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RESIDUAL STRENGTH OF FULL-DEPTH RECLAMATION

APPLIED RESEARCH & INNOVATION BRANCH

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16. Abstract

Cold recycling technologies such as Full-Depth Reclamation (FDR) are sustainable and cost-effective techniques for pavement rehabilitation. Previous research has found current mechanistic-empirical (M-E) default values to be non representative and overly conservative, leading to an underestimation of the true performance capabilities of FDR materials. To address this gap, this research analyzes the performance of 11 FDR sites constructed throughout Colorado and compares their long-term performance with M-E predictions. The objective of this report is to recommend input values for the M-E design of FDR base materials that result in reliable predictions of FDR long-term performance. Both initial values of the International Roughness Index (IRI) and resilient modulus were found to have a significant impact on M-E predictions and were calibrated in a two-step process. The proposed input parameters lead to a conservative design of FDR projects and result in improved IRI predictions than the ones derived from current design criteria. With the current design parameters, IRI predictions were on average overestimated by 51 in/mile, whereas the proposed input parameters allow to reduce this difference to 17 in/mile. Future research is needed to improve current models in M-E pavement design software to adequately model cold-in-place recycled layers such as FDR.

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Executive Summary

In the last years, transportation agencies are placing increasing emphasis on the use of environmentally friendly and cost-effective techniques in pavement construction and rehabilitation such as Full-Depth Reclamation (FDR). Limited information is currently available on the material properties of FDR mixtures and previous research has concluded that default values used in Mechanistic Empirical (ME) pavement design are non representative and overly conservative. The main objective of this research is to establish standard modulus values for FDR base materials that can be confidently and effectively used by CDOT pavement designers as reliable input to designing pavements utilizing the ME pavement design.

To achieve this objective, the research first synthesized the methods used by other State Transportation Agencies (STAs) to assign structural strength to FDR materials. This research found that states have not locally calibrated ME coefficients for FDR design. Although some STAs recommend FDR ME coefficients in their pavement design manuals, these coefficients have been calibrated at the national level. CDOT is a leading agency aiming to recommend ME coefficients based on state-level FDR projects.

To better understand the performance of FDR, this research analyzed 11 FDR sites constructed throughout Colorado. Cored samples were extracted for experimental analysis and Falling Weight Deflectometer (FWD) testing was performed to estimate the structural strength of the different layers of the FDR pavements. Although the work plan considered an ambitious plan to characterize FDR sections from cores samples, the research team encountered challenges that limited the final amount of information available. Back-calculated stiffness from FWD was the only test that could be performed in all of the sites and was thus considered as the most reliable measure of stiffness.

Current design inputs are found to be conservative and leading to an underestimation of the actual performance of FDR projects. With current design input parameters (strength modulus for FDR layer of 30,000 psi and initial IRI of 62 in/mile), IRI is on average overestimated by 51 in/mile in the 10 years design-period.

Both initial IRI and stiffness modulus were found to have a significant impact on ME predictions. These two parameters were calibrated in a two-step process. First, the initial IRI is calibrated for each of the sites to minimize the vertical offset in IRI predictions and ensure that the predicted deterioration at the beginning of the analysis period is similar to the actual deterioration. Once the initial IRI is calibrated for each of the sites, the strength modulus is calibrated to ensure the deterioration trend is similar to actual values.

The research team recommends using an initial IRI of 42 in/mile in future FDR designs. The resilient modulus of FDR layers are recommended to be increased from 30 ksi to 37.3 ksi for non-stabilized FDR and to 160 ksi for emulsion-stabilized FDR.

This research has highlighted the need for future research in several areas including the need to continue monitoring the performance of FDR sections so that a larger database would be available in the future to understand the long-term performance of FDR projects. Because of the limitations of current AASHTOWare version to model FDR performance, research is needed to develop deterioration models of FDR sections. Finally, CDOT may consider further research to optimize the design of stabilized FDR in Colorado to explore the use of chemical stabilizers and further deploy the superior performance of emulsion-stabilized FDR compared to non-stabilized FDR.

Implementation

This research recommends using an initial IRI value for future ME FDR design of 42 in/mi. In terms of resilient modulus values, CDOT pavement designers are encouraged to consider 37.3 ksi for non-stabilized FDR and to 160 ksi for emulsion-stabilized FDR. Designers could also consider using a range of resilient modulus values to account for different project characteristics. In this case, the recommended range of resilient modulus values would be 16 to 50 ksi for non-stabilized FDR and 100 to 256 ksi for emulsion-stabilized FDR. These values differ from current default values (i.e., initial IRI 62 in/mi and resilient modulus of 30 ksi). The proposed input parameters lead to a conservative design of FDR projects and result in improved IRI predictions than the ones obtained with current design criteria.

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Chapter 1. Introduction

Background and Objectives

In the last years, State Transportation Agencies (STAs) are placing increasing emphasis on incorporating asphalt mixtures using cold-recycling technologies in their rehabilitation strategies. Cold-recycling technologies, such as Full-Depth Reclamation (FDR), are effective rehabilitation strategies that are proven to reduce environmental impacts, as well as construction costs and time (Schwartz et al. 2017). FDR involves pulverizing and reusing materials from distressed pavements in place, ultimately producing a new base mixture. In order to improve the performance of the FDR mixture, three different types of stabilization methods can be considered (Reeder et al. 2017): (1) mechanical stabilization, consisting of adding granular materials to the FDR mixture to produce the required degree of structural support; (2) chemical stabilization, which incorporates materials such as Portland cement, lime, or fly ash, among others; and (3) bituminous stabilization, which incorporates an emulsified asphalt or foamed (expanded) asphalt. Of the three categories of stabilization, this study focuses on non-stabilized (also referred as "dry") and emulsion-stabilized FDR mixes (also referred as "wet FDR").

Unfortunately, limited information is available on the material properties of these asphalt mixtures to facilitate the structural design of pavements incorporating base materials produced with FDR. This information is critical in the mechanistic-empirical (ME) design of pavements using the AASHTOWare software. Several authors (May 2008; Apeagyei and Diefenderfer 2013) have pointed out that default values in previous AASHTOWare software versions for cold recycled materials were overly conservative and bordered on non representative. This leads to an underestimation of the true performance capabilities of these materials, resulting in a significant loss of potential for cost and resource saving or even reluctance to specify these techniques (Apeagyei and Diefenderfer 2013).

Several studies have recently been completed to address this limitation, such as the comprehensive NCHRP report 863 "*Material Properties of Cold In-Place Recycled and Full-Depth Reclamation Asphalt Concrete*" (Schwartz et al. 2017). Different STAs, such as the ones in Maryland and Virginia, have also completed projects to assess the mechanistic structural properties of stabilized base materials (Schwartz and Khosravifar 2013; Khosravifar et al. 2015; Amarh et al. 2017). The results obtained from these studies have advanced the design of FDR base mixtures by providing reference values of the modulus of FDR materials. However, these studies have not compared the proposed modulus values and predicted deterioration with actual performance of FDR sites. This research covers this gap by analyzing the performance of 11 FDR sites across Colorado and deriving recommendations to ensure future FDR design is better aligned with the actual performance of FDR projects.

Similarly than the aforementioned STAs, CDOT faces the need to better quantify the properties of FDR base materials. By analyzing the performance of various FDR sections built throughout the state, this project aims to establish standard modulus values for various FDR base materials that can be confidently and effectively used by CDOT pavement designers as reliable input to designing pavements utilizing the CDOT Pavement Mechanistic-Empirical Design (PMED).

Research Objectives

The main objective of this research was to *establish standard modulus values for various FDR base materials that can be confidently and effectively used by CDOT pavement designers as reliable input to designing pavements utilizing the PMED*. To achieve this objective, this research considered the following sub-objectives:

- Synthesize the methods used by other STAs to assign structural strength to FDR materials.
- Determine the modulus values of FDR base materials used in existing CDOT projects.
- Correlate the back-calculated modulus values with pavement performance data.
- Establish a range of reliable modulus values for FDR materials.
- Confirm the validity of the current predictive equations in PMED for CDOT FDR materials.
- Identify best practices that could improve the performance of pavements constructed with FDR materials.

Methods and Materials

Research Method

To achieve these objectives, the research plan considered nine tasks (Figure 1).

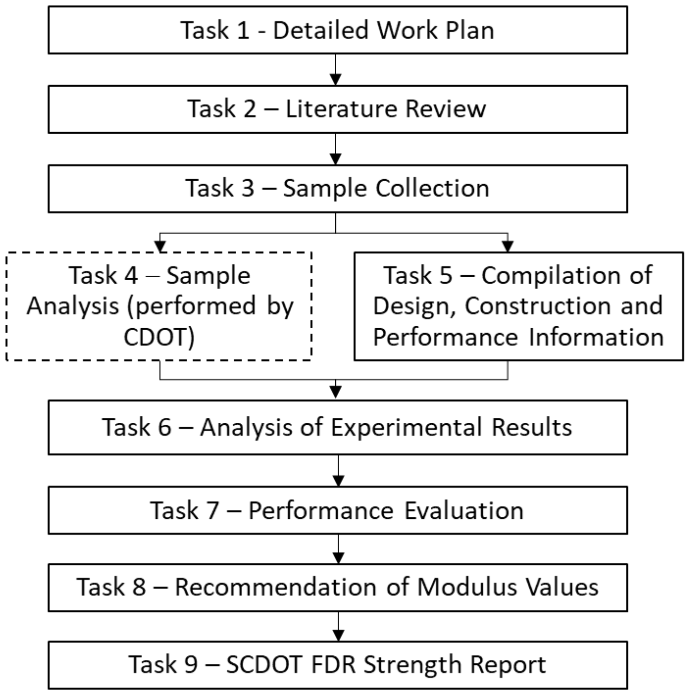


Figure 1. Research plan

Task 1 – Detailed Work Plan: Based on the comments received from the pre-kickoff meeting (May 4, 2018) and from the kickoff meeting (May 29, 2018), the research team prepared a detailed work plan that was submitted to the Study Panel on June 19, 2018.

Task 2 – Literature Review: The research team performed a literature review on published information regarding the methods other STAs are using/have used to assign structural strength to FDR materials.

Task 3 – Sample Collection: The research team collected samples from 11 FDR projects constructed throughout Colorado and submitted the samples to CDOT labs for testing. During this sample collection, CDOT personnel performed Falling Weight Deflectometer (FWD) testing to estimate the structural strength of the different layers of FDR pavements.

Task 4 – Sample Analysis: CDOT performed a set of experimental testing to the materials collected by the research in Task 3. Although the initial plan for sample analysis included several tests, the condition of samples and different limitations encountered by CDOT resulted on a partial completion of the sample analysis plan.

Task 5 – Compilation of Design, Construction and Performance Information: For each of the test sites considered in this project, the research team compiled performance information from CDOT's pavement management system. CDOT also provided the research team with backcalculated elastic modulus obtained from Falling Weight Deflectometer (FWD) data. In addition to the performance information, the research team compiled information about the design and construction of the test sites from the CDOT Headquarters' Project Support Branches and Regional offices.

Task 6 – Analysis of Experimental Results: As part of this task, the research team analyzed the results obtained from laboratory testing. Due to the challenges encountered in laboratory testing and the limited experimental results obtained, this task had a limited scope to the one initially envisioned. Initially, this plan included an outlier analysis to identify potentially abnormal data on modulus values and relationships between the stiffness obtained from the field cores and the project characteristics (e.g., recycling thickness, stabilizer, moisture susceptibility, etc.). The final analysis focused on the analysis of back-calculated modulus obtained from FWD.

Task 7 – Performance Evaluation: This task was initially envisioned to validate the recommended modulus obtained from laboratory analysis. However, due to the limited experimental results, this task was used to compare the predictive distresses obtained from AASHTOWare and the actual pavement performance data obtained from CDOT Pavement Management Program Unit.

Task 8 – Recommendation of Modulus Values: Based on the results obtained in the previous phases of the project, the research team proposed a range of reliable modulus values for FDR materials to be considered as level 2 input data on ME pavement design. The correlation analysis developed in this research will allow CDOT to predict the strength values that should be considered in future projects based on the project characteristics. Finally, level 3 input data will be recommended as a default value to consider in future FDR projects at CDOT.

Task 9 – CDOT FDR Strength Report: As a last step, the research team provided recommendations for the design of rehabilitation projects using FDR at CDOT. These recommendations include the input values to be used in the ME design of future FDR projects. It also includes recommendations that could improve the performance of pavements constructed with FDR materials in Colorado and make this technology more feasible and cost-effective to CDOT rehabilitation projects. As part of this task, the team made a preliminary presentation to the CDOT Study Panel summarizing the results of the research work. Based on the feedback obtained from this preliminary presentation, the research team developed a final report addressing the project objectives and documenting the entire research effort.

Experimental Materials

Experimental materials needed in this research consisted on field cores taken from 10 FDR projects constructed throughout Colorado. The location and basic characteristics of these projects are shown in Figure 2 and Table 1. As shown in Table 1, most of the sites consist on non-stabilized FDR sections (also referred as “dry sections”). Only three sites (sites 5, 6, and 11) are emulsion-stabilized FDR.

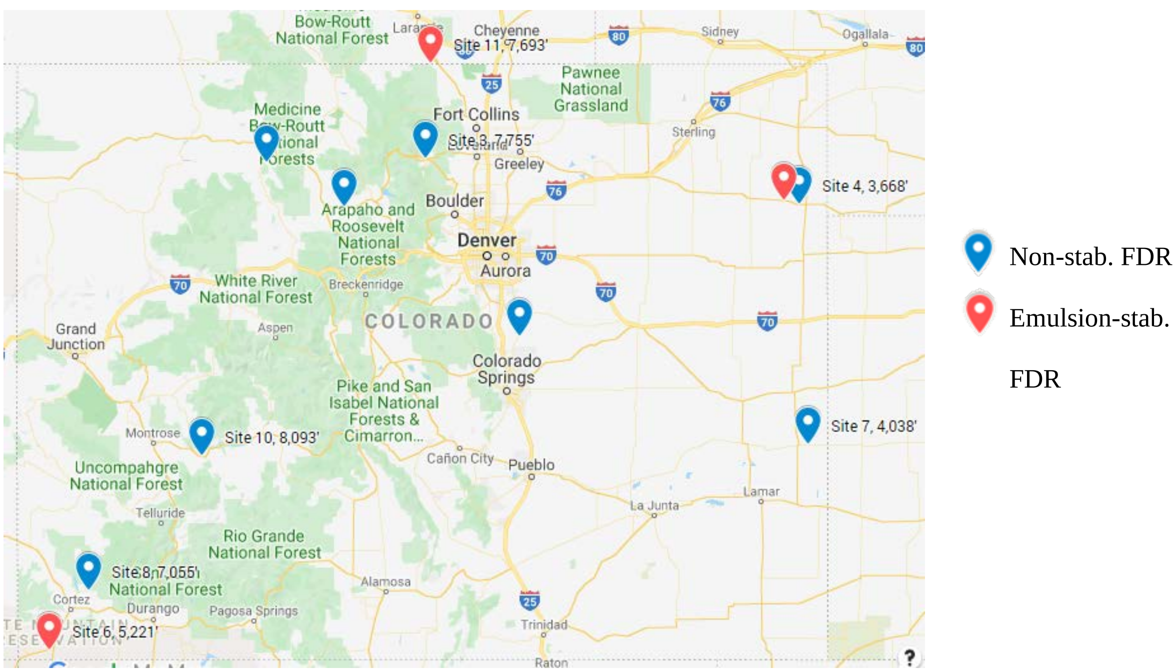


Figure 2. Location of FDR sites across Colorado

Table 1. Basic characteristics of FDR sites

ID	Location	State	Hwy	Direction	Site		1,000 ft test section		FDR Stabilization
					BMP	EMP	BMP	EMP	
1	SH 83 El Paso County Line North	CO	083A	N	33.1	41.2	35.6	35.8	Non-stab.
2	East & West of Parshall	CO	040A	E	191	201.0	192.8	193.0	Non-stab.
3	US 34: Estes Pk to Rky Mtn Pk	CO	034A	E	57.9	62.4	61.2	61.4	Non-stab.
4	US 34: Eckley East & West	CO	034B	E	231.0	241.0	245.8	246.0	Non-stab.
5	US 34: Eckley East & West	CO	034B	E	231.0	241.0	238.5	238.7	Emulsion-stab.
6	US 160 East Of Aztec Creek Ph	CO	160A	E	11.5	18.0	14.6	14.8	Emulsion-stab.
7	SH 96 Sheridan Lake East	CO	096D	E	193.7	200.0	197.8	198.0	Non-stab.
8	SH 145 - Dolores Northeast	CO	145A	N	12.6	16.2	13.8	14.0	Non-stab.
9	SH-131 Choke Cherry Lane South	CO	131B	N	58.3	61.5	59.8	60.0	Non-stab.
10	US 50 Blue Creek West	CO	050A	E	117.6	121.7	119.1	119.3	Non-stab.
11	Laramie - Colorado State Line	WY	287	S	419.3	425.4	423.3	423.5	Emulsion-stab.

