80.5
G21	9.8	121.1	103.6	---	---	(264)	---	---	35.2	58.7	66.0	79.7
G22	8.4	116.8	99.9	---	[61]	---	---	352	5.9	43.2	55.6	74.0
G23	9.2	116.1	99.3	---	(223)	---	---	---	7.3	43.0	56.8	76.9
G24	10.1	118.0	100.9	(186)	---	---	---	418	17.7	45.3	56.1	74.9
G25	11.4	116.8	99.9	---	---	(173)	---	---	23.5	47.1	57.8	76.0
G26	12.1	114.7	98.1	---	---	(236)	---	---	20.9	47.1	58.7	78.1
G27	11.1	115.5	98.8	---	(223)	---	297	---	19.8	58.9	69.8	86.0
G28	10.8	116.7	99.8	---	[38]	---	---	---	12.5	50.7	62.1	79.6
G29	10.1	114.8	98.2	---	---	(350)	215	---	8.6	52.5	65.0	82.2
G30	10.6	114.3	97.8	---	---	[52]	---	---	10.0	42.3	54.6	76.9
G31	11.5	111.0	95.0	---	---	(409), [40]	---	---	12.2	50.6	64.4	81.0
G32	10.6	113.4	97.0	---	---	---	223	---	13.9	48.1	59.2	77.2
G33	11.7	114.7	98.1	---	(251)	---	253	---	11.6	60.1	73.8	88.7
G34	9.4	114.9	98.3	---	---	(270)	358	---	9.9	46.9	58.6	77.4
G35	9.4	117.9	100.9	---	---	---	219	---	13.8	48.2	61.1	83.7

-- Proctor specimen H by D equals 6.0 by 6.0 in, and all UC strengths were adjusted to an H/D ratio of 2.0 according to ASTM C39.
-- Gradations include cement; therefore, fines contents should be reduced by 5% for more appropriate interpretation.

Table 4.5. Strength Variability of Table 4.4 Data

	UCS - Proctor			UCS - Core		S_t - Proctor	
	7 day	14 day	28 day	7 day	14 day	14 day	28 day
n	2	8	14	6	2	5	4
Mean	174	217	282	261	385	48	65
Min	162	144	173	215	352	36	40
Max	186	270	409	358	418	64	87
St. Dev.	17.2	38.2	66.2	56.7	47.3	13.4	22.9
COV	10	18	23	22	12	28	35

4.3 Results of Laboratory Testing on Laboratory Prepared Specimens

Of the three gradations evaluated in this section, only one did not make use of washed material sampled from US 49 (the fine gradation). It is the only gradation that would be suitable for any sort of direct comparison with information collected on site during construction or over time during service. Also, recall that all specimens tested by MSU for strength were gyratory compacted. Note that the design strength after 7 days of room temperature curing (including five hours of submerged curing) for chemically stabilized base layers within MDOT standard protocols is 300 psi. Another item to recall is the testing performed referenced MDOT's since-discontinued cement index (C_I), rather than cement content by mass (C_W). Table 4.6 is an equivalency between C_I and Cw for the combinations evaluated in this section. Note that UCS properties in this section are as measured on 1.15:1 aspect ratio specimens since this was standard MDOT protocol during the time frame of this project. UCS values in this section should be reduced by 10% to correspond with 2:1 aspect ratio data presented elsewhere in this report.

Table 4.6. Relationship Between C_I and C_W

	C_W by Gradation		
C_I	Coarse	Average	Fine
5	3.7	3.8	3.9
6	4.5	4.6	4.8
7	5.3	5.4	5.6

4.3.1 Proctor Compaction Results – Laboratory Prepared Specimens

Table 4.7 summarizes Proctor test results performed on specimens batched as shown in Table 3.6. A constant G_{sb} value of 2.41 was used for adjustment purposes, which was taken from MDOT's construction report for US 49 (Strickland 2010). Testing performed later as part of State Study 250 resulted in +12.5 mm aggregates from US 49 with a G_{sb} of 2.48. Use of a G_{sb} of 2.48 as opposed to 2.41 would increase adjusted Proctor values by 0.7, 0.4, and 0.2 pcf for coarse, average, and fine gradations, respectively. Adjusted values in Table 4.7 were used as density targets for laboratory SGC specimen production.

Table 4.7 γ_d values are considerably higher than those provided by MDOT and BCD in Section 4.2. An initial investigation was carried out to explore discrepancies between BCD and MDOT data relative to MSU Proctor data; this investigation was conducted prior to learning BCD G1 to G35 gradations, on which MSU coarse, average, and fine gradations were based, contained cement. In the initial investigation, four single point Proctor specimens were prepared from material processed in the same manner as the specimens represented in Table 4.7. These specimens included combinations of taller molds, 6% cement by mass, hold times up to 1 hr and mixing by hand. Dry densities were 121.9 to 124.6 pcf (unadjusted) for the average gradation. These values were at, to modestly below, Table 4.7 values; thus, the initial investigation suggested compaction protocols were not likely the result of the density discrepancies between Sections 4.2 and 4.3.

Table 4.7. MSU Proctor Compaction Test Results

MDOT Protocol	Gradation	C_w (%)	+12.5 mm (%)	Unadjusted γ_d (pcf)	Unadjusted OMC (%)	Adjusted γ_d (pcf)	Adjusted OMC (%)
MT-8	Coarse	0	22	126.9	6.7	131.3	5.3
	Average	0	14	125.4	7.2	128.2	6.3
	Fine	0	6	123.1	7.6	124.4	7.2
MT-9	Coarse	4.5	22	126.4	7.2	130.9	5.7
	Average	4.6	14	124.7	7.2	127.7	6.3
	Fine	4.8	6	121.2	8.6	122.6	8.1

Note: MDOT protocols (MT-8 and MT-9) were not followed in all cases; see Section 3.3.2.4.
Note: All cement Proctors were performed only at C_I = 6% due to limited material. For purposes of specimen preparation, all compaction targets were based on Table 4.7, which is an approximation but was deemed reasonable.

It was eventually assumed the most likely explanation for density differences between Sections 4.2 and 4.3 was the sieving and handling processes employed for laboratory prepared specimens. Section 4.3 Proctor testing exhibited higher γ_d values and lower optimum moisture contents, which could possibly occur from more handling and washing by dislodging particles prior to compaction. When correspondence with BCD revealed that BCD gradations contained cement, even more clarity was achieved; Section 4.3 γ_d values were likely higher because the gradations tested had approximately 5% more fines relative to BCD and MDOT data. Section 4.5 provides additional discussion of results considering all compaction data available.

4.3.2 Wheel Tracking Results – Laboratory Prepared Specimens

Table 4.8 provides APA test results for the six specimens tested; two replicate specimens were tested per gradation in one track of the APA device. Dry testing (8,000 cycles) was performed first, and thereafter, submerged testing (8,000 additional cycles) was performed on the same specimens. Rut depths reported for submerged testing are only that which occurred after dry testing was concluded (i.e. rut depths were zeroed after dry testing). Rut depths were insignificant as shown in Table 4.8 and Figure 4.2, which suggests the cement-stabilized FDR materials are not susceptible to rutting in wet or dry conditions. This behavior is not surprising.

Table 4.8. APA Test Results

Gradation	$N_{G\text{-}Avg}$	$w_{\%\text{-}M}$ (%)	$\gamma_{\%\text{-}Avg}$	C_w (%)	Dry Rut (mm)	Wet Rut (mm)
Coarse	26	8.0	97.3[a]	4.5	0.1	<0.1
Average	17	7.4	98.6	4.6	0.5	0.4
Fine	12	9.6	99.7	4.8	0.5	1.5

a: This value was slightly below the density tolerance, but specimens were not re-made due to low rut depths.
-- As-prepared moisture content and dry density values are provided, and all values are C_I of 6%.

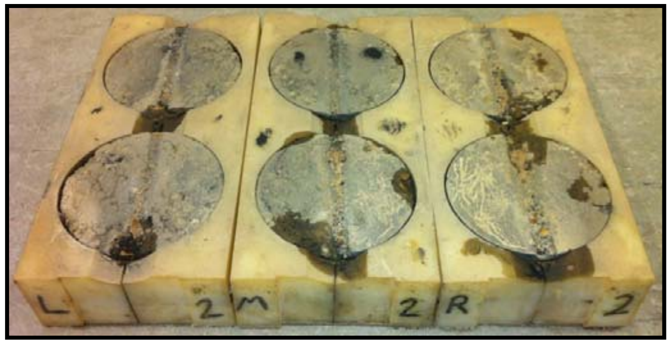


Figure 4.2. Photos of FDR Specimens After Submerged APA Testing

4.3.3 Strength Versus Time Results – Laboratory Prepared Specimens

Figure 4.3 plots strength versus time results. The specimens tested to produce Figure 4.3 had the following properties: $N_{G\text{-}Avg}$ of 29, $w_{\%\text{-}M}$ of 6.6, $\gamma_{\%\text{-}Avg}$ of 100.3 pcf, D_{Avg} of 100.1 mm, and H_{Avg} of 114.5 mm. One-day strength was just below 300 psi, and after 540 days (approximately 18 months), strength had increased to 671 psi. The 18-month strength was 1.44 times higher than the 28-day (around 1 month) strength, which is a manageable amount of strength gain over time. Excessive strength gain over time can indicate cracking concerns.

