DEPARTMENT OF TRANSPORTATION

Field Investigation of Stabilized Full-Depth Reclamation (SFDR)

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Research Project
Final Report 2018-33

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16. Abstract (Limit: 250 words)

The objectives of this study were to document the performance of roads using full-depth reclamation (FDR) and stabilized FDR (SFDR) in Minnesota, help develop SFDR design parameters appropriate for Minnesota, provide information on FDR/SFDR design procedures and specifications from beyond Minnesota, share current Minnesota FDR practices, and catalog the characteristics of some FDR/SFDR roads.

A comprehensive literature review of FDR/SFDR projects and case studies was conducted, and an online survey was distributed to Minnesota local road agencies to determine the stabilizing agents used for SFDR projects. Eighteen FDR/SFDR test sections from eight counties were then selected for a case study, and performance data and core samples were collected for the sections. Minnesota gravel equivalency (GE) analysis was performed to back-calculate the granular equivalent factor for FDR/SFDR layers based on the design equivalent single axle loads (ESALs) and R-values for subgrade soils.

The back-calculated GE values indicate that designers have likely been using GE values for FDR/SFDR layers that are consistent with current recommendations. It is recommended that the current GE values be generally retained for FDR/SFDR design. However, when slower-moving vehicles are the critical design consideration, relatively robust performance of FDR/SFDR layers may be expected. Visual distress surveys indicated that the FDR/SFDR bases studied are performing well in terms of destroying crack patterns that are often reflected through traditional hot mix asphalt (HMA) overlays. Therefore, decision makers may want to consider the use of FDR/SFDR as a base for reasons other than structural capacity.

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FIELD INVESTIGATION OF STABILIZED FULL-DEPTH RECLAMATION (SFDR)

FINAL REPORT

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EXECUTIVE SUMMARY

This report summarizes the research efforts performed to fulfill the goal of the Local Road Research Board (LRRB) INV981 research project entitled "Field Investigation of Stabilized Full-Depth Reclamation," Minnesota Department of Transportation (MnDOT) Contract 99004, Work Order No. 21, based on Research Needs Statement 369 for the Minnesota LRRB.

Full-depth reclamation (FDR) is a pavement rehabilitation technique in which the full flexible pavement section and a predetermined portion of the underlying materials are uniformly pulverized and blended together to produce a homogeneous stabilized base course. FDR and stabilized FDR (SFDR), which is FDR with the use of stabilizing additives, are widely used in Minnesota as an effective way to recycle aggregate and binder and provide a stabilized base for a roadway. In Minnesota, the interest in FDR is increasing due to the following reasons: highway network deterioration, maintenance funding reduction, sustainability concerns, and a desire to mitigate reflective cracking. There are several challenges with this technology, including a lack of knowledge and experience on how to design roads that include FDR and SFDR layers and a lack of information regarding the long-term performance of FDR/SFDR sections in Minnesota. Therefore, a more systematic effort to investigate the performance of these roads is desirable.

The objectives of this study are to document the performance of Minnesota FDR and SFDR roads and to aid in developing SFDR design parameters that are appropriate for Minnesota. Additional objectives are to provide information on FDR and SFDR design procedures and specifications from beyond Minnesota, share current FDR practices in Minnesota, and catalog the locations and characteristics of some FDR and SFDR roads. These objectives will facilitate the comparison of FDR experiences among various agencies in order to help the agencies approach FDR and SFDR road rehabilitation in a more informed manner.

In this study, a comprehensive literature review of various FDR and SFDR research projects and case studies was conducted. Representative FDR and SFDR road sections were documented. An online survey was distributed to Minnesota local road agencies, including counties and cities. According to the survey, BaseOne, asphalt emulsion, fly ash, cement, and foamed asphalt were the various types of stabilizing agents used for SFDR projects. Among them, BaseOne, asphalt emulsions, foamed asphalt, and cement had only occasional use. The local road agencies that were identified as having roads that could be case study road candidates were contacted by the research team. MnDOT also provided historical documents for some state trunk highways that have FDR and SFDR test sections and would be of potential research interest. A total of 18 test sections were selected from the candidate road list for consideration in the case study.

Based on dynamic cone penetrometer testing, FDR bases stabilized with asphalt emulsion, BaseOne, CSS1+cement, and engineered emulsion were noticeably more resistant to penetration than non-stabilized FDR or FDR stabilized with foamed asphalt or a fly ash and cement mixture. Visual observations of distress revealed low amounts of distress with apparent success in mitigating cracking and little rutting. Resilient modulus tests and analyses indicated that FDR and SFDR specimens were less stiff than typical hot mix asphalt (HMA) specimens under typical design assumptions. However, FDR and

SFDR specimens lost less apparent stiffness under low-frequency loads in comparison to typical hot mix asphalt specimens. This would suggest that FDR and SFDR layers would retain their ability to resist low-speed heavy loads well relative to hot mix asphalt. Minnesota gravel equivalency (GE) analysis was performed to back-calculate the granular equivalent factor for FDR and SFDR layers based on the design equivalent single axle loads (ESALs) and R-values for subgrade soils. The back-calculated GE values are between 1.16 and 1.53, indicating that designers have likely been using GE values for FDR or SFDR layers that are consistent with current recommendations.

Based on the foregoing, it is recommended that the current GE values be generally retained for FDR and SFDR design. However, for cases where slower-moving vehicles are the critical design consideration, it may be warranted to expect relatively robust performance of FDR and SFDR layers. Note that such slow-moving vehicles could compromise the performance of any HMA. In addition to providing support for overlaying pavement layers, FDR and SFDR bases are well known for destroying crack patterns and other road defects that are often reflected through traditional HMA overlays. The visual distress surveys indicate that the FDR and SFDR bases under study in this report have been performing well in this regard; therefore, decision makers may want to consider the use of FDR and SFDR as a base for reasons other than structural capacity.

CHAPTER 1: INTRODUCTION

A literature review on full-depth reclamation (FDR) technology was conducted to understand the economic and environmental benefits, design and construction methods, and concerns regarding the implementation of this pavement rehabilitation treatment. Specifications and guidelines used by various states were reviewed to document the states' experiences with FDR. The findings presented in this chapter cover the main aspects of FDR technology, including, among other topics, candidate road selection, costs and life expectancy, stabilization type selection, structural design, mix design, and quality control and quality assurance measures.

1.1 CONSTRUCTION PROCESS

An FDR construction process consists of four steps, including sizing, stabilization, shaping, and compaction (Morian et al. 2012) (Figure 1). Sizing and stabilization are usually performed with a single-unit reclaimer using a two-pass method.

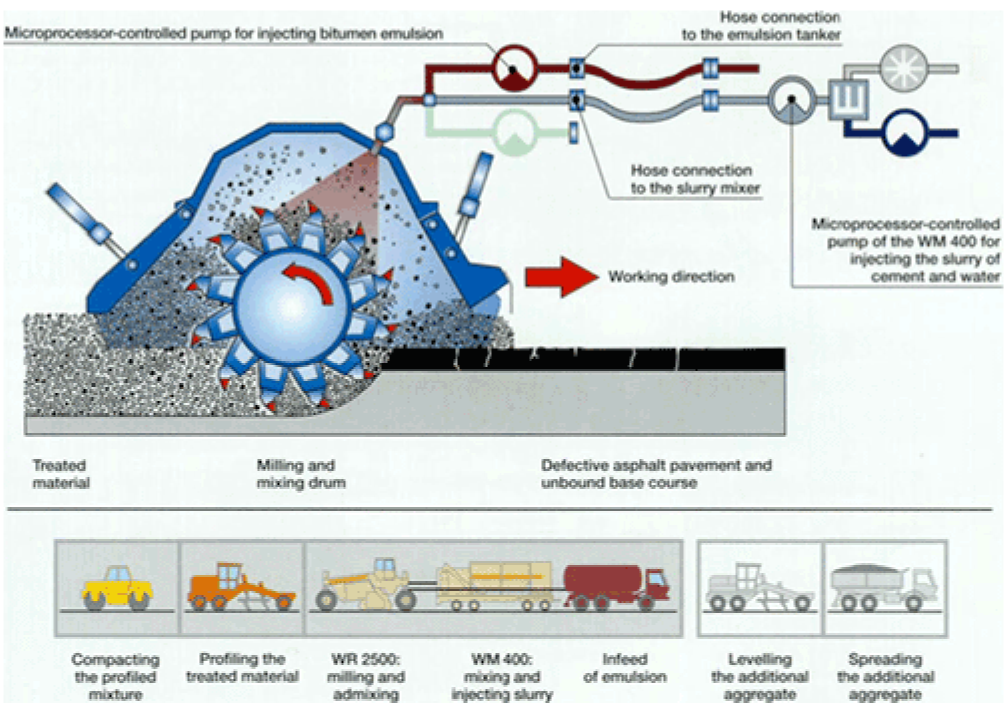

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Figure 1. FDR process and equipment.

In the first pass, the existing pavement is pulverized to a specified depth and particle size. Then, additional aggregates and/or stabilization agents are added to the road surface if needed. A second pass is then performed to mix the additional materials with the pulverized in-place pavement materials to produce a homogeneous mixture. After the second pass, the road is shaped with a motor grader to restore the surface profile and compacted to the target density. In addition to this single-unit two-pass operation, other procedures (Kandhal and Mallick 1997) using a two-unit or multiple-unit recycling train can also be used for FDR construction, although these methods are less commonly used than the single-unit reclaimer method.

1.2 BENEFITS AND LIMITATIONS

Stabilized FDR (SFDR) recycles 100% of the old pavement material and reuses the recycled pavement at the same location (i.e., in place) immediately. This process requires minimal virgin materials, hauling, stockpiling, traffic control, and time for construction, which provides economic and environmental benefits associated with the conservation of material, fuel, and energy. Figure 2 shows that SFDR saves more than 90% of new materials and 80% of diesel fuel in comparison to a base reconstruction project (PCA 2005).

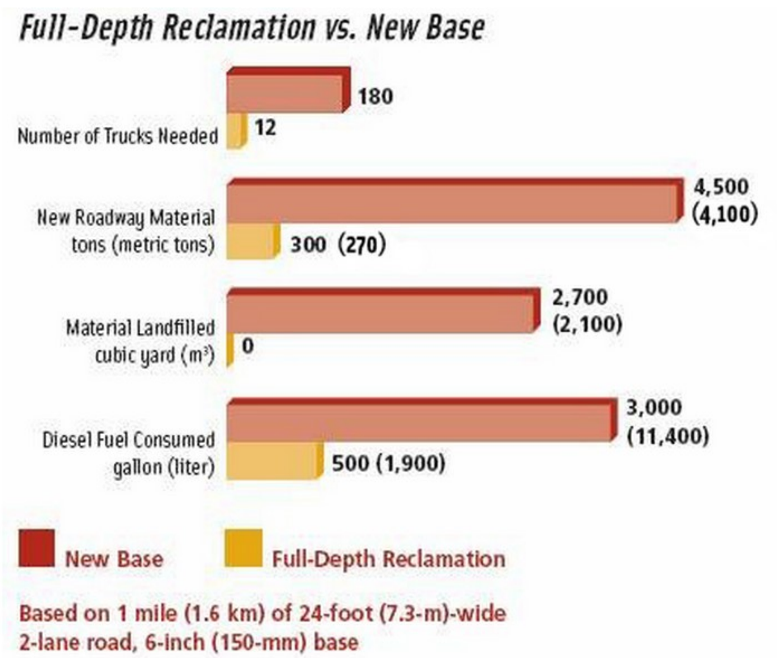


Figure 2. Energy use and materials needed for FDR and base reconstruction.

A life-cycle cost analysis was conducted by Diefenderfer et al. (2011a) to compare the life-cycle costs of SFDR and a conventional mill and overlay method over a 50-year period. A series of 2- or 4-inch overlay treatments using conventional mill and overlay methods and SFDR with a 3-inch overlay treatment were scheduled for two hypothetical roads, respectively. The treatment cost and schedule were assumed based on the experiences of the Virginia Department of Transportation (VDOT). The results indicate that the life-cycle maintenance cost for a road using SFDR is about 16% less than it is for a road using a conventional overlay method. Another investigation conducted by the Georgia Department of Transportation (GDOT) showed that construction costs were reduced by 42% when the agency used cement-stabilized FDR instead of a conventional base reconstruction method (Lewis et al. 2006). In addition to economic and environmental benefits, SFDR is effective in correcting various surficial and structural distresses, restoring pavement profile, and improving structural capacity. The deep pulverization process allows complete elimination of any cracking pattern, which mitigates the development of reflective cracking. The addition of chemical stabilizers, such as cement, lime, or calcium chlorides, decreases the moisture susceptibility of base materials. Using the process typically results in an increased base thickness, which usually results in a road with enhanced structural capacity.

The primary limitations for SFDR implementation are issues with curing and subgrade support during construction and the lack of a design method that is accepted nationwide . During the stabilization process, most stabilization agents require water to be added to facilitate mixing or compaction. After construction of SFDR is complete, the SFDR material needs to be cured to reduce the moisture content to a level that is less susceptible to moisture damage before a surface layer is placed. Any volatile content can also be decreased during curing (Kandhal and Mallick 1997). This requires that the construction is performed in warm and dry weather conditions, limiting the construction season for SFDR. During curing, excessive traffic loads should be prohibited. Rainfall or high humidity can prolong the curing process, causing delay to the construction of the surface layer. The strength of the SFDR layer immediately after construction is usually low. Pulverization crushes stiff hot mix asphalt (HMA) or concrete layers and produces a relatively soft base material. An SFDR project in Madison County, Mississippi, shows that without enough structural support, the weight of heavy construction equipment can cause significant damage to the subgrade and SFDR base. The SFDR base can gain 15% to 45% of its strength in one year (Diefenderfer et al. 2011b). However, before the ultimate strength is achieved, heavy traffic should be limited unless a surface course providing sufficient structural capacity is constructed.

Although many organizations have developed mix design methods for SFDR materials, an approach that has been adopted nationwide is still absent. The existing methods vary in sample preparation process, specimen size, testing methods, and physical property requirements. Many SFDR designs rely on the experience of the designer. The lack of a predominant widely accepted design method limits the application of SFDR in areas less experienced with this technology.

1.3 APPLICABILITY EVALUATION AND STABILIZATION ADDITIVE SELECTION

FDR and SFDR are most effective for correcting distresses that cannot be addressed with surface treatments. A pavement condition survey should be conducted to evaluate the applicability of FDR for a given project. The Pennsylvania Department of Transportation (PennDOT) developed a table (Table 1) to aid in the evaluation of whether or not a road is a good candidate for an FDR treatment.

Table 1. Selection table of FDR

Pavement Distress	X Denotes FDR would be Appropriate
Surface Defects	
• Raveling	
• Flushing	
• Slipperiness	
Deformation	
• Corrugations	
• Ruts-shallow	
• Rutting Deep[1]	X[2, 3]
Cracking (Load Associated)	
• Alligator	X
• Longitudinal	
• Wheel Path	X
• Pavement Edge	X
• Slippage	
Cracking (Non-Load Associated)	
• Block (Shrinkage)	X
• Longitudinal (Joint)	
• Transverse (Thermal)	X
• Reflection	X
Maintenance Patching	
• Spray	X[4]
• Skin	X[4]
• Pothole	X
• Deep Hot Mix	X
Weak Base or Subgrade	X
Ride Quality/Roughness	
• General Unevenness	
• Depressions (Settlement)	X[5]
• High Spots (Heaving)	X[6]

[1] Rutting originating from the lower portion of the pavement (below surface course and includes base and subgrade).

[2] The addition of new aggregate may be required for unstable mixes.

[3] The chemical stabilization of the subgrade may be required if the soil is soft, wet.

[4] In some instances, spray and skin patches may be removed by cold planning prior to these treatments (considered if very asphalt rich, bleeding).

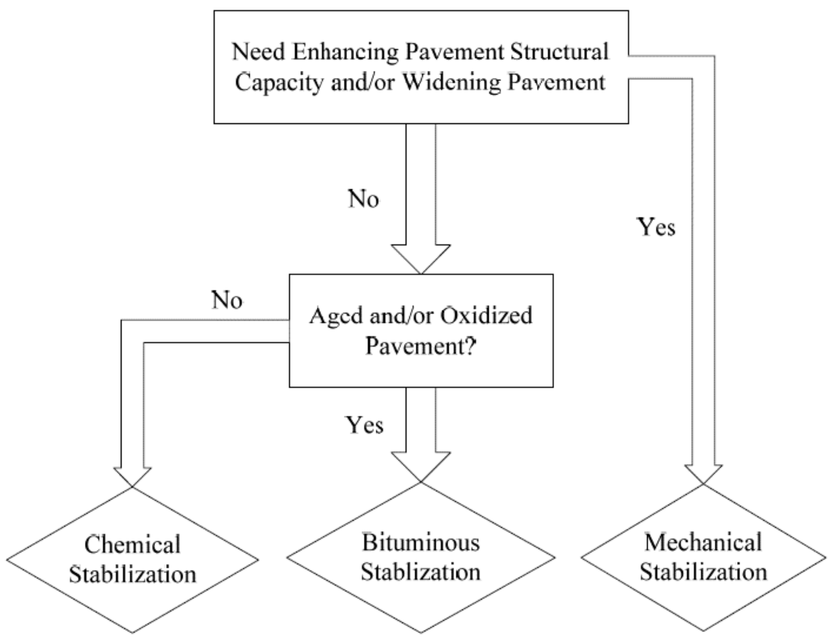
[5] Used if depressions due to a poor subgrade condition.

[6] Used if high spots caused by frost heave or swelling of an expansive subgrade soil.

Source: Morian et al. 2012

A typical FDR thickness is 6 to 9 inches and seldom over 12 inches. Therefore, pavements with a thick asphalt concrete (AC) layer that could result in an FDR layer that exceeds 12 inches thick are usually not good candidates for FDR treatment. A road that carries high traffic volumes may have a thick asphalt layer in order to provide sufficient structural support. Such a situation may cause construction difficulties for an FDR treatment. The New York State Department of Transportation (NYSDOT) prohibits the use of FDR for roads with more than 1 million equivalent single axle loads (ESALs) of traffic (NYSDOT 2015.

Decisions regarding the selection of an appropriate stabilization method and additive for a road should be made based on a careful assessment of the road's traffic level, the soundness of the pavement structure, and material properties using various in situ and laboratory tests. Commonly used in situ test methods include the dynamic cone penetrometer (DCP), light weight deflectometer (LWD), and falling weight deflectometer (FWD) tests (Morian et al. 2012). The primary objective of conducting these in situ tests is to evaluate the subgrade strength and modulus for estimating pavement structural capacity (Morian et al. 2012, Caltrans Division of Maintenance 2012). Additional evaluation of base and subgrade materials may be performed in the laboratory using sieve analysis, plasticity test, hydrometer particle size analysis, moisture content test, and proctor test (Morian et al. 2012). Mechanical stabilization should be considered if additional aggregate materials are needed for widening a road or enhancing a road's structural capacity due to a high level of design traffic. Bituminous stabilization is often used if the pavement contains asphalt that has experienced considerable aging and oxidization. A conceptual flow diagram (Figure 3) was developed for this decision process. The selection of a stabilization agent is also affected by the base materials' physical properties.



Morian et al. 2012

Figure 3. Conceptual flow diagram for selecting stabilization method.

PennDOT has developed a decision table to aid in the selection of a stabilizer based on the gradation, plastic index, and fines content of the pulverized base materials (Table 2).

Table 2. Selection table for stabilization additives

Percent Passing No. 200 Sieve	Plastic Index	Stabilizer	Soil Type												
			Granular Material								Silt-Clay Material				
			Well-graded gravel	Poorly graded gravel	Silty gravel	Clayey gravel	Well-graded sand	Poorly graded sand	Silty sand	Clayey sand	Silt, Silt with sand	Lean clay	Organic silt/Organic lean clay	Elastic silt	Fat clay, fat clay with sand
			GW	GP	GM	GC	SW	SP	SM	SC	ML	CL	OL	MH	CH
			A-1-a	A-1-a	A-1-b	A-1-b or A-2-6	A-1-b	A-3 or A-1-b	A-2-4 or A-2-5	A-2-6 or A-2-7	A-4 or A-5	A-5	A-4	A-5 or A-7-5	A-7-6
<25	<6	Bituminous													
	<10	Cement													
	>10	Lime													
>25	<10	Cement													
	10–30	Lime													
	>30	Lime + Cement													

Source: Morian et al. 2012

The California Department of Transportation (Caltrans) selects the type of stabilization agent to be used based on the gradation of the pulverized base material, the plasticity index of the base and subgrade, and the R-value of the subgrade. The selection criteria are summarized in Table 3.

Table 3. Caltrans SFDR material test minimum targets for various types of stabilizers

Stabilization Agent	Material/Layer	Gradation	Plasticity Index	R-Value[3]
Cement	Existing AC + underlying material[1]	%passing #200: 5% - 15%	-	-
	Base[2]	-	PI < 12	-
	Subgrade	-	PI < 40	> 5
Asphalt Emulsion	Existing AC + underlying material	%passing #200 ≤ 20%	PI < 6	-
	Base	-	PI < 6	-
	Subgrade	-	PI < 40	> 5
Foamed Asphalt	Existing AC + underlying material	%passing #200: 5% - 15%	-	-
	Base	-	PI < 12	-
	Subgrade	-	PI < 12	> 20

[1]Sample and blend proportionally according to the preliminary design FDR depth. Minimum 2 inches underlying material.
[2]Layer may not be present.
[3]If mechanistic-empirical analysis is used for structure design, R-value testing is not required.

Sources: Caltrans Division of Maintenance 2012, Caltrans Division of Maintenance 2013

Virgin materials can be introduced to modify the recycled asphalt pavement (RAP) material properties if the requirements for a stabilization agent are not met. The use of various stabilization agents or rehabilitation methods other than SFDR should be considered if adding new material cannot be accomplished economically.