For each of the test sites, core samples were collected in a 1,000 feet test section. The location of the 1,000 feet test section was defined by CDOT in collaboration with the research team considering two criteria: visibility and representativeness. Visibility was considered to ensure a safe environment for core sampling and FWD testing; whereas pavement performance data was considered to ensure the 1,000 feet sections was representative of the whole test site. Table 1 shows the beginning and ending mile posts of the 1,000 ft section considered in each site.

Although the initial scope of work considered taking 10 core samples from each test section and at least 30 pounds of aggregate base and subgrade soil (60 lb total), the research team recommended to collect additional cores to ensure having enough samples to perform the laboratory testing. After reviewing and discussing the scope of work with the Study Panel, it was decided that a total of 18 cores (10 cores 6’’ diameter and 8 cores 4’’ diameter) would be collected from each test site. Figure 3 shows how these samples were distributed in the 1,000 foot test section. As far as possible, samples were equally distributed over the 1,000 foot section length. Four cores (2 cores 6’’ diameter and 2 cores 4’’ diameter) every 250 ft were collected in the right wheel path. Similarly, 14 cores (8 cores 6’’ diameter and 6 cores 4’’ diameter) every 70 ft were collected between wheel paths. Base and sub-base granular material (30 lb each) were taken from two of the 6’’ cores made between wheel paths.

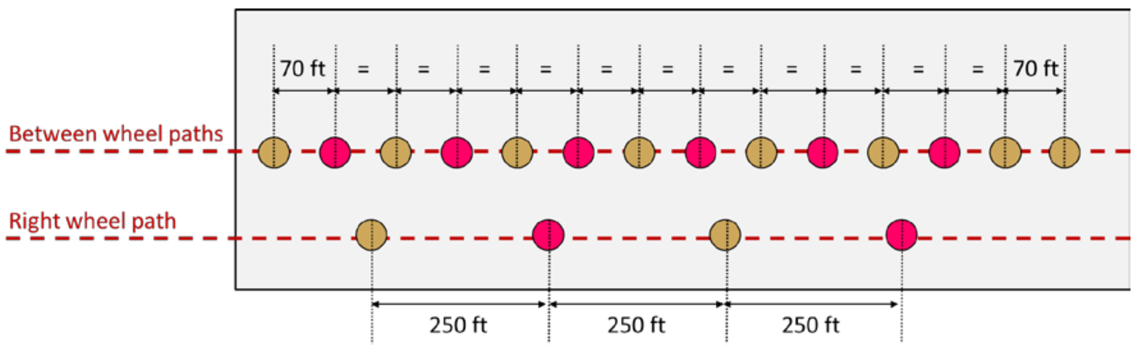


Figure 3. Sample location in 1,000 feet section

Coring was performed in 2018, with the first section being cored in July 16, 2018 and the last section cored in August 8, 2018. Figure 4 and Figure 5 show pictures of this sample collection.

Figure 4. Core sampling

Figure 5. Granular material sampling

Deflection testing was performed by CDOT personnel using a JILS truck mounted FWD built by Foundation Mechanics. The FWD was equipped with nine sensors and the load plate was located in approximately the center of each lane during testing. Deflection testing was conducted at 100-ft intervals, resulting on a total of 11 measurements for each test site. Deflection data were analyzed by CDOT using WinJILS to obtain back-calculated elastic modulus from FWD data.

Report Organization

The report is organized as follows: Chapter 2 summarizes the results found in the literature review and synthesizes past research completed on this topic. Chapter 3 presents the characteristics of the 11 FDR sections included in this study. This chapter includes laboratory results obtained from cores samples, back calculated elastic module obtained from FWD data, information on the section design, construction, and maintenance, and long-term performance. Chapter 4 recommends the input variables that should be considered for the ME design of FDR projects in Colorado. Finally, chapter 5 presents the overall conclusions and recommendations of this research.

Chapter 2. Current Methods Used for FDR Design

Introduction

Some STAs have actively sought to implement FDR technologies in their pavement rehabilitation strategies. Realizing the full benefits of these mixtures requires that local state data on material properties be available, in order to optimize FDR pavement designs. A key challenge is that local and reliable data is limited, leading STAs to use non-representative design values and coefficient modulus that yield overly conservative designs (Apeagyei and Diefenderfer 2013; May 2008).

In order to evaluate the current practice related to local calibration of FDR ME coefficients, the authors developed a systematic review of STAs' Construction Standards and Specifications (CS&S), Pavement Design Manuals, and ME manuals. This review was performed with the objective of finding relevant information published on methods and results used by STAs to calibrate and assign structural strength to FDR materials.

Review Methodology

The research team reviewed both the current practice of STAs (also known as current state of the practice) and scientific literature (also referred as the state of the art) on the recommendation of stiffness values for the ME design of FDR projects.

The review the current practice consisted on a systematic approach that searched for FDR publications directly on each of the fifty official STA's websites. The guiding principles of our methodology established that if STAs across the United States were using FDR, then FDR construction specifications and design guidelines would be contained in their CS&S, Pavement Design Manuals, and ME manuals. If a given STA did not show FDR references in its CS&S, it was assumed the agency did not use FDR actively or at all, since it would not seemingly have mechanisms in place to contract such work with third parties and contractors. However, if an agency did show references of FDR in its CS&S, we assumed that it would be more likely—but still inconclusive—that the agency actively used FDR in its projects, and further, that the agency might have calibrated ME coefficients for FDR design. The review of current state of the art searched for scientific papers and reports related to the ME design of FDR projects.

Results

Current State of the Practice

The systematic review developed in this study showed that several STAs include FDR references within their CS&S and Pavement Design Manuals (Table 2).

Table 2. FDR references found in STAs CS&S and Pavement Design Manuals

	State Transportation Agency (STA)									
Manual Type	**AL**	**AK**	**AZ**	**AR**	**CA**	**CO**	**CT**	**DE**	**FL**	**GA**
CS&S	✓				✓	✓				✓
PDM					✓	✓				✓
	HI	**ID**	**IL**	**IN**	**IA**	**KS**	**KY**	**LA**	**ME**	**MD**
CS&S			✓		✓				✓	
PDM		✓		✓					✓	✓
	MA	**MI**	**MN**	**MS**	**MO**	**MT**	**NE**	**NV**	**NH**	**NJ**
CS&S			✓			✓		✓	✓	
PDM			✓			✓			✓	
	NM	**NY**	**NC**	**ND**	**OH**	**OK**	**OR**	**PA**	**RI**	**SC**
CS&S	✓			✓				✓		
PDM	✓			✓				✓		
	SD	**TN**	**TX**	**UT**	**VT**	**VA**	**WA**	**WV**	**WI**	**WY**
CS&S	✓	✓	✓		✓	✓			✓	
PDM						✓				

Note: CS&S refers to Construction Standards and Specifications; PDM refers to Pavement Design Manual.

Although FDR is commonly referred in the STAs' CS&S and Pavement Design Manuals, this research found that no states have locally calibrated ME coefficients for FDR design. As STAs across the US have adopted the ME pavement design, their main focus seems to have been on calibrating default ME coefficients to local design parameters and pavement characteristics other than FDR. Table 3 provides a correlating summary on STAs that have published ME manuals and those that recommend FDR ME coefficients. It is worth noting that, when recommended, the FDR ME coefficients have been calibrated at the national level. No STAs were found that suggested the use of state-calibrated FDR coefficients in their ME design manuals.

Table 3. States including FDR references in their ME manuals and ME coefficients

	STA						
Manual Type	**CA**	**CO**	**IN**	**MI**	**MN**	**UT**	**VA**
Reference of FDR in ME Manual	✓	✓	✓	✓	✓	✓	✓
ME Coefficients for FDR design (Nationally Calibrated)	✓	✓					✓

Current State of the Art

Several studies have been completed in order to address limitations associated with FDR designs, such as the comprehensive NCHRP report 863 "*Material Properties of Cold In-Place Recycled and Full-Depth Reclamation Asphalt Concrete*" (Schwartz et al. 2017; Diefenderfer et al. 2016; Diefenderfer and Bowers 2015; Bowers et al. 2015). Different STAs such as Maryland and Virginia have also completed projects to assess the mechanistic structural properties of stabilized base materials (Amarh et al. 2017; Khosravifar et al. 2015; Schwartz and Khosravifar 2013). The results obtained from these studies have contributed new information towards the design of FDR mixtures, but the information gap is still wide, particularly for designing non-stabilized and emulsion-stabilized FDR mixtures. Furthermore, the literature is limited on recommending a range of stiffness values associated with FDR mixtures resulting from FWD measures. Research using FWD data has mainly focused on determining other pavement characteristics such as the Structural Number (Diefenderfer and Apeagyei 2011a; b), which is utilized for designs following the AASHTO 1993 Standard, not ME pavement design.

Three studies were found that did focus on recommending stiffness values from FWD data. All of them analyzed emulsion-stabilized FDR (Amarh et al. 2017; Kroge et al. 2009; Mallick et al. 2002), whereas only one evaluated non-stabilized FDR mixtures (Mallick et al. 2002). This highlights the need to conduct more research on non-stabilized FDR and how it compares with emulsion-stabilized FDR, which this study addresses.

Mallick et al. (2002) evaluated the performance of non-stabilized and emulsion-stabilized FDR mixes, conducting FWD testing to determine the back-calculated elastic modulus (E_{FWD}). This study found that E_{FWD} values were 80 ksi and 78 ksi (552,515 kPa and 535,333 kPa) for non-stabilized and emulsion-stabilized, respectively. Kroge et al. (2009) validated the benefits of FDR with engineered emulsion using preconstruction and post-construction FWD data, and further back-calculated E_{FWD} values, offering an average of 180 ksi (1,240 MPa) for the emulsion-stabilized base resulting from the tests. Amarh et al. (2017) back-calculated the elastic modulus for design purposes using ME pavement design and investigated the trends of three in-service pavements rehabilitated with FDR during the 2008 construction season in Virginia. This study compared FDR roads constructed with foamed asphalt, asphalt emulsion, and Portland cement as stabilizing agents. E_{FWD} associated with the emulsion-stabilized FDR mix ranged between 36 to 485 ksi (250 to 3,350 MPa), with the long-term average resulting in 174 ksi (1,200 MPa). This wide range was attributed to strong seasonal variations.

The results obtained from these previous studies have advanced the design of FDR base mixtures by providing reference values of the modulus of FDR materials. However, these studies have not compared the proposed modulus values and predicted deterioration with actual performance of FDR sites. This research covers this gap by analyzing the performance of 11 FDR sites across Colorado and deriving recommendations to ensure future FDR design is better aligned with the actual performance of FDR projects.

Conclusions

Although FDR is commonly referred in the STAs' CS&S and Pavement Design Manuals, this research found that no states have locally calibrated ME coefficients for FDR design. Although some STAs recommend FDR ME coefficients in their ME pavement design manuals, these coefficients have been calibrated at the national level. No STAs were found that suggested the use of state-calibrated FDR coefficients in their ME design manuals. Therefore, Colorado is a leading state aiming to recommend ME coefficients based on state-level FDR projects.

Chapter 3. Characterization of FDR Sections

Introduction

To characterize the FDR sections included in this study, the research team collected data from different sources. First, we collected data related to the design, construction and maintenance from CDOT. Second, cored samples were extracted and delivered to CDOT laboratory for experimental testing. The initial testing plan could only be partially completed mainly because of the challenge of keeping intact samples. Third, back-calculated modulus were obtained from Falling Weight Deflectometer. Finally, the research team analyzed distress data from the 11 sites including International Roughness Index (IRI), rutting, and cracking—which included fatigue, transverse and longitudinal cracking. This chapter summarizes the results obtained from these different sources of information and the recommended values considered in the ME simulations for the recommendation of input values.

Design, Construction, and Maintenance Information

CDOT provided the research team with the design, construction, and maintenance information for each of the test sections. Figure 6 depicts the range of thicknesses of the different layers present in the test sections. Test sections consist of a HMA layer ranging from 4 to 6'' thickness followed by a FDR layer of 8 to 12'' thickness. Appendix A provides detailed information of each of the layers in the test sites.

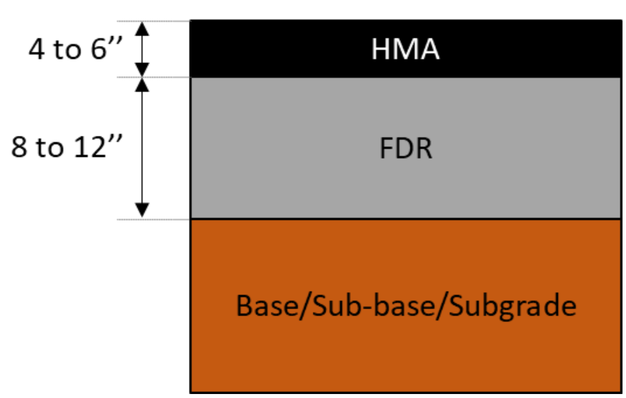


Figure 6. Layer thicknesses in typical test section

With respect to maintenance information, only 3 out of the 11 sections were recorded to receive maintenance treatments. These treatments consisted of chip seals performed in 2012 and 2011 in sites 7 and 8, respectively; and an overlay performed in 2013 on site 1.