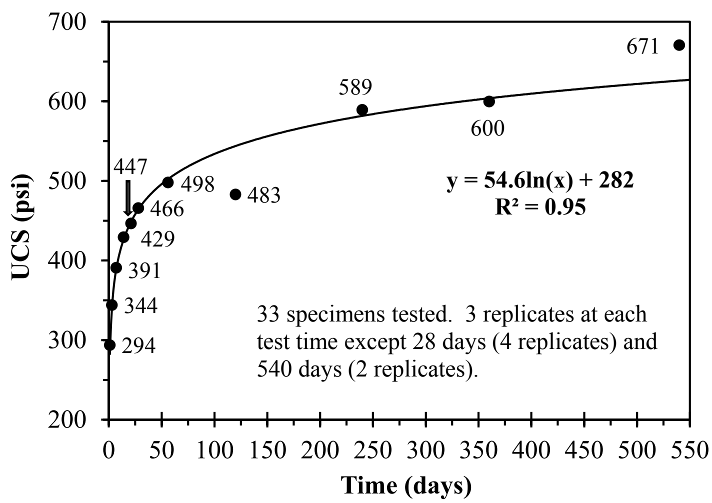


Figure 4.3. Strength Versus Time Test Results: Average Gradation, C_I of 6%

4.3.4 Gradation Effects Results – Laboratory Prepared Specimens

Table 4.9 provides all gradation effects results. As seen, all cases but the average gradation at a 6% cement index had a test replication of 10, while the average gradation at a cement index of 6% had a test replication of 30 (or three sets of 10 replicates each). All of the average gradation data at a 6% cement index data is provided in Table 4.9. The number of gyrations to achieve Proctor density increased as the gradation became coarser, which is expected. Averaging all data from a given gradation led to the fine, average, and coarse gradations being represented by 21, 27, and 50 gyrations, respectively. UCS COV values were very manageable at 3 to 8%.

Table 4.9. Gradation Effects Test Results for SGC Compacted Specimens

G	n	Test Day	$\gamma_{T\text{-}Avg}$ (pcf)	$\gamma_{\%\text{-}Avg}$ (%)	C_I (%)	$w_{\%\text{-}M}$ (%)	$N_{G\text{-}Avg}$	UCS_{Avg} (psi)	UCS_{COV} (%)
Average	10	7	136.4	100.5	5	6.0	23	347	3
	10	7	136.1	100.2	6	6.0	34	382	7
	10	7	136.3	100.4	6	7.1	26	383	6
	10	7	136.2	100.3	6	7.1	25	369	8
	30	*7*	*136.2*	*100.3*	*6*	*6.7*	*28*	378	*7*
	10	7	136.3	100.4	7	6.1	25	441	7
Fine	10	7	133.8	101.0	5	8.8	22	369	4
	10	7	133.6	100.8	6	8.0	21	414	6
	10	7	133.5	100.7	7	7.9	19	482	6
Coarse	10	7	138.4	100.1	5	6.4	52	383	5
	10	7	138.4	100.0	6	6.2	51	433	5
	10	7	138.4	100.0	7	5.1	47	504	4

--D_{Avg} was 100.0 to 100.1 mm and H_{Avg} was 113.9 to 114.6 mm

Figure 4.4 plots gradation effects UCS results presented in Table 4.9. The coarse gradation had the highest strength, even though the cement content by mass (Table 4.6) was 0.1 to 0.3% lower than the other gradations for a constant cement index. The maximum ranges of average UCS were 36, 51, and 63 psi for cement indices of 5, 6, and 7%, respectively. Practically speaking, expecting 40 to 60 psi difference in compressive strength at 7 days seems reasonable to expect for an FDR material similar to US 49.

Considering a typical MDOT design strength was easily exceeded in all Figure 4.4 cases, this level of strength variability due to extreme changes in gradation is manageable. Typical design strength was exceeded by around 50 psi for the worst case tested, which was 1% below the design value in terms of cement index. Figure 4.4 suggests gradation variability's effect on strength is not a first order concern.

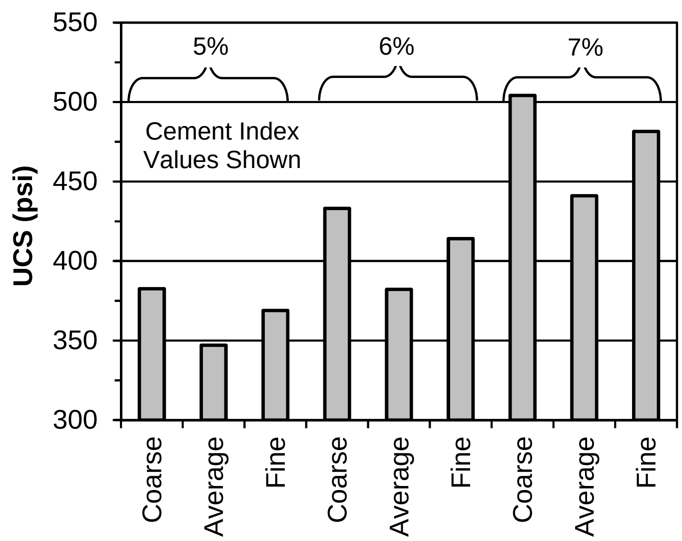
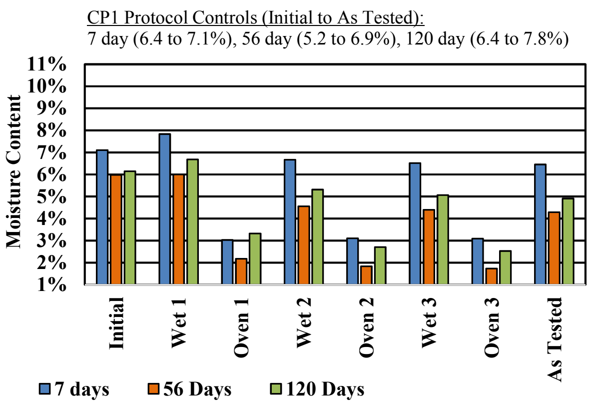


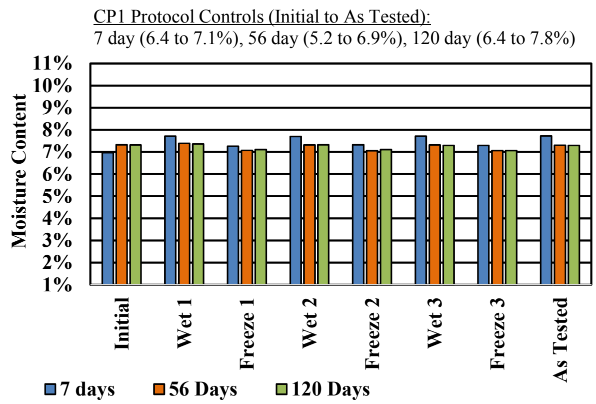
Figure 4.4. Gradation Effects Test Results

4.3.5 Curing Durability Results – Laboratory Prepared Specimens

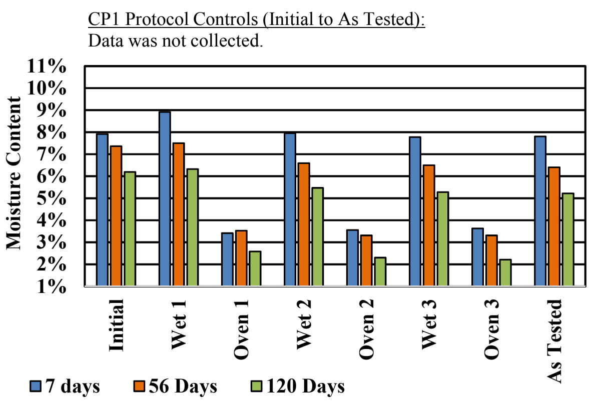
Figure 4.5 plots an estimate of moisture content over the duration of each curing protocol. As mentioned previously, initial moisture contents were taken on small samples and thus, are variable. The remaining values were determined by weighing specimens and referencing the initial moisture content, so they too would be subject to variability.