1.4 STRUCTURAL DESIGN

SFDR is treated as a cement or bituminous stabilized base course in the structural design process of many state highway agencies. Research has also been performed to evaluate the structural capacity of an SFDR layer. The typical structural coefficients for stabilized base courses and structural coefficients for SFDR recommended by various researchers are shown in Table 4.

Table 4. SFDR structural layer coefficients

Source	Material Type	Structural Coefficient
Alabama (Peters-Davis and Timm 2009)	Bituminous treated base	0.30
	Cement or lime treated base	0.23
Pennsylvania (Morian et al. 2012)	Bituminous SFDR	0.25 – 0.30
	Chemical SFDR	0.30 – 0.37
	Calcium chloride SFDR	0.14
Virginia (VDOT 2003)	Cement treated aggregate base	0.20
	Lime treated soil	0.18
Marquis et al. (2003)	Foamed asphalt SFDR	0.22 – 0.35
Nantung et al. (2011)	Cement + emulsion SFDR	0.16 – 0.22
Romanoschi et al. (2004)	Foamed asphalt SFDR	0.18

The findings in Table 4 show that the structural coefficient of an SFDR layer varies within a wide range. In Minnesota, a typical gravel equivalency (GE) of 1.5 inches is used for SFDR thickness design. This value was selected based on the findings from a research project that investigated the results of FWD tests conducted on SFDR roads with varying structures, stabilizers, and project locations (Tang et al. 2012). Large variations in the GE values for SFDR were found from project to project in the investigation. However, the test results clearly showed that the stiffness of an SFDR layer is considerably higher than the stiffness of a non-stabilized FDR layer or a typical aggregate base.

1.5 MIX DESIGN METHODS

1.5.1 Asphalt-Stabilized FDR

Various mix design methods for cold recycled asphalt materials have been developed by modifying the traditional mix design methods for HMA. These methods are usually used for the design of partial-depth cold recycling (cold in-place recycling [CIR]) material. They can also be used to design full-depth reclamation material using foamed asphalt or asphalt emulsion as a stabilization agent.

In this section, the Asphalt Institute, Chevron, Colorado, Illinois, Pennsylvania, South Carolina, and the US Army Corps of Engineers (USACE) mix design methods for SFDR using asphalt emulsion or foamed asphalt are summarized. For these methods, samples are prepared using a Marshall, Hveem, or Superpave gyratory compactor (SGC). The asphalt application rate is selected based on the results of the Marshall or Hveem stability test, indirect tensile strength (ITS) test, volumetric tests, and others.

1.5.1.1 Asphalt Institute Method (Asphalt Institute 1983)

The Asphalt Institute method uses an aggregate surface area formula to determine the application rate of asphalt. The aggregate surface area formula is shown in Equation 1.

$$P_c = \frac{0.035A + 0.045B + KC + F}{R} \qquad \text{Eq. 1}$$

P_c is the calculated percent of total asphalt material by weight of total mix. A, B, and C are the weight fractions of aggregate retained on the No.8 sieve, passing the No.8 and retained on the No.200 sieve, and passing the No.200 sieve, respectively. K is 0.15 if the aggregate has a fine content (percent aggregate passing the No.200 sieve) between 11% to 15%. If the fine content is between 6% to 10%, a K value of 0.18 should be used. In the case that the fine content is less than 5%, the K value is 0.2. F is a parameter that depends on aggregate absorption. The typical value of F ranges from 0.7 to 1. The F value cannot exceed 2%. R is the percent of asphalt binder in the stabilization agent. The R value is equal to 1 for foamed asphalt and 0.6 to 0.65 if asphalt emulsion is used.

Equation 2 can be used to determine the application rate of virgin asphalt in SFDR.

$$P_r = P_c - \frac{P_a \times P_p}{R} \qquad \text{Eq. 2}$$

The amount of virgin asphalt needed (P_r) is calculated using the total asphalt content calculated from Equation 1 minus the asphalt in the RAP material. P_p is the RAP content in the recycled mix, and P_a is the asphalt content of the RAP material. Field adjustments are required for this method.

1.5.1.2 Chevron Method (Chevron USA 1982)

The Chevron method calculates a starting binder content using an aggregate surface area formula or the centrifuge kerosene equivalent test. Equation 3 is the surface area formula.

$$P_c = (0.05A + 0.1B + 0.5C) - P_a \times \frac{P_p}{R} \qquad \text{Eq. 3}$$

P_c in Equation 3 is the required asphalt emulsion content for recycling. Just as in Equation 1 and Equation 2, the A, B, and C variables in Equation 3 characterize the gradation of the recycled material. The P_a, P_p, and R variables are the RAP asphalt content, RAP content, and asphalt residual content in emulsion, respectively.

Equation 4 can be used if the centrifuge kerosene equivalent test result is available.

$$P_c = \frac{P_R - P_a \times P_p}{R} \qquad \text{Eq. 4}$$

P_R in Equation 4 represents the asphalt demand estimated from the centrifuge kerosene equivalent test. An additional amount of new aggregates is desirable if the starting asphalt content is less than 2%. After the starting asphalt content is selected, samples for laboratory testing are prepared at three emulsion contents, including the starting emulsion content and two emulsion contents above and below the starting emulsion content. The specimens are compacted using the California kneading compactor and tested for the Hveem stability, resistance (R-value), resilient modulus, and cohesiometer values. An acceptable job formula should meet the criteria shown in Table 5.

Table 5. SFDR design criteria for Chevron method

Test Method	Sample Curing Condition	Criteria
Percent Coating		≥ 75%
Resistance (tested at room temperature)	Initial cure[1]	≥ 70
	Final cure[2]	≥ 78
Cohesiometer (tested at room temperature for initial cured and water soaked samples and 60°C for final cured samples)	Initial cure	≥ 50
	Final cure	≥ 100
	Final cure + water soak[3]	≥ 100
Resilient Modulus (tested at room temperature)	Final cure	150 to 500 ksi
Stabilometer (tested at 60°C)	Final cure	≥ 30

[1] Cure in the mold at room temperature for 24 hours.

[2] Cure in the mold at room temperature for 72 hours.

[3] Vacuum saturation at 100 mm of mercury.

1.5.1.3 Colorado Method

The Colorado Department of Transportation (CDOT) specifies a mix design method for SFDR and CIR using asphalt emulsion. The Colorado method requires that SFDR samples be prepared using aggregate meeting a specified gradation criterion with a specified moisture content. The gradation criteria are shown in Table 6.

Table 6. RAP sample gradation requirements

Sieve Size	% Passing, %
1.25 inch	100
1 inch	90 to 100
3/4 inch	80 to 97
No. 4	30 to 55
No. 30	5 to 15

The moisture content of the sample should be controlled at 50% to 75% of the optimum moisture content (OMC) determined using the modified proctor test (AASHTO T 180 Method D) for aggregate with a sand equivalent that is less than 30 and 40% to 65% for aggregate with a sand equivalent greater than 30. Two to three percent moisture content should be used if the OMC cannot be determined from the proctor test. Then, the RAP samples are mixed with asphalt emulsion and cured in plastic containers that are 6 inches in diameter for 30 to 45 minutes at 40°C. Compaction is performed with a SGC for 30 gyrations. The samples are then tested for volumetrics, the Hveem cohesiometer value, resilient modulus, and ITS. The job formula acceptance criteria are provided in Table 7.

Table 7. Colorado performance test criteria for selecting optimum emulsion content

Test Method	Sample Curing Condition	Criteria	
		For mixtures containing <8% passing #200 sieve	For mixtures containing >8% passing #200 sieve
Short-term strength test, 1 hour – modified cohesiometer, AASHTO T 246 (Part 13), g/25mm of wITSh	60 minutes at 25°C	>175	>150
Indirect Tensile Strength (ITS), ASTM D4867, Part 8.11.1, 25°C, psi	72 hours at 40°C	>40	>35
Conditioned ITS, ASTM D4867, psi		>25	>20
Resilient modulus, ASTM D4123, 25°C, 1000psi		>150	>120
Thermal cracking (ITS), AASHTO T 322		< -20ºC	< -20ºC

1.5.1.4 Illinois Method

The Illinois Department of Transportation (IDOT) specifies a SGC mix design method. Samples are prepared at the OMC determined from the modified proctor test (ASTM D1557 Method C). Specimens are compacted with a SGC using 30 gyrations with 600 kPa compaction pressure and tested for short-term strength (STS) (ASTM D1560) and ITS (ASTM D4867). The acceptance criteria for the job formula are summarized in Table 8.

Table 8. Illinois performance test criteria for selecting optimum binder content

Property	Criteria	
	For mixtures containing <8% passing #200 sieve	For mixtures containing >8% passing #200 sieve
Short-term strength test, ASTM D1560	>175	>150
Indirect Tensile Strength (ITS), ASTM D4867, psi	>40	>35
Conditioned ITS, ASTM D4867, psi	>25	>20

IDOT also established criteria for asphalt emulsion selection. The asphalt emulsion type selection criteria are shown in Table 9.

Table 9. Illinois emulsified asphalt material specification for FDR

Test	Procedure	Minimum	Maximum
Viscosity, Saybolt Furol, at 25°C, SFS	AASHTO T 59	20	100
Sieve Test, No. 20 (850μm), retained on sieve, %	AASHTO T 59		0.1
Storage Stability Test, 24 hours, %	AASHTO T 59		1
Distillation Test, Residue from distillation to 175°C, %	AASHTO T 59	64	
Oil distillate by volume, %	AASHTO T 59		1
Penetration, 25°C, 100g, 5s, dmm	AASHTO T 49	75	200

1.5.1.5 Pennsylvania Method (Morian et al. 2012)

PennDOT developed different guidelines for the design of SFDR materials using asphalt emulsion and foamed asphalt. Both methods require SGC-compacted samples for the ITS test and determination of the optimum asphalt content. However, the mix design method for foamed asphalt stabilization also involves determination of the optimum water content for foaming. Samples of SFDR using asphalt emulsion are prepared at the OMC determined from the modified proctor test (ASTM D558). A moisture content of 2% to 3% is used if a reasonable OMC cannot be determined. Samples are cured at 40°C for 30 minutes after mixing and at room temperature for 48 hours after compaction. Compaction is performed with 600 kPa pressure and 30 gyrations. The pressure continues for 10 seconds after the last gyration is completed. The selected emulsion content should be able to produce samples that have ITS values greater than 50 psi and ITS ratios greater than 0.7. The samples need to have air voids between 6% and 8%. Samples for SFDR using foamed asphalt are prepared at 85% of the OMC determined from the standard (AASHTO T 99) or modified (AASHTO T 180) proctor test. The samples are cured at 40°C for 30 minutes and compacted into 4- or 6-inch specimens. The asphalt content of the sample that has the highest ITS value is selected as the design asphalt content. A volume expansion test is also performed for foamed asphalt. The optimum foaming water content is determined using the intercept between the half-life volume expansion curve and the expansion ratio curve.

1.5.1.6 South Carolina Method (SCDOT 2012)

The South Carolina Department of Transportation (SCDOT) requires that the asphalt emulsion has a minimum asphalt residue content of 63% and a penetration grade between 55 and 95. The SFDR aggregate material needs to meet the gradation requirements in Table 10.

Table 10. SCDOT gradation requirements for SFDR aggregate

Sieve Size	Percent Passing
2 inch sieve	100
1.75 inch sieve	97-100
No. 200 sieve	0-20

The OMC is determined by following AASHTO T 180. If a well-defined dry density curve cannot be produced, 3% OMC should be used. A sand equivalent test is also performed to determine the moisture content for sample preparation. Some of the following details are similar to those of the Colorado method; however, some of the limits are different. The Colorado method has a slightly more tolerant moisture content range for preparing samples. Sample mixes for the South Carolina method should be prepared at 60% to 75% of the OMC for aggregates that have a sand equivalent less than 30 and 45% to 65% of the OMC for aggregates with a sand equivalent greater than 30. A fixed moisture content between 2% and 3% should be used if the aggregate has a fine content less than 4% or no peak value is found for the dry density curve. Sample mixes are prepared by mixing with the required amount of water for 60 seconds. Asphalt emulsion is added after the curing is completed. Compaction is performed using a SGC at room temperature with 600 kPa compaction pressure and 30 gyrations. The pressure is maintained for 10 seconds after the final gyration. Then, the specimens are tested for stability, flow, and ITS. The job formula acceptance criteria are shown in Table 11.

Table 11. SCDOT job formula acceptance criteria

Test Method	Sample Curing Condition	Criteria
Flow (AASHTO T 245)	Final cure[1]	0.1 to 0.25 inch
Stability (AASHTO T 245)	Initial cure[2]	≥ 1500 lbs
	Final cure	≥ 3000 lbs
Gyratory Quotient		150 to 500 ksi
ITS	Dry conditioned	≥ 45 psi
	Moisture conditioned	≥ 25 psi

[1] Cure in a forced draft oven for 48 hours at 60°C and cooled at room temperature for 24 hours.
[2] Cure at room temperature for 24 hours.

1.5.1.7 USACE Method (Salomon and Newcomb 2000, Cross and Ramaya 1995)

Under the USACE method, the optimum binder content is selected based on ITS test results from specimens that are compacted using a SGC or a Marshall hammer. The Marshall compaction process requires 50 blows on each side of the specimen. The SGC compaction process applies 90 psi (620 kPa) pressure for 150 gyrations. The job formula is accepted if the shear strength of the sample is greater than 14 psi (100 kPa) and the elastic plastic index is less than 1.54.

1.5.2 Cement-Stabilized FDR

A design for SFDR using cement or other chemical stabilizers is usually based on the results of the unconfined compressive strength (UCS) test. However, for various state highway agencies, the mix design methods vary in terms of sample preparation and strength criteria. In this section, mix design methods used by Georgia, Pennsylvania, and New York for SFDR using cement and other chemical stabilizers for are summarized.

1.5.2.1 Georgia Method (Lewis et al. 2006)

The Georgia Department of Transportation (GDOT) used a proctor compaction method for a pilot project in 2001. This mix design method was adopted later by GDOT specifications for the design of cement-stabilized base material because of the success of the pilot project. The method requires determination of the maximum dry density and the OMC using the standard proctor test (AASHTO T 99). The samples are mixed at the OMC and at varying cement contents. Then, a 4-inch mold and a standard proctor hammer are used to compact the samples into test specimens. The compaction should be done in three layers with 25 blows per layer. After compaction, the specimens are sealed in plastic bags and cured for 7 days. Then, the specimens are tested for UCS. The job formula is accepted if the UCS of the sample is greater than 450 psi.

1.5.2.2 Pennsylvania Method (Morian et al. 2012)

The Pennsylvania method requires samples for UCS tests to be prepared following ASTM D1633 Method A. Samples are cured in plastic wrap for 7 days after mixing and compaction. For roads that are to be overlaid with an overlay that will be thicker than 3 inches, the method requires samples to have an average UCS between 200 and 500 psi. In cases where the overlay thickness is less than 3 inches, a mix design can be accepted if an average UCS between 300 and 500 psi is achieved. The Pennsylvania method can be also used to develop an SFDR design that uses lime or fly ash as a stabilization agent. For the design of a lime- or fly ash-stabilized FDR, UCS samples are prepared and tested following ASTM D5102 Procedure B. A curing temperature of 40°C should be maintained for 7 days after mixing and compaction. The job formula is accepted if the average UCS is greater than 200 psi.

1.5.2.3 New York Method (NYSDOT 2015)

The NYSDOT cement-stabilized FDR design method is very similar to the Pennsylvania method. The only difference is the job formula acceptance criteria. The NYSDOT requires the average UCS to be greater than 350 psi and less than 800 psi.

1.6 COSTS AND LIFE EXPECTANCY

SFDR prolongs the service life of a road by eliminating distress patterns and improving base layer thickness and strength. A successfully constructed SFDR road provides 7 to 20 years of service life (Maher et al. 2005, Wu et al. 2010). The material and installation costs for SFDR vary from $4 to $7 per square meter and are dependent on the SFDR thickness and type of stabilizer (Federal Lands Highway Division of the Federal Highway Administration 2005). The cost for SFDR using asphalt emulsion is slightly higher than the cost for SFDR using cement.

1.7 QUALITY CONTROL/ASSURANCE

Quality materials, sufficient compaction, and proper curing are important to the success of SFDR construction and are the primary aspects of quality control/assurance measures. Many state highway agencies establish requirements that specify the pulverized particle size, weather conditions, compaction criteria, and curing criteria.

1.8 PULVERIZED PARTICLE SIZE

The presence of large RAP particles in SFDR often affects the ability of the reclaimer to handle and produce a uniform mix. The gradation of the pulverized RAP material is usually specified by the state highway agencies that have established specifications for SFDR. Some of the RAP size requirements are shown in Table 12.

Table 12. RAP size requirements for SFDR

State	Requirements
Iowa (Iowa DOT 2012)	98 to 100% passing 1.5-inch sieve; 90 to 100 passing 1-inch sieve
Minnesota (MnDOT 2018a)	100% passing 3-inch sieve; 97 to 100% passing 2-inch sieve
Pennsylvania (Morian et al. 2012)	95% passing 2-inch sieve

1.9 WEATHER CONSTRAINTS FOR CONSTRUCTION

Many state specifications require SFDR to be constructed in certain weather conditions and at certain times of the year. The reason is to ensure that the construction is done during favorable weather conditions when proper curing can be achieved and excessive moisture retention can be prevented. The weather constraints for SFDR construction specified by NYSDOT and PennDOT are summarized in Table 13. In Minnesota, the construction weather constraints are not specified.

Table 13. Weather constraints for SFDR construction

Type of Additive	Climate Limitation for Construction	
	New York	Pennsylvania
Cement or Cement Fly-Ash	Air temperature in the shade should be no less than 4°C (39°F) and rising. Completion of stabilization should be at least one month before the first hard freeze.	Air temperature above 7°C.
Lime, Fly Ash or Lime-Fly Ash	Air temperature in the shade should be no less than 4°C and rising. Completion of stabilization should be at least one month before the first hard freeze. Two weeks minimum of warm to hot weather is desirable after completing the stabilization work.	
Calcium Chloride	Air temperature in shade should be no less than 4°C and rising. Complete stabilization should be at least one month before the first hard freeze.	
Asphalt Emulsion or Foamed Asphalt	Air temperature in the shade should be no less than 15°C (59°F) and rising. Avoid construction in foggy days or when the humidity is above 80%. Warm to hot dry weather is preferred for curing.	Air temperature above 15°C and humidity lower than 80%.

Sources: Morian et al. 2012, NYSDOT 2015

1.10 COMPACTION REQUIREMENTS

The level of compaction for the SFDR layer is usually measured by field material density readings from a nuclear density gage. The field material density can be corrected for the moisture content and compared with the density requirements. The Iowa Department of Transportation (Iowa DOT) requires the field density at 75% of the reclaimed material depth to be higher than 94% of the dry density of the laboratory-compacted sample for Interstate and primary roads and 92% of the laboratory sample density for other roads (Iowa DOT 2012). The field density of the reclaimed material at a depth of 2 inches should be no less than 97% of the density at 75% of the reclaimed material depth. In Minnesota, a DCP test is conducted to verify the level of compaction (MnDOT 2018a). The specification requires an SFDR layer to achieve a penetration index of 0.4 inches and a setting value of 1.5 inches. PennDOT determines compaction criteria by constructing a test strip (Morian et al. 2012). If the material is under-rolled, sufficient compaction through the reduction of air voids in the material cannot be achieved. If the material is over-rolled, shear failure could damage the material and reduce density. Therefore, it is important to establish a rolling pattern that will provide the maximum density that can be reasonably produced. The field density is specified to be higher than 95% of the average density of the test strip.

1.11 CURING REQUIREMENTS

Proper curing is critical in order for an SFDR layer to gain strength and achieve the desired moisture content. Through hydration or evaporation of moisture, initial curing allows the SFDR layer to develop the minimum strength at which the road can be opened to traffic. Excessive moisture in the SFDR layer can cause moisture damage and increased moisture sensitivity for the pavement base. IDOT requires that the moisture content of an SFDR layer be less than 2.5% or 50% of the optimum moisture content determined from the proctor test (IDOT 2012). Minnesota SFDR specifications do not specify a curing criterion.

CHAPTER 2: IMPLEMENTATION OF SFDR IN MINNESOTA

A comprehensive investigation was conducted to locate representative FDR and SFDR road sections in Minnesota. This part of the study consisted of two phases. In the first phase, an online survey was distributed to Minnesota local road agencies, including counties and cities. The intent of the online survey was to collect data on the current status of SFDR roads in Minnesota and identify those that are potential candidates for a case study. The local road agencies that were identified as having roads that could be case study road candidates were contacted by the researchers.

MnDOT also provided historical documents for some state trunk highways that are SFDR roads and that were identified as roads that could be included among the case study roads. A total of 13 sections were selected from the candidate road list for possible case studies. The selection process considered the availability of historical project documents, the age of the road, and the road's suitability from a research perspective. The goal was to select roads that have similar traffic and subgrade conditions, such that the influences of these factors on pavement performance could be controlled. In addition, it was important that the roads have a sufficiently high age so that long-term performance could be evaluated.

The online survey questions are summarized in Appendix A. There were 28 respondents, including 23 from counties and 5 from cities. Twenty-three respondents from both counties and cities claimed to have experience with using full-depth reclamation for pavement rehabilitation, including four respondents that have used non-stabilized full-depth reclamation (NSFDR), seven respondents that have used SFDR, and twelve respondents that have constructed projects using both NSFDR and SFDR technology. BaseOne and asphalt emulsion were the most commonly used stabilization agents, which were used by 42% and 47% of the respondents who had SFDR experience, respectively. The other types of stabilization agents that were mentioned include fly ash (five respondents), cement (three respondents), and foamed asphalt (one respondent).