Laboratory Results of Cored Samples

Layer thickness

The thickness of the different layers in cored samples were measured by the research team before delivering these samples to CDOT laboratory. Table 4 summarizes the thicknesses of the different layers in each site and how it compares with the information obtained from design and construction. Detailed thicknesses for each core are provided in Appendix A. Table 4 shows that, in general terms, the layer thickness measurements from cored samples are consistent with the design and construction information. It is worth noting, however, that site 2 shows a significant difference. This is probably due to the fact that it was not possible to extract intact cores from site 2 for the whole depth indicated in the design and construction information, with cores breaking at a depth of approximately 4.5 inches.

Table 4. Layer thickness

Site	Thickness measured from cored samples		Information from design and construction	
	HMA (inches)	FDR (inches)	HMA (inches)	FDR (inches)
1	7	6	6	8
2	4.5	6	8	8
3	4	6	4	8
4	5	5	4	8
5	4	6	4	8
6	4	5	4	9
7	4	10	4	12
8	5	10	5	10
9	5	6	5	8
10	5	12	5	8
11	4	8	4	9

Figure 7. Examples of dry (site 2) and emulsion-stabilized FDR (site 6) cored samples

One of the main challenges encountered when coring was to obtain intact cores of the FDR. The asphalt emulsion FDR shown a better stability because the emulsion acted as a bonding agent. However, it was more difficult to obtain intact cores from dry FDR sections because they do not have nothing to bind the material. Figure 7 shows an example of this fact, with cores from site 2 (dry FDR) breaking after the HMA layer and cores from site 6 (emulsion stabilized) standing for a significant larger depth.

Laboratory Sample Results

As introduced in the Research Method section, the initial plan for lab testing included several experiments. However, the condition of samples and different limitations encountered by CDOT resulted on a partial completion of this plan (Table 5).

Among the tests that could not be performed, it is worth noting the importance of dynamic modulus (AASHTO TP62) and resilient modulus (AASHTO T307), as these are key input parameters to model FDR layers in ME design. Resilient modulus was initially planned to be performed on dry FDR cores (sites 1 to 4 and 7 to 10). However, none of this testing was finally performed. Dynamic modulus was performed on wet FDR cores (sites 5, 6, and 11), although testing in cores from site 6 was not successful.

Table 5. Initial testing plan and summary of performed tests

Design Type	Initial Testing Plan	Performed Testing
All layers	Back-calculated layer modulus (AASHTO T256 and ASTM D5858)	Yes. All sites
HMA Layer	Dynamic Modulus (AASHTO TP62)	No
	Indirect Tensile Strength at 14F (psi) (AASHTO T322)	
	Creep Compliance (1/psi) (for various loading times and temperatures) (AASHTO T322)	
	Lottman (CP-L 5109)	
	Asphalt Content (Burn Off) (AASHTO T164/T170/T319)	
	Air Voids (%) (AASHTO T166)	
	Aggregate Specific Gravity (AASHTO T84 and T85)	
	Gradation (AASHTO T27)	
	Unit Weight (pcf) (AASHTO T166)	
	Voids filled with asphalt (VFA) (AASHTO T209)	
FDR Wet Layer (3 sites)	Dynamic Modulus (AASHTO TP62)	Yes (2 out of 3)
	Indirect Tensile Strength at 14F (psi) (AASHTO T322)	No
	Creep Compliance (1/psi) (for various loading times and temperatures) (AASHTO T322)	
	Lottman (CP-L 5109)	
	Asphalt Content (Burn Off) (AASHTO T164/T170/T319)	
	Air Voids (%) (AASHTO T166)	Yes (2 out of 3)
	Aggregate Specific Gravity (AASHTO T84 and T85)	No
	Gradation (AASHTO T27)	
	Unit Weight (pcf) (AASHTO T166)	
	Voids filled with asphalt (VFA) (AASHTO T209)	Yes (2 out of 3)
FDR Dry Layer (7 sites)	Classification (CP31 and CP32 AASHTO T89/T90)	No
	Maximum Dry Density (AASHTO T180)	
	Optimum Moisture Content (AASHTO T180)	
	Resilient Modulus (AASHTO T307)	
	R-Value (AASHTO T190)	
Base-Course	Classification (CP31 and CP32 AASHTO T89/T90)	Yes (8 out of 11)
	Maximum Dry Density (AASHTO T180)	No
	Optimum Moisture Content (AASHTO T180)	
	Resilient Modulus (AASHTO T307)	
	R-Value (AASHTO T190)	
Subgrade	Classification (CP23 and CP31 AASHTO T89/T90)	Yes (8 out of 11)
	Maximum Dry Density (AASHTO T99)	No
	Optimum Moisture Content (AASHTO T99)	
	Resilient Modulus (AASHTO T307)	
	R-Value (AASHTO T190)	

Results of dynamic modulus from sites 5 and 11 are shown in Table 6 and Table 7. These values were not satisfactory because the recommended dynamic modulus values failed to achieve singularity with the binder modulus values (G*) when used in the AASHTOWare software. As a result, the dynamic modulus values of emulsion stabilized FDR sections could not be used in the ME models.

Table 6. Dynamic modulus E* (ksi) of FDR layer in site 5

Freq (Hz)	Temperature				
	14°F	40°F	70°F	100°F	130°F
25.0	463.3	461.3	448.2	386.1	217.0
10.0	463.1	460.4	442.1	359.2	168.4
5.0	463.0	459.4	435.9	334.3	133.6
1.0	462.4	455.9	414.3	261.8	69.0
0.5	462.1	453.7	400.9	225.8	49.8
0.1	460.8	445.5	357.0	141.7	22.7

Table 7. Dynamic modulus E* (ksi) of FDR layer in site 11

Freq (Hz)	Temperature				
	14°F	40°F	70°F	100°F	130°F
25.0	463.6	463.6	463.0	294.0	44.7
10.0	463.6	463.6	462.7	248.5	44.1
5.0	463.6	463.6	462.4	213.7	43.8
1.0	463.6	463.6	461.2	142.4	43.3
0.5	463.6	463.6	460.3	118.8	43.2
0.1	463.6	463.6	456.9	81.2	43.0

Although CDOT provided information on gradation and R-values for 8 out of the 11 sites, it was not clear which layer these values corresponded to (i.e., base, sub-base, or dry FDR). As a result of this, the research team did not feel confident using these values in the analysis either.

Back-Calculated Modulus

Deflection data were analyzed by CDOT using WinJILS to obtain back-calculated elastic modulus from FWD data. Results from this analysis are shown in Table 8. Table 8 shows that emulsion-stabilized FDR layers (site 5, 6, and 11) are generally stiffer than dry sections.

Table 8. Back-calculated modulus data from FWD (ksi)

Site	Layer 1 (HMA Layer)	Layer 2 (FDR Layer)	Layer 3 (ABC Layer)	Layer 4 (Subgrade Layer)
1	775.4	24.2		34.5
2	827.3	23.0		14.1
3	1.524.8	66.0		34.4
4	1.087.5	81.7		29.1
5	880.0	413.3		30.1
6	451.2	160.7	107.6	22.8
7	592.3	146.5		13.9
8	974.4	31.7		17.0
9	621.2	79.8		21.7
10	727.6	28.8		19.2
11	436.0	200.3	50.4	33.7

Back-calculated stiffness from FWD was the only test that could be performed for all of the sites. With so limited experimental data on stiffness modulus from resilient and dynamic modulus, the research team decided to rely on back-calculated stiffness modulus from FWD testing as the most reliable experimental measure of stiffness.

Long-Term Performance

This section reports on the analysis of Pavement Management System (PMS) data. This data include annual distresses measured every 0.1 mile for each of the test sites. Sections located in Colorado (sites 1 to 10) include information of five distresses: average International Roughness Index (IRI), average rutting, and fatigue, transversal and longitudinal cracking. Data from section 11, located in Wyoming, only includes IRI and rutting.

The purpose of this analysis was to propose and select the performance curves that will be considered for the recommendation of FDR modulus values. Because this data will be compared with the performance values resulting from ME simulations, it is essential that the PMS data and the associated distresses be the most representative of the actual pavement conditions found in the respective sites. In the analysis of the performance data provided by CDOT, the research team encountered some data inconsistencies, which are summarized in this section, together with our interpretation and recommendation to address them. Based on these recommendations, a set of performance curves for IRI, rutting, and transversal, fatigue and longitudinal cracking are proposed for each of the sites. These curves will be analyzed and compared with ME predictions to derive recommendations on the ME input values to be used to design FDR projects.

Preliminary Analysis of Pavement Distress Data

The research team started performing an analysis of outliers using box plots. These graphs show a compact view of a variable's distribution with quartiles and outliers, and, as such, ultimately helping to guide the selection of the most representative value of centrality in such distributions. Figure 9 shows an illustrative example of such outlier box plots for site 1. Similar graphs were produced for all 11 sites (See Appendix A).

From the analysis of outliers box plots, the research team found that mean values were very sensitive to outliers and therefore recommended the use of median values as a reliable measure to characterize the deterioration of sites over time.

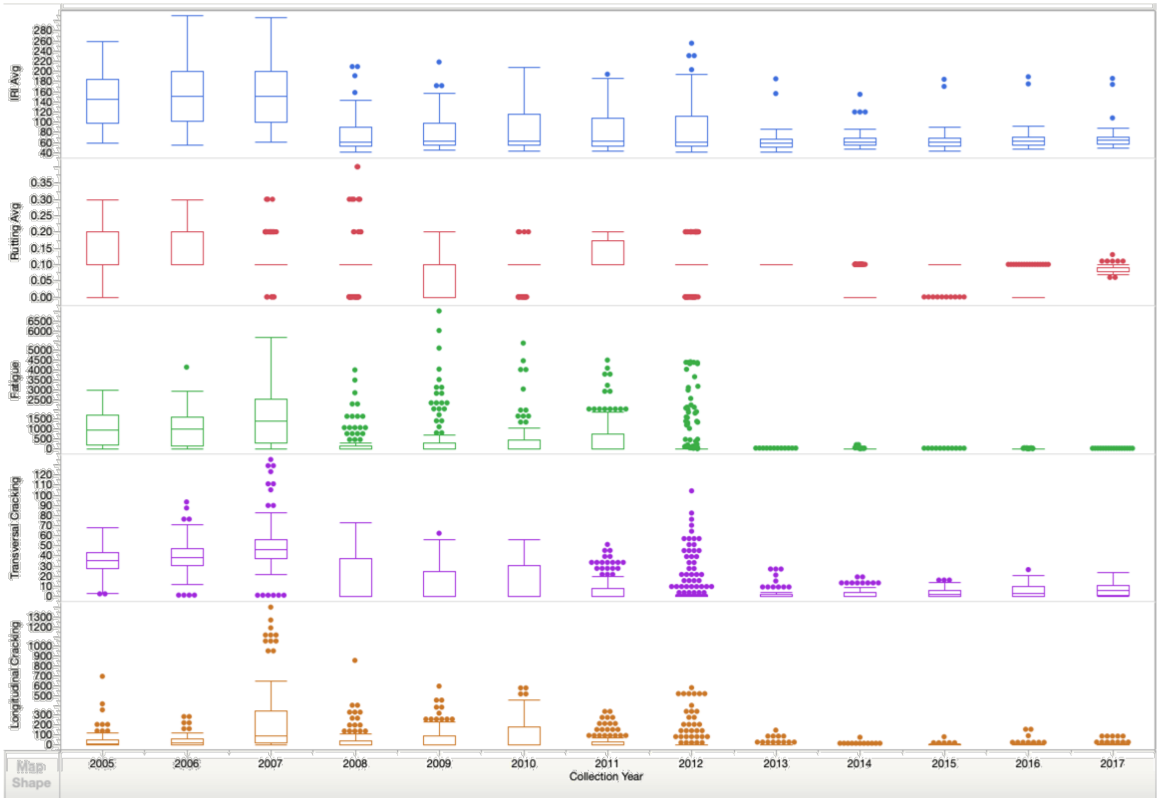


Figure 8. Outlier box plot of distresses in site 1

Inconsistencies in the PMS Data

The analysis of PMS found six patterns of data inconsistencies, in which recorded PMS and maintenance history data are not consistent with expected trends of deterioration. In the description of these inconsistencies, the year in which the FDR section is constructed is referred as "FDR Date". These inconsistencies are:

1. Distresses after the construction of the FDR section (i.e., FDR Date + 1 year) show unexpected high values for the next year or two, then values decline
2. Distresses unexpectedly improve with no records of maintenance (unexplained declining trends)
3. Random null distresses create disruptions on typically expected trends
4. Random high distresses create disruptions on typically expected trends
5. Rutting shows inconsistent trends throughout the years
6. Distress values after "Ad Date + 1 year" show null median values overtime

Appendix A includes a detailed description and illustrative examples of each of these issues. It is worth noting that not all issues occur on every site. That is, different sites may show different data inconsistencies. Table 9 summarizes the six data inconsistencies found by site. This summary shows that some of the issues (i.e., inconsistencies 1, 5, and 6 in Table 9) are encountered in the majority of sections, whereas other inconsistencies (i.e., inconsistencies 2, 3, and 4 in Table 9) are found in only one or two sites. From the analysis of each of these inconsistencies, the research team suggests possible interpretations and a recommendation to address them (Table 10).

Table 9. Summary of inconsistences encountered by site

Data inconsistency		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	Distresses after "FDR Date + 1 year" show unexpected high values for next one or two years, then values decline					x	x			x	x	
2	Distresses unexpectedly improve with no records of maintenance; unexplained declining trends		x									
3	Random null distresses create disruptions on expected trends		x									
4	Random high distresses create disruptions on expected trends			x		x						
5	Rutting shows inconsistent trends throughout the years	x	x	x	x	x	x		x	x	x	
6	Distress values after "FDR Date + 1 year" show null values overtime	x			x	x	x	x	x	x		

Table 10: Interpretations and recommendations for PMS data inconsistencies

Data inconsistency		**Interpretations**	**Recommendations**
1	Distresses after "FDR Date + 1 year" show unexpected high values for next one or two years, then values decline	"FDR Date" was mistakenly documented	Analyze data from the year that all distresses start increasing from the lowest values

2	Distresses unexpectedly improve with no records of maintenance; unexplained declining trends	Maintenance performed but perhaps not documented on the PMS records	Assume minor maintenance was performed yet not documented
3	Random null distresses create disruptions on typically expected trends	Data may not have been collected	Ignore this data and extrapolate trend as it would be expected
4	Random high distresses create disruptions on typically expected trends	Instrument error when measuring at site	Ignore this data and extrapolate trend as it would be expected
5	Rutting shows inconsistent trends throughout the years	Rutting scale is too tight, so small variations seemingly display large discrepancies	Request information on type(s) of equipment(s) used and check instrument tolerances
6	Distress values after "FDR Date + 1 year" show null values overtime	Distresses did not significantly develop overtime	Analyze data as is (distress may not have developed)

Based on these recommendations, a set of performance curves for IRI, rutting, and transversal, fatigue and longitudinal cracking were proposed for each of the sites. A detailed description of each of these curves is included in Appendix A.