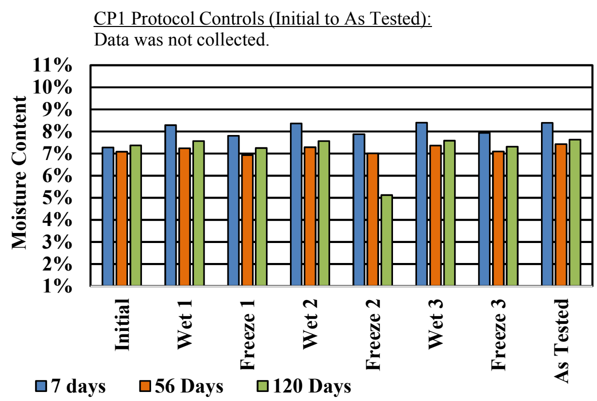
a) Coarse Gradation-CP2



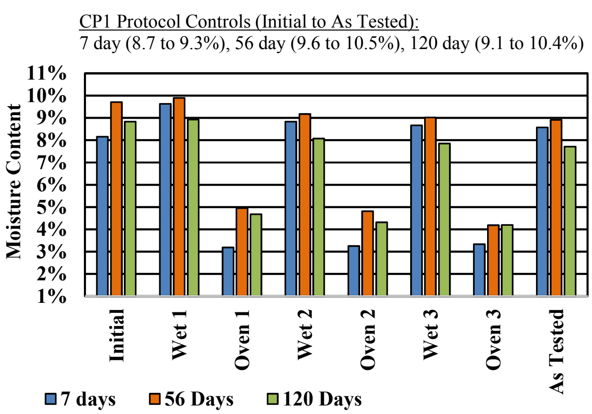
b) Coarse Gradation-CP3



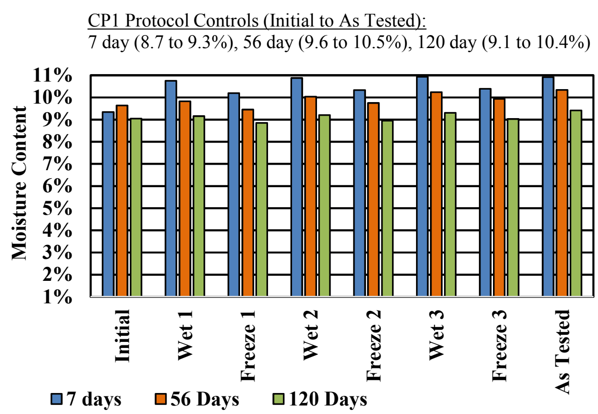
c) Average Gradation-CP2



d) Average Gradation-CP3



e) Fine Gradation-CP2



f) Fine Gradation-CP3

Figure 4.5. Curing Durability Moisture Content Results

Where available, data collected on the same type of specimens during the reference CP1 (i.e. 100% humidity room at room temperature) was shown to provide a perspective for the amount of moisture changes that were occurring throughout the different curing protocols. CP1 represents traditional curing, and moisture content increased, on average, 1.1% (range of 0.6 to 1.7%) from the range of curing times and gradations represented in Figure 4.5. CP2 evaluated effects of relatively rapid wetting and drying. Moisture content changes were pronounced between wetting/drying cycles as seen in Figure 4.5. CP3 evaluated effects of relatively rapid freezing/thawing. In general, moisture contents throughout CP3 did not change relative to initial moisture contents.

Table 4.10 provides temperature-time factors (*TTF*s) for the curing protocols incorporated into this section. *TTF*s are shown to provide an estimate of maturity. *TTF* calculation used a reference temperature of 0 °C since cement hydration effectively does not occur below 0 °C. The freezer's contribution to *TTF* was taken to be zero since negative temperatures would not reverse cement hydration, it would only prohibit hydration. ASTM C1074 Equation 1 was used for calculations. Table 4.10 shows that, for specimens that are not negatively affected durability wise by wetting and drying or freezing and thawing, CP2 should be stronger than CP1, which should be stronger than CP3 (CP1 should be only be slightly stronger than CP3 and the gap should decrease with cure time). If there are durability problems, the fine gradation would, generally speaking, be expected to be the most susceptible to problems.

Table 4.10. CP *TTF* Values

Cure Time	***TTF* (°C-days)**		
(days)	**CP1**	**CP2**	**CP3**
7	168	309	96
56	1344	1485	1272
120	2880	3021	2808

UCS test results from curing durability testing are provided in Table 4.11. In every case, the trend of CP2 being considerably stronger than CP1 and CP1 being slightly to modestly stronger than CP3 held true. UCS increased over time in all cases. There was no evidence the wetting/drying (with heat) or freezing/thawing had any detrimental effects on the specimens. The key Table 4.11 finding is that controlled laboratory testing on specimens screened and carefully re-combined to a desired gradation indicate the FDR materials from US 49 were not susceptible to strength loss from durability issues such as wetting and drying or freezing and thawing.

Table 4.11. Curing Durability Test Results of SGC Compacted Specimens

G	n	Test Day	$\gamma_{\%\text{-Avg}}$ (%)	C_I (%)	$w_{\%\text{-M}}$ (%)	$N_{G\text{-Avg}}$	Curing Protocol	UCS_{Avg} (psi)
Coarse	3	7	99.5	6	6.4	50	CP1	395
		7	99.6	6	7.1	59	CP2	529
		7	99.8	6	7.0	56	CP3	362
		56	100.2	6	5.2	39	CP1	537
		56	99.7	6	6.0	38	CP2	690
		56	99.8	6	7.3	51	CP3	489
		120	100.1	6	6.4	44	CP1	577
		120	100.4	6	6.2	35	CP2	771
		120	100.4	6	7.3	48	CP3	510
Average	3	7	100.2	6	6.1	33	CP1	391
		7	100.1	6	7.9	26	CP2	519
		7	100.2	6	7.3	30	CP3	283
		56	100.4	6	6.3	30	CP1	498
		56	100.1	6	7.4	26	CP2	682
		56	100.1	6	7.1	25	CP3	460
		120	100.3	6	7.2	29	CP1	483
		120	100.4	6	6.2	22	CP2	710
		120	100.5	6	7.4	26	CP3	439
Fine	3	7	100.9	6	8.7	31	CP1	356
		7	100.9	6	8.2	35	CP2	638
		7	100.8	6	9.3	30	CP3	290
		56	100.5	6	9.6	22	CP1	465
		56	100.6	6	9.7	23	CP2	774
		56	100.5	6	9.6	23	CP3	400
		120	100.9	6	9.1	24	CP1	562
		120	101.0	6	8.8	22	CP2	799
		120	100.9	6	9.0	26	CP3	478

--D_{Avg} was 100.0 to 100.3 mm, and H_{Avg} was 114.0 to 114.8 mm.
--Note that Average gradations with CP1 are identical to Strength vs. Time specimens, so these 9 specimens were not remade. This Table has 81 specimens, but only 72 unique specimens, 9 were re-used from Strength vs. Time specimens.

4.4 Field Monitoring Results of Field Testing Performed After Construction

Field monitoring is organized into three activities: FWD testing, automated profiler distress surveys, and coring with associated laboratory testing. Results from field core testing are presented first since some results were used to assist FWD analysis and discussion. FWD testing took place at 24, 28, 34, 40, and 53 months. All times referenced are with respect to the complete opening of US 49 to public traffic. US 49 was profiled for roughness at 10 months, and a full distress survey was conducted at 53 months. Coring occurred at 53 months.

4.4.1 Results From Characterization of Field Cores

4.4.1.1 Subgrade Properties

Table 4.12 presents properties of the two composite subgrade samples taken from the bottom of core holes. Both samples classified as low plasticity clay soils, which are A-6 soils by the AASHTO classification system. Table 11-10 in the *MEPDG* Manual (2008) provides typical M_R values for A-6 soils ranging from 14 to 17 ksi.

Table 4.12. Properties of Subgrade under US 49 FDR

Sample	1	2
Description	Brown Fine Grained Soil	Grayish-Brown Fine Grained Soil
Liquid Limit	33	37
Plastic Limit	21	20
Plasticity Index	12	17
P_{200} (%)	91.2	96.5
Unified Soil Classification	CL	CL
AASHTO Classification	A-6	A-6

Note: CL refers to low-plasticity clay

4.4.1.2 Layer Thicknesses

Table 4.13 presents a summary of US 49 layer thicknesses measured on the 12 FDR cores obtained. AC surface lift thickness ranged from 1.3 to 2.0 inches with an average of 1.6 inches, which is similar to the target thickness of 1.5 inch on average. AC base lift thickness ranged from 2.5 to 3.4 inches with an average of 2.8 inches, which is slightly lower on average than the 3 inch target thickness.