All respondents were satisfied with the overall performance of NSFDR and SFDR projects. Three respondents reported occasional issues with subgrade support caused by wet subgrade soil, especially clay soil, and weak spots in the subgrade during construction. The gradation of the pulverized old asphalt material was also a concern in some projects. Two respondents reported the existence of large chunks of old asphalt that did not get pulverized sufficiently and failed to meet the Minnesota full-depth reclamation specifications (MnDOT 2018a) for RAP gradation and the gradation requirements for BaseOne stabilization. However, this issue did not appear to negatively influence the overall performance of the roads. Some local agencies indicated concern about the road profile change caused by applying a 3- to 5-inch asphalt overlay on top of the SFDR base. The elevated road profile may cause difficulties in matching the driveways and drainage, as well as narrowing the cross section of the road top.

More than half of the respondents with FDR experience have used this reclamation technology for more than 5 years, including five respondents that had over 10 years of experience using FDR. Approximately 43% of the local agencies recently started using FDR.

The local road agencies and MnDOT provided historical construction documents for 56 projects, including both county roads and state trunk highways. Typical FDR construction in Minnesota (based on observations from the 56 projects) involves a 3- to 12-inch reclaimed layer and 3- to 5.5-inch asphalt overlay. The reclamation depth mainly depends on the thickness of the existing bituminous material. Usually, 1 to 4 inches of aggregate base are also reclaimed with the overlaying asphalt pavement. The existing pavement surface was sometimes milled by 2 to 4 inches before reclaiming if the existing asphalt layer was too thick for reclamation. In some cases when the reclaimed layer was relatively thick, stabilization agents, especially asphalt emulsion or foamed asphalt, were only applied to the top 3 to 5 inches of the reclaimed base.

2.1 CASE STUDY CANDIDATES SELECTION

The research team received historical construction documents for 56 FDR projects from local road agencies and MnDOT. A list of these projects is shown in Appendix B. From these projects, 13 test sections on three trunk highways and two county roads that could fulfill the goals of this investigation were selected for case study analysis. These test sections involve various stabilization technologies, including asphalt emulsion and cement, engineered emulsion (EE), foamed asphalt, and BaseOne, as well as non-stabilized FDR treatments. The test sections on TH55, TH65, and TH30 could be used to investigate the influence of various types of stabilization methods on pavement performance and compare the cost-effectiveness of SFDR and NSFDR. The Goodhue County highways were to be studied to evaluate the short- and long-term performance of FDR/SFDR roads.

2.2 CONCLUSION AND RECOMMENDATION

Chapter 2 of this report summarized the investigation of the implementation of SFDR technology in Minnesota. The results indicate that SFDR has been successfully used in Minnesota by both city and county road agencies in various situations on several projects. Various types of stabilization agents and techniques have been applied. Local road agencies are generally satisfied with the performance of the SFDR projects. An inventory of certain FDR and SFDR projects was created, and a total of 13 sections on 5 roads were recommended for inclusion in the case study list.

CHAPTER 3: FIELD INVESTIGATION

The research team collected performance data and core samples from the selected case study sections. A pavement condition survey was conducted to evaluate the roads' performance by quantifying the distresses on the pavement surface. According to the historical construction documents, 18 test sections from 8 counties were selected out of 50 FDR projects from local road agencies and MnDOT. The locations of these 8 counties are shown in Figure 4.

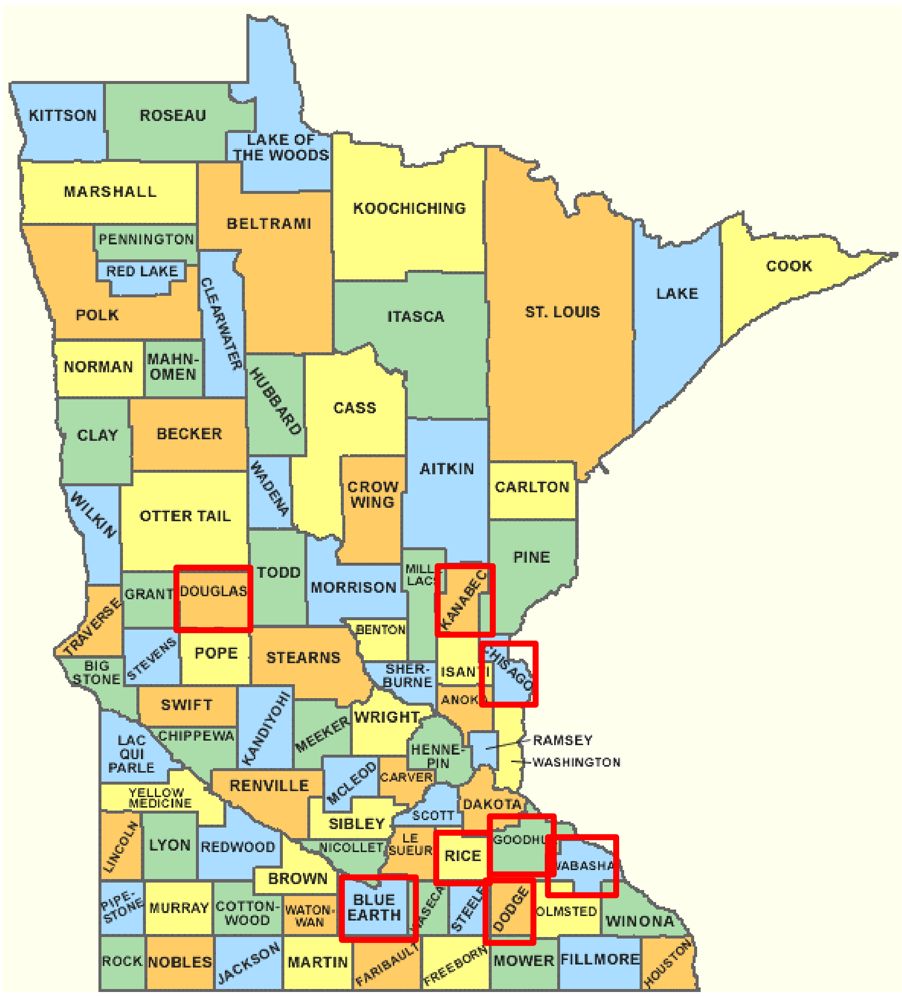


Figure 4. Minnesota counties map showing eight counties with test sections.

Field investigation sections and their information are presented in Table 14. These test sections involve various stabilization technologies, including asphalt emulsions and cement, engineered emulsions, foamed asphalt, and BaseOne, as well as non-stabilized FDR treatments.

Table 14. Field investigation sections

Location	Section	Start/End	Location GPS	Description of Pavement and Base	Additional Detail
Douglas County, TH 55	Test Section 1	Start	N45.80619, W95.75600	CCS-1 Emulsion	48.27 to 52.7
		End	N45.80006, W95.74364		
	Test Section 2	Start	N45.79781, W95.73904	EE	
		End	N45.79068, W95.72472		
	Test Section 3	Start	N45.79781, W95.73904	Foamed Asphalt	
		End	N45.78254, W95.70755		
	Test Section 4	Start	N45.78254, W95.70755	Base 1	
		End	N45.77802, W95.69679		
	Test Section 5	Start	N45.77564, W95.69030	Control	
		End	N45.77245, W95.68037		
Kanabec County, TH 65	Section 3	Start	N45.572480, W93.200385	EE	Southbound CSAH 27 to 300th Ave.
		End			
		Start	N46.042917, W93.284399		
		End			
	Section 2	Start	N45.996414, W93.282972	CSS1	Southbound 300th to 250th Ave.
		End			
		Start	N45.996414, W93.282972		
		End			
	Section 1	Start	N45.996414, W93.282972	FDR Control	Southbound 250th to 220th Ave.
		End			
		Start	N45.996414, W93.282972		
		End			
Goodhue County, County 7 Blvd. North	Test Section 1	Start	44.457644, -92.742007	FDR from 2005	Between CSAH 1 and CSAH 9
	Test Section 2				
		End	44.398512, -92.701402		
Goodhue County CSAH 7 South	Test Section 1	Start	44.398512, -92.701402	FDR from 1998	Between CSAH 9 and TH 52
	Test Section 2				
	Test Section 3				
		End	44.324617, -92.711542		
Goodhue County, CSAH 11 Blvd.	Test Section 1	Start	N44.225104, W92.812648	FDR from 2012	Between County Blvd. 1 and County Blvd. 13
		End			
	Test Section 2	Start	N44.224436, W92.791824		
		End			
	Test Section 3	Start	N44.218432, W92.849133		
		End			

Location	Section	Start/End	Location GPS	Description of Pavement and Base	Additional Detail
Dodge County, CSAH 19	Test Section 1	Start	N44.195693, W92.758123	Only the westernmost 2 miles stabilized with BaseOne, that is where the test sections shall be located, 2006 pavement	NE border of Dodge county, also known as 27, from 240 Ave. to 2 miles east
		End			
	Test Section 2	Start	N44.195929, W92.741329		
		End			
	Test Section 3	Start	N44.196027, W92.750816		
		End			
Dodge County, CSAH 10	Test Section 1	Start	N44.007426, W92.984520	FDR, 2010 pavement	CSAH 10 (650th St) from CSAH 3 (130th Ave.) to CSAH 5 (160th Ave.)
		End			
	Test Section 2	Start	N44.007461, W92.959059		
		End			
	Test Section 3	Start	N44.007461, W92.959059		
		End			
Wabasha County CSAH 9	Test Section 1	Start	N44.405221, W92.269540	FDR with approx 4 in. aggregate base modified class 5 (2 in.) added during FDR in 2014	North/south road from CSAH 33 to Lake City Limits, aka W Lakewood Ave.
		End			
	Test Section 2	Start	N44.392522, W92.269688		
		End			
	Test Section 3	Start	N44.374231, W92.269907		
		End			
Wabasha County CSAH 16 (aka County 16 Blvd.)	Test Section 1	Start	N44.367660, W92.391777	FDR stabilized with BaseOne upper 4 in. - 2011	Located west of TH 63 for 5.3 miles, aka County 16 Blvd.
		End			
	Test Section 2	Start	N44.368517, W92.420748		
		End			
	Test Section 3	Start	N44.368633, W92.446878		
		End			
Rice County, CSAH 26	Test Section 1	Start	N44.286512, W93.060588	12 in. FDR with 4 in. bituminous surfacing	from TH 60 to TH 246 in Nerstrand, Lamb Ave.
		End			
	Test Section 2	Start	N44.299273, W93.060793		
		End			
	Test Section 3	Start	N44.320078, W93.060666		
		End			

Location	Section	Start/End	Location GPS	Description of Pavement and Base	Additional Detail
Rice County, CSAH 29	Test Section 1	Start	N44.337684, W93.209279	10 in. FDR with 4 in. bituminous surfacing	TH 3 to CSAH 20, aka 158th St. E, aka Cabot Ave.
		End			
	Test Section 2	Start	N44.345505, W93.217988		
		End			
	Test Section 3	Start	N44.352077, W93.235863		
		End			
Chisago County, CSAH 81, aka Reed Ave.	Test Section 1	Start	N45.540472, W92.759370	Emulsion stabilized	Between CSAH 12 (Park Trail) and 810th St.
		End			
	Test Section 2	Start	N45.540472, W92.753970		
		End			
	Test Section 3	Start			
		End			
Chisago County, CSAH 74 aka 347 St.	Test Section 1	Start	N45.450185, W92.878031	Emulsion stabilized	between Lincoln Rd. and Oasis Rd.
		End			
	Test Section 2	Start	N45.450158, W92.878031		
		End			
	Test Section 3	Start	N45.450740, W92.854223		
		End			
Chisago County, CSAH 11, aka 375 St.	Test Section 1	Start	45.491551, -92.812268	Fly ash/cement-stabilized	Oasis Rd. CSAH 9 to Oriole Ave. (C.R. 70)
		End			
	Test Section 2	Start			
		End			
	Test Section 3	Start			
		End			
Chisago County, CSAH 19 aka Stacy Trail	Test Section 1		45.404538, -92.905036	Fly ash/cement-stabilized	Between Ivywood Trail and Lofton Ave.
	Test Section 2				
	Test Section 3				
Chisago County, CSAH 77 aka Lofton Ave.	Test Section 1	Start	45.408214, -92.882992	FDR	Between Lincoln Rd. (14) and Stacy Trail (19)
		End			
	Test Section 2	Start			
		End			
	Test Section 3	Start			
		End			
Chisago County CSAH 20, aka Furuby Rd.	Test Section 1	Start	N45.433226, W92.820123	FDR	Between Oasis Rd. (9) and Park Trail (12)
		End			
	Test Section 2	Start			
		End			
	Test Section 3	Start			
		End			

Location	Section	Start/End	Location GPS	Description of Pavement and Base	Additional Detail
Blue Earth County, TH 30	Test Section 1		N43.921733, W-94.021258	6 in. Foamed asphalt	

3.1 IN SITU TESTING – CORING

Pavement coring was performed to establish the thickness of the pavement and its condition (Figure 5). The cores were collected from non-crack locations to establish HMA pavement thickness and pavement condition and transferred to Iowa State University for further laboratory performance testing.

Figure 5. Pavement coring apparatus.

3.2 IN SITU TESTING – DYNAMIC CONE PENETRATION TEST

The dynamic cone penetration test provides a measure of a material's in situ resistance to penetration. According to the standard test method for use of the dynamic cone penetrometer in shallow pavement applications (ASTM D6951/D6951M-09), the test is performed by driving a metal cone into the ground by repeated strikes with a 17.6 lb (8 Kg) weight with a drop distance of 2.26 feet (575 mm). The penetration of the cone is measured after each blow and is recorded to provide a continuous measure of shearing resistance up to 5 feet below the ground surface. Test results can then be used to estimate the California bearing ratio (CBR), in situ density, resilient modulus, and bearing capacity.

The DCP apparatus consists of a 5/8-inch diameter steel rod with a 60 degree conical tip. The rod is topped with an anvil that is connected to a second steel rod. This rod is used as a guide to allow the weight to be repeatedly raised and dropped and hit the anvil. The connection between the two rods consists of an anvil to allow for quick connections between the rods and for efficient energy transfer from the falling weight to the penetrating rod (Figure 6).

Figure 6. DCP testing apparatus.

Once the test apparatus is assembled, the DCP is placed at the test location, and the initial penetration of the rod is recorded to provide a zeroing scale. While the rod is held vertically, the weight is raised to the top of the rod, 575 mm above the anvil, and dropped. The penetration of the rod is measured after each drop.

Several correlations exist between common engineering parameters and the dynamic penetration index (DPI). In this project, the DCP test results were used to estimate the CBR for future design purposes (ASTM D6951). Equation 5 was used for this purpose.

$$CBR(\%) = 292/DPI^{\mathbf{1.12}} \qquad \text{Eq. 5}$$

Some of the correlations between DPI and CBR are presented as follows.

The DPI for each drop was used to calculate an average DPI for both the upper 75 mm (3 inches) and 150 mm (6 inches). The first seating drop was not used in the calculations.

The average DPIs were then used to calculate the in situ CBR value using equations recommended by the US Army Corps of Engineers.

Equation 5 can be used when the CBR is greater than 10%. The DPI units are in mm/blow counts. Since all CBR's were greater than 10%, Equation 5 was used in all cases.

The DCP results for the test sections and the correlated in situ CBR values are presented in Tables 15 through 62. CBR percentages greater than 100 are reported as 100.

Table 15. DCP testing results, Chisago County, CSAH 81, River Road, emulsion stabilized

				Base layer extracted with core				Base layer extracted with core			
Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	14.7		2	0	29		3	0	17.7	
	5	16.2	1.5		5	33.6	4.6		5	20.7	3
	10	17.3	2.6		10	35.3	6.3		10	22	4.3
	15	18	3.3		15	36.9	7.9		15	23.6	5.9
	20	18.5	3.8		20	37.8	8.8		20	24.9	7.2
	25	18.9	4.2		25	39	10		25	26.4	8.7
	30	19.8	5.1		30	39.9	10.9		30	27.9	10.2
	35	19.9	5.2		35	41	12		35	29.7	12
	40	20	5.3		40	42.2	13.2		40	30.4	12.7
	45	20.4	5.7		45				45	32	14.3
	50	20.9	6.2		50				50	33.5	15.8

Table 16. Average DCP testing results, Chisago County, CSAH 81, River Road, emulsion stabilized

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							1.24
5	1.50	15.00	3.00	1	3.00	85.3	
10	2.60	11.00	2.20	1	2.20	100	
15	3.30	7.00	1.40	1	1.40	100	
20	3.80	5.00	1.00	1	1.00	100	
25	4.20	4.00	0.80	1	0.80	100	
30	5.10	9.00	1.80	1	1.80	100	
35	5.20	1.00	0.20	1	0.20	100	
40	5.30	1.00	0.20	1	0.20	100	
45	5.70	4.00	0.80	1	0.80	100	
50	6.20	5.00	1.00	1	1.00	100	

Table 17. DCP testing results, Chisago County, CSAH 11, 375th Street, fly ash/cement stabilized

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	14.9		2	0	15.2		3	0	13.4	
	5	16.8	1.9		5	17.5	2.3		5	16.4	3
	10	17.9	3		10	18.2	3		10	17.4	4
	15	18.8	3.9		15	19.2	4		15	17.9	4.5
	20	19.5	4.6		20	20	4.8		20	18.4	5
	25	20	5.1		25	20.2	5		25	18.6	5.2
	30	20.6	5.7		30	20.7	5.5		30	19	5.6
	35	21.1	6.2		35				35	19.4	6
	40	21.4	6.5		40				40	19.8	6.4
	45	21.8	6.9		45				45		
	50				50				50		

Table 18. Average DCP testing results, Chisago County, CSAH 11, 375th Street, fly ash/cement stabilized

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							1.53
5	2.40	24.00	4.80	1	4.80	50.4	
10	3.33	9.33	1.87	1	1.87	100	
15	4.13	8.00	1.60	1	1.60	100	
20	4.80	6.67	1.33	1	1.33	100	
25	5.10	3.00	0.60	1	0.60	100	
30	5.60	5.00	1.00	1	1.00	100	
35	6.10	5.00	1.00	1	1.00	100	
40	6.45	3.50	0.70	1	0.70	100	
45	6.90	4.50	0.90	1	0.90	100	
50							

Table 19. DCP testing results, Chisago County, CSAH 20, Furuby Road, FDR control

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	19.6		2	0	22.4		3	0	23	
	5	22.1	2.5		5	25	2.6		5	25.7	2.7
	10	23.6	4		10	26.9	4.5		10	27.4	4.4
	15	25.1	5.5		15	28.4	6		15	29	6
	20	26.8	7.2		20	30	7.6		20	30.9	7.9
	25	28.7	9.1		25	31.7	9.3		25	32.8	9.8
	30	29.9	10.3		30	33.4	11		30	34.8	11.8
	35	31.4	11.8		35	35	12.6		35	36.8	13.8
	40	33	13.4		40	36.8	14.4		40	38.4	15.4
	45	34.8	15.2		45	38.7	16.3		45	39.8	16.8
	50	36.3	16.7		50	40.2	17.8		50	41.5	18.5

Table 20. Average DCP testing results, Chisago County, CSAH 20, Furuby Road, FDR control

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	2.60	26.00	5.20	1	5.20	46.1	3.53
10	4.30	17.00	3.40	1	3.40	74.2	
15	5.83	15.33	3.07	1	3.07	83.2	
20	7.57	17.33	3.47	1	3.47	72.6	
25	9.40	18.33	3.67	1	3.67	68.1	
30	11.03	16.33	3.27	1	3.27	77.6	
35	12.73	17.00	3.40	1	3.40	74.2	
40	14.40	16.67	3.33	1	3.33	75.8	
45	16.10	17.00	3.40	1	3.40	74.2	
50	17.67	15.67	3.13	1	3.13	81.3	

Table 21. DCP testing results, Kanabec County, TH 65, Section 1, FDR control, type FDR control rut

Broke early; second asphalt layer not removed											
Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
	0	15			0	24.3			0	23.7	
	5	19.8	4.8		5	26.6	2.3		5	26.6	2.9
	10	21.6	6.6		10	28.1	3.8		10	28.2	4.5
	15	22.6	7.6		15	29.6	5.3		15	29.9	6.2
	20	23.5	8.5		20	30.5	6.2		20	31	7.3
1	25	24.4	9.4	2	25	31.3	7	3	25	31.8	8.1
	30	25	10		30	32.3	8		30	32.8	9.1
	35	25.6	10.6		35	33	8.7		35	33.5	9.8
	40	26.3	11.3		40	33.7	9.4		40	34.1	10.4
	45	26.9	11.9		45	34.5	10.2		45	34.7	11
	50	27.5	12.5		50	35	10.7		50	35.3	11.6

Table 22. Average DCP testing results, Kanabec County, TH 65, Section 1, FDR control, type FDR control rut

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	2.60	26.00	5.20	1	5.20	46.1	
10	4.15	15.50	3.10	1	3.10	82.2	
15	5.75	16.00	3.20	1	3.20	79.4	
20	6.75	10.00	2.00	1	2.00	100	
25	7.55	8.00	1.60	1	1.60	100	2.23
30	8.55	10.00	2.00	1	2.00	100	
35	9.25	7.00	1.40	1	1.40	100	
40	9.90	6.50	1.30	1	1.30	100	
45	10.60	7.00	1.40	1	1.40	100	
50	11.15	5.50	1.10	1	1.10	100	

Table 23. DCP testing results, Kanabec County, TH 65, Section 2, CSS1 stabilized, type CSS1 rut

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	29.6		2	0	29.2		3	0	30.6	
	5	31.7	2.1		5	32.1	2.9		5	33.2	2.6
	10	33.2	3.6		10	33.8	4.6		10	34.7	4.1
	15	34.5	4.9		15	35.1	5.9		15	35.8	5.2
	20	36	6.4		20	36.4	7.2		20	36.9	6.3
	25	37	7.4		25	37.4	8.2		25	37.8	7.2
	30	37.8	8.2		30	38.4	9.2		30	38.7	8.1
	35	38.8	9.2		35	39.2	10		35	39.7	9.1
	40	39.4	9.8		40	40.2	11		40	40.5	9.9
	45	40.4	10.8		45	41.1	11.9		45	41.3	10.7
	50	41.4	11.8		50				50	42.2	11.6

Table 24. Average DCP testing results, Kanabec County, TH 65, Section 2, CSS1 stabilized, type CSS1 rut

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							2.34
5	2.53	25.33	5.07	1	5.07	47.4	
10	4.10	15.67	3.13	1	3.13	81.3	
15	5.33	12.33	2.47	1	2.47	100	
20	6.63	13.00	2.60	1	2.60	100	
25	7.60	9.67	1.93	1	1.93	100	
30	8.50	9.00	1.80	1	1.80	100	
35	9.43	9.33	1.87	1	1.87	100	
40	10.23	8.00	1.60	1	1.60	100	
45	11.13	9.00	1.80	1	1.80	100	
50	11.70	5.67	1.13	1	1.13	100	

Table 25. DCP testing results, Kanabec County, TH 65, Section 3, EE stabilized, type EE rut 0

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	29		2	0	24		3	0	25	
	5	31	2		5	26.2	2.2		5	27	2
	10	32.5	3.5		10	27.5	3.5		10	28.5	3.5
	15	33.8	4.8		15	28.9	4.9		15	29.5	4.5
	20	34.8	5.8		20	29.7	5.7		20	30.7	5.7
	25	35.9	6.9		25	30.5	6.5		25	31.7	6.7
	30	36.7	7.7		30	31.3	7.3		30	32.6	7.6
	35	37.8	8.8		35	32.7	8.7		35	33.5	8.5
	40	38.9	9.9		40	33.5	9.5		40	34.5	9.5
	45	39.7	10.7		45	34.3	10.3		45	35	10
	50	40.2	11.2		50	35	11		50	36	11
	55	41.3	12.3		55	35.9	11.9		55	36.9	11.9
	60	41.9	12.9		60	36.9	12.9		60	37.7	12.7
	65				65	37.7	13.7		65	38.7	13.7
	70				70	38.7	14.7		70		

Table 26. Average DCP testing results, Kanabec County, TH 65, Section 3, EE stabilized, type EE rut 0

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							2.21
5	2.07	20.67	4.13	1	4.13	59.6	
10	3.50	14.33	2.87	1	2.87	89.8	
15	4.73	12.33	2.47	1	2.47	100	
20	5.73	10.00	2.00	1	2.00	100	
25	6.70	9.67	1.93	1	1.93	100	
30	7.53	8.33	1.67	1	1.67	100	
35	8.67	11.33	2.27	1	2.27	100	
40	9.63	9.67	1.93	1	1.93	100	
45	10.33	7.00	1.40	1	1.40	100	
50	11.07	7.33	1.47	1	1.47	100	

Table 27. DCP testing results, Douglas County, TH 55, Section 5, FDR control, put 1/10 in.