Proposed Performance Curves

From the analysis of performance data, the research team found that only IRI showed a consistent and reasonable behavior over time. Rutting and cracking (longitudinal, transversal, and fatigue) did not show reasonable deterioration trends over time (Figure 10). Figure 10 shows median distress values for all sites (“median all”), for non-stabilized sites (“median NS”), and for emulsion-stabilized sites (“median ES”). For comparative purposes, Figure 10 also shows the maximum and minimum distress values of all sites for each year. Fatigue cracking, for example, had null values for all the sites except for site 2, which presented values that decreased over time without having any reference of a maintenance activity being performed (Figure 10 b). After analyzing the distress trends over time for rutting and cracking, the research team concluded that these unreasonable trends may be caused by errors in data collection or unreported maintenance activities. Based on these observations, the research team decided to use IRI as the leading indicator for pavement deterioration. IRI is therefore the distress used to compare with ME predictions and derive recommendations on ME input data.

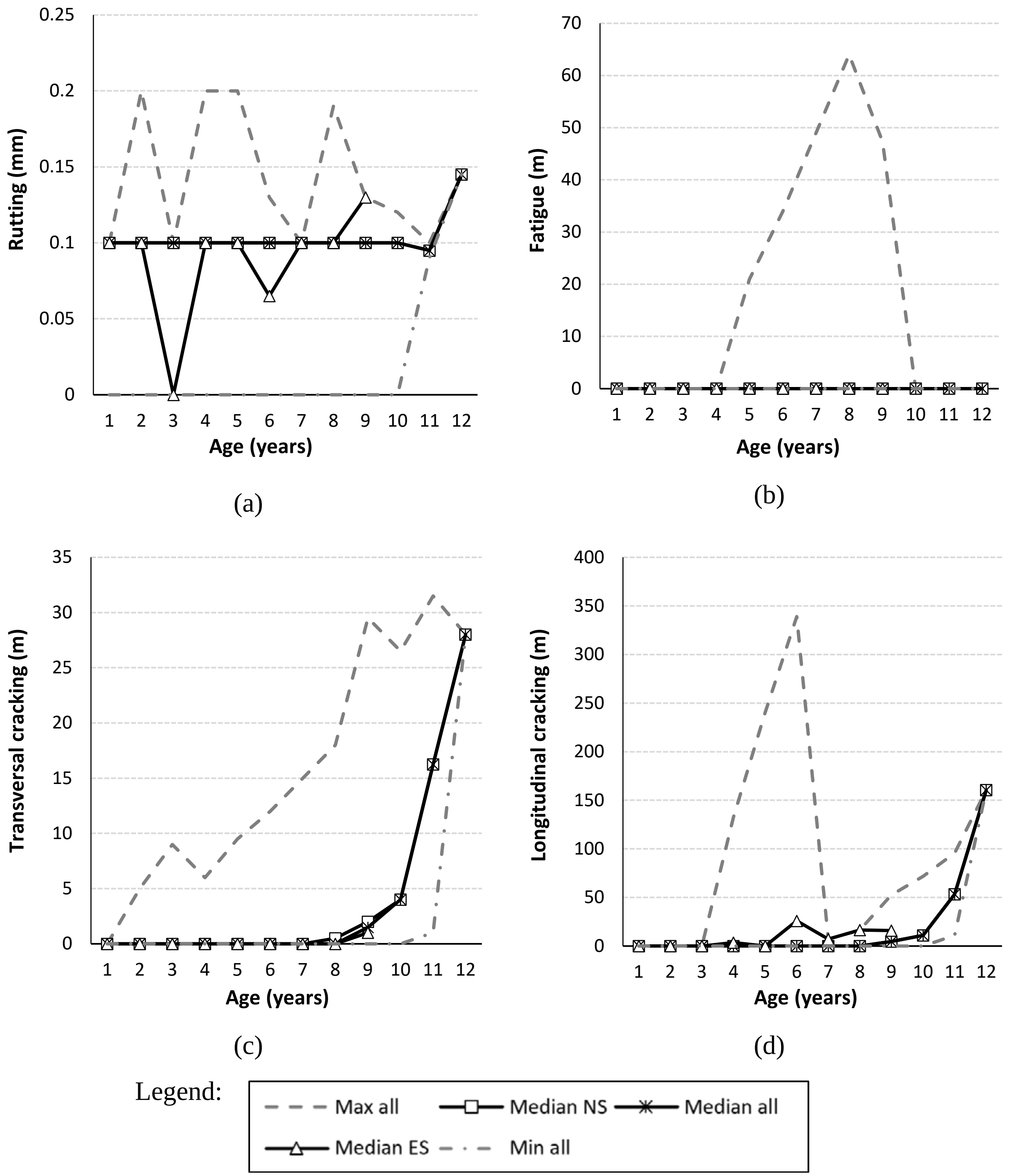


Figure 9. Median values of actual distresses of FDR sections over time (a) rutting, (b) fatigue, (c) transversal cracking, and (d) longitudinal cracking

Figure 11 summarizes the deterioration of FDR sections over time, measured in terms of IRI. It shows that the IRI of emulsion-stabilized sections (i.e., wet FDR) is lower than the one of non-stabilized sites (i.e., dry FDR). Therefore, emulsion-stabilized sites perform better over time than non-stabilized sections. In general terms, FDR sites show a good performance over time, with a low increase of IRI in the 10 years design period (i.e., in median values, IRI increases from 54 to 65 in/mile in 10 years).

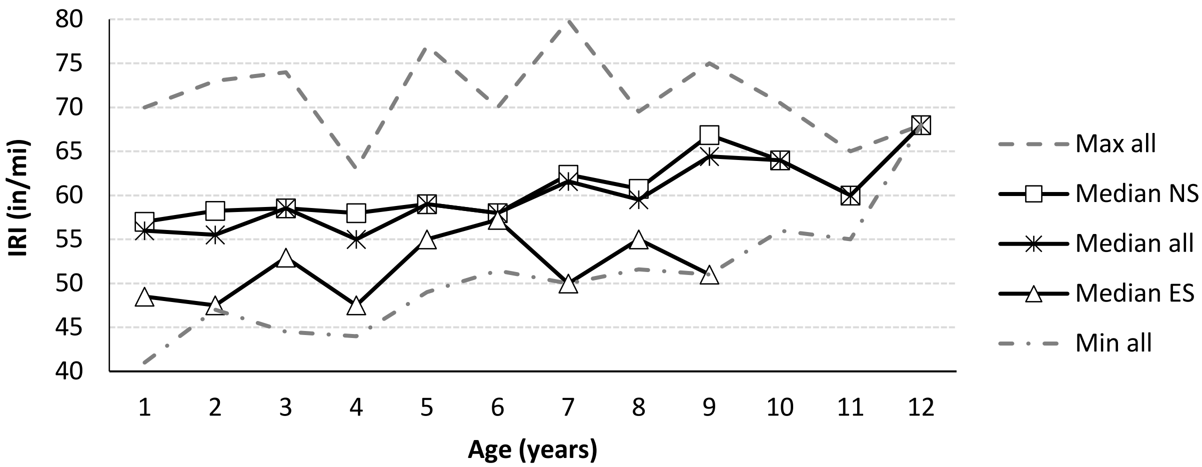


Figure 10. Median values of actual IRI of FDR sections over time

Although the deterioration trend of median IRI values in non-stabilized sections show reasonable values that increase with pavement age, emulsion-stabilized sections show a more erratic trend in which IRI values even decrease in certain periods of time (e.g. age 4, 7 and 9 in Figure 11). These unexpected results are probably due to the small sample size - there are only 3 sites with emulsion-stabilized FDR; and for some of these sections IRI data was collected for only one year. Although median values in Figure 11 for emulsion-stabilized sections show a non-intuitive trend, individual values for each of the sections (Figure 12) seem reasonable.

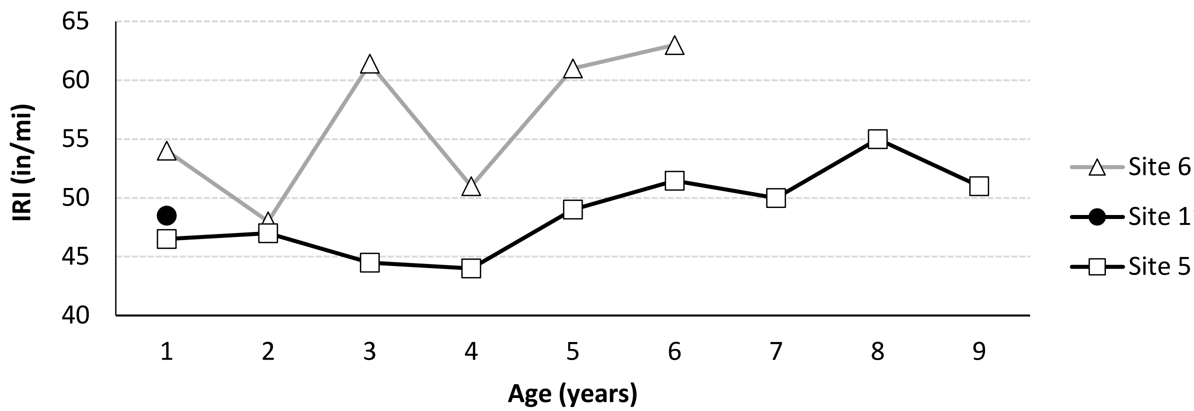


Figure 11. Actual IRI values of emulsion-stabilized FDR sections over time

The detailed deterioration curves for IRI values in all the non-stabilized sections are shown in Figure 13. The research team considered the deterioration trends shown in Figure 12 and Figure 13 as representative values for each of the test sections. These curves are the ones compared with ME predictions for the recommendation of modulus values.

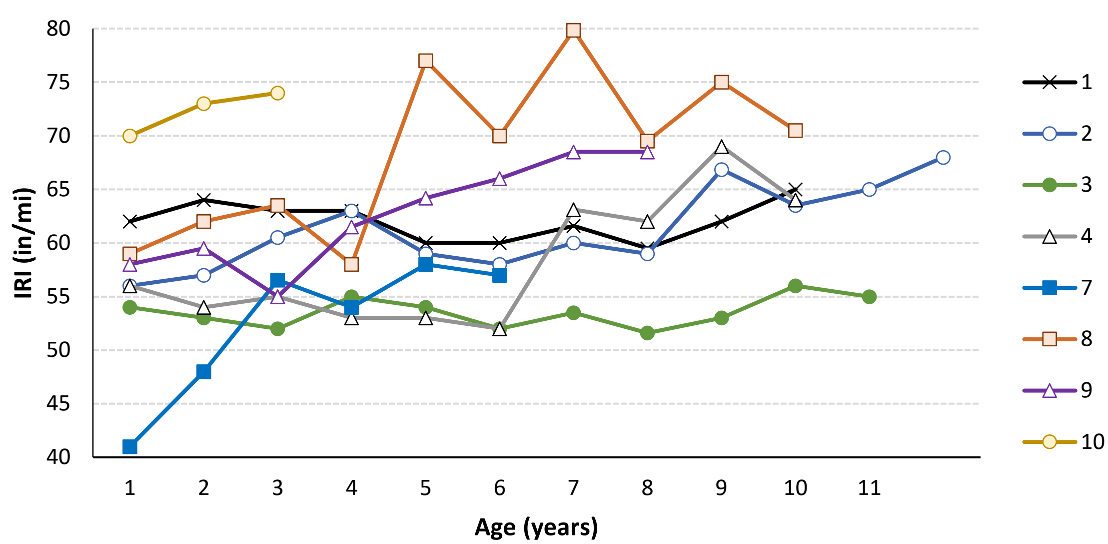


Figure 12. Actual IRI values of non-stabilized FDR sections over time

Conclusions and Recommendations

Although the work plan considered an ambitious plan to characterize FDR sections, the research team encountered several challenges that limited the final amount of information available. The major challenge was related to the difficulty of obtaining intact cores of the FDR, which limited the laboratory testing that could be performed. The asphalt emulsion FDR shown a better stability because the emulsion acted as a bonding agent. However, it was more difficult to obtain intact cores from dry FDR sections because they do not have nothing to bind the material. Among the tests that could not be performed, it is worth noting the importance of dynamic modulus (AASHTO TP62) and resilient modulus (AASHTO T307), as these are key input parameters to model FDR layers in ME design. Resilient modulus was initially planned to be performed on dry FDR cores (sites 1 to 4 and 7 to 10) but none of this testing was finally performed. Dynamic modulus was performed on wet FDR cores (sites 5, 6, and 11), although testing in cores from site 6 was not successful.

Back-calculated stiffness from FWD was the only test that could be performed for all of the sites. With so limited experimental data on stiffness modulus from resilient and dynamic modulus, the research team decided to rely on back-calculated stiffness modulus from FWD testing as the only experimental measure of stiffness. Back-calculated stiffness values of non-stabilized FDR were between 23 and 146 ksi, while values in stabilized sections ranged from 160.7 and 413 ksi. These values are similar to the ones found in the literature, except from the back-calculated value in site 5 (emulsion-stabilized), which is significantly higher. Back-calculated elastic modulus of non-stabilized projects show less variability and values similar to the one found in the literature.

From the analysis of long-term performance data, the research team found inconsistent values in deterioration trends for rutting and cracking. IRI values shown consistent values for both non-stabilized and emulsion-stabilized sections. The analysis shows that emulsion-stabilized sites perform better over time than non-stabilized sections in terms of IRI. In general terms, FDR sites show a good performance over time, with a low increase of IRI in the 10 years design period (i.e., in median values, IRI increases from 54 to 65 in/mile in 10 years). From the analysis of IRI data, the research team proposed a set of IRI deterioration trends for each of the 11 sections in the project. These curves are the ones compared with ME predictions for the recommendation of modulus values.

Chapter 4. Recommendation of Modulus Values

Introduction

The objectives of this chapter are to establish a range of reliable modulus values for FDR materials and to confirm the validity of the current predictive equations in ME pavement design for CDOT FDR materials. To achieve these objectives, the research team analyzed the actual performance of the 11 FDR sites considered in this study and compared it with the predicted deterioration obtained from ME simulations considering the design parameters currently used by CDOT (referred hereafter as current design parameters). If significant differences become apparent in this comparison, the research team would derive recommendations on the input data to consider in ME so that this difference is minimized.

Methodology for the Recommendation of Modulus Values

To derive recommendations of input values for the design of FDR projects, the research team considered the following methodology:

<u>Definition of actual performance:</u> The actual long-term performance of FDR sites was determined from Pavement Management System (PMS) data. As explained in Chapter 3, the research team considered IRI as the most reliable metric to characterize the long-term performance of the FDR sites considered in this study. Therefore, actual performance is defined by the IRI deterioration curves obtained in Chapter 3. This actual performance will serve as a benchmark to determine the goodness of ME predictions.

<u>Long-term difference between actual performance and current design:</u> For each of the 11 FDR sites, the research team run the AASHTOWare software to determine ME predictions of future IRI deterioration. In these predictions, the research team used state calibrated parameters included in CDOT ME Design Guide (CDOT 2019) and characterized the stiffness of the FDR layers based on the back-calculated modulus obtained from FWD testing. As depicted in Figure 14, the research team evaluated the long-term difference between actual performance and ME current design

prediction by quantifying the area between these two curves. By measuring the area, this metric allows to quantify the differences in IRI prediction over the design period of FDR projects.