FDR thickness was 15.3 inches on average, which is relatively similar to the 16 inch target thickness. However, the overall range in FDR thickness 7.5 inches from 12.0 to 19.5 inches. Similarly, the FDR layer thickness 95% confidence interval ranges from 11.1 to 19.5 inches. Practically speaking, this range is nearly half of the overall target thickness.

Table 4.13. US 49 Cored Layer Thicknesses

Property	**AC Surface**	**AC Base**	**FDR**
Mean (in)	1.6	2.8	15.3
Min (in)	1.3	2.5	12.0
Max (in)	2.0	3.4	19.5
St. Dev. (in)	0.26	0.31	2.07
COV (%)	16	11	14

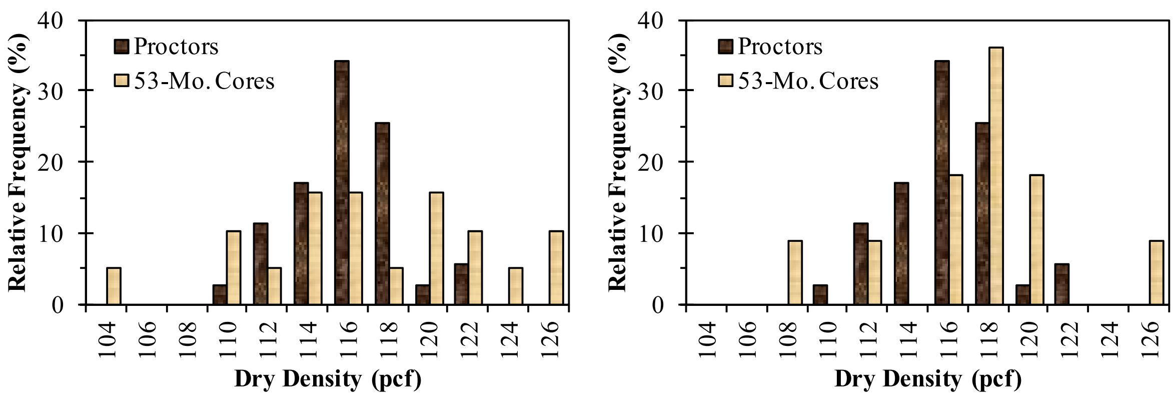
4.4.1.3 FDR Density

Density was measured only on UCS specimens, as described in Section 3.5, and was analyzed two ways. First, all UCS specimens were considered individually. In this case, multiple pairs existed where two UCS specimens were sliced from the same core, one from the top portion (approximately 0 to 6 inches from the top of the FDR layer) and one from the bottom portion (approximately 6 to 12 inches from the top of the FDR layer). Second, UCS specimens that formed top-portion and bottom-portion pairs were considered jointly to approximate density for the original core.

In the first analysis where all UCS specimens were independently considered, $\gamma_{dry\text{-}T331}$ ranged from 103 to 125 pcf and was 116 pcf on average. Paired *t*-tests were conducted and found top-layer $\gamma_{dry\text{-}T331}$ was significantly higher (*p-value* was less than 0.01) than bottom-layer $\gamma_{dry\text{-}T331}$ by 7.1 pcf on average. This finding is logical since compaction would be less effective at greater depths. In the second analysis where top- and bottom-layer densities were averaged and jointly considered, $\gamma_{dry\text{-}T331}$ ranged from 108 to 124 pcf and was 116 pcf on average. COV for $\gamma_{dry\text{-}T331}$ decreased from 5% to 4% when $\gamma_{dry\text{-}T331}$ values were combined where possible to form average core densities.

Figure 4.6 presents γ_{dry} distributions for Table 4.4 Proctor specimens and 53-month cores. Table 4.4 $\gamma_{dry\text{-}1\ pt}$ ranged from 110 to 121 pcf and averaged 115 pcf. Table 4.4 $\gamma_{dry\text{-}1\ pt}$ values were, within reason, similar to MDOT Proctor γ_d values (Table 4.1), which ranged from 111 to 121 pcf and averaged 117 pcf. In Figure 4.6a, 53-month core densities, when considered independently, demonstrated a wider spread than Table 4.4 Proctor densities. Figure 4.6b core density distribution, when jointly considered, was similar to that of Table 4.6 Proctor densities, indicating overall field-compacted density variability was on the order of Proctor-compacted density variability, suggesting overall observed variability (e.g. range of 110 to 121 pcf) is associated within the FDR material.

US 49 full-pay density was 97% of standard Proctor density, which would be 113.4 pcf using Table 4.1 average data and 111.6 pcf using Table 4.6 average data. In general, 53-month core densities met 97% Proctor density. The few exceptions in Figure 4.6 were influenced by low bottom-layer $\gamma_{dry\text{-}T331}$. Despite significant density gradients, overall compaction and density was satisfactory based on a Proctor density reference.



a) Distribution if Top and Bottom Layers Separated *b) Distribution if Bulk Core Evaluated*

Figure 4.6. Distribution of $\gamma_{dry\text{-}T331}$ (Cores) Compared to $\gamma_{dry\text{-}1\ pt}$ (Proctors)

4.4.1.4 FDR Strength and Durability

Table 4.14 presents properties of all 53-month cores except for those testing in the APA. Figure 4.7 shows core UCS and *E* results. Unlike Proctor UCS results in Table 4.4, cores were considerably affected by density. Density effects yielded an observed range of UCS and *E* that are not necessarily a result of excessive strength/stiffness gain over time or meaningful material degradation. Therefore, density effects were normalized using normalization factors which were a function of density (Figures 4.7b and 4.7d).

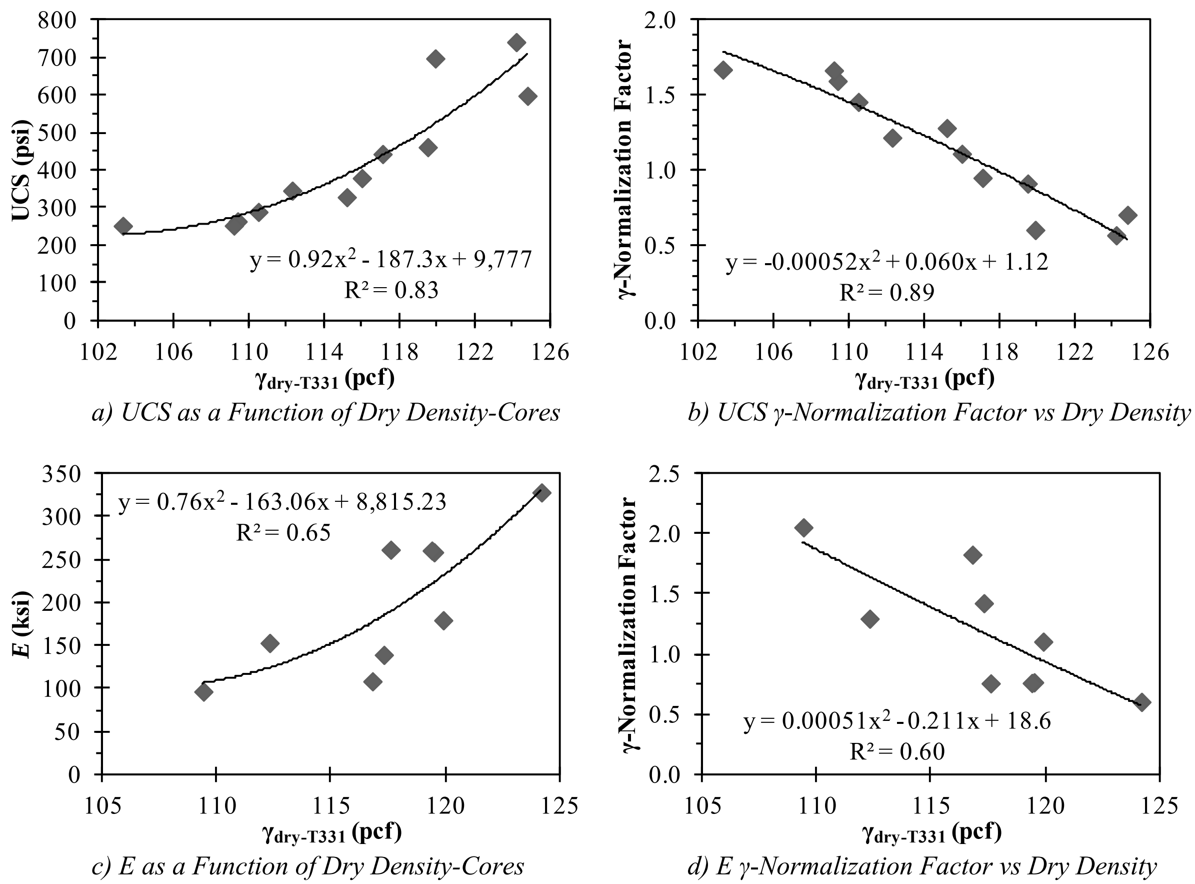
E and UCS in Table 4.14 were on average around 200 ksi and 400 psi, respectively. *E* and UCS COVs were considerably lower after density normalization, while the average did not change considerably. A paired *t*-test was conducted on as-measured UCS values and verified top-layer UCS was significantly higher than bottom-layer UCS (*p-value* = 0.02 and average difference of 252 psi), which was largely driven by density differences.