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	1	17.2		2	1	16.1		3	1	16.8	
	5	19.8	2.6		5	19	2.9		5	19.8	3
	10	21.5	4.3		10	20.9	4.8		10	21.7	4.9
	15	23.5	6.3		15	22.6	6.5		15	24	7.2
	20	25.7	8.5		20	24.8	8.7		20	26	9.2
	25	27.5	10.3		25	26.8	10.7		25	27.8	11
	30	28.9	11.7		30	28.9	12.8		30	29.2	12.4
	35	30.5	13.3		35	30.8	14.7		35	30.4	13.6
	40	32.2	15		40	32.2	16.1		40	31.6	14.8
	45	34.2	17		45	33.5	17.4		45	32.8	16
	50	36.4	19.2		50	34.5	18.4		50	34.4	17.6
	55				55	35.5	19.4		55	36.5	19.7
	60				60				60	38.2	21.4

Table 28. Average DCP testing results, Douglas County, TH 55, Section 5, FDR control, put 1/10 in.

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0	0.00						3.68
5	2.83	28.33	5.67	1	5.67	41.8	
10	4.67	18.33	3.67	1	3.67	68.1	
15	6.67	20.00	4.00	1	4.00	61.8	
20	8.80	21.33	4.27	1	4.27	57.5	
25	10.67	18.67	3.73	1	3.73	66.8	
30	12.30	16.33	3.27	1	3.27	77.6	
35	13.87	15.67	3.13	1	3.13	81.3	
40	15.30	14.33	2.87	1	2.87	89.8	
45	16.80	15.00	3.00	1	3.00	85.3	
50	18.40	16.00	3.20	1	3.20	79.4	

Table 29. DCP testing results, Douglas County, TH 55, Section 2, EE stabilized, type EE rut 0

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	22.2		2	0	18.1		3	0	24.1	
	5	24	1.8		5	20.7	2.6		5	26.3	2.2
	10	25.3	3.1		10	21.8	3.7		10	27.9	3.8
	15	26.7	4.5		15	22.8	4.7		15	29	4.9
	20	28	5.8		20	23.9	5.8		20	30.6	6.5
	25	29.4	7.2		25	24.7	6.6		25	31.7	7.6
	30	32.5	10.3		30	25.5	7.4		30	33.4	9.3
	35	33.4	11.2		35	26.6	8.5		35	35	10.9
	40	35.2	13		40	27.7	9.6		40	36.7	12.6
	45	36.1	13.9		45	28.8	10.7		45	38	13.9
	50	37	14.8		50	30	11.9		50	39.1	15
	55	38	15.8		55	31.5	13.4		55	40.3	16.2
	60	39	16.8		60	32.8	14.7		60		

Table 30. Average DCP testing results, Douglas County, TH 55, Section 2, EE stabilized, type EE rut 0

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							2.78
5	2.20	22.00	4.40	1	4.40	55.6	
10	3.53	13.33	2.67	1	2.67	97.3	
15	4.70	11.67	2.33	1	2.33	100	
20	6.03	13.33	2.67	1	2.67	97.3	
25	7.13	11.00	2.20	1	2.20	100	
30	9.00	18.67	3.73	1	3.73	66.8	
35	10.20	12.00	2.40	1	2.40	100	
40	11.73	15.33	3.07	1	3.07	83.2	
45	12.83	11.00	2.20	1	2.20	100	
50	13.90	10.67	2.13	1	2.13	100	

Table 31. DCP testing results, Douglas County, TH 55, Section 1, CSS1 emulsion stabilized, chip seal two years

Layer Extracted with core				*Layer at bold line				*Layer at bold line			
Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	29		2	0	13.8		3	0	14	
	5	31.5	2.5		5	16	2.2		5	15	1
	10	34.6	3.1		10	17.4	3.6		10	16	2
	15	37	2.4		15	18.5	4.7		15	16.5	2.5
	20	39.8	2.8		20	19.3	5.5		20	17.1	3.1
	25	43	3.2		25	20.2	6.4		25	17.3	3.3
	30	45.4	2.4		30	21.5	7.7		30	17.9	3.9
	35	46.9	1.5		35	23	9.2		35	18.4	4.4
	40	48.5	1.6		40	24.9	11.1		40	19.1	5.1
	45	50	1.5		45	26.5	12.7		45	19.3	5.3
	50	52.5	2.5		50	28	14.2		50	19.9	5.9

Table 32. Average DCP testing results, Douglas County, TH 55, Section 1, CSS1 emulsion stabilized, chip seal two years

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	1.60	16.00	3.20	1	3.20	79.4	2.01
10	2.80	12.00	2.40	1	2.40	100	
15	3.60	8.00	1.60	1	1.60	100	
20	4.30	7.00	1.40	1	1.40	100	
25	4.85	5.50	1.10	1	1.10	100	
30	5.80	9.50	1.90	1	1.90	100	
35	6.80	10.00	2.00	1	2.00	100	
40	8.10	13.00	2.60	1	2.60	100	
45	9.00	9.00	1.80	1	1.80	100	
50	10.05	10.50	2.10	1	2.10	100	

Table 33. DCP testing results, Douglas County, TH 55, Section 3, foamed asphalt stabilized, type FA rut 0

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	35.3		2	0	34.5		3	0	34.9	
	5	37.7	2.4		5	36.9	2.4		5	37	2.1
	10	39.5	4.2		10	38.4	3.9		10	38.3	3.4
	15	40.6	5.3		15	39.2	4.7		15	39.5	4.6
	20	41.9	6.6		20	40.4	5.9		20	40.5	5.6
	25	42.4	7.1		25	41.6	7.1		25	41.1	6.2
	30	43.2	7.9		30	42	7.5		30	42.4	7.5
	35	44.6	9.3		35	42.8	8.3		35	43.6	8.7
	40	46.1	10.8		40	44.1	9.6		40	45.5	10.6
	45	48.1	12.8		45	45.9	11.4		45	47.4	12.5
	50	50	14.7		50	47.8	13.3		50	49.6	14.7

Table 34. Average DCP testing results, Douglas County, TH 55, Section 3, foamed asphalt stabilized, type FA rut 0

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	2.30	23.00	4.60	1	4.60	52.9	2.85
10	3.83	15.33	3.07	1	3.07	83.2	
15	4.87	10.33	2.07	1	2.07	100	
20	6.03	11.67	2.33	1	2.33	100	
25	6.80	7.67	1.53	1	1.53	100	
30	7.63	8.33	1.67	1	1.67	100	
35	8.77	11.33	2.27	1	2.27	100	
40	10.33	15.67	3.13	1	3.13	81.3	
45	12.23	19.00	3.80	1	3.80	65.5	
50	14.23	20.00	4.00	1	4.00	61.8	

Table 35. DCP testing results, Douglas County, TH 55, Section 4, BaseOne stabilized, type BaseOne rut 0

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	18.4		2	0	18		3	0	17	
	5	20.2	1.8		5	20.3	2.3		5	19.1	2.1
	10	21.2	2.8		10	21.1	3.1		10	20.8	3.8
	15	22.5	4.1		15	22	4		15	21.5	4.5
	20	23.3	4.9		20	22.5	4.5		20	22.4	5.4
	25	24.4	6		25	23.5	5.5		25	23	6
	30	25	6.6		30	24.4	6.4		30	24.1	7.1
	35	25.8	7.4		35	25	7		35	24.8	7.8
	40	26.5	8.1		40	25.6	7.6		40	25.8	8.8
	45	27.5	9.1		45	26.5	8.5		45	26.5	9.5
	50	28.1	9.7		50	27	9		50	27.3	10.3
	55	28.5	10.1		55	27.7	9.7		55	27.9	10.9
	60	29.2	10.8		60	28.5	10.5		60	28.7	11.7
	65	30.1	11.7		65	29	11		65	29.5	12.5

Table 36. Average DCP testing results, Douglas County, TH 55, Section 4, BaseOne stabilized, type BaseOne rut 0

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							1.93
5	2.07	20.67	4.13	1	4.13	59.6	
10	3.23	11.67	2.33	1	2.33	100	
15	4.20	9.67	1.93	1	1.93	100	
20	4.93	7.33	1.47	1	1.47	100	
25	5.83	9.00	1.80	1	1.80	100	
30	6.70	8.67	1.73	1	1.73	100	
35	7.40	7.00	1.40	1	1.40	100	
40	8.17	7.67	1.53	1	1.53	100	
45	9.03	8.67	1.73	1	1.73	100	
50	9.67	6.33	1.27	1	1.27	100	

Table 37. DCP testing results, Chisago County, CSAH 74, 347th Street, emulsion stabilized

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	27.8		2	0	27.4		3	0	28.1	
	5	29.2	1.4		5	29.8	2.4		5	31	2.9
	10	30.7	2.9		10	31.1	3.7		10	32.5	4.4
	15	31	3.2		15	32.5	5.1		15	34.3	6.2
	20	32.2	4.4		20	33.6	6.2		20	36	7.9
	25	33.6	5.8		25	34.7	7.3		25	37	8.9
	30	34.4	6.6		30	35.8	8.4		30	38.4	10.3
	35	35.2	7.4		35	36.9	9.5		35	39.7	11.6
	40	36.4	8.6		40	37.9	10.5		40	40.7	12.6

Table 38. Average DCP testing results, Chisago County, CSAH 74, 347th Street, emulsion stabilized

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							2.64
5	2.23	22.33	4.47	1	4.47	54.6	
10	3.67	14.33	2.87	1	2.87	89.8	
15	4.83	11.67	2.33	1	2.33	100	
20	6.17	13.33	2.67	1	2.67	97.3	
25	7.33	11.67	2.33	1	2.33	100	
30	8.43	11.00	2.20	1	2.20	100	
35	9.50	10.67	2.13	1	2.13	100	
40	10.57	10.67	2.13	1	2.13	100	

Table 39. DCP testing results, Chisago County, CSAH 24, Lofton Ave, FDR control

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
	0	25.5			0	22.4			0	18.8	
	5	27.8	2.3		5	24.4	2		5	21	2.2
	10	29.7	4.2		10	26	3.6		10	22.9	4.1
	15	31.4	5.9		15	27.3	4.9		15	24.1	5.3
	20	32.5	7		20	28.8	6.4		20	25.7	6.9
1	25	34	8.5	2	25	30.8	8.4	3	25	27.2	8.4
	30	35.2	9.7		30	32	9.6		30	28.7	9.9
	35	36.7	11.2		35	33.3	10.9		35	30.1	11.3
	40	38.5	13		40	34.8	12.4		40	31.5	12.7
	45	40.2	14.7		45	36.2	13.8		45	32.7	13.9
	50	42.5	17		50	38.1	15.7		50	34.4	15.6

Table 40. Average DCP testing results, Chisago County, CSAH 24, Lofton Ave, FDR control

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	2.17	21.67	4.33	1	4.33	56.5	
10	3.97	18.00	3.60	1	3.60	69.6	
15	5.37	14.00	2.80	1	2.80	92.2	
20	6.77	14.00	2.80	1	2.80	92.2	
25	8.43	16.67	3.33	1	3.33	75.8	3.22
30	9.73	13.00	2.60	1	2.60	100	
35	11.13	14.00	2.80	1	2.80	92.2	
40	12.70	15.67	3.13	1	3.13	81.3	
45	14.13	14.33	2.87	1	2.87	89.8	
50	16.10	19.67	3.93	1	3.93	63.0	

Table 41. DCP testing results, Chisago County, CSAH 19, Stacy Trail, fly ash/cement stabilized, rut = 0

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	14.5		2	0	14.8		3	0	14.8	
	5	19.1	4.6		5	17.7	2.9		5	20.5	5.7
	10	23	8.5		10	18.4	3.6		10	22	7.2
	15	27	12.5		15	18.8	4		15	23.6	8.8
	20	30.3	15.8		20	19.8	5		20	25	10.2
	25	32.5	18		25	20.3	5.5		25	26.5	11.7
	30	35	20.5		30	20.5	5.7		30	29	14.2
	35	38.4	23.9		35	20.8	6		35	32	17.2
	40	43.6	29.1		40	21.3	6.5		40	37	22.2
	45	51	36.5		45	21.4	6.6		45	42.5	27.7
	50	63	48.5		50	21.5	6.7		50	48.5	33.7

Table 42. Average DCP testing results, Chisago County, CSAH 19, Stacy Trail, fly ash/cement stabilized, rut = 0

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	4.40	44.00	8.80	1	8.80	25.6	5.93
10	6.43	20.33	4.07	1	4.07	60.7	
15	8.43	20.00	4.00	1	4.00	61.8	
20	10.33	19.00	3.80	1	3.80	65.5	
25	11.73	14.00	2.80	1	2.80	92.2	
30	13.47	17.33	3.47	1	3.47	72.6	
35	15.70	22.33	4.47	1	4.47	54.6	
40	19.27	35.67	7.13	1	7.13	32.3	
45	23.60	43.33	8.67	1	8.67	26.0	
50	29.63	60.33	12.07	1	12.07	17.9	

Table 43. DCP testing results, Wabasha County, CSAH 9, FDR with 4 in. aggregate base modified class 5 (2 in.), recently sealed summer 2017

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	19.7		2	0	18.3		3	0	15.7	
	5	21.4	1.7		5	20.4	2.1		5	17.5	1.8
	10	22.6	2.9		10	22	3.7		10	18.8	3.1
	15	24	4.3		15	23.2	4.9		15	19.8	4.1
	20	25	5.3		20	24.7	6.4		20	20.4	4.7
	25	26.1	6.4		25	25	6.7		25	21.3	5.6
	30	27.5	7.8		30	26	7.7		30	22	6.3
	35	28.4	8.7		35	27.6	9.3		35	23.2	7.5
	40	29.3	9.6		40	28.5	10.2		40	24	8.3
	45	29.9	10.2		45	29.4	11.1		45	25	9.3
	50	30.5	10.8		50	30.4	12.1		50	25.9	10.2

Table 44. Average DCP testing results, Wabasha County, CSAH 9, FDR with 4 in. aggregate base modified class 5 (2 in.), recently sealed summer 2017

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							2.21
5	1.87	18.67	3.73	1	3.73	66.8	
10	3.23	13.67	2.73	1	2.73	94.7	
15	4.43	12.00	2.40	1	2.40	100	
20	5.47	10.33	2.07	1	2.07	100	
25	6.23	7.67	1.53	1	1.53	100	
30	7.27	10.33	2.07	1	2.07	100	
35	8.50	12.33	2.47	1	2.47	100	
40	9.37	8.67	1.73	1	1.73	100	
45	10.20	8.33	1.67	1	1.67	100	
50	11.03	8.33	1.67	1	1.67	100	

Table 45. DCP testing results, Wabasha County, CSAH 16, 12 in. FDR stabilized with BaseOne upper 4 in., recently slurry sealed fall 2016

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	21.9		2	0	22.1		3	0	17	
	5	25	3.1		5	24.2	2.1		5	19	2
	10	27.2	5.3		10	25.7	3.6		10	19.9	2.9
	15	28.6	6.7		15	26.7	4.6		15	20.7	3.7
	20	29.5	7.6		20	27.6	5.5		20	21.6	4.6
	25	30	8.1		25	28.9	6.8		25	22.4	5.4
	30	31.4	9.5		30	29.4	7.3		30	23.3	6.3
	35	32.2	10.3		35	30.2	8.1		35	24.5	7.5
	40	33.3	11.4		40	30.9	8.8		40	25.2	8.2
	45	34.2	12.3		45	31.8	9.7		45	26.1	9.1
	50				50	32.5	10.4		50	27	10

Table 46. Average DCP testing results, Wabasha County, CSAH 16, 12 in. FDR stabilized with BaseOne upper 4 in., recently slurry sealed fall 2016

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							2.04
5	2.40	24.00	4.80	1	4.80	50.4	
10	3.93	15.33	3.07	1	3.07	83.2	
15	5.00	10.67	2.13	1	2.13	100	
20	5.90	9.00	1.80	1	1.80	100	
25	6.77	8.67	1.73	1	1.73	100	
30	7.70	9.33	1.87	1	1.87	100	
35	8.63	9.33	1.87	1	1.87	100	
40	9.47	8.33	1.67	1	1.67	100	
45	10.37	9.00	1.80	1	1.80	100	
50	10.20	-1.67	-0.33	1	-0.33		

Table 47. DCP testing results, Blue Earth County, TH 30, 6 in. (15 cm) foamed asphalt stabilized base, rut = 1/10 in.

Base Extracted with Core											
Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	27.2		2	0	21.8		3	0	18	
	5	29.1	1.9		5	23.7	1.9		5	19.8	1.8
	10	30.8	3.6		10	25	3.2		10	20.9	2.9
	15	32.1	4.9		15	26.5	4.7		15	21.7	3.7
	20	33.9	6.7		20	27.9	6.1		20	23.2	5.2
	25	35.4	8.2		25	29.4	7.6		25	24.3	6.3
	30	37.2	10		30	31.1	9.3		30	25.2	7.2
	35	39.4	12.2		35	32.7	10.9		35	26.8	8.8
	40	41	13.8		40	34.3	12.5		40	27.7	9.7
	45	42.6	15.4		45	36.4	14.6		45	29	11
	50	43.9	16.7		50	38	16.2		50	30.4	12.4

Table 48. Average DCP testing results, Blue Earth County, TH 30, 6 in. (15 cm) foamed asphalt stabilized base, rut = 1/10 in.