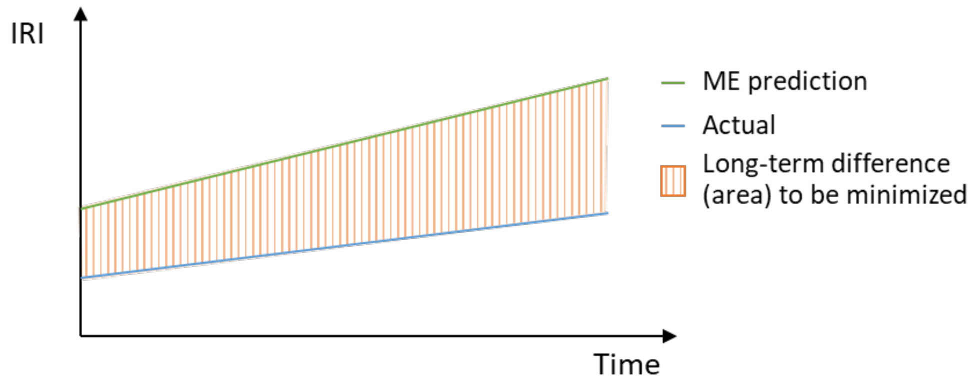


Figure 13. Long-term difference between actual performance and ME prediction

Sensitivity Analysis of IRI predictions to input ME data: To better understand the sensitivity of ME predictions to input variables, the research team quantified the changes in the predicted IRI when changing input variables such as the FDR layer stiffness (i.e., backcalculated modulus) and initial IRI. The results obtained in this analysis provided valuable insights to derive recommendations on the input values that should be considered in future FDR design to achieve a better alignment between actual performance and ME predictions.

Recommendations of ME input data at the project level: For each of the 11 FDR sites, the research team derived recommendations for ME input variables. The recommended values are aimed at minimizing the difference between actual and predicted ME IRI performance while providing reasonable values to characterize the stiffness of FDR materials.

Recommendations of ME input data for future FDR design in Colorado: The research team summarized the recommendations derived for each of the 11 FDR sites and derive more general recommendations for future FDR design in Colorado.

Long-Term Difference between Actual Performance and Current Design

The main goal of this task was to assess the difference between the actual performance and the ME predicted performance using current input design values, which consider a stiffness value of the FDR layer of 30 ksi and an initial IRI of 62 in/mile. Figure 15 shows an example of this comparison for site 4.

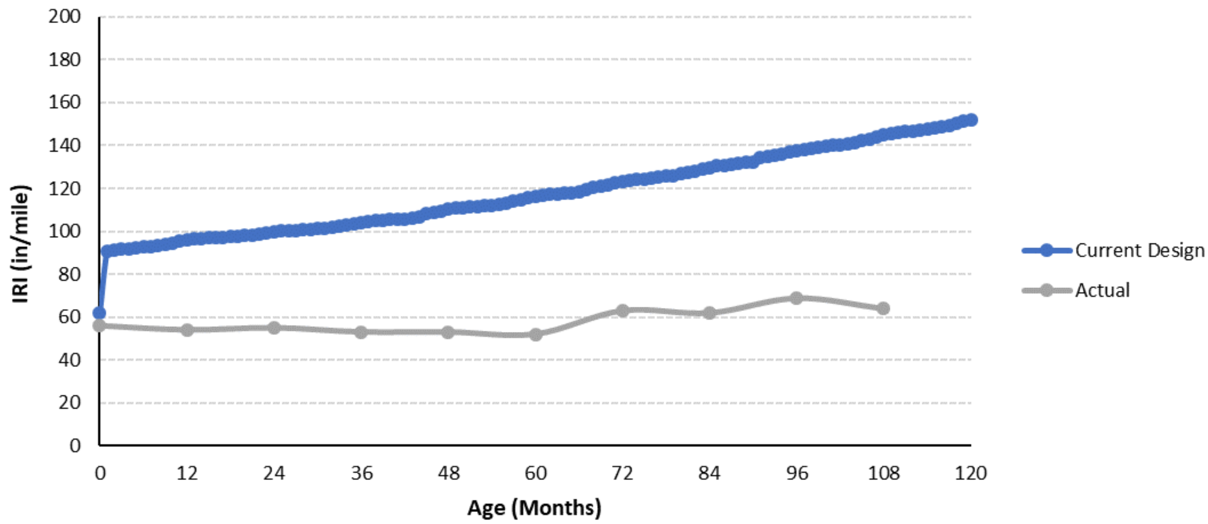


Figure 14. Actual performance and ME prediction with current design inputs for site 4

Figure 15 shows that the predicted performance obtained with current design inputs is conservative when compared to the actual performance. Predicted performance does not exceed the ultimate IRI (200 in/mi) at the end of the design period (10 years). The predicted and actual IRI values of other sites vary from site 4 (plots for other sites are included in Appendix B), but all the sites exhibit a similar pattern in which the predicted IRI is higher than the actual IRI. Because of this conservative prediction, the long-term difference between predicted performance based on current design inputs and actual performance is positive in all the sites (Table 11). This means that in all the sites, the predicted deterioration is larger than the actual deterioration. To provide a more intuitive metric, Table 11 also shows the average difference in terms of IRI. On average, current design parameters overestimate IRI by 51 in/mile. These results are aligned with previous research that have pointed out that default values in AASHTOWare software versions for cold recycled materials were overly conservative and bordered on non representative (May 2008; Apeagyei and Diefenderfer 2013).

Table 11. Long-term (LTD) and average IRI difference between current design and actual performance[1]

Difference design - actual	Site										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
LTD	5,333	5,278	5,721	6,097	9,055	3,513	2,961	3,294	4,308	532	NA
Average IRI difference	50	44	48	57	95	59	50	31	52	23	NA

Figure 15 also shows that the difference in deterioration is characterized by two facts. First, there is a significant difference at the beginning of the analysis with IRI predictions suffering an important jump in month 1 of the simulation. Second, there is a difference in the deterioration trend over time, with the predicted deterioration having a steeper slope than actual deterioration. These two facts are related to two input values in ME, the initial IRI and the strength modulus of the pavement layers. To better understand the influence of these input design parameters in the predicted deterioration obtained from ME, the research team performed a sensitivity analysis that is described in the next section.

Sensitivity Analysis of IRI predictions to ME input data

The goal of this task was to determine the sensitivity of IRI predictions to changes in two ME input parameters: initial IRI (I-IRI) and strength modulus (E). The former corresponds to the initial IRI considered in ME prediction, while the latter is related to the slope of the deterioration curve, with lower strength modulus leading to a more rapid deterioration and therefore steeper slope in the curve.

To analyze the impact of these two input parameters (I-IRI and E) in the predicted IRI, the research team defined three levels (i.e., low, medium, high) for each of these parameters based on the actual performance data and the backcalculated modulus values (E_{FWD}). To determine the low, medium, and high levels for I-IRI and E_{FWD}, the research team defined three groups with the medium group comprised of data within the 25th and 75th percentile of data, the high group comprised of data

[1] Note: Long-term difference is estimated based on the area depicted in Figure 14.

above the 75th percentile, and the low group comprised of values below the 25th percentile of the data. The threshold values defining each of these groups are summarized in Table 12.

Table 12. Grouping criteria to determine low, medium, and high values of initial IRI and strength modulus

Group	I-IRI (in/m)					E_{FWD} (ksi)				
Low	I-IRI	<	54			E_{FWD}	<	19		
Medium	54	≤	I-IRI	≤	59	19	≤	E_{FWD}	≤	95
High	I-IRI	>	59			E_{FWD}	>	95		

For each of these groups, average values were obtained to determine representative low, medium, and high values of I-IRI and E_{FWD}. The results of these calculations are summarized in Table 13, with the last column summarizing the average values that represent low, medium, and high values of initial IRI and strength modulus in the dataset.

Table 13. Low, medium, and high values of initial IRI and strength modulus

Parameter	Group	Site 1	2	3	4	5	6	7	8	9	10	11	Average
E_{FWD} (ksi)	Low	15	14								18		16
	Medium			41	51			91	20	49			50
	High					256	100					124	160
I-IRI (in/m)	Low					46		41				NA	44
	Medium		56	54	56		54			58		NA	56
	High	62							59		69	NA	63

NOTE: NA is not available

Sensitivity Analysis to Initial IRI Values

Based on the low, medium, and high values of initial IRI obtained above, the research team performed ME simulations to understand the sensitivity of IRI predictions to initial IRI values. In these simulations, the strength modulus values (E) were held constant at the value currently considered for design (E_{design} = 30 ksi).

Figure 16 shows an example of this simulation for site 4, showing that changes in initial IRI values lead to vertical offsets of deterioration curves. The performance curves of other sites were generally similar to the one depicted in Figure 16 for site 4, characterized by a sudden increase (i.e., 'jump') in initial IRI values. Some of the sites (i.e., 1, 6, and 9) have multiple 'jumps' in the predicted IRI values (e.g., site 1 plotted in Figure 17). Based on conversations with ARA, the consulting company in charge of the technical support of AASHTOWare, these sudden increases in IRI predictions are a result of adjustments made by the software to ensure the reliability levels of the simulation are met. In our simulations, reliability is 90%, as per CDOT recommendations for ME design (CDOT 2019).

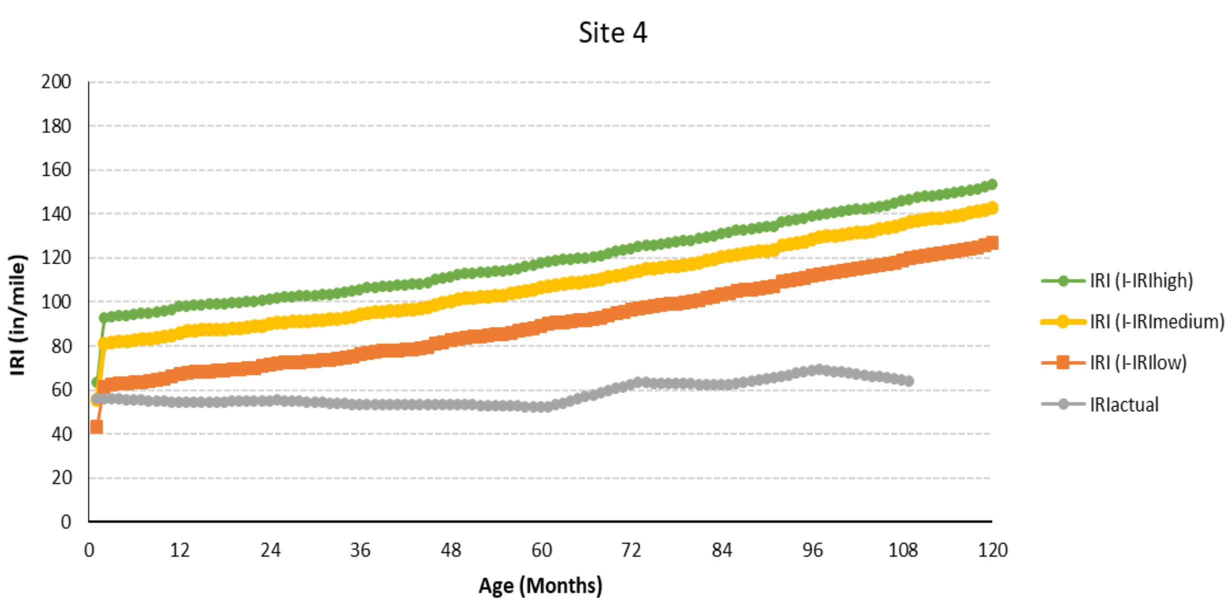


Figure 15. Sensitivity analysis of IRI predictions to changes in initial IRI - Site 4

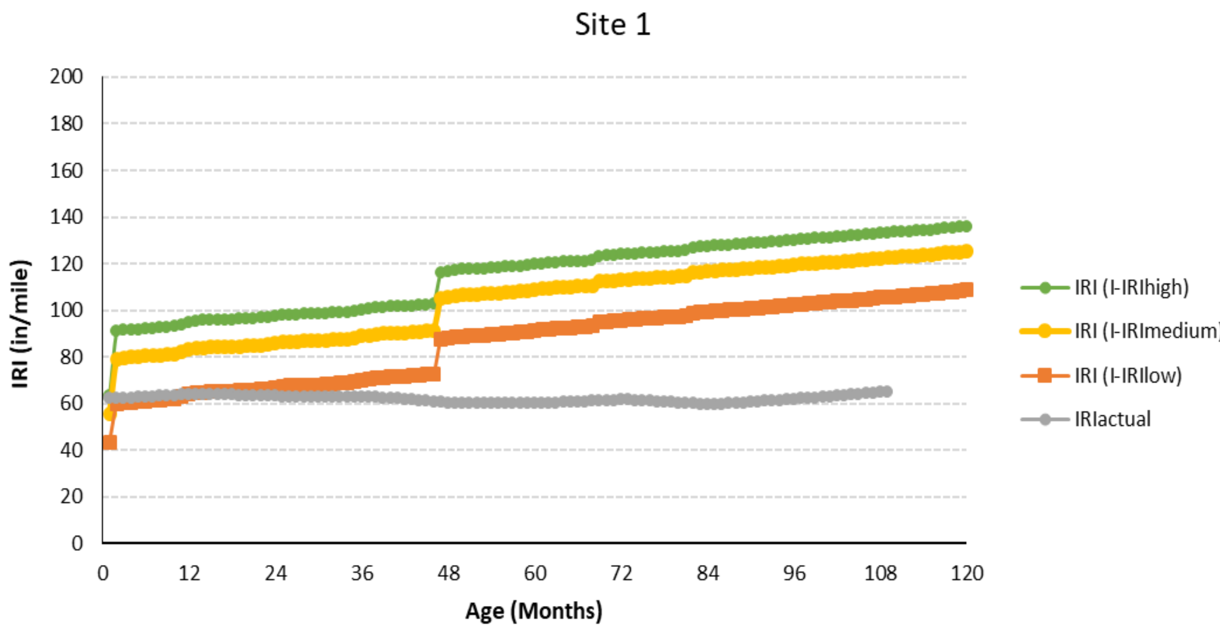


Figure 16. Sensitivity analysis of IRI predictions to changes in initial IRI - Site 1

The results obtained in this sensitivity analysis suggest that initial IRI values can be calibrated based on actual IRI measures so that the predicted IRI using ME matches the actual deterioration of each of the sites.

Sensitivity Analysis to Strength Modulus Values

To analyze the sensitivity of IRI predictions to strength modulus values, the research team run three models for each site, each of them considering low, medium, and high values of the strength modulus as summarized in Table 13 and keeping initial IRI to the actual value at the beginning of the analysis period.