Table 4.14. US 49 Core Properties

	E (ksi)		UCS (psi)		S_t (psi)	$w_{\%\text{-}M}$ (%)
	As-measured	γ-Normalized	As-measured	γ-Normalized	As-measured	As-measured
Mean	199	212	421	406	75.1	4.3
n	9	9	12	12	7	27
Min	97	133	252	290	19	2.9
Max	328	301	741	569	165	5.9
St. Dev.	81.2	55.8	172	66	50.0	1.2
COV (%)	41	26	41	16	67	28

-- All UCS values were adjusted to an H/D ratio of 2.0 according to ASTM C39.

Density normalization was not performed for S_t since density was not measured for those specimens. S_t COV is considerably high, which is likely tied to density variability as well. When as-measured S_t is compared to as-measured UCS, COV is noticeably higher, which is similar to the trend observed in Table 4.5.

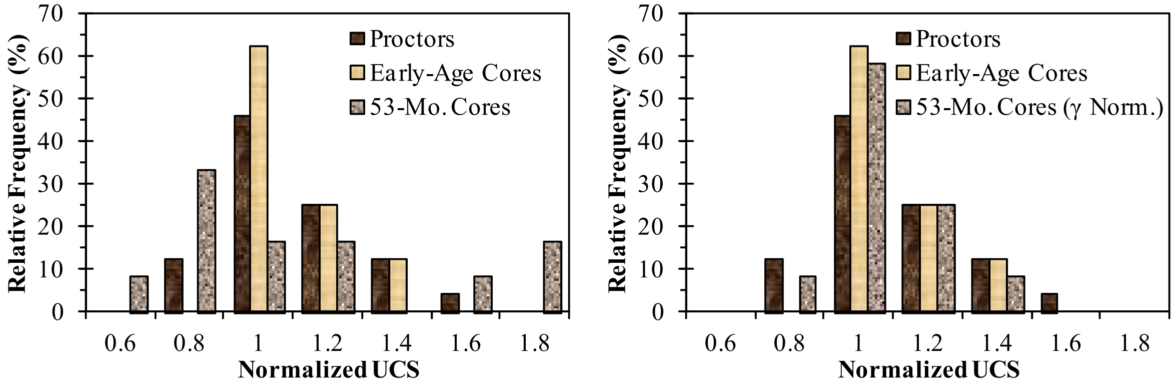


a) UCS as a Function of Dry Density-Cores *b) UCS γ-Normalization Factor vs Dry Density*

c) E as a Function of Dry Density-Cores *d) E γ-Normalization Factor vs Dry Density*

Figure 4.7. Dry Density Relationships for Core Properties

Figure 4.8 shows normalized UCS distributions of Proctor specimens (Table 4.4), early-age cores (Table 4.4), and 53-month cores for one-to-one comparison. UCS was normalized to one for each specimen type and day (e.g. 14-day Proctor specimens were normalized by dividing by the average 14-day Proctor UCS). Note that this normalization was independent of density normalization. Specimens of each type were grouped together for general variability comparison. Figure 4.8a shows the distribution prior to density normalization; Figure 4.8b shows the distribution after density normalization.



a) UCS Distribution Absent γ-Normalization *b) UCS Distribution After γ-Normalization*

Figure 4.8. Normalized UCS Distributions for Variability Assessment

Practically, variability was similar between all three Figure 4.8b data sets. This suggests that, other than gaining a modest amount of strength relative to Table 4.4, the 53-month field cores do not appear to have changed considerably from the first few weeks after construction. Noticeable increases in variability over time could suggest incomplete cement mixing leading to relative strength gain differences due to hydration and/or degradation due to cracking. However, variability increases were not observed at 53 months.

APA tests were conducted as described in Section 3.3.4.2 and Section 3.5. Six replicates were tested in a dry state and then retested in a submerged, or wet, state. Results on 53-month field cores were similar to laboratory-compacted FDR results presented in Section 4.3.2 in that rutting was insignificant and negligible. Dry-test APA rut depths were all less than 1 mm, and wet-test APA rut depths were all less than 2 mm.

4.4.2 FWD and Structural Capacity Results

Twelve FDR locations were tested over time with the FWD, and they are labeled FWD1 to FWD12 and can be seen in Figure 3.1. FWD tests were conducted five times: November 2012, March 2013, September 2013, March 2014, and June 2015. FWD testing times are further denoted FWD Phases 1 to 5, respectively. Just after the FWD Phase 5 testing, five of these twelve locations were cored directly below the position of the FWD load plate (i.e. the spot was marked prior to FWD testing and a core cut in the middle of the area where the FWD load plate was positioned). These locations were FWD4, FWD6, FWD8, FWD10, and FWD12.

Procedures documented in the *1993 AASHTO Pavement Design Guide* (AASHTO 1993), which is hereafter referred to as the *1993 Guide*, were used to analyze US 49 FWD data since a structural number (SN) approach was used within MDOT at the time the project was constructed. For each FWD location and phase, deflections were normalized to 9 kips using linear regression of data at all available applied FWD loads (target FWD loads ranged from 6 to 18 kips). In accordance with the *1993 Guide*, the deflection under the center of loading was also corrected for temperature effects (the other measurements were not temperature corrected). Figure L5.5 of (AASHTO 1993) was used to determine temperature correction factors (*C*). Measured asphalt surface temperatures were used as the Figure L5.5 input. This approach, while not ideal for temperature correction for US 49, incorporated cement stabilized base that was 10 in thick with an elastic modulus of 850 ksi (5.86 GPa).

Effective structural number (SN_{eff}) was the primary output of the *1993 Guide* and was used with known layer thicknesses and typical AC layer coefficients (a_1) to calculate FDR layer coefficients (a_2). Equation 5.15 (AASHTO 1993) was used to iteratively solve for the effective pavement modulus which was used in Equation 5.6 (AASHTO 1993) to calculate SN_{eff}. The FDR layer coefficient (a_2) was calculated using the *1993 Guide* SN equation presented herein as Equation 4.1.

$$SN = a_1 D_1 + a_2 D_2 m_2 \tag{4.1}$$

Where,
SN = structural number
a_1 = layer coefficient of asphalt layer (MDOT currently uses 0.44)
a_2 = layer coefficient of FDR layer
D_1 = thickness of asphalt layer (in)
D_2 = thickness of FDR layer (in)
m_2 = drainage coefficient of FDR layer (MDOT currently uses 1.0)

Table 4.15 presents FWD results for the five locations previously mentioned where a core was obtained directly underneath the FWD load plate position. Thicknesses shown are those directly measured from cores. Although these thicknesses are likely more representative than plan thicknesses or thicknesses from pavement management data, measurements are approximate in some cases (e.g. when layers separated at the layer interface during coring, in the case of stripping of underlying layers, etc.).

AASHTO SN_{eff} averaged 7.8 for all FWD locations and phases. In order of observation frequency, 40% of observations were between 8 and 9, 36% were between 7 and 8, and 20% were between 6 and 7. AASHTO a_2 averaged 0.36 for all FWD locations and phases. In order of observation frequency, 40% of observations were between 0.34 and 0.38, 32% were between 0.38 and 0.42, and 20% were between 0.28 and 0.30. In comparison, cement-stabilized FDR a_2's presented in Chapter 2 generally ranged from 0.28 to 0.32 generally support measured US 49 a_2's.