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	1.85	18.50	3.70	1	3.70	67.5	2.86
10	3.05	12.00	2.40	1	2.40	100	
15	4.20	11.50	2.30	1	2.30	100	
20	5.65	14.50	2.90	1	2.90	88.6	
25	6.95	13.00	2.60	1	2.60	100	
30	8.25	13.00	2.60	1	2.60	100	
35	9.85	16.00	3.20	1	3.20	79.4	
40	11.10	12.50	2.50	1	2.50	100	
45	12.80	17.00	3.40	1	3.40	74.2	
50	14.30	15.00	3.00	1	3.00	85.3	

Table 49. DCP testing results, Dodge County, CSAH 10, FDR in 2010

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	13.9		2	0	14.7		3	0	15.8	
	5	15.8	1.9		5	16.5	1.8		5	17.4	1.6
	10	16.7	2.8		10	17.7	3		10	18.4	2.6
	15	17.7	3.8		15	18.5	3.8		15	19.7	3.9
	20	18.4	4.5		20	19.7	5		20	20.8	5
	25	19.5	5.6		25	20.6	5.9		25	21.9	6.1
	30	20.6	6.7		30	21.9	7.2		30	22.6	6.8
	35	21.8	7.9		35	23.2	8.5		35	23.7	7.9
	40	22.8	8.9		40	24.6	9.9		40	25	9.2
	45	24	10.1		45	26	11.3		45	26.1	10.3
	50	25.3	11.4		50	27.6	12.9		50	27.4	11.6

Table 50. Average DCP testing results, Dodge County, CSAH 10, FDR in 2010

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							2.39
5	1.77	17.67	3.53	1	3.53	71.0	
10	2.80	10.33	2.07	1	2.07	100	
15	3.83	10.33	2.07	1	2.07	100	
20	4.83	10.00	2.00	1	2.00	100	
25	5.87	10.33	2.07	1	2.07	100	
30	6.90	10.33	2.07	1	2.07	100	
35	8.10	12.00	2.40	1	2.40	100	
40	9.33	12.33	2.47	1	2.47	100	
45	10.57	12.33	2.47	1	2.47	100	
50	11.97	14.00	2.80	1	2.80	92.2	

Table 51. DCP testing results, Rice County, CSAH 29, 10 in. FDR with 4 in. bituminous surfacing, no visible cracks on two-year overlay

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	18		2	0	16		3	0	16.2	
	5	20.3	2.3		5	18.6	2.6		5	18.3	2.1
	10	22.4	4.4		10	20.6	4.6		10	20.4	4.2
	15	24.6	6.6		15	22	6		15	22	5.8
	20	27.3	9.3		20	23.8	7.8		20	24	7.8
	25	30	12		25	25.8	9.8		25	26.5	10.3
	30	33	15		30	27.7	11.7		30	29.4	13.2
	35	38	20		35	30.2	14.2		35	32.8	16.6
	40	39.6	21.6		40	32.9	16.9		40	35.7	19.5
	45	41.1	23.1		45	35.5	19.5		45	37.4	21.2
	50				50	37.4	21.4		50	38.9	22.7

Table 52. Average DCP testing results, Rice County, CSAH 29, 10 in. FDR with 4 in. bituminous surfacing, no visible cracks on two-year overlay

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	2.33	23.33	4.67	1	4.67	52.0	4.41
10	4.40	20.67	4.13	1	4.13	59.6	
15	6.13	17.33	3.47	1	3.47	72.6	
20	8.30	21.67	4.33	1	4.33	56.5	
25	10.70	24.00	4.80	1	4.80	50.4	
30	13.30	26.00	5.20	1	5.20	46.1	
35	16.93	36.33	7.27	1	7.27	31.7	
40	19.33	24.00	4.80	1	4.80	50.4	
45	21.27	19.33	3.87	1	3.87	64.2	
50	22.05	7.83	1.57	1	1.57	100	

Table 53. DCP testing results, Rice County, CSAH 26, 12 in. FDR with 4 in. bituminous surfacing

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	14.6		2	0	16.1		3	0	18.4	
	5	16.3	1.7		5	18.5	2.4		5	20.3	1.9
	10	17.3	2.7		10	19.2	3.1		10	21.7	3.3
	15	18.6	4		15	20.3	4.2		15	23	4.6
	20	19.3	4.7		20	21.3	5.2		20	24	5.6
	25	20.5	5.9		25	22.6	6.5		25	25.2	6.8
	30	21.4	6.8		30	23.6	7.5		30	26.4	8
	35	22.7	8.1		35	24.5	8.4		35	27.5	9.1
	40	24	9.4		40	25.2	9.1		40	28.4	10
	45	25.4	10.8		45	26.4	10.3		45	29.5	11.1
	50	26.5	11.9		50	27.4	11.3		50	30.2	11.8

Table 54. Average DCP testing results, Rice County, CSAH 26, 12 in. FDR with 4 in. bituminous surfacing

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	2.00	20.00	4.00	1	4.00	61.8	2.33
10	3.03	10.33	2.07	1	2.07	100	
15	4.27	12.33	2.47	1	2.47	100	
20	5.17	9.00	1.80	1	1.80	100	
25	6.40	12.33	2.47	1	2.47	100	
30	7.43	10.33	2.07	1	2.07	100	
35	8.53	11.00	2.20	1	2.20	100	
40	9.50	9.67	1.93	1	1.93	100	
45	10.73	12.33	2.47	1	2.47	100	
50	11.67	9.33	1.87	1	1.87	100	

Table 55. DCP testing results, Dodge County, Co Hwy 27, 2006 pavement stabilized with BaseOne

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	25.1		2	0	16		3	0	15.8	
	5	26.7	1.6		5	18	2		5	18.1	2.3
	10	27.7	2.6		10	19	3		10	19.6	3.8
	15	28	2.9		15	20.1	4.1		15	20.7	4.9
	20	28.8	3.7		20	21.4	5.4		20	21.7	5.9
	25	29.9	4.8		25	22.1	6.1		25	22.6	6.8
	30	30.6	5.5		30	22.9	6.9		30	23.2	7.4
	35	31	5.9		35	23.8	7.8		35	24.2	8.4
	40	31.6	6.5		40	24.9	8.9		40	25.1	9.3
	45	32.4	7.3		45	25.7	9.7		45	26.4	10.6
	50	33	7.9		50	26.4	10.4		50	27.5	11.7

Table 56. Average DCP testing results, Dodge County, Co Hwy 27, 2006 pavement stabilized with BaseOne

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	1.97	19.67	3.93	1	3.93	63.0	2.00
10	3.13	11.67	2.33	1	2.33	100	
15	3.97	8.33	1.67	1	1.67	100	
20	5.00	10.33	2.07	1	2.07	100	
25	5.90	9.00	1.80	1	1.80	100	
30	6.60	7.00	1.40	1	1.40	100	
35	7.37	7.67	1.53	1	1.53	100	
40	8.23	8.67	1.73	1	1.73	100	
45	9.20	9.67	1.93	1	1.93	100	
50	10.00	8.00	1.60	1	1.60	100	

Table 57. DCP testing results, Goodhue County, CSAH 11, FDR from 2012

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	16.2		2	0	15.2		3	0	16	
	5	17.9	1.7		5	17.4	2.2		5	17.4	1.4
	10	19.3	3.1		10	18.8	3.6		10	18.7	2.7
	15	20.3	4.1		15	20.2	5		15	19.9	3.9
	20	21.4	5.2		20	21.4	6.2		20	21.2	5.2
	25	22.5	6.3		25	22.9	7.7		25	22.8	6.8
	30	23.8	7.6		30	24.6	9.4		30	24.1	8.1
	35	25.2	9		35	26	10.8		35	25.2	9.2
	40	26.7	10.5		40	27.4	12.2		40	26.7	10.7
	45	28.3	12.1		45	28.8	13.6		45	28.2	12.2
	50	29.7	13.5		50	29.9	14.7		50	29.4	13.4

Table 58. Average DCP testing results, Goodhue County, CSAH 11, FDR from 2012

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	1.77	17.67	3.53	1	3.53	71.0	2.77
10	3.13	13.67	2.73	1	2.73	94.7	
15	4.33	12.00	2.40	1	2.40	100	
20	5.53	12.00	2.40	1	2.40	100	
25	6.93	14.00	2.80	1	2.80	92.2	
30	8.37	14.33	2.87	1	2.87	89.8	
35	9.67	13.00	2.60	1	2.60	100	
40	11.13	14.67	2.93	1	2.93	87.5	
45	12.63	15.00	3.00	1	3.00	85.3	
50	13.87	12.33	2.47	1	2.47	100	

Table 59. DCP testing results, Goodhue County, CSAH 7 North, FDR from 2005

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	20.4		2	0	20		3	0	19.1	
	5	22	1.6		5	21.9	1.9		5	20.7	1.6
	10	23.1	2.7		10	23.9	3.9		10	22.4	3.3
	15	24.7	4.3		15	25.2	5.2		15	24.3	5.2
	20	26.2	5.8		20	26.7	6.7		20	25.4	6.3
	25	27.4	7		25	28.2	8.2		25	26.6	7.5
	30	28.8	8.4		30	29.8	9.8		30	27.8	8.7
	35	30.1	9.7		35	31.2	11.2		35	28.9	9.8
	40	31.3	10.9		40	32.5	12.5		40	29.9	10.8
	45	32.2	11.8		45	33.6	13.6		45	31	11.9
	50	33.4	13		50	34.9	14.9		50	32.1	13

Table 60. Average DCP testing results, Goodhue County, CSAH 7 North, FDR from 2005

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	1.70	17.00	3.40	1	3.40	74.2	2.73
10	3.30	16.00	3.20	1	3.20	79.4	
15	4.90	16.00	3.20	1	3.20	79.4	
20	6.27	13.67	2.73	1	2.73	94.7	
25	7.57	13.00	2.60	1	2.60	100	
30	8.97	14.00	2.80	1	2.80	92.2	
35	10.23	12.67	2.53	1	2.53	100	
40	11.40	11.67	2.33	1	2.33	100	
45	12.43	10.33	2.07	1	2.07	100	
50	13.63	12.00	2.40	1	2.40	100	

Table 61. DCP testing results, Goodhue County, CSAH 7 South, FDR from 1998, rut 1/10 in.

Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative	Section #	# Blows	Depth (cm)	Cumulative
1	0	20.8		2	0	18.3		3	0	16.8	
	5	23	2.2		5	20.9	2.6		5	19	2.2
	10	25.1	4.3		10	22.9	4.6		10	20.8	4
	15	27	6.2		15	24.4	6.1		15	22	5.2
	20	28.8	8		20	26	7.7		20	23.4	6.6
	25	31	10.2		25	27.6	9.3		25	24.9	8.1
	30	33	12.2		30	29.9	11.6		30	26.5	9.7
	35	35.3	14.5		35	32.2	13.9		35	28.2	11.4
	40	37.7	16.9		40	34.3	16		40	29.9	13.1
	45	39.4	18.6		45	37.6	19.3		45	32.2	15.4
	50	41	20.2		50	40.1	21.8		50	34.1	17.3

Table 62. Average DCP testing results, Goodhue County, CSAH 7 South, FDR from 1998, rut 1/10 in.

# Blows	Depth (cm)	Penetration Between Readings mm	Penetration per Blow mm	Hammer Factor	DCP Index mm/blow	CBR %	Average mm/blow
0							
5	2.33	23.33	4.67	1	4.67	52.0	3.95
10	4.30	19.67	3.93	1	3.93	63.0	
15	5.83	15.33	3.07	1	3.07	83.2	
20	7.43	16.00	3.20	1	3.20	79.4	
25	9.20	17.67	3.53	1	3.53	71.0	
30	11.17	19.67	3.93	1	3.93	63.0	
35	13.27	21.00	4.20	1	4.20	58.5	
40	15.33	20.67	4.13	1	4.13	59.6	
45	17.77	24.33	4.87	1	4.87	49.6	
50	19.77	20.00	4.00	1	4.00	61.8	

A summary of the DCP results for different locations and different stabilization treatments is presented in Table 63.

Table 63. Summary of DCP results for stabilization treatments at different locations

Treatment	County	Average DCP Index (mm/blow)	CBR%
Asphalt emulsion	Chisago CSAH 81	1.24	229.5*
Fly ash+ Cement	Chisago CSAH 11	1.53	181.4*
FDR control	Chisago CSAH 20	3.53	71.1
FDR control	Kanabec section 1	2.23	118.9*
CSS1+Cement	Kanabec section 2	2.34	112.7*
EE	Kanabec section 3	2.21	120.1*
FDR control	Douglass TH 55 section 5	3.68	67.9
EE	Douglass TH 55 section 2	2.78	92.9
CCS1+Cement	Douglass TH55 section 1	2.01	133.6*
Foamed asphalt	Douglass TH55 section 3	2.85	90.4
BaseOne	Douglass TH55 section 4	1.93	139.8*
Asphalt emulsion	Chisago CSAH 74	2.64	98.4
FDR control	Chisago CSAH 20	3.22	78.8
Fly ash+ Cement	Chisago CSAH 19	5.93	39.8
FDR with 4 in. AB	Wabasha CSAH 9	2.21	120.1*
BaseOne	Wabasha CSAH 16	2.04	131.4*
Foamed asphalt	Blue Earth TH 30	2.86	90.0
FDR in 2010	Dodge CSAH 10	2.39	110.0*
10 in. FDR+4 in. bituminous surface	Rice CSAH 29	4.41	55.4
12 in. FDR+4 in. bituminous surface	Rice CSAH 26	2.33	113.2*
BaseOne in 2006	Dodge CSAH 19	2.00	134.3*
FDR 2012	Goodhue CSAH 11	2.77	93.3
FDR 2005	Goodhue CSAH 7N	2.73	94.8
FDR 1998	Goodhue CSAH 7S	3.95	62.7

* CBR percentages greater than 100 are beyond the range of most correlations between the DCP index and CBR% and therefore may not be reliable.

CBR values resulting from the correlation between the DCP index and CBR percentage for some of the base layers are greater than 100%. Most such correlations were developed for CBR percentages of less than 100; therefore, CBR percentage values that exceed 100% may not be reliable.

Average DCP vales (mm/blow) were measured for each stabilization treatment and are presented in Table 64.

Table 64. Sorted average CBR values for different stabilization treatments

Treatment	Treatment Rank	Average DCP (mm/blow)
Asphalt emulsion	1	1.94
BaseOne	2	2.00
CSS1+cement	3	2.175
Engineered emulsion	4	2.495
Foamed asphalt	5	2.855
FDR control	6	3.04
Fly ash+ cement	7	3.73

The values are in ascending order, meaning that SDFR with asphalt emulsion shows the highest resistance against penetration, which implies the highest value for stiffness, while SFDR with fly ash and cement shows the lowest resistance to penetration, which implies the lowest stiffness.

Based on the resulting DCP data in Tables 15 through,64, it appears that emulsions, emulsion/cement hybrids, and BaseOne roads exhibited the highest in situ stiffness as indicated by the DCP index comparison. Fly ash/cement, EE, foamed asphalt, and FDR roads exhibited the lowest in situ stiffness as indicated by the DCP index comparison.

3.3 VISUAL DISTRESS IDENTIFICATION

Visual distress identification was performed based on the Long-Term Pavement Performance (LTPP) Program specifications, which identify the distress by its location and length or thickness depending on the distress type. The severity of the distress is a number from 1 to 3, with 3 indicating the highest severity (Elkins et al. 2003). Each distress is identified by a code between 1 and 9. Table 65 presents the distress evaluation criteria used by the LTPP manual. The results are reported in terms of distress type and severity.

Table 65. Distress evaluation criteria

Distress Type	Code	Unit of Measure	Thickness Unit
Transverse cracking	1	ft	mm
Longitudinal cracking	2	ft	mm
Fatigue	3	ft^2	mm
Pothole	4	ft	N/A
Patching	5	ft^2	N/A
Bleeding	6	ft^2	N/A
Loss of aggregate cover	7	ft^2	N/A
Rutting	8	in.	N/A
Raveling	9	ft^2	N/A

It should be mentioned that distress identification and pavement condition evaluation are typically performed for pavements that are sufficiently old, and not all of the sections in this study were old enough to be evaluated (that is, they did not exhibit any distress). Some of the older pavement sections had been maintained with overlays; therefore, some of the underlying distress was not visible.

The results of a pavement surface condition evaluation are presented in Table 66.

Table 66. Distress evaluation results

Name	Section	Distress Code	Location (ft)	Measurement	LTTP Severity Low - High (1–3)
Chisago	CSAH 81 River Road	3	0–24	10 ft^2	1
		1	7	10	3
		1	25	5	2
		1	31	5	2
		1	76	20	sealed
		3	92–124	5 ft^2	
		1	118	8	2
		1	124	5	2
		1	131	10	2
		1	143	6	2
		1	147	6	2
		1	174	8	3
		1	188	10	3
		1	212	7	2
		1	234	10	sealed
		1	262	6	2
Chisago	CSAH 11, 375th	1	104	12	3
		1	161	12	3
		2	190–200	10	1
		–	104–240	36	1
		1	261	12	3
		2	300–316	16	1
		2	324–350	26	1
		1	341	12	3
		1	468	12	3
Kanabec	TH 65, Section 1	1	67	2	1
		1	96	24	2
		1	225	24	3
		1	318	24	3
		1	389	24	3
		1	469	18	3
Kanabec	TH 65, Section 2	1	50	10	2
		CL	0–90	90	1
		1	108	24	3
		1	167	24	3
		1	237	24	3
		1	275	24	3
		1	325	24	3
		1	350	18	2
		1	394	7	3
		1	430	24	3
		1	489	24	3

Name	Section	Distress Code	Location (ft)	Measurement	LTTP Severity Low - High (1–3)
Kanabec	TH 65, Section 3	1	4	24	3
		1	65	13	3
		1	97	24	3
		CL	121–410	289	1
		1	174	24	3
		1	243	24	3
		1	335	24	3
		1	422	24	3
		1	450	12	3
		1	320	4	3
		1	310	12	3
Douglas	TH 55, Section 5	1	113	4	1
		1	259	24	2
		1	315	24	3
		1	382	13	2
Douglas	TH 55, Section 2	1	62	9	1
		1	72	9	1
		1	78	12	2
		9	164	1	2
		1	498	24	3
Douglas	TH 55, Section 1	1	74	24	3
		1	178	24	2
		1	318	24	3
		1	447	24	2
Douglas	TH 55, Section 3	1	321–362	47	1
		1	391–408	17	1
Douglas	TH 55, Section 4	1	36	24	3
		1	175	24	3
		1	218	24	2
		CL	312–350	38	1
		1	350	10	3
Blue Earth	TH 30	1	14	12	1
		1	158	12	1
		1	341	12	1
Dodge	CSAH 19	1	20	12	1
		1	86	9	1
		2	90–120	30	2
		1	102	6	1
		1	123	9	1
		1	135	4	2
		2	144–250	106	1
		1	217	12	1
		1	294	12	1
		2	334–338	4	2
		2	364–376	12	2
		1	400	12	1
		2	400–431	31	2
		1	442	6	1
		1	481	12	1

Name	Section	Distress Code	Location (ft)	Measurement	LTTP Severity Low - High (1–3)
Goodhue	CSAH 11	1	37	12	1
		1	153	12	1
		1	271	12	1
		1	432	12	1
Goodhue	CSAH 7 North	1	16	12	1
		1	92	3	1
		1	117	CL	2
		1	131	4	2
		1	206	12	1
		1	276	3	2
		1	302	12	2
		1	307	3	1
		1	323	8	2
		1	335	3	1
		1	341	12	2
		1	365	6	1
		1	377	6	1
		1	382	3	1
		1	394	3	1
		1	400	3	2
		1	430	2	1
		1	492	10	2
Goodhue	CSAH 7 South	9	0–500	Between Wheelpaths	
		1	159	1	2
		1	195	12	1
		1	365	3	2
Chisago	CSAH 19	2	38–63	25	1
		2	100–123	23	1
		1	113	12	1
		1	115	12	2
		1	180	12	1
		1	183	12	1
		1	272	12	1
		1	305	12	1
		1	341	12	1
		1	415	12	1
		1	451	12	1

Rutting was measured for all pavements in the wheel path, with values between 0 and 0.1 inches. Pavement sections Chisago CSAH 74, Chisago CSAH 24, Wabasha CSAH 9, Wabasha CSAH 16, Rice CSAH 29, Rice CSAH 26, and Dodge CSAH 19 are not included in Table 66; these roads had been maintained with recent surface treatments, and it was therefore not possible to see the cracks so that a crack inspection could be performed.

Observed distresses were mostly transverse cracks of light to medium severity, with occasional examples of longitudinal cracking and high-severity transverse cracks. Very little distress was observed on the roads on the primary system. The most distress observed on a primary road was TH 55 Section 2 in Kanabec County, where an average of one transverse crack was noted every 50 feet; however, many of these were classified as high severity. The most cracks were noted on Goodhue County CSAH 7 North, which had a mix of low- and medium-severity transverse cracks averaging every 28 feet. Many of the roads had been maintained with surface treatments and overlays, and no detectable defects were observable. In such situations, it seems reasonable to surmise that these roads are performing well, except in the case where a thick HMA overlay was placed to cover relatively severe distress and the distress had not had time to reflect through the overlay. Based on the research team’s observations, such was not the case.

CHAPTER 4: LABORATORY TESTING – DYNAMIC MODULUS IN INDIRECT TENSION TESTING MODE

According to the American Association of State Highway and Transportation Officials (AASHTO), pavement performance is the ability of a pavement to satisfactorily serve traffic over time. In order to measure and predict pavement performance, a repeatable, well-established, and field-calibrated condition evaluating system is required. There are several ways to evaluate pavement (layer) performance, including rut measurement, dynamic modulus, and so on (Ghasemi et al. 2018a, Ghasemi et al. 2018b).

The dynamic modulus |E*| is a complex number that describes the relationship between stress and strain for a linear viscoelastic material under sinusoidal loading. It is defined as the ratio of the amplitude of the sinusoidal stress and sinusoidal strain in a steady state response (Ghasemi et al. 2016).

The dynamic modulus is a performance-related property that can be used for mixture evaluation and characterizing the stiffness of a material (SFDR cores in the present study) for use in mechanistic-empirical pavement design. The indirect tension (IDT) mode dynamic modulus test is performed according to the protocol specified by Kim et al. (2004) using 6-inch (152.4 mm) diameter, 1.5-inch (38.1 mm) thick specimens. Sinusoidal loading is applied in a controlled stress mode. Horizontal and vertical deformations are measured from two loose core–type miniature linear variable differential transformers (LVDTs) with a 50.8mm gauge length located on each side of a specimen's face (Figure 7).