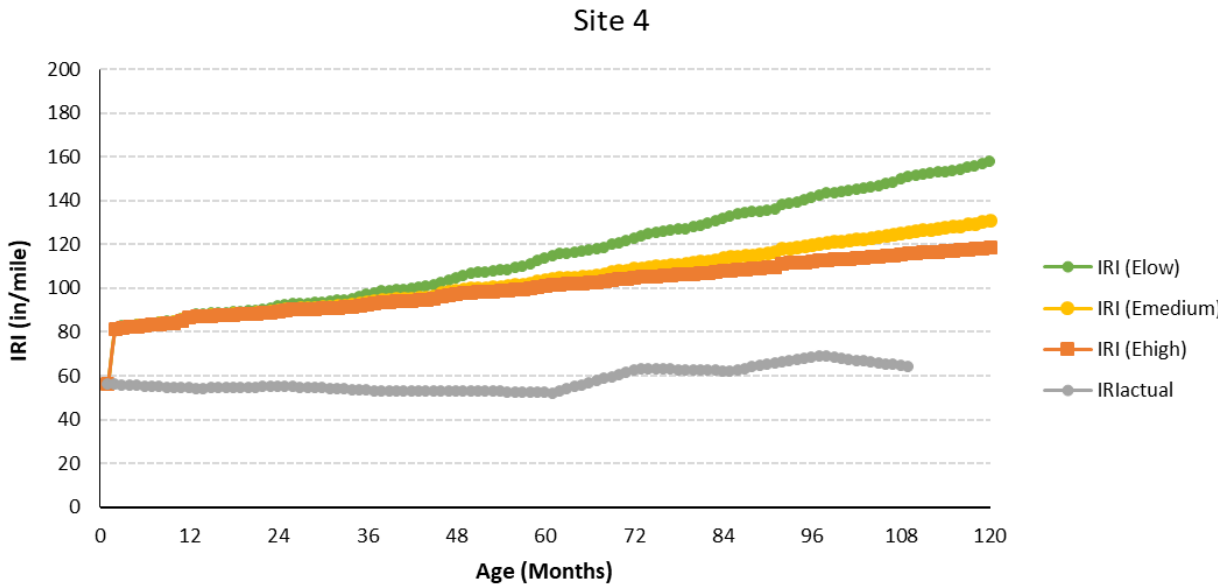


Figure 17. Sensitivity analysis of IRI predictions to changes in FDR modulus - Site 4

The results from the sensitivity analysis of IRI predictions to changes in the strength value of the FDR layer (Figure 18) support the intuition that changes in the strength values result in different slopes of the deterioration curve. As expected, higher strength values result in a slower IRI deterioration characterized by a gentle slope in the deterioration curve.

Recommended ME Input Values

Although the initial objective of this project was to derive recommendations of strength values for FDR layers, the sensitivity analysis showed that ME predictions are impacted not only by the strength modulus but also by the initial IRI parameter. It is therefore needed to recommend both input parameters (i.e., initial IRI and strength modulus) as input values for the ME design of FDR projects.

The research team performed a two-step calibration. First, the initial IRI was calibrated for each of the sites to minimize the vertical offset in IRI predictions and ensure that the predicted deterioration at the beginning of the analysis period was close to the actual deterioration. Once the initial IRI was calibrated for each of the sites, the strength modulus was calibrated to ensure the deterioration trend was closer to actual values.

Recommended Initial IRI

The results from the sensitivity analysis informed the initial IRI values to recommend for each site. These values were obtained by interpolation to ensure that the predicted IRI at the beginning of the analysis period was similar to the actual IRI value. As mentioned before, some of the sites had a sudden increase (i.e., jump) in the predicted IRI during the first month. To account for this and minimize its impact in the recommendation of input parameters, initial IRI was calibrated based on the IRI values in month 1. Following this method, the recommended initial IRI values for each of the sites are summarized in Table 14.

Table 14. Proposed initial IRI (I-IRI) values for each of the sites

Site	**$I\text{-}IRI_{proposed}$ (inch/mile)**
1	45.04
2	42.01
3	41.09
4	39.99
5	33.68
6	41.46
7	31.39
8	44.4
9	42.21
10	48.9
11	N/A
Median	41.74
Average	41.02

A statistical analysis of the initial IRI recommended values for each of the sites shows that the median and average values are close and therefore, the distribution of I-IRI is fairly symmetrical. This indicates that the median value of 42 (41.74 rounded up) is a good representative of all the sites. Hence, an I-IRI value of 42 inch/mile is recommended to be used in ME design of flexible pavements rehabilitated with dry or emulsion based FDR layer.

Strength Modulus Minimizing Long-Term Difference

To recommend strength modulus values, the research team calculated through interpolation which would need to be the strength modulus that minimize the long-term difference between predicted and actual IRI. This long-term difference was to be estimated based on the area between actual and predicted deterioration (Figure 14). Table 15 summarizes the strength modulus values for the FDR layer that minimize the long-term difference between actual and predicted IRI.

Table 15. Strength modulus values minimizing long-term difference in IRI prediction

Site	1	2	3	4	5	6	7	8	9	10	11
$E_{min\ LTD}$ (ksi)	2,042	838	64	233	138	332	58	385	375	119	N/A

From the results obtained in this calibration, it can be concluded that the strength values needed to minimize the difference between actual and predicted IRI are significantly higher than the strength modulus value currently used for FDR design (E_{design} = 30 ksi). In some cases, the strength values may even be considered unrealistic (e.g., site 1). This is probably due to the fact that this recommendation is derived from a mathematical perspective (i.e., these values are the ones minimizing the area between the actual and predicted IRI) but do not take into account the fact that this values may not seem reasonable from an engineering perspective. The research team further analyzed these results to derive more realistic recommendations by analyzing the overall calibration and recommended values.

Recommended Values at the Project Level

New simulations of predicted deterioration were run considering the input parameter values obtained in the two-steps calibration. In these simulations, an initial IRI value of 42 in/mi was considered for all the sites, whereas the input strength modulus value were the ones summarized in Table 15. These simulations were performed to check the validity of the two-steps calibration approach.

Table 16. Long-term difference (LTD) between actual performance, current design, and calibrated input parameters[2]

Difference	Site										
	1	2	3	4	5	6	7	8	9	10	11
$LTD_{design-actual}$	5,333	5,278	5,721	6,097	9,055	3,513	2,961	3,294	4,308	532	NA
$LTD_{calib.-actual}$	2,097	1,440	1,712	2,130	2,719	1,697	1,001	-92	1,597	-248	NA

The long-term difference between predicted and actual deterioration has improved significantly when compared to the current design, but actual deterioration is still lower than the ME predicted values (Table 16). These results indicate that the proposed strength values summarized in Table 15 are still underestimating the performance of FDR projects resulting in higher deterioration predictions than actual measures. To improve this prediction, the recommended values of strength modulus would need to be even higher to the ones listed in Table 15, which can already be considered as unrealistic because of their high values. This is because the deterioration trend has low sensitivity to strength modulus values. High changes in the strength modulus only result in small changes in the deterioration trend and, therefore, the area under the curve. Figure 19 shows an example of this for site 4 and how an increase of the strength modulus by more than 400% results in a change in the area of only 8%.

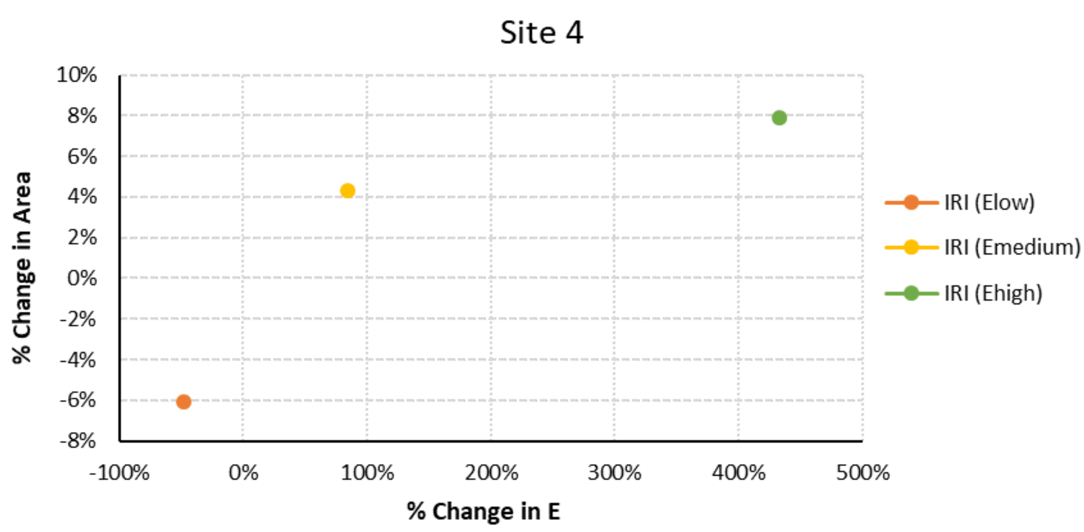


Figure 18. Percentage change in the area under the curve with changes in modulus

[2] Note: Long-term difference is estimated based on the area depicted in Figure 14.

From these results it can be concluded that, under the current models in ME, the strength modulus of the FDR layer would need to be increased to unreasonable values to ensure the ME predicted IRI matches the actual IRI deterioration of FDR sites. This may be due to the fact that current AASHTOWare software does not have a module to include FDR layers in pavement design. Because of this limitation, pavement designers need to model FDR layers either as a granular layer or a Hot Mix Asphalt (HMA) layer. Because of their configuration, the behavior of FDR layers can be expected to lay within these two models (i.e., granular or HMA). Non-stabilized FDR layers (i.e., dry layers) would probably show a deterioration closer to the one of granular layers; whereas the deterioration of emulsion-stabilized FDR would probably be closer to HMA layers. However, the simulations performed in this study have shown the limitations of current AASHTOWare models to adequately predict the deterioration of FDR layers and the need of future research in this topic.

Because of the limitations of current AASHTOWare software to model FDR layers, it seems reasonable to base the recommendations of FDR strength modulus values on the back-calculated modulus obtained from FWD testing, as these are the most realistic values available. Following AASHTO's recommendations (2015), the back-calculated modulus obtained from deflection data for the FDR layer (Table 8) were adjusted to laboratory conditions of resilient modulus for M-E design by applying a coefficient of 0.62. These values are summarized in Table 17. This table also summarizes relevant project characteristics (i.e., FDR stabilization, FDR thickness, and backcalculated modulus of the subgrade) that will be explored in the following section to understand their impact on FDR resilient modulus.

Table 17. Resilient modulus values for FDR sites

ID	M_r for FDR M-E design (ksi)	FDR Stabilization	FDR thickness (inches)	E_{FWD} subgrade (ksi)
1	15.0	Non-stab.	6	34.5
2	14.3	Non-stab.	6	14.1
3	40.9	Non-stab.	6	34.4
4	256.2	Non-stab.	5	29.1

ID	M_r for FDR M-E design (ksi)	FDR Stabilization	FDR thickness (inches)	E_{FWD} subgrade (ksi)
5	50.7	Emulsion-stab.	6	30.1
6	99.6	Emulsion-stab.	5	107.6
7	90.8	Non-stab.	10	13.9
8	19.7	Non-stab.	10	17.0
9	49.5	Non-stab.	6	21.7
10	17.9	Non-stab.	12	19.2
11	124.2	Emulsion-stab.	8	50.4

Project Characteristics and their Impact on ME Input Values

This section analyzes the impact of project characteristics on the resilient modulus of FDR layers. Specifically, three characteristics are analyzed: the FDR stabilization, FDR thickness, and the modulus values of the subgrade. The research team was not able to analyze the impact of the milled HMA pavement (i.e., performance grade and asphalt percentage) because this information was not available form the laboratory testing.

FDR Stabilizer

Figure 20 shows that stabilizers have a significant impact on the resilient modulus values of FDR layers. Emulsion-stabilized FDR result on stiffer layers than non-stabilized projects (also known as dry projects). From these results it can be concluded that the recommended resilient modulus values should be different for non-stabilized and emulsion-stabilized FDR.

Figure 19: Resilient modulus of FDR layers with different stabilizers

FDR thickness

This project found no significant effect of FDR thickness on resilient modulus values. The results of this analysis (summarized in Figure 20) show that there is not a clear trend on how FDR thickness impact modulus values. Resilient modulus values seem to increase with FDR thickness in dry FDR projects, but decrease for emulsion-stabilized FDR. No clear conclusions can be derived on this issue, partially because of the small size of the sample—the majority of projects were built with the same thickness (6‘’), with only two projects having a thickness of 10’’ and two more projects having a thickness of 12’’. More data would be needed to analyze the impact of FDR thickness on modulus values.

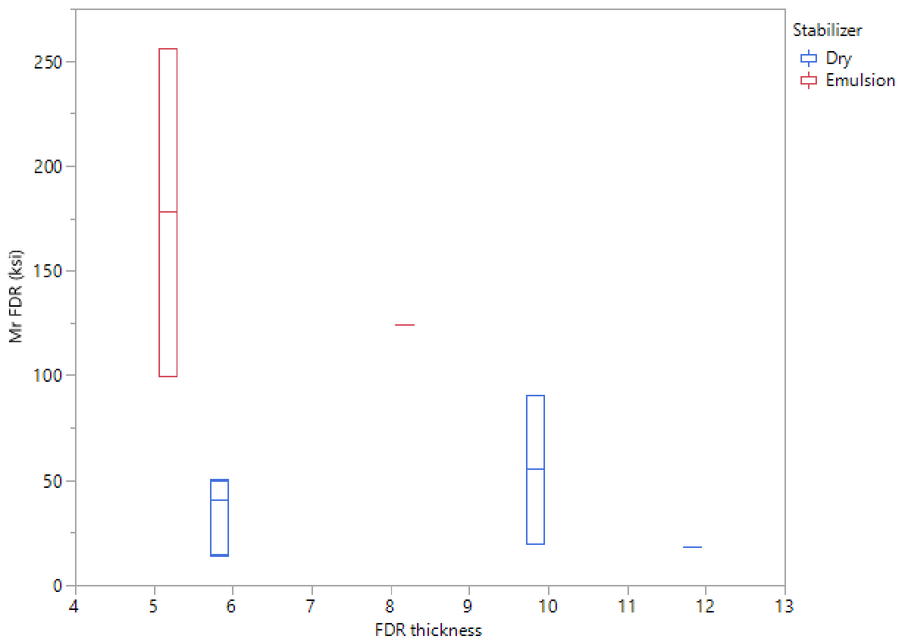


Figure 20. Resilient modulus of FDR layers with different stabilizers and FDR thickness

Subgrade characteristics

Finally, the research team analyzed the impact of the subgrade characteristics on the FDR modulus. This analysis is important because in the FDR construction process, part of the subgrade is milled and incorporated to the new FDR layer. Therefore, the characteristics of the subgrade can be expected to have a significant impact on the characteristics of the resulting FDR layer.