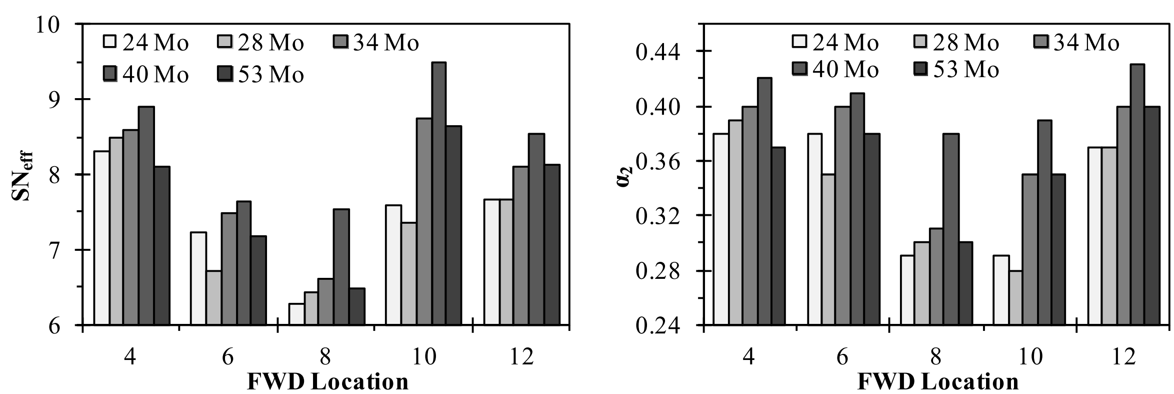
Figure 4.9 plots SN_{eff} and a_2 values from Table 4.15 over time. A general increase in both properties is observed over time. On average, SN_{eff} has increased at a rate of approximately 0.2 units per year, while a_2 has increased at a rate of approximately 0.013 units per year. Average trends would have been slightly more pronounced if only 24 to 40 month data was considered. Testing at 53 months exhibited SN_{eff} and a_2 decreases at all FWD locations. Given structural capacity trends up to 40 months, this behavior is interesting but not overly concerning.

Figure 4.10 plots FWD results versus core properties. SN_{eff} increased as the FDR gradation fineness increased, or as density decreased (neither behavior is intuitive nor understood). Layer coefficient (a_2) was generally unaffected by either gradation or density and remained around 0.36 on average. Overall, differences in FWD results by FWD location could not be reliably connected to core properties such as gradation or density.

Table 4.15. FWD Results at Locations Where Cores Were Taken

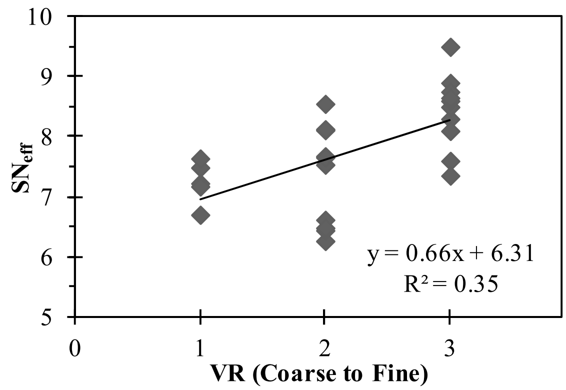
	Thickness (in)			FWD		Deflection (mils)								M_R	AASHTO	
ID	D_p	D_1	D_2	**Phase**	C	$d_{0\text{-}68}$	d_8	d_{12}	d_{18}	d_{24}	d_{36}	d_{48}	d_{60}	**(ksi)**	SN_{eff}	a_2
FWD4	21.0	4.5	16.5	1	1.03	2.9	2.3	2.2	2.1	2.0	1.8	1.3	1.0	34.2	8.3	0.38
				2	1.06	2.6	2.1	1.9	1.8	1.7	1.5	1.3	1.0	40.2	8.5	0.39
				3	0.94	2.6	2.3	2.1	1.8	1.9	1.6	1.3	1.0	38.2	8.6	0.40
				4	1.07	2.5	2.2	2.0	1.9	1.8	1.6	1.4	1.0	36.9	8.9	0.42
				5	0.94	2.9	2.4	2.2	2.0	1.9	1.6	1.4	1.1	36.8	8.1	0.37
FWD6	18.3	4.0	14.3	1	1.03	3.3	2.8	2.6	2.5	2.2	1.9	1.6	1.3	31.5	7.2	0.38
				2	1.06	3.7	2.9	2.7	2.5	2.2	1.9	1.6	1.3	31.6	6.7	0.35
				3	0.94	3.2	2.9	2.7	2.3	2.5	2.0	1.7	1.4	30.5	7.5	0.40
				4	1.07	3.1	2.8	2.6	2.4	2.3	1.9	1.6	1.4	30.9	7.6	0.41
				5	0.98	3.6	3.0	2.8	2.6	2.5	2.1	1.8	1.5	28.3	7.2	0.38
FWD8	19.3	4.8	14.5	1	1.03	4.2	3.7	3.2	2.6	2.2	1.8	1.5	1.3	33.6	6.3	0.29
				2	1.09	4.0	3.0	2.7	2.5	2.2	1.8	1.5	1.3	32.6	6.4	0.30
				3	0.98	3.9	3.4	2.7	2.1	2.3	1.8	1.6	1.3	32.9	6.6	0.31
				4	1.10	3.2	3.8	3.5	2.8	2.2	1.8	1.5	1.3	32.6	7.5	0.38
				5	0.98	4.3	4.1	3.9	2.7	2.5	2.1	1.7	1.4	28.4	6.5	0.30
FWD10	23.8	4.3	19.5	1	1.02	4.1	3.2	2.9	2.8	2.5	2.1	1.8	1.5	27.9	7.6	0.29
				2	1.03	4.0	3.2	2.9	2.5	2.2	1.8	1.5	1.2	33.3	7.4	0.28
				3	0.88	3.3	3.2	2.9	2.5	2.7	2.1	1.8	1.4	27.9	8.8	0.35
				4	1.04	2.8	3.1	2.8	2.6	2.4	2.0	1.7	1.4	29.9	9.5	0.39
				5	0.90	3.2	2.8	2.7	2.5	2.4	2.0	1.7	1.4	29.8	8.6	0.35
FWD12	19.9	5.1	14.8	1	1.02	3.5	3.1	2.9	2.8	2.6	2.2	1.7	1.4	27.5	7.7	0.37
				2	1.03	3.5	3.2	3.0	2.9	2.7	2.2	1.8	1.5	27.0	7.7	0.37
				3	0.89	3.2	3.3	3.1	2.6	2.9	2.2	1.8	1.5	27.2	8.1	0.40
				4	1.05	3.1	3.3	3.2	2.9	2.8	2.4	1.9	1.6	25.3	8.6	0.43
				5	0.85	3.3	3.3	3.1	2.9	2.7	2.3	1.9	1.6	26.4	8.1	0.40

-- D_p = total pavement thickness (in), D_1 = thickness of asphalt layer (in), D_2 = thickness of FDR layer (in)
-- $d_{0\text{-}68}$ = deflection under the center of loading (d_0) adjusted to reference temperature of 68 °F
-- d_r = deflection (mils) normalized to 9 kips where r is the radial distance (in) from the center of loading
-- M_R = subgrade resilient modulus according to AASHTO Eq. 5.23 (AASHTO 1993) using d_{36} as the deflection
-- SN_{eff} = effective structural number of in situ pavement calculated by AASHTO Eq. 5.6 (AASHTO 1993)
-- a_2 = layer coefficient of FDR layer

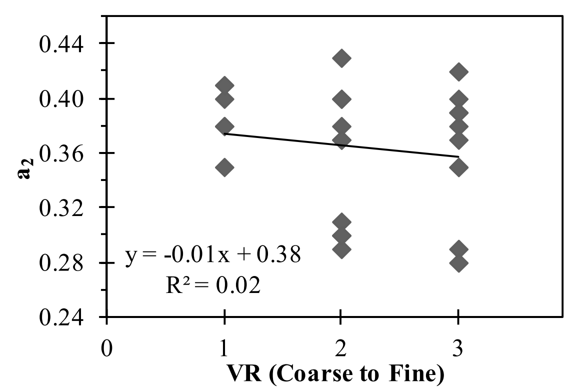


a) SN_{eff} by FWD Location with Time *b) a_2 by FWD Location with Time*

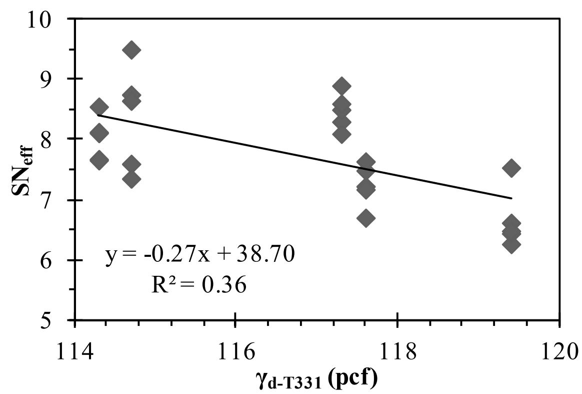
Figure 4.9. SN_{eff} and a_2 throughout 53 Month Monitoring Period