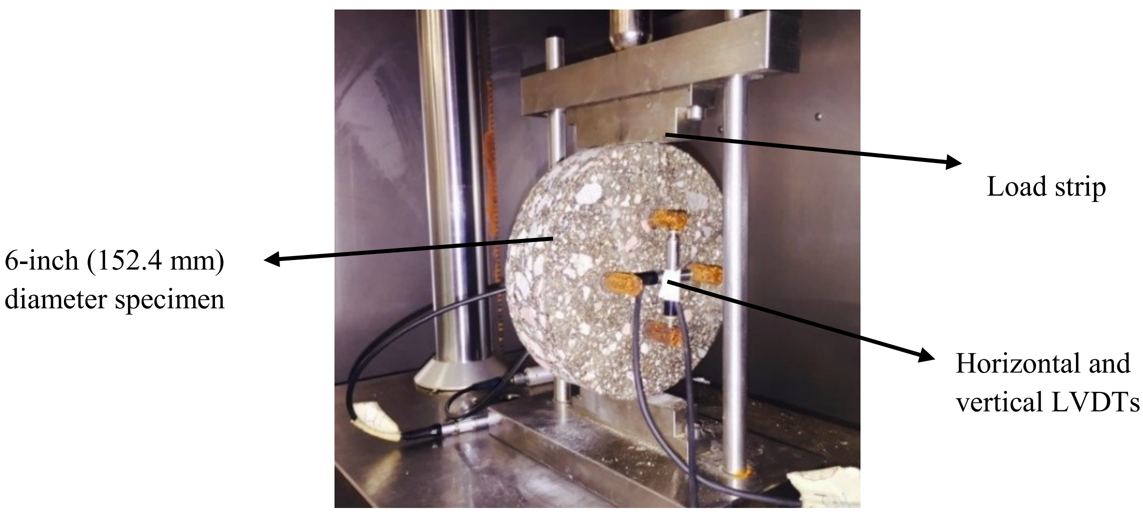


Figure 7. IDT dynamic modulus test setup

Based on the AASHTO TP 62-07 specification, testing must take place on at least two replicate specimens at five temperatures between 14°F and 130°F (-10°C and 54.4°C) and six loading rates between 0.1 and 25 Hz. In order or the measurement to remain linear viscoelastic, Kim et al. (2004) presented a linear viscoelastic solution and calculated coefficients for Poisson's ratio and dynamic modulus for different specimen diameters and gage lengths. Based on these results, the target horizontal tensile stain is 40 to 60 microstrains and the target vertical compressive strain should be under 100 microstrains (Kim et al. 2004).

One of the important parameters used in mechanistic-empirical pavement design for asphalt concrete is the dynamic modulus. This property represents the temperature and frequency-dependent or time-dependent stiffness characteristic of the pavement material. It is used in the AASHTOWare Pavement ME Design software to determine the temperature and rate-dependent behavior of an asphalt concrete layer.

In the present study, the dynamic modulus test in the IDT mode was performed at three temperatures (4.4, 21.1, and 37.8°C) each at seven frequencies (10, 5, 2, 1, 0.5, 0.2, and 0.1 Hz). Differences in vehicle speed were simulated by varying the number of cycles of load per unity time that were placed on the specimen, with high-speed vehicles simulated with high-frequency cycle loads and low-speed vehicles simulated with low-frequency cycle loads. Typically, when HMA pavement specimens are tested under this protocol, stiffness decreases as the load cycle frequency decreases. FDR/SFDR specimens were found to be generally less stiff in comparison to HMA specimens but experienced less stiffness loss with low-frequency cycle loads. Linear viscoelastic theory was used in this study, which considers the time rate of stress and strain in the asphalt concrete (Kim et al. 2004). It was assumed that the asphalt concrete was homogenous and isotropic, with the same modulus values in both tension and compression.

Collected cores were transferred to Iowa State University. Field cores were evaluated for testing. Twenty cores were considered initially for preparation and testing. Table 67 presents the specimen IDs.

Table 67. Specimen IDs

No.	Specimen	Description of Pavement and Base	Group No.
1	Blue Earth TH 20 #1	6 in. foamed asphalt	1
2	Kanabec TH 65 Sec 2-3	CSS1 3.5%+1.5% cement	2
3	Goodhue CSAH 7N #3	FDR from 2005	3
4	Chisago 347th St. #1	Emulsion stabilized	4
5	Kanabec TH 65 Sec 1-3	FDR control	5
6	Kanabec TH 65 Sec 1-2	FDR control	5
7	Chisago CSAH 20 #2	FDR	6
8	Kanabec TH 65 Sec 3-2	Engineered emulsion	7
9	Kanabec TH 65 Sec 2-2	CSS1 3.5%+1.5% cement	2
10	Wabasha CSAH 16 #2	FDR stabilized with BaseOne upper 4 in. - 2011	8
11	Douglas TH 55 Sec 3-1	Foamed asphalt	9
12	Kanabec TH 65 Sec 3-3	Engineered emulsion	7
13	Chisago CSAH 74 #2	Emulsion stabilized	4
14	Kanabec TH 65 Sec 2-1	CSS1 3.5%+1.5% cement	2
15	Douglas TH 55 Sec 3-3	Foamed asphalt	9
16	Goodhue CSAH 7N #1	FDR from 2005	3
17	Goodhue CSAH 7N #2	FDR from 2005	3
18	Chisago CSAH 81 #2	Emulsion stabilized	10
19	Kanabec Sec 3-1	Engineered emulsion	7
20	Chisago CSAH 11 #3	Emulsion stabilized	4

Among these 20 specimens, No. 10 (Wabasha CSAH 16 #2) broke during the test, and specimens Chisago 374th St.#1, Kanabec Sec 3-2, Kanabec Sec 2-2, Douglas Sec 3-1, and Douglas Sec 3-3 (Nos. 4, 8, 9, 11, and 15, respectively) were not qualified for testing because they were not sufficiently intact. Therefore, the remaining 14 specimens were prepared for testing. Table 68 lists these specimens with their descriptions.

Table 68. Prepared specimens for laboratory testing

Group No.	Specimen	County	District	Test Section	Description of Pavement and Base
1	TH 30 #1	Blue Earth	1	1	6 in. foamed asphalt
2	TH 65 Sec 2-3	Kanabec	8	2	CSS1 3.5%+1.5% cement
3	CSAH 7N #3	Goodhue	2	1	FDR from 2005
	CSAH 7N #1	Goodhue			
	CSAH 7N #2	Goodhue			
4	CSAH 74 #2	Chisago	8	1	Emulsion stabilized
	CSAH 74 #3				
5	TH 65 Sec 1-3	Kanabec	8	1	FDR control
	TH 65 Sec 1-2				
6	CSAH 20 #2	Chisago	8	1	FDR
7	TH 65 Sec 3-3	Kanabec	8	3	Engineered emulsion
	TH 65 Sec 3-1				
10	CSAH 81 #2	Chisago	8	1	Emulsion stabilized

In cases where more than one specimen was obtained for one location, the specimens were placed in groups. The groups are numbered 1 through 10, with some groups only containing one specimen; Groups 8 and 9 are missing because the specimens placed in those groups were not sufficiently intact to qualify for testing or broke before testing. Group 3 had three replicates, while groups 4, 5, and 7 each had two replicates. Groups 1, 2, 4, 6, and 10 had one replicate each.

Table 69 presents the dynamic modulus values obtained from laboratory testing.

Table 69. Dynamic modulus data

Conditions		Sample, modulus in MPa													
Temp, °C	Freq, Hz	1	2	3	5	6	7	12	13	14	16	17	18	19	20
4.4	10	9865	5314	12090	10256	10793	4920	5508	9946	4418	7369	10046	19553	9333	5554
4.4	5	9064	5148	11547	9777	10422	4806	5259	9555	4242	6856	10061	14135	8919	5404
4.4	2	8217	5005	10993	9411	10022	4549	4999	8877	4065	5943	9685	13704	8624	5149
4.4	1	7630	4826	10377	9065	9545	4298	4764	8467	3907	4945	9087	13200	8364	4874
4.4	0.5	6980	4702	9829	8682	9113	4013	4520	8160	3749	4009	8481	12792	7993	4652
4.4	0.2	6251	4537	9172	8188	8579	3762	4216	7653	3605	3619	7916	12258	7653	4293
4.4	0.1	5735	4579	8744	8001	8185	3331	3940	7215	3477	3393	7592	11934	7427	4083
21.1	10	5926	5537	7548	7458	5790	3048	3719	4615	2960	8376	6150	12554	6570	4132
21.1	5	4070	5384	6915	6961	5396	2849	3405	7058	2820	7717	5713	11786	6202	3890
21.1	2	3446	5113	6277	6459	4908	2587	3135	6431	2637	7309	5191	11221	5858	3573
21.1	1	2992	4796	5678	6095	4474	2371	2880	5273	2567	6662	4630	10637	5392	3250
21.1	0.5	2580	4557	5115	5653	4158	2194	2636	4919	2399	6015	4170	10034	5057	2960
21.1	0.2	2159	4284	4554	5164	3788	1947	2392	4502	2157	5536	3677	9395	4759	2570
21.1	0.1	1858	4048	4179	4898	3471	1850	2203	4231	2042	5164	3369	8768	4489	2282
37.8	10	2313	2837	4449	5529	5504	1914	2497	4209	2298	4726	3792	7332	4596	2825
37.8	5	2023	2698	4057	5091	5116	1824	2320	3720	2104	4297	3411	6799	4144	2471
37.8	2	1655	2494	3553	4618	4637	1613	2060	3260	1926	3974	3017	6273	3811	2155
37.8	1	1396	2305	3156	4209	3944	1362	1824	2887	1674	3528	2651	5720	3507	1824
37.8	0.5	1202	2143	2803	3764	3518	1263	1618	2534	1544	3214	2334	5177	3268	1559
37.8	0.2	1002	1944	2460	3332	3138	1101	1412	2178	1447	2826	1992	4617	3030	1286
37.8	0.1	879	1804	2280	3013	3277	1040	1308	1798	1291	2570	1766	4188	2803	1119

According to the AASHTO *Mechanistic-Empirical Pavement Design Guide* (2015), the stiffness of HMA at all levels of temperature and time rate of load is determined from a master curve constructed at a reference temperature (generally taken as 70°F). Master curves are constructed using the principle of time-temperature superposition. The data at various temperatures are shifted with respect to time until the curves merge into a single smooth function. The master curve of dynamic modulus as a function of time formed in this manner describes the time dependency of the material. The amount of shifting at each temperature required to form the master curve describes the temperature dependency of the material. The greater the shift factor, the greater the temperature dependency (temperature susceptibility) of the mixture.

The master modulus curve can be mathematically modeled by a sigmoidal function described as follows:

$$\log|E^*| = \delta + \frac{\alpha}{a + e^{\beta+\gamma(\log t_r)}}$$

where,

t_r = reduced time of loading at reference temperature
δ = minimum value of E*
δ + α = maximum value of E*
β, γ = parameters describing the shape of the sigmoidal function.

The shift factor can be shown in the following form:

$$a(T) = t/t_r$$

where,

a(T) = shift factor as a function of temperature
t = time of loading at desired temperature
t_r = reduced time of loading at reference temperature
T = temperature of interest.

Using a sigmoid function, dynamic modulus master curves are created and presented in Figures 8 to 15. Twenty-one degrees is selected as the reference temperature, and all other data points are shifted according to this reference temperature for the master curves.

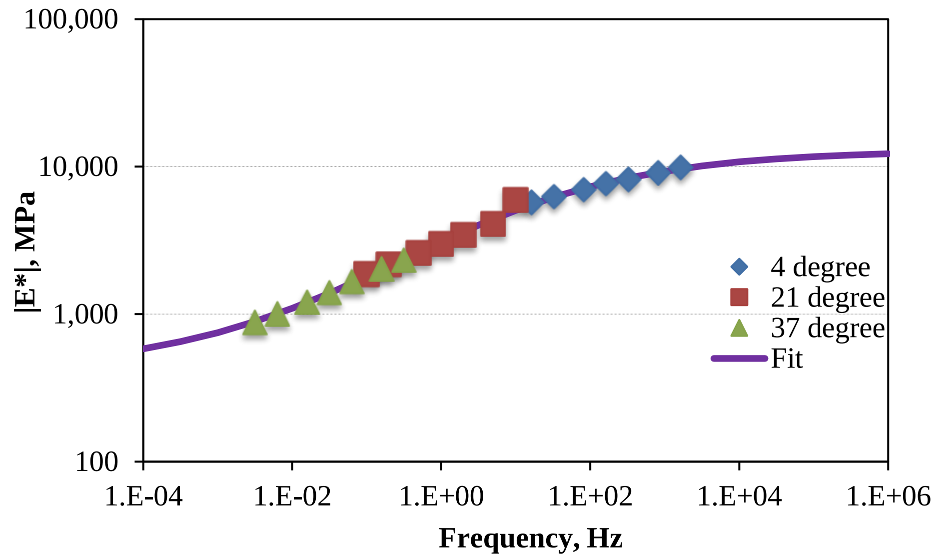


Figure 8. Dynamic modulus master curve for group 1 stabilized with foamed asphalt.

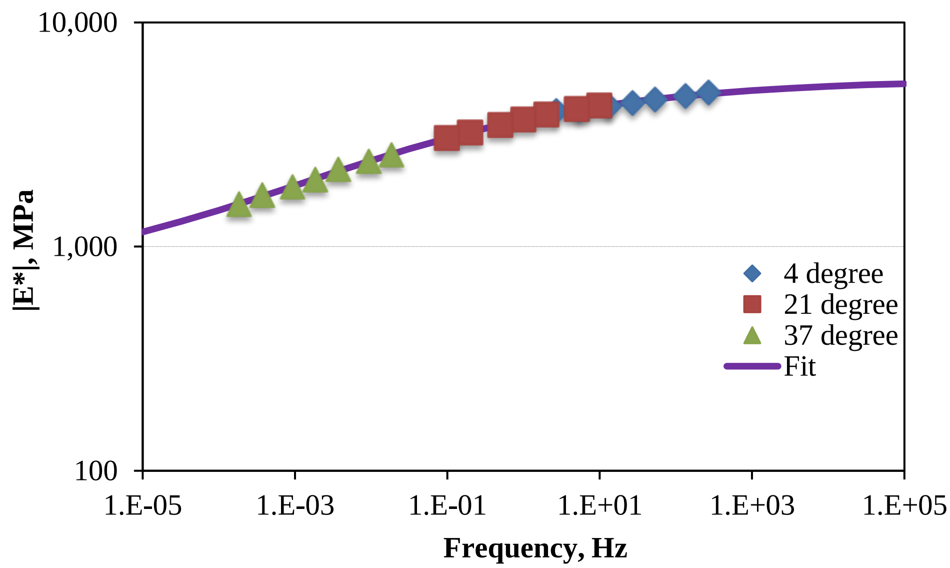


Figure 9. Dynamic modulus master curve for group 2 stabilized with CSS1 3.5%+1.5% cement.

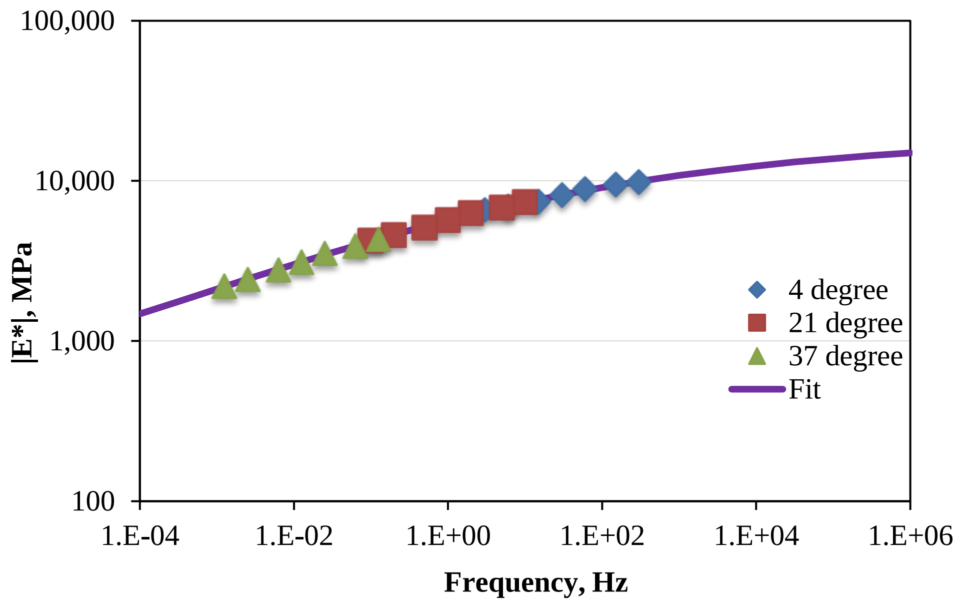


Figure 10. Dynamic modulus master curve for group 3 with FDR.

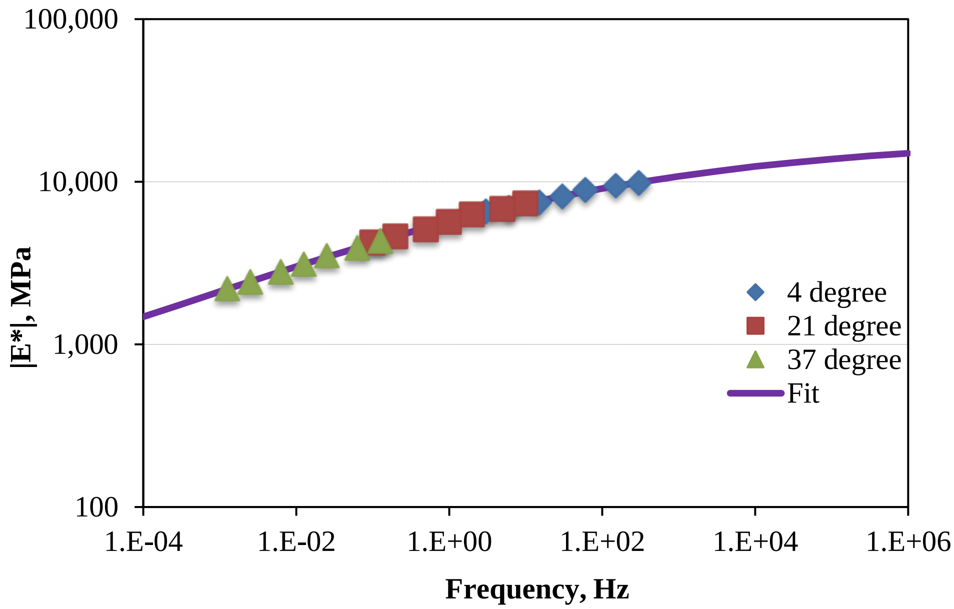


Figure 11. Dynamic modulus master curve for group 4 stabilized with emulsion.

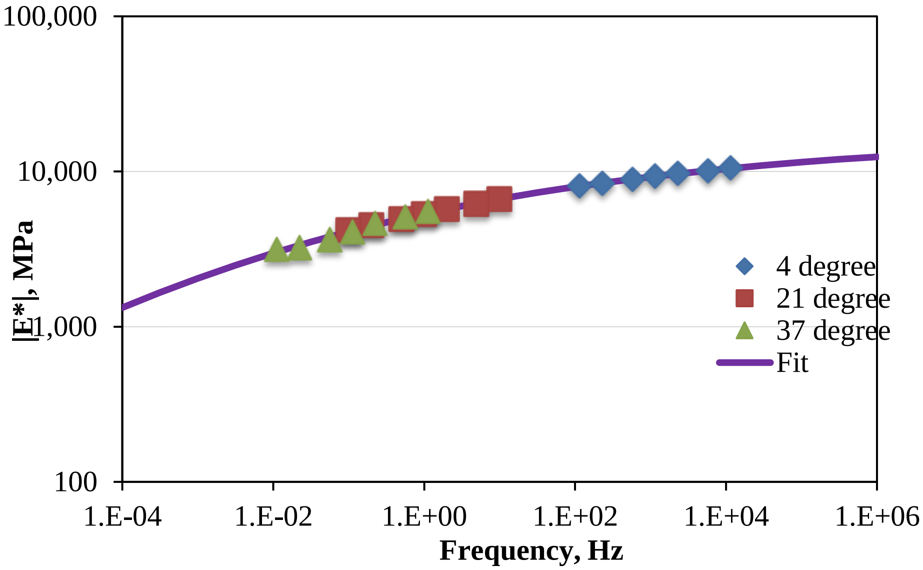


Figure 12. Dynamic modulus master curve for group 5 with FDR control.

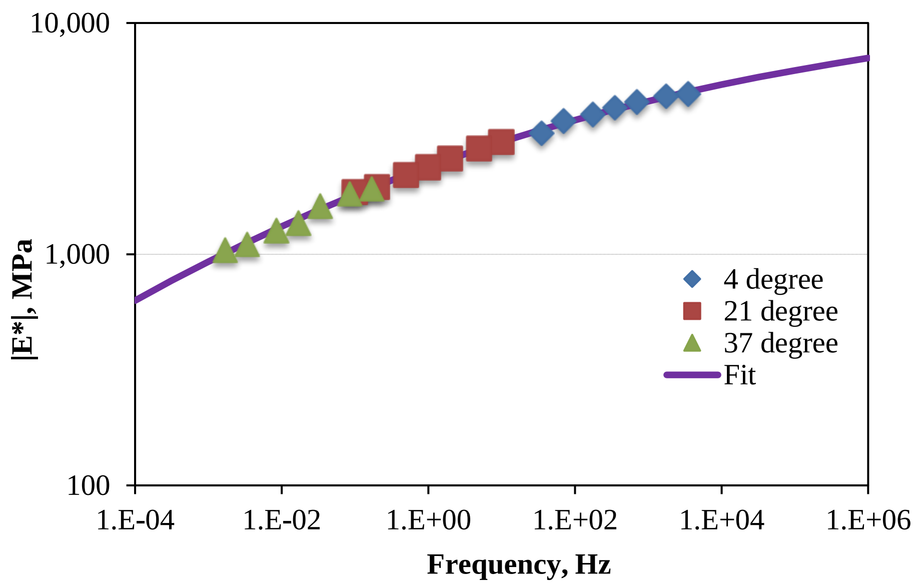


Figure 13. Dynamic modulus master curve for group 6 with FDR.