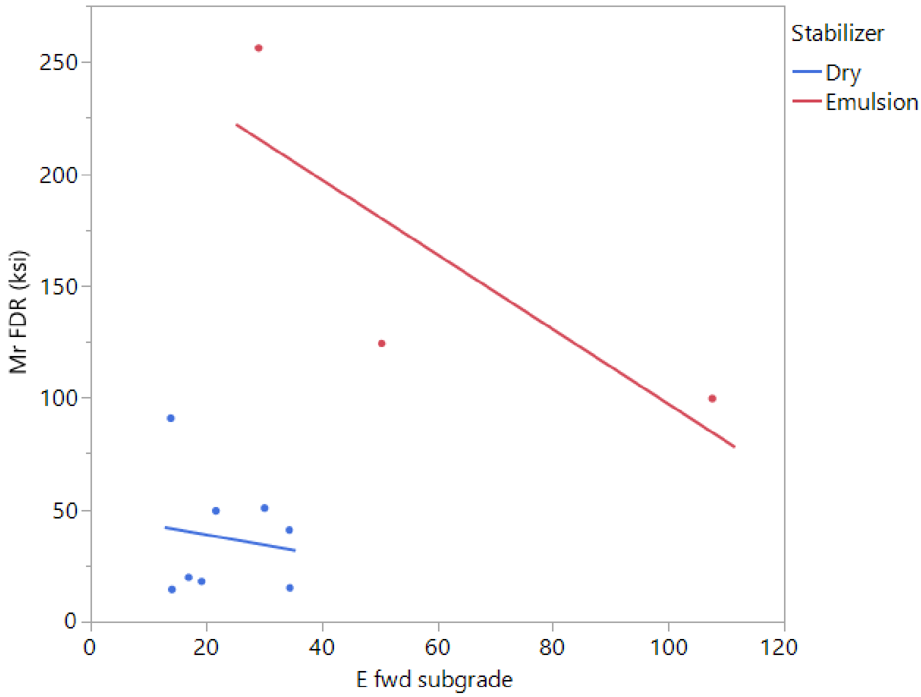


Figure 21. Relation between the resilient modulus of the subgrade and FDR layers

Based on the construction process of FDR, one would expect that higher subgrade modulus will result in higher FDR modulus. This is, however, not the trend found in this research. Actually, the results plotted in Figure 21 do not suggest a reasonable correlation between the subgrade and FDR modulus. Both dry and emulsion-stabilized projects show a negative correlation between subgrade and FDR modulus. These results are counterintuitive and may derive from errors in the characterization of the subgrade modulus. These errors are suggested by the fact that emulsion-stabilized FDR projects (depicted in red in Figure 21) show higher values of subgrade modulus than non-stabilized projects (depicted in blue in Figure 21). From an engineering perspective, emulsion-stabilized projects should not necessarily have higher subgrade modulus. In fact, the opposite result may seem more reasonable—as emulsion-stabilization would be applied to improve the characteristics of a poor subgrade. The underlying reasons for the mischaracterization of the subgrade modulus, may relate to the method used to determine the subgrade modulus. These values were backcalculated form FWD deflections. Therefore, the modulus results of the subgrade layer may have been affected by the deflections in the FDR layers. Due to the limited laboratory results to characterize the subgrade, it is not possible to derive clear reasons for these results nor conclude on the validity of the backcalculated subgrade modulus.

Considering these limitations and the counterintuitive results obtained in this analysis, it is not possible to derive recommendations on how the subgrade characteristics may affect the resilient modulus values of FDR projects.

Recommendation of ME Input Values for Future FDR Projects

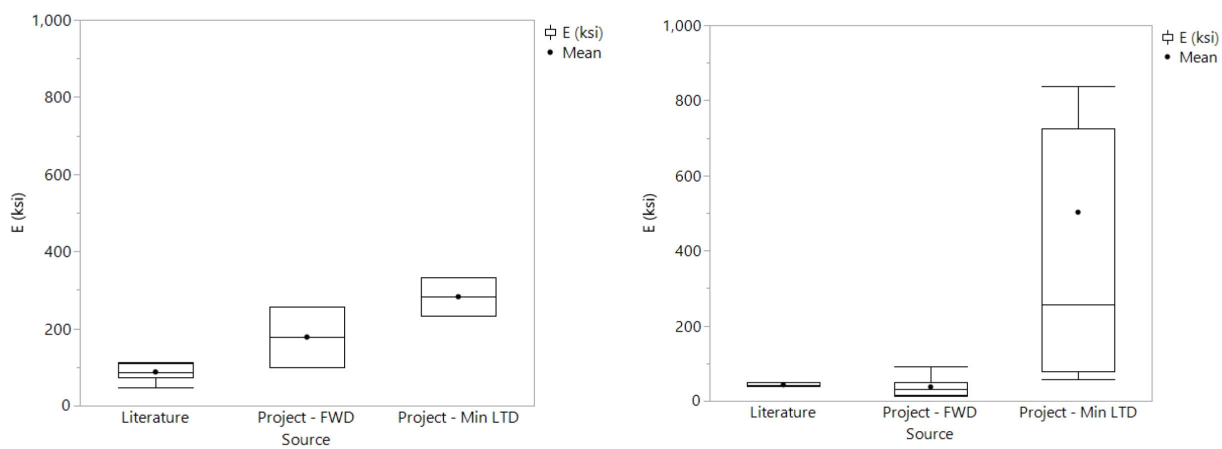
Based on these results, the research team recommends using back-calculated strength values for the future design of FDR layers in Colorado. The analysis of project characteristics showed that stabilizers resulted in significant differences in modulus values. Therefore, different recommendations are derived for non-stabilized and emulsion-stabilized FDR. The analysis of the impact of thickness and subgrade modulus were inconclusive and are thus not included in the recommendation.

The recommended input values for the ME design of FDR layers are summarized in Table 17. This recommendation consists on an average value and a range obtained from the 25th and 75th percentile of the backcalculated values. When compared to the current values used by CDOT in ME, the initial IRI is recommended to be reduced from 62 to 42 in/mile; whereas the strength of FDR layers are recommended to be increased from 30,000 psi to 37,300 psi for non-stabilized FDR and to 160,000 psi for emulsion-stabilized FDR. Although the proposed values recommended above are the average values, designers could consider using a range of resilient modulus to account for different site characteristics. In this case, the recommended range of resilient modulus values would be 16,000 to 50,000 for non-stabilized FDR and 100,000 to 256,000 for emulsion-stabilized FDR.

Table 18. Recommended input parameters for ME design of FDR projects in Colorado

Input ME parameter	Non-stabilized FDR (i.e., dry FDR)	Emulsion-stabilized FDR (i.e., wet FDR)
Initial IRI (in/mi)	42	42
Mr average (psi)	37,300	160,000
Mr range (psi)	16,000 – 50,000	100,000 – 256,000

The proposed values of strength modulus were compared with values found in the literature (Amarh et al. 2017; Johanneck and Dai 2013; Kroge et al. 2009; Mallick et al. 2002) and the calibrated values that minimized the long-term difference of predicted and actual IRI (summarized in Table 15). This comparison is plotted in Figure 20. It is worth mentioning that the strength values included in Figure 20 correspond to values for ME input. Therefore, the back-calculated modulus values (E_{FWD}) obtained from deflection data are adjusted to laboratory conditions of resilient modulus by applying a coefficient of 0.62, as recommended by AASHTO (2015). The values obtained from the literature were also weighted by the same coefficient in those cases in which the modulus was obtained from deflection measures. The comparative analysis (Figure 20) shows that the recommended values (depicted by the mean value in the "Project-FWD" box plot in Figure 20) are slightly higher than the ones found in the literature but seem more reasonable than the values obtained through the minimization of the long-term deterioration (Project - Min LTD in Figure 20).



a) Emulsion-stabilized FDR

b) Non-stabilized FDR

Figure 22. Comparative analysis of modulus values in literature and this project

To summarize the improvements in the ME prediction derived from the proposed input parameters, Table 18 summarizes the difference between actual IRI and M-E predictions obtained with current design parameters and the proposed parameters recommended in this study. With the current design parameters, IRI predictions were on average overestimated by 51 in/mile. The proposed input parameters allow to reduce this difference to 17 in/mile. In terms of the long-term difference

(LTD in Table 18), the current design is underestimating the performance of FDR by an average of 88%, whereas the proposed parameters reduce this long-term difference to 30%.

Table 19. Comparison of actual IRI and IRI predictions obtained from current design and recommended input parameters

Site	Area under IRI curve (A)			Long-term difference (LTD)		Average IRI difference	
	A_{design}	$A_{proposed}$	A_{actual}	$LTD_{design-actual}$	$LTD_{proposed-actual}$	$IRI_{design-actual}$	$IRI_{proposed-actual}$
1	12,012	8,823	6,679	5,333	2,144	50	20
2	12,566	8,938	7,288	5,278	1,650	44	14
3	12,136	8,452	6,415	5,721	2,037	48	17
4	12,350	9,008	6,253	6,097	2,755	57	26
5	13,731	7,326	4,676	9,055	2,650	95	28
6	6,872	5,074	3,359	3,513	1,715	59	29
7	6,148	4,273	3,187	2,961	1,086	50	18
8	10,729	7,415	7,435	3,294	-20	31	0
9	9,563	7,071	5,255	4,308	1,816	52	22
10	2,272	1,515	1,740	532	-225	23	-9
11	NA	NA	NA	NA	NA	NA	NA
Average	9,838	6,790	5,229	4,609	1,561	51	17

NOTE: Results are not available for site 11 because only one data point of actual IRI is available

Figure 21 summarizes actual IRI and IRI predictions obtained with the proposed ME input parameters (summarized in Table 17) and current design for site 4. Similar figures for all the sites are included in Appendix B. Based on these results, it can be concluded that the proposed input parameters lead to a conservative design of FDR projects and result in improved IRI predictions than the ones obtained with current design.

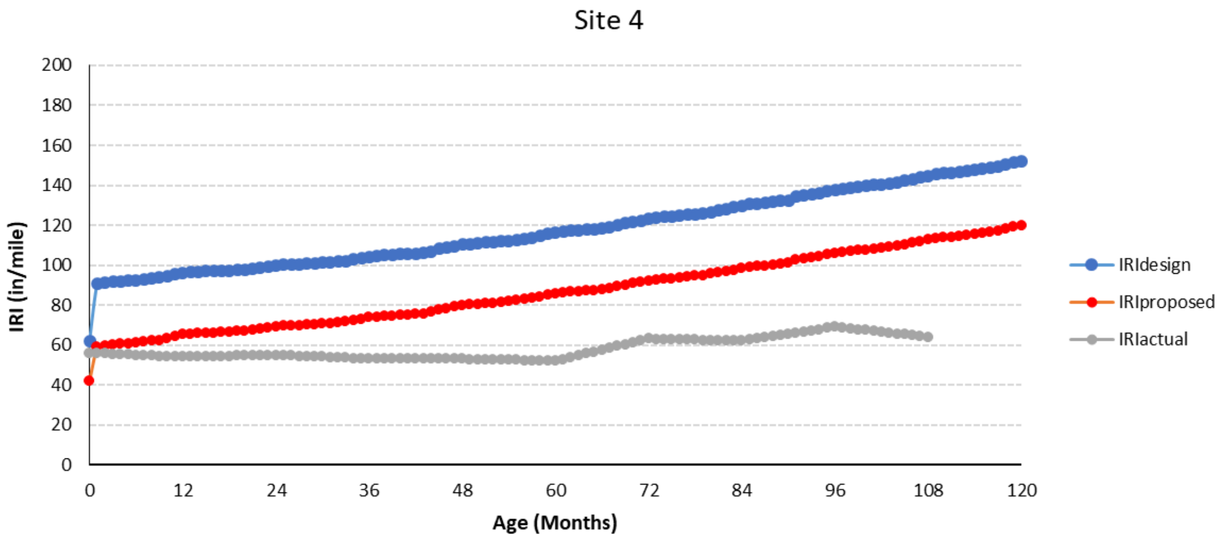


Figure 23. Comparison of actual IRI and predictions from current design and proposed parameters

Conclusions and Recommendations

From the results obtained in this study it can be concluded that current design inputs are conservative and lead to an underestimation of the actual performance of FDR projects. With current design input parameters (strength modulus for FDR layer of 30,000 psi and initial IRI of 62 in/mile), IRI is on average overestimated by 51 in/mile in the 10 years design-period. These results are aligned with the results obtained by previous research.

The research team analyzed the sensitivity of ME predictions to changes in two input parameters: initial IRI (I-IRI) and strength modulus (E). The results from this sensitivity analysis informed the procedure to follow for the calibration of input parameters, which consisted of a two-step process. First, the initial IRI was calibrated for each of the sites to minimize the vertical offset in IRI predictions and ensure that predicted IRI at the beginning of the analysis period was similar to the actual deterioration. Once the initial IRI was calibrated for each of the sites, the strength modulus was calibrated to ensure a deterioration trend similar to actual values.

Following this procedure, initial IRI values were calibrated for all the sites. The median value of the initial IRI obtained in this calibration (42 in/mile) was representative for all the sites and is recommended as a new input parameter for future FDR design. The calibration of the strength modulus seeking to minimize the difference between actual and predicted IRI led to very high values of strength modulus that even seemed unrealistic. This is because the deterioration trend has low sensitivity to strength modulus values. As a result, high changes in the strength modulus only result in small changes in the deterioration trend and, therefore, the area under the curve.

From these results it can be concluded that, under the current models in ME, the strength modulus of the FDR layer would need to be increased to unreasonable values to ensure the ME predicted IRI matches the actual IRI deterioration of FDR sites. This may be due to the fact that current AASHTOWare software does not have a module to include FDR layers in pavement design. The simulations performed in this study have shown the limitations of current AASHTOWare models to adequately predict the deterioration of FDR layers and the need of future research in this topic.

Because of the limitations of current AASHTOWare software to model FDR layers, it seems reasonable to base the recommendations of FDR strength modulus values on the back-calculated modulus obtained from FWD testing, as these are the most realistic values available.

The research team analyzed the impact of project characteristics (namely stabilizer, FDR thickness, and subgrade modulus) in the resilient modulus of FDR layers. Additional features such as the characteristics of the milled HMA in the FDR performance were not analyzed due to the lack of relevant information to characterize this material (e.g., performance grade, asphalt content, etc.) The analysis found stabilizers have a significant impact on the resilient modulus values of FDR layers. No statistical significant differences were found on the analysis of FDR thickness and modulus subgrade. Therefore, it was no possible to derive conclusions nor recommendations on the impact of FDR thickness nor subgrade modulus on the resilient modulus of FDR layers.

Based on these results, the research team recommends to use rounded average back-calculated strength values for the future design of FDR layers in Colorado. The strength modulus of FDR layers are recommended to be increased from 30,000 psi to 37,300 psi for non-stabilized FDR and to 160,000 psi for emulsion-stabilized FDR. Designers could also consider using a range of resilient modulus values to account for different project characteristics. In this case, the recommended range of resilient modulus values would be 16,000 to 50,000 for non-stabilized FDR and 100,000 to 256,000 for emulsion-stabilized FDR.

The recommended values of strength modulus are slightly higher than values found in the literature and allow to improve IRI predictions. Based on these results, it can be concluded that the proposed input parameters lead to a conservative design of FDR projects and result in improved IRI predictions than the ones obtained with current design criteria.

Chapter 5. Conclusions and Recommendations

Conclusions

The main objective of this research was to establish standard modulus values for various FDR base materials that can be confidently and effectively used by CDOT pavement designers as reliable input to designing pavements utilizing the ME pavement design.

To achieve this objective, the research first synthesized the methods used by other State Transportation Agencies (STAs) to assign structural strength to FDR materials. Although FDR is commonly referred in the STAs' Construction Standards and Specifications and Pavement Design Manuals, this research found that no states have locally calibrated ME coefficients for FDR design. Although some STAs recommend FDR ME coefficients in their pavement design manuals, these coefficients have been calibrated at the national level. No STAs were found that suggested the use of state-calibrated FDR coefficients in their ME design manuals. Therefore, Colorado is a leading state aiming to recommend ME coefficients based on state-level FDR projects.