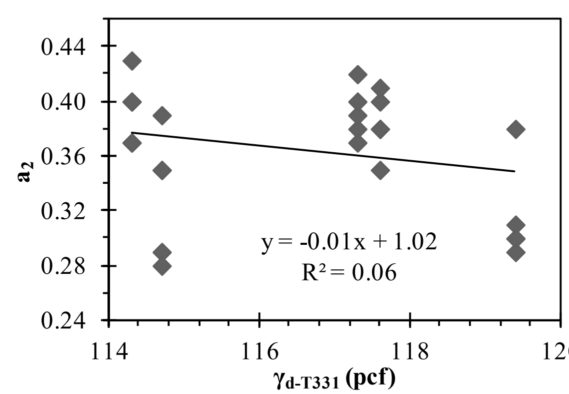
a) SN_{eff} versus Visual Gradation Rating (VR)



b) a_2 versus Visual Gradation Rating (VR)



c) SN_{eff} versus $\gamma_{dry\text{-}T331}$



d) a_2 versus $\gamma_{dry\text{-}T331}$

Figure 4.10. SN_{eff} and a_2 Correlations with Core Properties

Table 4.16 presents Analysis of Variance (ANOVA) multiple comparison *t*-group rankings of Table 4.15. Four ANOVAs were conducted, two per response variable (i.e. SN_{eff} or a_2). Two randomized complete block ANOVAs were performed per response variable. First, FWD phase was used as the block while location was the treatment, and vice versa for the second. Either can be the block or treatment in this case. Multiple comparison rankings assign each treatment ID (e.g. FWD location) a *t*-group letter; two treatments with different letters are significantly different from each other.

Table 4.16. ANOVA Rankings of AASHTO SN_{eff} and a_2

SN_{eff}						a_2					
FWD Location (*p-value* < 0.001)			**FWD Phase (*p-value* < 0.001)**			**FWD Location (*p-value* < 0.001)**			**FWD Phase (*p-value* < 0.001)**		
***t*-Group**	**ID**	**Mean**	***t*-Group**	**ID**	**Mean**	***t*-Group**	**ID**	**Mean**	***t*-Group**	**ID**	**Mean**
A	4	8.5	A	4	8.4	A	12	0.39	A	4	0.41
AB	10	8.4	B	3	7.9	A	4	0.39	B	3	0.37
B	12	8.0	BC	5	7.7	A	6	0.38	BC	5	0.36
C	6	7.2	C	1	7.4	B	10	0.33	C	1	0.34
D	8	6.7	C	2	7.3	B	8	0.32	C	2	0.34

ANOVA results were always significant. SN_{eff} exhibited statistically significant differences with FWD location; nearly every FWD location is significantly different from all others, suggesting there is notable variability with respect to structural capacity across US 49

FDR sections. Note that statistically significant variability does not necessarily equate to practically significant variability, though it may. SN_{eff} demonstrated significant increases from Phases 1 and 2 to Phase 3 and then again to Phase 4. Phase 5 SN_{eff} statistically ranked in the middle of Phases 1, 2, and 3. For a_2, there were two groups of FWD locations. FWD4, FWD6, and FWD12 yielded a_2 in the high 0.30's; whereas, a_2 for FWD8 and FWD10 was in the low 0.30's. FWD phases ranked identically for a_2 and for SN_{eff}.

Table 4.17 presents FWD results were cores were not obtained. Since directly-measured layer thicknesses were not available at Table 4.17 FWD locations, $d_{0\text{-}68}$ was temperature-corrected using Table 4.15's average D_1. SN_{eff} and a_2 were not calculated in Table 4.17 since individual layer thicknesses were not known.

Table 4.17. FWD Results at Locations Where Cores Were Not Taken

ID	FWD Phase	*C*	Deflection (mils) $d_{0\text{-}68}$	d_8	d_{12}	d_{18}	d_{24}	d_{36}	d_{48}	d_{60}	M_R (ksi)
FWD1	1	1.03	3.4	3.1	3.0	3.0	2.8	2.4	2.0	1.6	24.8
	2	1.07	3.4	3.0	3.0	3.0	2.8	2.4	2.0	1.6	25.2
	3	0.95	3.6	3.4	3.3	3.1	3.3	2.6	2.1	1.7	23.5
	4	1.08	3.4	3.3	3.2	3.2	3.1	2.5	2.0	1.6	24.1
	5	0.96	4.6	4.6	4.6	3.9	3.5	2.9	2.3	1.9	20.6
FWD2	1	1.03	2.9	2.5	2.3	2.2	2.0	1.7	1.4	1.2	35.4
	2	1.06	2.8	2.2	2.1	2.0	1.9	1.6	1.4	1.1	36.6
	3	0.94	2.9	2.7	2.5	2.2	2.4	1.9	1.5	1.2	32.4
	4	1.07	2.6	2.3	2.2	2.0	1.9	1.6	1.4	1.1	37.3
	5	0.99	3.2	2.8	2.6	2.4	2.2	1.9	1.6	1.2	31.6
FWD3	1	1.03	2.5	2.3	2.2	2.1	1.9	1.6	1.4	1.1	37.3
	2	1.06	2.7	2.2	2.0	1.9	1.7	1.5	1.3	1.1	39.4
	3	0.94	2.6	2.5	2.3	2.0	2.2	1.7	1.4	1.1	36.3
	4	1.07	2.4	2.3	2.2	2.0	1.9	1.6	1.4	1.2	37.3
	5	0.98	2.7	2.4	2.3	2.1	2.0	1.7	1.4	1.2	35.6
FWD5	1	1.03	4.2	3.8	3.7	3.6	3.4	3.2	2.8	1.8	18.5
	2	1.05	5.0	4.7	4.7	4.7	4.6	4.5	4.3	3.4	13.2
	3	0.94	4.8	4.7	4.6	4.2	4.5	3.9	3.4	2.7	15.5
	4	1.07	5.4	5.1	5.2	5.3	5.4	5.6	5.8	1.5	10.7
	5	0.98	5.7	5.8	5.9	6.1	6.2	6.6	7.0	1.4	9.1
FWD7	1	1.03	5.6	4.7	4.2	3.8	3.3	2.7	2.1	1.7	22.5
	2	1.04	6.3	4.8	4.3	3.8	3.3	2.5	2.0	1.5	23.6
	3	0.91	4.6	4.3	3.9	3.1	3.6	2.6	2.1	1.7	23.0
	4	1.06	4.9	4.9	4.5	4.0	3.6	2.9	2.3	1.8	20.5
	5	0.92	4.5	4.1	3.8	3.4	3.1	2.6	2.1	1.6	23.3
FWD9	1	1.03	4.9	4.6	4.4	4.3	4.0	3.6	2.9	2.2	16.7
	2	1.04	4.8	4.1	3.9	3.7	3.4	3.1	2.7	2.3	19.6
	3	0.89	4.7	4.9	4.6	4.1	4.4	3.5	3.0	2.5	17.0
	4	1.05	4.5	4.8	4.5	4.2	4.0	3.5	3.0	2.5	17.1
	5	0.87	4.4	4.4	4.1	3.9	3.7	3.2	2.8	2.4	18.5
FWD11	1	1.03	3.6	3.1	2.9	2.8	2.5	2.2	1.9	1.6	27.2
	2	1.02	4.9	4.0	3.5	3.1	2.7	2.3	1.9	1.6	26.3
	3	0.87	4.0	4.1	3.8	3.2	3.5	2.6	2.1	1.8	23.3
	4	1.03	3.6	3.7	3.4	3.2	3.0	2.5	2.0	1.7	24.1
	5	0.87	3.4	3.4	3.2	3.0	2.8	2.4	2.0	1.7	24.8

-- C determined using average D_1 from all cores obtained.

-- FWD5 deflections are questionable.

Another randomized complete block ANOVA (FWD phases were used as the blocks) was conducted to determine if Table 4.15 FWD data, where cores were available, was representative of all 12 FWD locations based on $d_{0\text{-}68}$ and d_{36}, which were the primary deflections used in *1993 Guide* calculations. Significant differences were observed among FWD locations; however, with fairly low deflections for all 12 FWD locations, these differences are not all that meaningful from a practical perspective. For example, though FWD7 exhibited the highest deflections, a_2, if calculated using the range of Table 4.15 D_p's observed, ranged from 0.27 to 0.30, which is still within reason similar to a_2 values in Table 4.15.

Figure 4.11 illustrates Table 4.17 $d_{0\text{-}68}$ values over time. The trendline plotted shows that deflections were, on average, constant between 24 and 53 months. Some variability was present (enough for the ANOVA to indicate statistically significant differences as previously mentioned); however, deflections were low overall, ranging from approximately 2 to 6 mils.