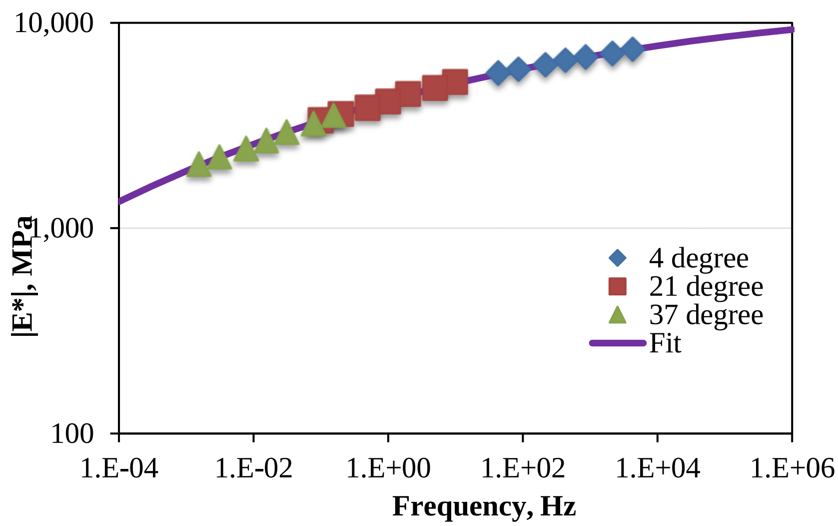


Figure 14. Dynamic modulus master curve for group 7 stabilized with EE.

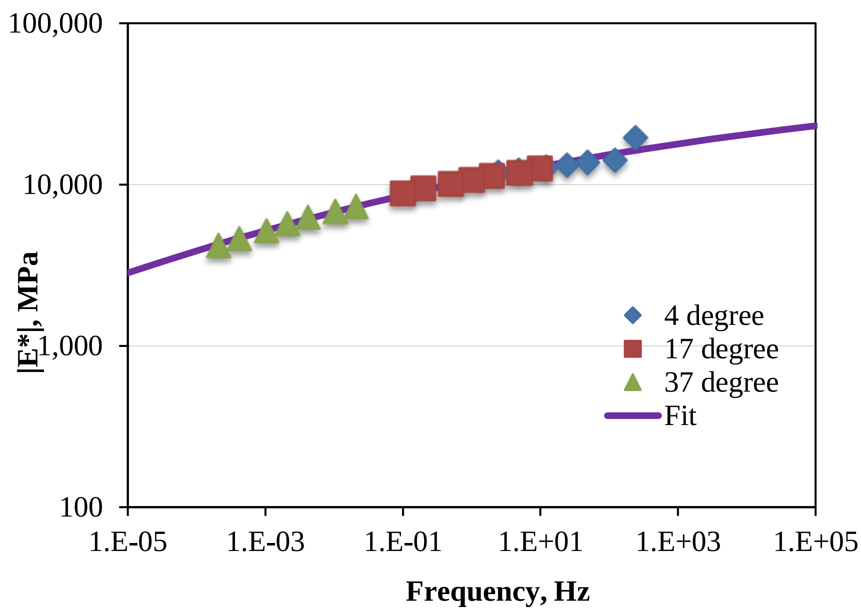


Figure 15. Dynamic modulus master curve for group 10 stabilized with emulsion.

Among the tested specimens, the specimen from Chisago County CSAH 81 (Read Ave.) #2 (group 10), which is stabilized with emulsion, had the highest dynamic modulus value and thus, by inference, the highest stiffness. Sections with FDR and foamed asphalt had the next highest dynamic modulus values. The laboratory test results are also in a good agreement with in situ testing results. Figure 16 shows a comparison among the dynamic modulus master curves of the various FDR/SFDR layers. In addition, typical dynamic modulus master curves for two HMA pavements are also presented to show the difference between FDR/SFDR material behavior and that of HMA.

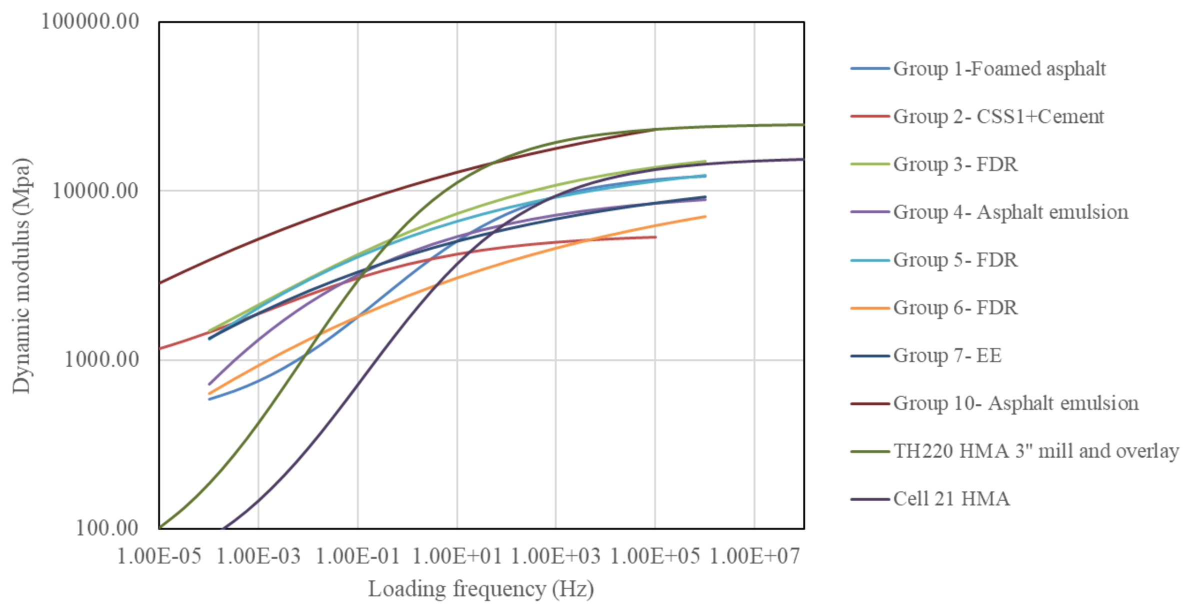


Figure 16. Comparing dynamic modulus of different treatments.

In order to be able to compare the SFDR sections' behaviors at various loading rates, the dynamic modulus master curves for two HMA sections, Cell 21 and TH 220, are also presented in Figure 16. Section TH 220 (District 2) was constructed in 2012 with a 3-inch mill and overlay construction method and was tested at Iowa State University using the same test setup as was used for this study. The data for HMA in Cell 21 of the MnRoad test track was taken from Tang et al. (2012), which used a compression setup for that dynamic modulus testing. Although the results imply that the Cell 21 HMA is less stiff than the TH 220 HMA, the shape of the master curve is similar to that of TH 220 and different from that of all of the FDR/SFDR specimens. Going forward in this report, comparisons between HMA and FDR/SFDR will be drawn between the TH 220 specimen and the specimens taken in this study, since they were both tested using the same setup in the same laboratory. The Cell 21 data are included herein only to confirm the shape of an HMA master curve compared to an FDR/SFDR master curve.

In general, it appears that the FDR/SFDR specimens retain a higher proportion of their dynamic modulus stiffness at low frequencies in comparison to the HMA specimens. In general, asphalt mixes are expected to exhibit less stiffness in response to slower traffic loads than in response to faster traffic loads. Low-frequency behavior for the dynamic modulus master curve is intended to represent material behavior for lower traffic speeds, while higher-frequency behavior is intended to represent higher traffic speeds. Therefore, it may follow that FDR/SFDR base layers may not experience as much performance degradation under low-speed traffic in comparison to HMA layers. This will be further discussed later.

At an intermediate temperature and load frequency, group 10 with asphalt emulsion exhibited the highest stiffness, followed by groups 3 and 5 (FDR), group 4 (asphalt emulsion), group 7 (engineering emulsion), group 2 (CSS1+Cement), group 1 (foamed asphalt), and group 6 (FDR). Table 70 summarizes the obtained ranking based on dynamic modulus values at 21.1°C (an intermediate temperature) and a loading frequency of 5 Hz (an intermediate frequency).

Table 70. Treatment performance ranking based on dynamic modulus value

Treatment (Number of Replicates)	Treatment Rank	Dynamic Modulus (MPa)	Dynamic Modulus (ksi)
Group 10-Emulsion (1)	1	11786	1709
Group 3-FDR (3)	2	6782	983
Group 5-FDR (2)	3	6179	896
Group 4-Emulsion (2)	4	5474	794
Group 7- EE (2)	5	4804	696
Group 2- CSS1+Cement (2)	6	4102	595
Group1 1-Foamed asphalt (1)	7	4070	590
Group 6-FDR (1)	8	2849	413

Recall that group 3 had three replicates and would be considered to provide the most reliable results. Groups 2, 4, 6, and 7 had two replicates and would be considered to have results with intermediate reliability, while groups 1, 6, and 10 have only one replicate and would be considered to have the least reliable results. A review of Table 70 shows that the groups with the lowest number of replicates are ranked the highest and lowest.

Mollenhauer et. al. (2009) developed an empirical relationship between the vehicle speed and the derived frequency. Their findings are presented in Table 71 and Figure 17.

Table 71. Vehicle speed and corresponding frequency

Speed (kph)	2.9	7.9	16.2	31.4	60	80	96.6	128.7
Speed (mph)	1.8	4.9	10.1	19.5	37.3	49.7	60	80
Frequency (Hz)	0.8	1.9	3.9	7.1	13.2	17.3	20.4	26.7

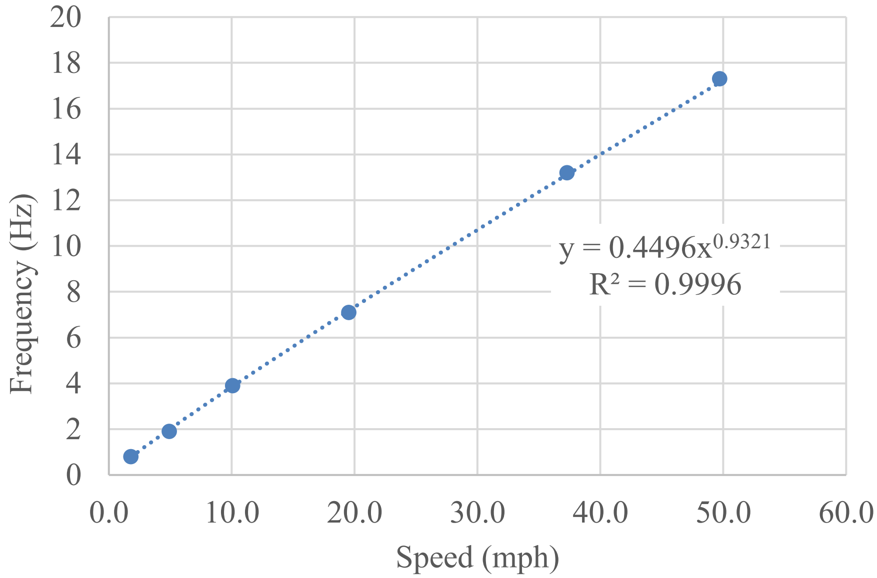


Figure 17. Frequency as a function of vehicle speed.

Using the empirical correlation between loading frequency and vehicle speed, it is possible to estimate the dynamic modulus stiffness for each group of FDR/SFDR specimens based on the master curves. Similar calculations are provided for the HMA master curves that are being used for comparison herein. The results are presented in Table 72. Columns are provided for vehicle speeds of 2, 10, 20, and 50 mph. For the purposes of the following analysis, 50 mph was taken as the baseline vehicle speed. For each of the slower vehicle speeds, the numbers in parentheses give the percent of dynamic modulus stiffness that each table cell represents in comparison to that at 50 mph.

Table 72. Comparing stabilizing agents for various vehicle speed zones

Treatment (number of replicates)	2 mph		10 mph		20 mph		50 mph	
	Predicted modulus – Mpa (% of 50 mph)	Treatment rank	Predicted modulus – Mpa (% of 50 mph)	Treatment rank	Predicted modulus – Mpa (% of 50 mph)	Treatment rank	Predicted modulus – MPa	Treatment rank
Group 10-Emulsion (1)	10,479 (78)	1	11,926 (89)	1	12,574 (93)	1	13,452	1
TH 220 HMA – 3-in. mill and overlay, for comparison only (not applicable)	6,158 (50)	2	9,128 (74)	2	10,494 (85)	2	12,299	2
Group 3-FDR (3)	5,558 (72)	3	6,621 (85)	3	7,099 (92)	3	7,744	3
Group 5-FDR (2)	5,244 (75)	4	6,093 (88)	4	6,464 (93)	4	6,955	4
Group 4-Emulsion (2)	4,221 (75)	5	4,937 (88)	5	5,238 (93)	5	5,624	5
Group 7- EE (2)	4,118 (78)	6	4,694 (89)	6	4,946 (94)	6	5,282	6
Group 2- CSS1+Cement (2)	3,637 (84)	7	4,012 (92)	7	4,160 (92)	8	4,341	8
Group1 1-Foamed asphalt (1)	2,993 (54)	8	4,179 (75)	8	4,765 (85)	7	5,585	7
Group 6-FDR (1)	2,347 (73)	9	2,770 (86)	9	2,964 (92)	10	3,228	10
Cell 21 HMA - for comparison with TH 220 HMA only (not applicable)	1,628 (38)	10	2,747 (64)	10	3,363 (78)	9	4,288	9
Weighted average (by replicates) of all FDR/SFDR sections	4,781 (75)	(n/a)	5,586 (87)	(n/a)	5994 (93)	(n/a)	6,422	(n/a)
Ratio of FDR/SFDR % to TH 220 %	1.49		1.17		1.09		(n/a)	

All dynamic modulus stiffness values for all of the materials in Table 72 decrease as the vehicle speed decreases. However, a greater reduction is calculated for the HMA materials in comparison to the FDR/SFDR materials: the TH 220 specimens saw a 50% reduction from 50 mph to 2 mph while the Cell 21 specimens saw a 75% reduction. As mentioned previously, since the TH 220 HMA specimens were tested in the same laboratory with the same setup as the FDR/SFDR specimens, the subsequent analysis will use the TH 220 HMA results for comparison with the FDR/SFDR results. For all speeds, the TH 220 HMA specimens were calculated to be stiffer in comparison to all FDR/SFDR specimens, except for group 10; note that group 10 is considered to have low reliability because there is only one replicate in this group. The results for the rest of the FDR/SFDR specimens are ranked according to calculated stiffness. The groups that are ranked 1 through 6 hold their same ranking regardless of vehicle speed, while there is some change in the 7th, 8th, and 9th ranked groups. At the bottom of the chart, a weighted average of stiffness and percent of stiffness at 50 mph is provided. The weighting is calculated by the number of replicates for each group. This weighted average implies that the calculated dynamic modulus stiffness for the FDR/SFDR specimens is 93%, 87%, and 75% for vehicle speeds of 20, 10, and 2 mph, respectively. By contrast, the percentage of retained stiffness for the TH 220 HMA is 85%, 74%, and 50% for the same vehicle speeds. The ratio of FDR/SFDR percentage to TH 220 percentage in the bottom row of Table 25 indicates that FDR/SFDR retains a greater amount of dynamic modulus stiffness as vehicle speed decreases in comparison to the TH 220 HMA specimens by 1.09, 1.17, and 1.48 for 20, 10, and 2 mph, respectively.

Although these calculations imply that the weighted average of the dynamic modulus stiffness for the FDR/SFDR specimens is never higher than that of the TH 220 HMA, it does imply that the FDR/SFDR retains more stiffness as vehicle speed decreases in each case. This suggests that an FDR/SFDR base will likely retain stiffness performance better in comparison to HMA for heavy, slow-moving vehicles, such as the agricultural vehicles that are common on lower-volume roads. If an FDR/SFDR base has an HMA overlay, the HMA overlay will have the same performance challenges for slow-moving vehicles, whether or not the overlay is placed over an FDR/SFDR base. However if the HMA layer is compromised by slow-moving heavy vehicles while the FDR/SFDR base is not, it may be possible to provide repairs by filling ruts or milling and filling at a relatively low cost in comparison to replacing an entire pavement section.

CHAPTER 5: DISCUSSION, CONCLUSIONS, AND RECOMMENDATIONS

5.1 1993 AASHTO PAVEMENT DESIGN PROCEDURE FOR FLEXIBLE PAVEMENTS

The AASHTO pavement design procedure (AASHTO 1993) is based on the structural number (SN), which is an index of pavement strength. SN depends on layer thickness and base material properties and is commonly used in pavement design practices. It expresses the capacity of pavements to carry loads for a given combination of soil support, estimated traffic, terminal serviceability, and environment.

The AASHTO design method incorporates several design variables such as traffic loading, environmental effects, serviceability, pavement layer thickness, and pavement layer materials. In addition, it also incorporates a level of uncertainty in the process to ensure that the design will last; the level of reliability must increase as the traffic volume increases. The AASHTO *Guide for Design of Pavement Structures* (AASHTO 1993) first requires the desired terminal serviceability to be determined. The serviceability is expressed as an index from 4.2 to 0, where 4.2 is a newly constructed flexible pavement and 2.0 is a pavement in need of rehabilitation. Next, the known traffic volumes must be converted to the number of 18 kip ESALs. Then the SN can be determined by using design charts or a computer program. After the SN is known, the layer coefficients are evaluated and the required layer thicknesses are computed.

To evaluate the structural effects of the different stabilization treatments used for an SFDR layer, the 1993 AASHTO pavement design procedure for flexible pavements is selected. The design procedure recommended by AASHTO is based on the results of the extensive AASHO Road Test conducted in the late 1950s and early 1960s. Empirical equations are used to relate observed or measurable pavement characteristics with pavement performance (outcomes). This empirical equation is widely used and has the following form as the final design equation for flexible pavement:

$$\log(W_{18}) = Z_R S_0 + 9.36\log(SN+1) - 0.20 + \frac{\log\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32\log M_R - 8.07$$

where W_{18} = number of 18 kip (80 kN) single axle load applications; Z_R = normal deviation for a given reliability R; S_0 = standard deviation; SN = structural number of the pavement ($SN = a_1D_1 + a_2D_2 + a_3D_3$), in which a_1, a_2, and a_3 are layer coefficients for the surface, base, and subbase, respectively, and D_1, D_2, and D_3 are the thicknesses of the surface, base, and subbase, respectively; ΔPSI = change in serviceability; and M_R = effective roadbed soil resilient modulus.

5.2 MNDOT GE METHOD

The granular equivalent factor is an indicator of the structural capacity of road materials used in the design of pavement structures in Minnesota. The GE factor represents the relative strength of a material compared to Class 5 and Class 6 base aggregate. Two road structure design methods are currently implemented by local agencies in Minnesota: the soil factor and R-value methods. The soil factor design method was adopted by MnDOT during the 1950s. Both methods allow users to determine a required GE value based on the design traffic and subgrade soil strength. The thickness of each layer can be calculated as follows:

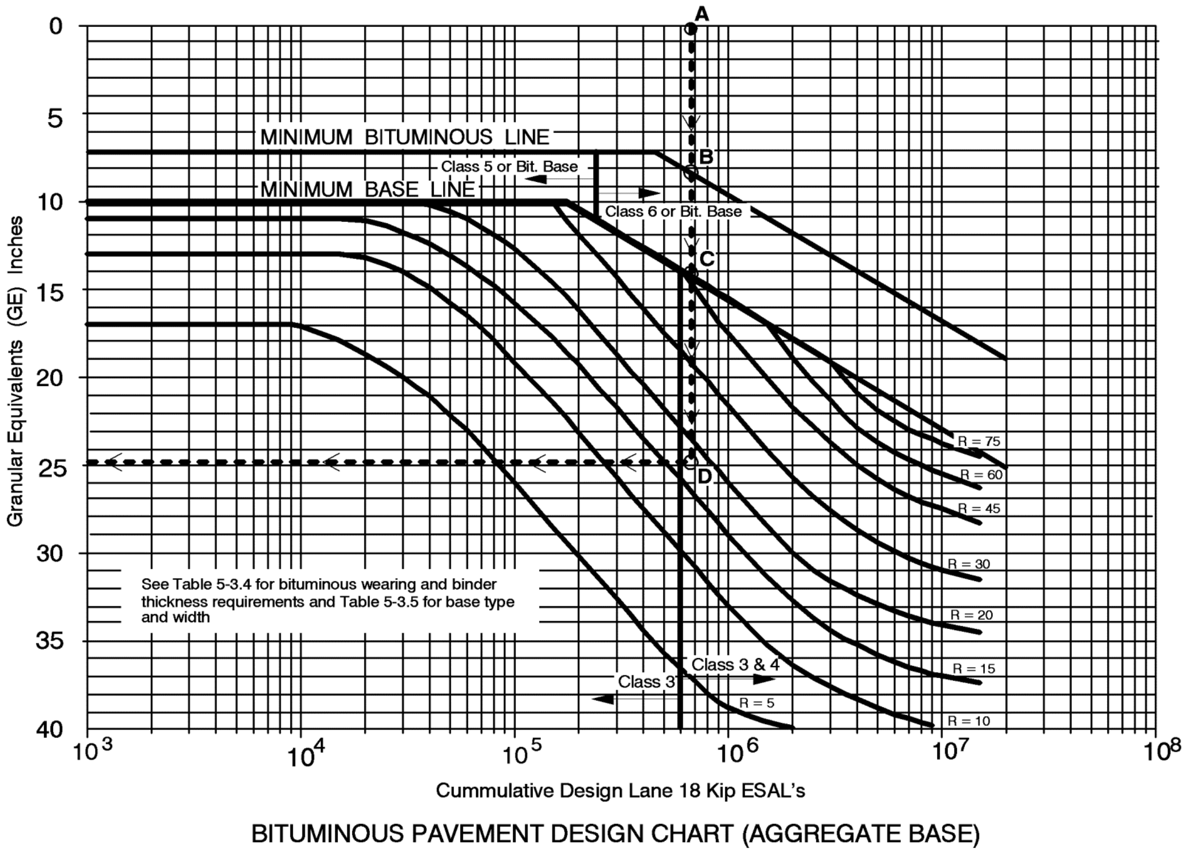
$$GE = a_1 D_1 + a_2 D_2 + a_3 D_3$$

where GE is the total granular thickness determined from Figure 18, D_1 is the thickness of the bituminous material (mm (in.)), D_2 is the thickness of the aggregate base material (mm (in.)), D_3 is the thickness of the aggregate subbase (mm (in.)), and a_1, a_2, a_3 are the GE factors shown in Table 73.

In many cases, the R-value is estimated from the subgrade soil resilient modulus (M_R) or CBR. Christopher et al. (2006) recommend the following equation to estimate the M_R value using the R-value.

$$M_R(psi) = 1000 + 555 \times (R - value)$$

Using Figure 18 and the available historical documents, layer coefficients for the FDR and SFDR layers are back-calculated and presented in Table 73. It should be noted that the GE factors for the HMA layer and aggregate base in Table 73 are taken from documents related to MnDOT's flexible pavement design guidance. The subgrade soil classes shown in Figure 19 and the subgrade soil properties (R-values) in Table 74 are taken from MnPAVE software and MnDOT's flexible pavement design guide, respectively. Based on the amount of traffic and the subgrade soil properties (R-values) and using Figure 19, the required amount of GE for the pavement section is obtained (Table 75). After subtracting the GE provided by the HMA layer and aggregate base layer, the remaining GE is provided by the FDR/SFDR layer.



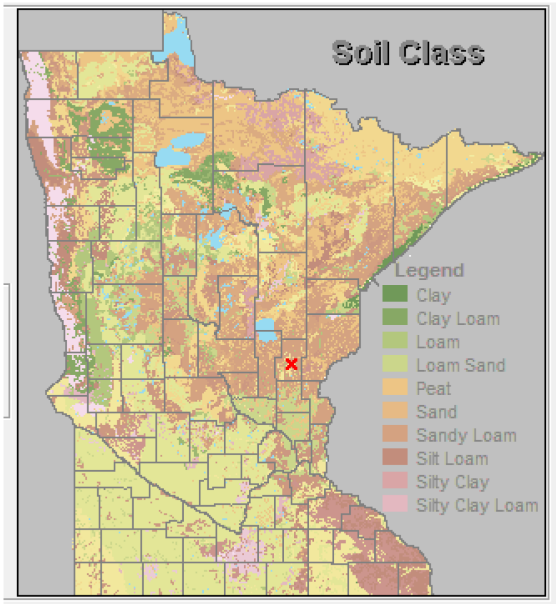
http://www.dot.state.mn.us/stateaid/projectdelivery/pdp/tools/r-value.pdf

Figure 18. Bituminous pavement design chart

Table 73. GE factors

Type of Material	Specification	GE Factors
Bituminous Pavement	2360	2.25
Cold In-Place Recycling (CIR)	2331	1.50
Pavement Breaking/Rubblized Concrete Pavement	2231	1.50
Bituminous Pavement Reclamation (FDR)	2231	1.00
Aggregate Base Class 5 and 6	3138	1.00
Aggregate Base Class 3 and 4	3138	0.75
Selected Granular Material	3149.2B2	0.50

Source: http://www.dot.state.mn.us/stateaid/projectdelivery/pdp/tools/9-ton-design.pdf



MnDOT 2018b

Figure 19. Minnesota soil map from MnPAVE software

Table 74. Soil classifications, MnDOT soil factor, and stabilometer R-values by soil type

MnDOT Textural Classification	AASHTO Classification	ASTM Unified Classification	MnDOT Soil Factor	R-value (assumed)
Gravel	A-1	GP-GM	50	75
Sand	A-1, A-3	SP-SM	75	70
Loamy Sand	A-2	SM, SC	75	70
Sandy Loam Slightly Plastic (<10% clay)	A-2	SM, SC	75	30
Sandy Loom Plastic (10–20% clay)	A-4	SM, SC	100	20
Loam	A-4	ML, MH	100	20
Silt Loam	A-4	ML, MH	100	20
Sandy Clay Loam	A-6	SC, SM	100	20
Clay Loam	A-6	CL	100	12
Silty Clay Loam	A-6	ML/CL	120	12
Sandy Clay	A-7	SC	120	12
Silty Clay	A-7	ML/CL	120	12
Clay	A-7	CL, CH	130	10

Notes:

Soil factors from state.

R-values from Table 5-3.3(a) of MnDOT Pavement Manual, July 2007, page 5-3.0(26).

R-values based on data collected by MnDOT through 1974.

Source: MnDOT 2017

Table 75. Back-calculated GE values for SFDR layers

Section	Treatment	HMA thickness (in)	FDR/SFDR thickness (in)	Aggregate base thickness (in)	R-value	Design ESALs	Required GE	Back-calculated SFDR GE
Kanabec TH 65 section #1	FDR	4	6	13	20	1360000	27.96	1.16
Kanabec TH 65 section #2	CSS1 3.5%+1.5% Cement	3.5	6	13	20	1360000	27.96	1.18
Kanabec TH 65 Section #3	EE	2.75	6	13	20	1360000	27.96	1.46
Wabasha CSAH 16	BaseOne	3.5	4 in. BaseOne+4 in. FDR	8	12	252532	22.88	1.25
Wabasha CSAH 9	FDR	3.5	4	13	12	228786	22.28	1.16
Blue Earth TH 30	Foamed asphalt	5.75	6	6	12	607000	28.22	1.55
Douglas TH 55 Section #4	BaseOne	5	5	9	15	452296	24.31	1.26

5.3 CONCLUSIONS AND RECOMMENDATIONS

During this study, information on 24 pavements was collected for multiple sections of each pavement from eight counties in Minnesota. In three of the cases, information from primary highways was collected, reflecting funding contributions from MnDOT. For the remaining cases, information from secondary highways was collected, reflecting funding contributions from the Minnesota LRRB. In situ tests including coring, DCP, and pavement distress identification were conducted on the test sections, with the results presented in this Task 3 report. Field cores were transferred to Iowa State University for further laboratory performance tests as part of the ensuing task.

Based on the resulting DCP data, it appears that emulsions, emulsion/cement hybrids, and BaseOne showed the highest in situ stiffness. Fly ash/cement showed the highest DCP index (mm/blow), which means it had the lowest in situ stiffness.

The visual distress survey results indicated that the roads are generally performing well. Low- to medium-intensity longitudinal cracks are the most common distress observed, with a minimum spacing of 28 feet on one of the oldest roads. One case of high-severity transverse cracks at a 50 ft spacing was observed on a primary highway; occasional longitudinal cracks were observed on various roads. In some cases, the roads had been recently covered with thin maintenance surface treatments (such as seal coats or slurry seals) or overlays. In such cases, no distresses were observed. It seems likely that these roads are not experiencing much distress, because higher severity distress would reflect through such thin treatments. Overall, it appears that this inventory of FDR/SFDR roads are performing well.

Dynamic modulus testing in the indirect tension mode was performed on the field cores at three temperatures (4.4, 21.1, and 37.8°C) each at seven frequencies (10, 5, 2, 1, 0.5, 0.2, and 0.1 Hz). Dynamic modulus master curves were created and used to evaluate and compare the performance of test specimens taken from the reclaimed layers, considering loads that would be imposed by vehicles traveling at various speeds. The analysis of the laboratory test results indicated that the dynamic modulus stiffness values of the FDR/SFDR specimens were generally less than that for HMA. However, for situations that are consistent with loads imposed by heavy, slow-moving vehicles, it was inferred that the FDR/SFDR base layers would retain performance to a greater degree than HMA. In cases where the critical design vehicle loads are slow-moving vehicles, it would be possible to expect a greater amount of support from the FDR/SFDR layers than is typical under faster-moving vehicles. However, if an HMA overlay is placed over the FDR/SFDR base layer, it could suffer performance problems under the slow-moving loads; however, if these problems were to occur only close to the surface, they would be relatively easy to correct.

Minnesota GE analysis was performed to back-calculate the granular equivalent factor for SFDR layers having the design ESALs and R-values for subgrade soils. GE values were determined for foamed asphalt stabilized FDR, engineering emulsion stabilized FDR, BaseOne stabilized FDR, and CSS1 3.5%+1.5% cement-stabilized layer. Foamed asphalt and EE provided the highest GE values of 1.55 and 1.46, respectively. BaseOne, CSS1+cement, and control FDR layers provided GE values of 1.1 to 1.2. The back-calculated GE values were between 1.16 and 1.53, indicating that designers have likely been using GE values for the FDR or SFDR layers consistent with current recommendations.

Based on the foregoing, it is recommended that the current GE values be generally retained for FDR/SFDR design. However, for cases where slower-moving vehicles are the critical design consideration, increasing the GE values after referring to Table 25 could be warranted. Note that such slow-moving vehicles could compromise the performance of any HMA overlays that are placed on top of the FDR/SFDR base; however, if the resulting performance issue is near the surface, the issue might be corrected with modest expense through maintenance or rehabilitation activities. In addition to providing support for overlaying pavement layers, FDR and SFDR bases are well known for destroying crack patterns and other road defects that are often reflected through traditional HMA overlays. The visual distress surveys indicate that the FDR/SFDR bases under study in this report have been performing well in this regard; therefore, decision makers may want to consider the use of FDR/SFDR as a base for reasons other than structural capacity.

REFERENCES

AASHTO. 1993. *Guide for Design of Pavement Structures*. Fourth Ed. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2015. *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice*. Second Edition. American Association of State Highway and Transportation Officials, Washington, DC.

Asphalt Institute. 1983. *Asphalt Cold-Mix Recycling.* MS-21. Asphalt Institute, College Park, MD.

Caltrans. 2013. *Full-Depth Reclamation Using Engineered Emulsion*. California Department of Transportation, Sacramento, CA. http://www.dot.ca.gov/hq/maint/Pavement/Offices/Pavement_Engineering/PDF/FDR_E_Design_Guide_August_2013.pdf.

Caltrans. 2012. *Full-Depth Reclamation Using Foamed Asphalt*. California Department of Transportation, Sacramento, CA. http://www.dot.ca.gov/hq/maint/Pavement/Offices/Pavement_Engineering/PDF/FDR-FA_Design_Guide_06-05-12_final.pdf.

Chevron USA. 1982. *Recycling Manual.* Chevron, San Ramon, CA.

Cross, S. A. and B. M. Ramaya. 1995. *Evaluation of Cold In-Place Recycling*. Final Report of Project KU-93-1. University of Kansas, Lawrence, KS.

Diefenderfer, B. K. and A. K. Apeagyei. 2011a. *Analysis of Full-Depth Reclamation Trial Sections in Virginia*. Virginia Center for Transportation Innovation and Research, Charlottesville, VA. http://www.virginiadot.org/vtrc/main/online_reports/pdf/11-r23.pdf.

Diefenderfer, B. K., and A. K. Apeagyei. 2011b. Time-Dependent Structural Response of Full-Depth Reclamation. *Transportation Research Record: Journal of the Transportation Research Board,* No. 2253, pp. 3–9.

Elkins, G. E., P. N. Schmalzer, T. Thompson, and A. Simpson. 2003. *Long-Term Pavement Performance Information Management System: Pavement Performance Database User Guide*. FHWA-RD-03-088. Federal Highway Administration, McLean, VA.

Federal Lands Highway Division of the Federal Highway Administration. 2005. *Roadway Surfacing Options Photo Album: Companion Document to Context Sensitive Roadway Surfacing Selection Guide*. Central Federal Lands Highway Division, Lakewood, CO.

Ghasemi, P., J. Podolsky, R. C. Williams, and E. Dave. 2016. Performance Evaluation of Coarse-Graded Field Mixtures Using Dynamic Modulus Results Gained from Testing in the Indirect Tension Mode. Paper presented at International Conference on Transportation and Development, June 26–29, Houston, TX.

Ghasemi, P., M. Aslani, D. K. Rollins, and R. C. Williams. 2018a. Principal Component Analysis-Based Predictive Modeling and Optimization of Permanent Deformation in Asphalt Pavement: Elimination of Correlated Inputs and Extrapolation in Modeling. *Structural and Multidisciplinary Optimization*, pp. 1–19.

Ghasemi, P., M. Aslani, D. K. Rollins, R. C. Williams, and V. R. Schaefer. 2018b. Modeling Rutting Susceptibility of Asphalt Pavement Using Principal Component Pseudo Inputs in Regression and Neural Networks. *International Journal of Pavement Research and Technology*.

IDOT. 2012. *Special Provision for Full-Depth Reclamation (FDR) with Emulsified Asphalt.* Illinois Department of Transportation, Springfield, IL. http://www.idot.illinois.gov/Assets/uploads/files/Doing-Business/Manuals-Guides-&-Handbooks/Highways/Local-Roads-and-Streets/Special-Provisions/LR400-4.pdf.

Iowa DOT. 2012. *Standard Specifications for Highway and Bridge Construction.* Iowa Department of Transportation, Ames, IA. https://www.iowadot.gov/specifications/Specificationsseries2012.pdf.

Kandhal, P. S. and R. B. Mallick. 1997. *Pavement Recycling Guidelines for State and Local Governments Participant's Reference Book*. FHWA-SA-98-042. Federal Highway Administration, Washington, DC. https://www.fhwa.dot.gov/pavement/recycling/98042/.

Kim, Y. R., Y. Seo, M. King, and M. Momen. 2004. Dynamic mosdulus testing of asphalt concrete in indirect tension mode. *Transportation Research Record: Journal of the Transportation Research Board,* No.1891, pp. 163–173.

Lewis, D. E., D. M. Jared, H. Torres, and M. Mathews. 2006. Georgia's Use of Cement-Stabilized Reclaimed Base in Full-Depth Reclamation. *Transportation Research Record: Journal of the Transportation Research Board,* No. 1952, pp. 125–133.

Maher, M., C. Marshall, F. Harrison, and K. Baumgaertner. 2005. *Context Sensitive Roadway Surfacing Selection Guide*. Federal Highway Administration, Central Federal Lands Highway Division, Lakewood, CO.

Marquis, B., D. Peabody, R. Mallick, and T. Soucie. 2003. *Determination of Structural Layer Coefficient for Roadway Recycling Using Foamed Asphalt*. Recycled Materials Resource Center, University of New Hampshire, Durham, NH. http://rmrc.wisc.edu/wp-content/uploads/2012/10/p26final.pdf.

MnDOT. 2018a. *Standard Specifications for Construction.* Minnesota Department of Transportation, St. Paul, MN.

MnDOT. 2018b. *MnPAVE Flexible.* Minnesota Department of Transportation, St. Paul, MN. http://www.dot.state.mn.us/app/mnpave/.

MnDOT. 2017. *Flexible Pavement Design Guidance*. Minnesota Department of Transportation, St. Paul, MN. http://www.dot.state.mn.us/stateaid/projectdelivery/pdp/tools/flexible-pavement-design-guidance.pdf.

Mollenhauer, K., M. Wistuba, and R. Rabe. 2009. Loading frequency and fatigue: In situ conditions and impact on test results. Paper presented at the 2nd European Workshop on Four Point Bending, September 24–25, University of Minho, Guimarães, Portugal.

Morian, D. A., M. Solaimanian, B. Scheetz, and S. Jahangirnewjad. 2012. *Developing Standards and Specifications for Full-Depth Pavement Reclamation.* Pennsylvania Department of Transportation, Harrisburg, PA.

Nantung, T., Y. Ji, and T. Shields. 2011. Pavement Structural Evaluation and Design of Full-Depth Reclamation (FDR) Pavement. Paper presented at the Transportation Research Board 90th Annual Meeting, January 23–27, Washington, DC.

NYSDOT. 2015. *Geotechnical Engineering Manual: Design and Construction Guidelines for Full-Depth Reclamation of Asphalt Pavement.* State of New York Department of Transportation, Albany, NY. https://www.dot.ny.gov/divisions/engineering/technical-services/technical-services-repository/GEM-27b.pdf.

Peters-Davis, K. and D. H. Timm. 2009. *Recalibration of the Asphalt Layer Coefficient*. NCAT Report 09-03. National Center for Asphalt Technology, Auburn University, Auburn, AL. http://www.eng.auburn.edu/files/centers/ncat/reports/2009/rep09-03.pdf.

PCA. 2005. *Full-Depth Reclamation: Recycling Roads Saves Money and Natural Resources.* Portland Cement Association, Skokie, IL. http://cncement.org/wp-content/uploads/2014/04/R_FDR_SR995.pdf.

Romanoschi, S. A., M. Hossain, A. Gisi, and M. Heitzman. 2004. Accelerated Pavement Testing Evaluation of the Structural Contribution of Full-Depth Reclamation Material Stabilized with Foamed Asphalt. *Transportation Research Record: Journal of the Transportation Board,* No. 1896, pp. 199–207.

Salomon, A. and D. E. Newcomb. 2000. *Cold In-Place Recycling Literature Review and Preliminary Mixture Design Procedure*. Minnesota Department of Transportation, St. Paul, MN.

SCDOT. 2012. *Standard Method of Test for Full-Depth Reclamation Using Asphalt Emulsion - Job Mix Formula Preparation*. South Carolina Department of Transportation, Columbia, SC.

Tang, S., Y. Cao, and J. F. Labuz. 2012. *Structural Evaluation of Asphalt Pavements with Full-Depth Reclaimed Base*. Minnesota Department of Transportation, St. Paul, MN.

VDOT. 2003. *Guidelines for 1993 AASHTO Pavement Design.* Materials Division, Pavement Design and Evaluation Section, Virginia Department of Transportation, Richmond, VA. http://www.virginiadot.org/business/resources/materials/bu-mat-moi-vi.pdf.

Wu, Z., J. L. Groeger, A. L. Simpson, and R. G. Hicks. 2010. *Performance Evaluation of Various Rehabilitation and Preservation Treatments*. FHWA-HIF-10-020. Federal Highway Administration, Washington DC.

APPENDIX A
ONLINE INFORMATION SURVEY QUESTIONS

Q1: What types of full-depth reclamation (FDR) experience has your county had? (check all that apply)		
1. No FDR experience (Thanks for your participation; you may have completed the survey.) 2. Non-stabilized FDR (No stabilization additives incorporated.) 3. Stabilized FDR (SFDR) (Stabilization additives such as emulsion, fly ash, or chemicals incorporated.)		
Q2: If your county has used SFDR before, what stabilization agents did you use? (check all that apply)		
1. Asphalt emulsion	2. Fly ash	3. Foamed asphalt
4. Lime	5. Cement	6. Others
Please provide more details if you have selected "Others."		
Q3: Please describe the SFDR mix design method your county has used.		
1. Marshall	2. Superpave gyratory compactor (SGC)	
3. Hveem	4. Others (Outside party such contractor, consultant, or testing lab)	
Please provide more details if you have selected "Others."		
Q4: Frequently encountered construction problem.		
1. None 2. Curing/low strength and load-induced failure shortly after construction 3. Failure to meet density and/or consistency specification 4. Insufficient subgrade and support for construction equipment 5. Others		
Please briefly explain the causes, consequences, and correction measures of the construction problems.		
Q5: Do you have any historical documentation regarding your FDR projects? (check all that apply)		
1. Mix design and material information	2. Performance records (IRI, PQI, etc.)	
3. Construction costs records	4. Maintenance records and maintenance costs	
5. QC/QA records	6. FWD or other strength testing	
Q6: For the projects that have any historical records as mentioned in the previous question, when were the projects constructed? (check all that apply)		
1. < 2 years	2. 2 – 5 years	
3. 5 – 10 years	4. > 10 years	
Q7: May we contact you by phone for further details?		
1. Yes	2. No	

APPENDIX B
FDR PROJECT INVENTORY

Location	Road	Stabilization Method	Year of Construction
Otter Tail	CSAH 65	None	2015
City of Duluth	Glenwood St	None	2010
City of Duluth	Arrowhead Rd	None	2011
City of Duluth	Swan Lake Rd	None	2011
City of Duluth	PECAN AVE	None	2015
City of Duluth	Carver Ave.	None	2013
City of Duluth	Skyline Pkwy and 7th St	None	2013
Hennepin	South Diamond Lake Rd	Cement	2012
Hennepin	South Diamond Lake Rd	Cement	2012
McLeod	CSAH 3	BaseOne	2012
McLeod	CSAH 33	BaseOne	2012
Watonwan	CSAH 3	None	2004
Goodhue	CSAH 7 South	None	1998
Goodhue	CSAH 7 North	None	2005
Goodhue	CSAH 11	None	2012
District 3	TH 65	None	2010
District 3	TH 65	3.5% emulsion; 1.5% cement	2010
District 3	TH 65	3.5% emulsion; 1.5% cement	2010
District 4	TH 55	CSS1 + cement	2009
District 4	TH 55	emulsion	2009
District 4	TH 55	foam	2009
District 4	TH 55	BaseOne	2009
District 4	TH 55	None	2009
District 6	TH 16	emulsion	2009
District 6	TH 248	foam	2009
District 7	TH 30	foam	2010
District 7	TH 30	none	2010
District 1	TH 70	emulsion + cement	2010
District 1	TH 70	geogrid	2010
District 1	TH 70	none	2010
District 1	TH 70	non-woven textile	2010
District 2	TH 72	emulsion	2011
District 2	TH 72	emulsion	2011
District 2	TH 72	mill + fill	2011
District 6	TH 76	none	2008
District 6	TH 109	none	2010
Pope	CSAH 1	none	2011
Pope	CSAH 8	none	2003
Pope	CSAH 11	none	2011
Pope	CSAH 16	none	2012
Pope	CSAH 18	none	2010
Pope	CSAH 22	none	2005
Pope	CSAH 22	none	2009
Pope	CSAH 27	none	2011
Pope	CSAH 28	none	2007
Pope	CSAH 28	none	2011
Pope	CSAH 28	emulsion	2015
Pope	CSAH 29	none	2004
Pope	CSAH 29	none	2011
Pope	CSAH 30	none	2013