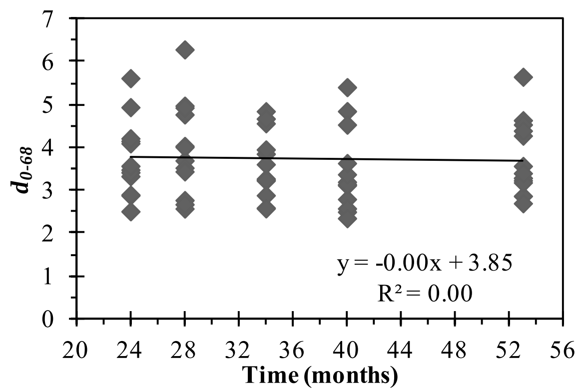


Figure 4.11. $d_{0\text{-}68}$ Data for All FWD Locations over Time

While a_2 is the primary FDR input needed for AASHTO design approaches, elastic modulus is a key *MEPDG* input. Table 4.14 provides E values that were measured on field cores and can be directly used as *MEPDG* Level 1 inputs. Table 4.14 values should be fairly reliable since they are aged field cores from a project that has been fairly well documented herein.

In addition to *MEPDG* Level 1 inputs, literature review and the data collected from US 49 also provide Level 2 input guidance. Syed (2009) found the relationship between FDR M_R and UCS to be around 900:1. The Table 11-7 relationship in (MEPDG 2008) for Level 2 input allows this relationship to be 1200:1. Table 4.14 values measured directly on US 49 with the *MEPDG*'s preferred method (ASTM C469) put the E to UCS relationship at around 520:1. This finding is meaningful since the Level 2 *MEPDG* input would have used a modulus for pavement design that is over twice the actual value after 53 months of service. Until further data is available, *MEPDG* users should use caution when predicting FDR E values where the material has a large amount of fine particles. Note that Sullivan et al. (2015) measured C469 E values on soil-cement in Mississippi (A-2-4 soil) and found the *MEPDG* Level 2 relationship of 1200:1 to reasonably represent the lower (or conservative) boundary of the data collected. The opposite was true for US 49 FDR.

4.4.3 Automated Distress Survey Results

Automated distress survey results served two purposes within State Study 250: 1) compare performance of CIR and FDR (see Volume 2 State Study 250 report); and 2) provide an overall performance assessment of FDR after 53 months of service. Table 4.18 provides the results of the distress survey performed by MDOT. The US 49 FDR was rated good according to MDOT's PCR criteria and four-lane route category. Mean roughness index (MRI) was low at 69 in/mile and did not change meaningfully from the profiling MDOT conducted in September of 2011 where MRI was 64 in/mile on average. Rutting was of no concern, and all observed block and fatigue cracking was low severity. FDR did, however, have around 30% low severity longitudinal cracking and around 20% low severity transverse cracking. Overall, FDR appears to be performing satisfactorily.

Table 4.18. Summary of US-49 FDR Distress Survey at 53 Months

Distress	Avg or Severity	FDR Results
PCR	Avg	87
MRI	Avg (in/mi)	69
	L (%)	83.7
	M (%)	15.1
	H (%)	1.1
Rutting	Avg (in)	0.05
	L (%)	97.1
	M (%)	2.5
	H (%)	0.4
Fatigue Cracking	L (%)	0.4
	M or H (%)	0.0
Block Cracking	L (%)	2.8
	M or H (%)	0.0
Longitudinal Cracking	L (%)	29.7
	M (%)	1.8
	H (%)	0.3
Transverse Cracking	L (%)	20.6
	M (%)	2.4
	H (%)	0.2

-- L = low, M = medium, H = high
-- For PCR, Very Good ≥ 89, 82 ≤ Good < 89, 73 ≤ Fair < 82, 63 ≤ Poor < 73, Very Poor < 63
-- For MRI, L: MRI < 150 in/mi, M: 150 < MRI < 300 in/mi, H: MRI > 300 in/mi
-- For rutting, L: 0.063 < Rut < 0.125 in, M: 0.125 < Rut < 0.250 in, H: Rut > 0.250 in
-- Fatigue and block cracking values were figured using 3.66 m lane widths
-- Edge cracking, patching, potholes, raveling, and bleeding were not detected

4.5 Discussion of Results

A brief discussion of results has been provided for items where between data set investigations was deemed potentially useful. For most of the analysis presented, the standalone assessments provided thus far have been sufficient. Rutting and density results are discussed in more detail in the remainder of this section.

With regard to rutting, laboratory prepared specimens and field obtained cores were in agreement that the FDR materials were not susceptible to rutting in a wet or a dry condition. Rut depths were below 2 mm for all cases evaluated. A rut depth below 2 mm in APA testing is well below typical levels of concern.

Data was collected from four sources for purposes of evaluating compaction and density. They were: MDOT data taken during construction; BCD data taken during construction; MSU laboratory data; and cores taken from US 49 around five years after construction. When all four sources of density data were considered, it was clear that the MSU laboratory prepared specimens had density values that were not representitive of field conditions. MDOT Proctor values were 111 to 121 pcf (117 pcf average), BCD Proctor results were 110 to 121 pcf (115 pcf average), and field cores were 108 to 124 pcf (116 pcf average). In contrast, MSU laboratory prepared Proctor values were 121 to 131 pcf. Items documented earlier in the report (in particluar additional fines batched into laboratory specimens since cement was in field gradations and more dispersed particles due to processing) seem to explain this differing trend.

To further evaluate the densities tested by MSU in the laboratory, gyrations required to achieve the aforementioned density values (i.e. Table 4.7) were investigated. Table 4.19 summarizes $N_{G\text{-}Avg}$ values for all four laboratory testing categories in Section 4.3. WT specimens are relatively short and there was minimal replication, so their results were reported in Table 4.19, but are largely ignored. A range of gyrations of 20 to 50 practically encompasses Table 4.19. Considering a typical design gyrations for in-place recycling is 30, this range does not suggest the MSU laboratory prepared specimens were compacted to an unreasonably large number of gyratrions. Table 4.19 generally supports the aforementioned statements that specimen processing and excessively added fine particles (around 5% too many) led to the increased densities relative to the rest of the data presented in this chapter.

Table 4.19. MSU Laboratory Prepared $N_{G\text{-}Avg}$ Summary

Category	Coarse Gradation	Average Gradation	Fine Gradation
WT	26[a]	17	12
ST	---	29	---
G-V	50	27	21
CD	47	27	26

a: only achieved 97.3% $\gamma_{\%\text{-}Avg}$

CHAPTER 5 – SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

5.1 Summary

This report's primary objective was to study FDR performed on US Highway 49 (US 49) in Madison county, Mississippi for purposes of evaluating properties and performance. Several aspects of this report are effectively a case study of high traffic FDR, while other aspects are a controlled parametric laboratory investigation not necessarily intended to interface directly with US 49, rather were performed to shed light on specific issues associated with high traffic FDR. Field data was presented for FDR activities from construction through 53 months of service. Laboratory data was presented for material samples collected during US 49 construction and evaluated in the laboratory. Three aspects make US 49 appealing as a case study: 1) the highly variable and large amount of particles finer than 75 μm; 2) the relatively deep reclaimed depth of 16 in; and 3) the presence of numerous fine particles in a relatively deep reclaimed layer used for a high-traffic application.

5.2 Conclusions

Laboratory and field testing concluded the US 49 FDR is performing well under high traffic. Specific conclusions are listed below.

1. US 49 field densities (as measured on cores) comfortably met 97% Proctor density requirements.
2. Density in the top 6 inches of the FDR layer was around 7 pcf higher than density in the lower six inches.
3. After 53 months of service, a representative unconfined compressive strength was around 400 psi, and a representative elastic modulus was around 200 ksi (this is a lower strength to modulus relationship than would be used in current Level 2 mechanistic-empirical pavement design).
4. Automated distress survey results after 53 months of service rated US 49 FDR "good" according to MDOT's PCR criteria for four-lane routes.
5. a reasonable a_2 layer coefficient suitable for the 1993 AASHTO Pavement Design Guide was found to be 0.30.

5.3 Recommendations

The primary recommendations from this study are provided in the following list, and they are mostly related to pavement design.

1. Use a_2 of 0.30 for cement stabilized FDR in MDOT pavement designs.
2. Do not use a 1200:1 relationship for relating elastic modulus to unconfined compressive strength for FDR where conditions are similar to US 49. A value closer to 500:1 was measured, and is recommended until more data is collected.
3. Continue to monitor US 49 every two to three years for the foreseeable future.

CHAPTER 6 - REFERENCES

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