Although the work plan considered an ambitious plan to characterize FDR sections from cores samples in 11 sites in Colorado, the research team encountered challenges that limited the final amount of information available. The major challenge was related to the difficulty of obtaining intact cores of the FDR, which limited the laboratory testing that could be performed. Asphalt emulsion FDR sections shown a better stability than dry FDR sections because the emulsion acted as a bonding agent. Among the tests that could not be performed, it is worth noting the lack of reliable results from dynamic modulus (AASHTO TP62) and resilient modulus (AASHTO T307), as these are key input parameters to model FDR layers in ME design.

Back-calculated stiffness from FWD was the only test that could be performed for all of the sites. With so limited experimental data on stiffness modulus from resilient and dynamic modulus, the research team decided to rely on back-calculated stiffness modulus from FWD testing as the only experimental measure of stiffness. Back-calculated stiffness values of non-stabilized FDR were between 23 and 146 ksi, while values in stabilized sections ranged from 160.7 and 413 ksi.

From the analysis of long-term performance data, the research team found that emulsion-stabilized sites perform better over time than non-stabilized sections in terms of IRI. In general terms, FDR sites show a good performance over time, with a low increase of IRI in the 10 years design period (i.e., in median values, IRI increases from 54 to 65 in/mile in 10 years). From the analysis of IRI data, the research team proposed a set of IRI deterioration trends for each of the 11 sections in the project. These curves are the ones compared with ME predictions for the recommendation of stiffness values.

From the comparison of actual IRI and ME predictions based on current design parameters it can be concluded that current design inputs are conservative and lead to an underestimation of the actual performance of FDR projects. With current design input parameters (strength modulus for FDR layer of 30,000 psi and initial IRI of 62 in/mile), IRI is on average overestimated in the 10 years design-period by 51 in/mile.

Both initial IRI and stiffness modulus were found to have a significant impact on ME predictions. These two parameters were calibrated in a two-step process. First, the initial IRI is calibrated for each of the sites to minimize the vertical offset in IRI predictions and ensure that the predicted deterioration at the beginning of the analysis period is similar to the actual deterioration. Once the initial IRI is calibrated for each of the sites, the strength modulus is calibrated to ensure the deterioration trend is similar to actual values.

The median value of the initial IRI obtained in this calibration (42 in/mile) was representative for all the sites and is recommended as a new input parameter for future FDR design. Because of the low sensitivity in IRI predictions to strength modulus values, the calibration of the strength modulus seeking to minimize the difference between actual and predicted IRI led to very high values of strength modulus that could even be seemed unrealistic.

From these results it can be concluded that, under the current models in ME, the strength modulus of the FDR layer would need to be increased to unreasonable values to ensure the ME predicted IRI matches the actual IRI deterioration of FDR sites. This may be due to the fact that current AASHTOWare software does not have a module to include FDR layers in pavement design. The simulations performed in this study have shown the limitations of current AASHTOWare models to adequately predict the deterioration of FDR layers and the need of future research in this topic.

Because of the limitations of current AASHTOWare software to model FDR layers, the recommendations of FDR strength modulus values were based on the back-calculated modulus obtained from FWD testing, as these are the most realistic values available on the strength of FDR layers. The research team recommends to use rounded average back-calculated strength values for the future design of FDR layers in Colorado. The strength modulus of FDR layers are recommended to be increased from 30,000 psi to 37,300 psi for non-stabilized FDR and to 160,000 psi for emulsion-stabilized FDR. The proposed input parameters lead to a conservative design of FDR projects and result in improved IRI predictions than the ones obtained with current design criteria.

Recommendations

Based on the results and findings of this study, the research team derives the following recommendations for CDOT consideration related to the future design of FDR:

- For future ME design of FDR sections in Colorado, the research team recommends to increase the modulus values of FDR layers from 30,000 psi to 37,300 psi for non-stabilized FDR and to 160,000 psi for emulsion-stabilized FDR.
- To reduce the offset in IRI predictions at the beginning of the analysis period, the research team recommends to reduce the initial IRI in FDR design from 62 in/mile to 42 in/mile.

The analysis of actual performance of FDR sites revealed some inconsistencies in deterioration trends. In some sections, condition improved with no record of maintenance or rehabilitation activities taking place. To overcome this, the research team would recommend to ensure maintenance and rehabilitation activities are properly recorded in CDOT's Pavement Management System. Special attention should also be paid to historical data on rutting and cracking, which showed inconsistent and null values for all the sites. The research team would recommend CDOT to review the calibration of the equipment used in pavement survey to minimize errors in data collection.

Most of the sites included in this study consisted of non-stabilized FDR sections (i.e., dry FDR). These sections showed higher deterioration levels than emulsion-based FDR sections. Based on these results, CDOT may consider extending the application of emulsion stabilizer agents in future FDR projects and even consider applying chemical stabilization (i.e., Portland cement, lime, etc.) to further enhance the properties of FDR projects.

Future Research

This research effort highlighted need for future research in several areas. The first one is related to the historical database available on FDR performance. CDOT has historically been a pioneer state in the construction of FDR rehabilitation projects, highlighting thus CDOT's commitment with pavement recycling and sustainability. To continue this effort, it would be worthy to continue monitoring the performance of FDR sections so that a larger database would be available in the future to understand the long-term performance of FDR projects.

Because of the limitations of current AASHTOWare version to model FDR performance, research is needed to develop deterioration models of FDR sections. There is therefore an opportunity to develop empirical models of IRI deterioration for FDR projects in Colorado based on the historical information available from CDOT projects. Different techniques could be used for the development of these methods: Markov models, regression models, etc. This would allow CDOT to have user-friendly, state-calibrated, and reliable models to predict FDR performance.

In this project, emulsion stabilized FDR showed better performance than traditional non-stabilized sections, which have commonly been applied in Colorado. To further explore the capabilities of stabilized FDR and the use of chemical stabilizers, CDOT may consider further research to optimize the design of chemically stabilized FDR in Colorado.

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Appendices

Appendix A: Design, Construction, and Maintenance Additional Information

Appendix B: Comparison of actual and ME predicted performance based on current design and proposed input parameters

Appendix A: Design, Construction, and Maintenance Additional Information

Table A-1: Construction data layer for FDR sites

Site	Layer 1 (Top layer)	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6	Subgrade Soil
1	2" SX 75 PG 64-28	2" PG 64-28	2" PG 64-22	8" FDR		3.5" HMA, 4" base, 6" Subbase (1968)	A-2-4(0) R = 78 (1968)
2	2" PG 58-34	2" PG 58-28	4" ABC Class 6	8" FDR (Estimated)	Seal Coat (1974)	6" Base Surfacing (1936)	
3	2" PG 58-34	2" PG 58-34	8" FDR			4" HMA (1958) 4" Base & 4" Sub Base (1958)	A-1-b(0) R = 80 (1958)
4	2" PG 64 - 28	2" PG 64-22	8'' FDR Estimated	2.25" Grading C (1993)	4" Cold-in-place recycling (1993)	2.5" HMA, 2.5" PMBB, over 4" existing HMA (1978)	A-2-4(0) Estimated
5	2" PG 64 - 28	2" PG 64-22	8' FDR Estimated	2.25" Grading C (1993)	4" Cold-in-place recycling (1993)		
6	2" PG 64-22	2" PG 64-22	Approx. 9" FDR with 1" ABC added			4" HMA, 7" ABC (1961)	A-1-a(0) (2010)
7	2" PG 64-28	2" PG 64-22	12" FDR	2" Grading CX (1993)		6" HMA, 4" Sub Base (1957)	A-6(10) CBR 4.6 (1957)
8	2" PG 58-28	3" PG 58-28	10" with Magnesium Chloride on surface			6" HMA, 4" Sub Base (1938)	No Data
9	2" 58-34	3" PG 58-28	8' FDR estimated	1.5" Grading CX (1994)	1.5" HMA Overlay (1980)	2" HMA, 4" Balast (1947)	A-4-7 (1947)
10	2" PG 64-28	3" PG 58-28	Approx. 8" FDR topped with about 2" ABC Class 6	1" Leveling Grading SX(76) AC-10 topped by 2" Grading SX(76) AC-20R (1997)		3" HMA, 8" Base, 18" Embankment (1968)	A-6(10) CBR 3.0 (1968)

11	2" PG 64-28	2" PG 64-28	9" FDR with emulsion			9" HMA and 6" Base	A-1-b(0) R = 77 (2014)

Table A-2: Core information from sites 1 to 4

Site 1			Site 2			Site 3			Site 4		
Core	**Depth to Subgrade (in)**	**HMA Thickness (in)**	**Core**	**Depth to Subgrade (in)**	**HMA Thickness (in)**	**Core**	**Depth to Subgrade (in)**	**HMA Thickness (in)**	**Core**	**Depth to Subgrade (in)**	**HMA Thickness (in)**
1-1	13	NA	2-1	11-1/4	NA	3-1	10	5	4-1	10-1/2	4
1-2	12	NA	2-2	7-7/8	NA	3-2	10	5-1/2	4-2	10	3-1/2
1-3	12	NA	2-3	11-3/4	NA	3-3	10	NA	4-3	10	3-1/2
1-4	12	NA	2-4	9-3/8	NA	3-4	9-3/4	6	4-4	10	3-1/2
1-5	12 - 1/2	NA	2-5	9-7/8	NA	3-5	10-1/2	6	4-5	9-1/2	4
1-6	12 - 1/2	NA	2-6	8-5/8	NA	3-6	9-1/2	4-1/2	4-6	10	4
1-7	13	NA	2-7	9-3/8	NA	3-7	9	4	4-7	10-1/2	5-1/2
1-8	13	NA	2-8	9	NA	3-8	9-1/2	4	4-8	10	4
1-9	13	NA	2-9	9-3/8	NA	3-9	9-1/2	4	4-9	9	3-3/4
1-10	12 - 1/2	NA	2-10	10-1/8	NA	3-10	9	4	4-10	10-1/2	4-3/4
1-11	13	NA	2-11	9-3/4	NA	3-11	10-3/4	4	4-11	8	3-3/4
1-12	12	NA	2-12	8-1/8	NA	3-12	9	4	4-12	10-1/2	4
1-13	13 - 1/2	NA	2-13	9-1/4	NA	3-13	10	4	4-13	11	3-1/2
1-14	12	NA	2-14	9-3/4	NA	3-14	10	5	4-14	11-1/2	4
1-15	12 - 1/2	NA	2-15	10-1/8	NA	3-15	9	4	4-15	11	4-3/4
1-16	12	NA	2-16	8-5/8	NA	3-16	10	4	4-16	10-1/2	4
1-17	12 - 1/2	NA	2-17	8-1/2	NA	3-17	8-1/2	4	4-17	11	4
1-18	12	NA	2-18	9-7/8	NA	3-18	10	4	4-18	11	4

Table A-3: Core information from sites 5 to 8

Site 5			Site 6			Site 7			Site 8		
Core	**Depth to Subgrade (in)**	**HMA Thickness (in)**	**Core**	**Depth to Subgrade (in)**	**HMA Thickness (in)**	**Core**	**Depth to Subgrade (in)**	**HMA Thickness (in)**	**Core**	**Depth to Subgrade (in)**	**HMA Thickness (in)**
5-1	10	5-1/2	6-1	10	4	7-1	13	5-1/2	8-1	NA	5
5-2	9	5	6-2	10	4-3/4	7-2	13-3/4	4-3/4	8-2	NA	4-1/8
5-3	10	4-3/4	6-3	10	4-3/8	7-3	14-1/2	4-1/2	8-3	NA	4-7/8
5-4	10	5	6-4	10	4-1/2	7-4	13-3/4	5	8-4	NA	5
5-5	9	5	6-5	10	NA	7-5	14-1/4	4-1/2	8-5	NA	4-1/2
5-6	9-1/2	4-3/4	6-6	10	NA	7-6	13-1/2	5	8-6	NA	6
5-7	8-1/2	5-1/8	6-7	10	4-1/2	7-7	15	4-1/2	8-7	NA	4-7/8
5-8	10	4-3/4	6-8	10	4	7-8	15	5	8-8	NA	5
5-9	10	4-7/8	6-9	10	4-5/8	7-9	14-1/2	4-3/4	8-9	NA	4-5/8
5-10	10	5-1/2	6-10	10	4	7-10	14	4-3/4	8-10	NA	5
5-11	10	5-1/4	6-11	10	4-1/2	7-11	15-1/4	5	8-11	NA	5
5-12	11	5	6-12	10	4-1/2	7-12	15	5	8-12	NA	4-3/8
5-13	10	5	6-13	10	4-3/4	7-13	15-3/4	5	8-13	NA	4-3/4
5-14	10	5-1/8	6-14	10	4-7/8	7-14	15	4-3/4	8-14	NA	4-1/2
5-15	9-1/2	5	6-15	10	4-1/8	7-15	14-1/2	4-1/2	8-15	NA	6-3/8
5-16	8-1/2	4-3/4	6-16	10	4-5/8	7-16	13-1/2	5	8-16	NA	5-1/8
5-17	10	5	6-17	10	4-3/8	7-17	13-1/2	4-1/2	8-17	NA	5
5-18	10	5	6-18	10	4-5/8	7-18	15	5-1/4	8-18	NA	4-3/4

Table A-4: Core information from sites 9 to 11

Site 9			Site 10			Site 11		
Core	Depth to Subgrade (in)	HMA Thickness (in)	Core	Depth to Subgrade (in)	HMA Thickness (in)	Core	Depth to Subgrade (in)	HMA Thickness (in)
9-1	NA	5-1/8	10-1	16-22	5-1/4	11-1	NA	4-3/4
9-2	NA	5-1/4	10-2	16-22	5-1/8	11-2	NA	3-3/4
9-3	NA	5-1/4	10-3	16-22	4-7/8	11-3	NA	4-5/8
9-4	NA	5-3/8	10-4	16-22	5	11-4	NA	4-3/4
9-5	NA	4-7/8	10-5	16-22	4-7/8	11-5	NA	5
9-6	NA	5-3/8	10-6	16-22	5-3/8	11-6	NA	4-3/8
9-7	NA	5-1/4	10-7	16-22	5-1/2	11-7	NA	4-7/8
9-8	NA	NA	10-8	16-22	5-1/4	11-8	NA	4-1/2
9-9	NA	5-3/8	10-9	16-22	5-1/4	11-9	NA	4-1/2
9-10	NA	5-1/8	10-10	16-22	5-1/4	11-10	NA	4-3/8
9-11	NA	5-3/8	10-11	16-22	5-1/8	11-11	NA	4-1/2
9-12	NA	5-38	10-12	16-22	5-1/4	11-12	NA	4-1/2
9-13	NA	5-5/8	10-13	16-22	5	11-13	NA	4-5/8
9-14	NA	5-1/8	10-14	16-22	5-3/8	11-14	NA	5
9-15	NA	5-1/2	10-15	16-22	5-1/8	11-15	NA	4-3/8
9-16	NA	4-7/8	10-16	16-22	5-3/8	11-16	NA	4-1/2
9-17	NA	4-7/8	10-17	16-22	5-1/4	11-17	NA	4-1/2
9-18	NA	5-3/4	10-18	16-22	5-1/8	11-18	NA	4-3/4

Figure A-1: Outlier box plot of distresses in site 1

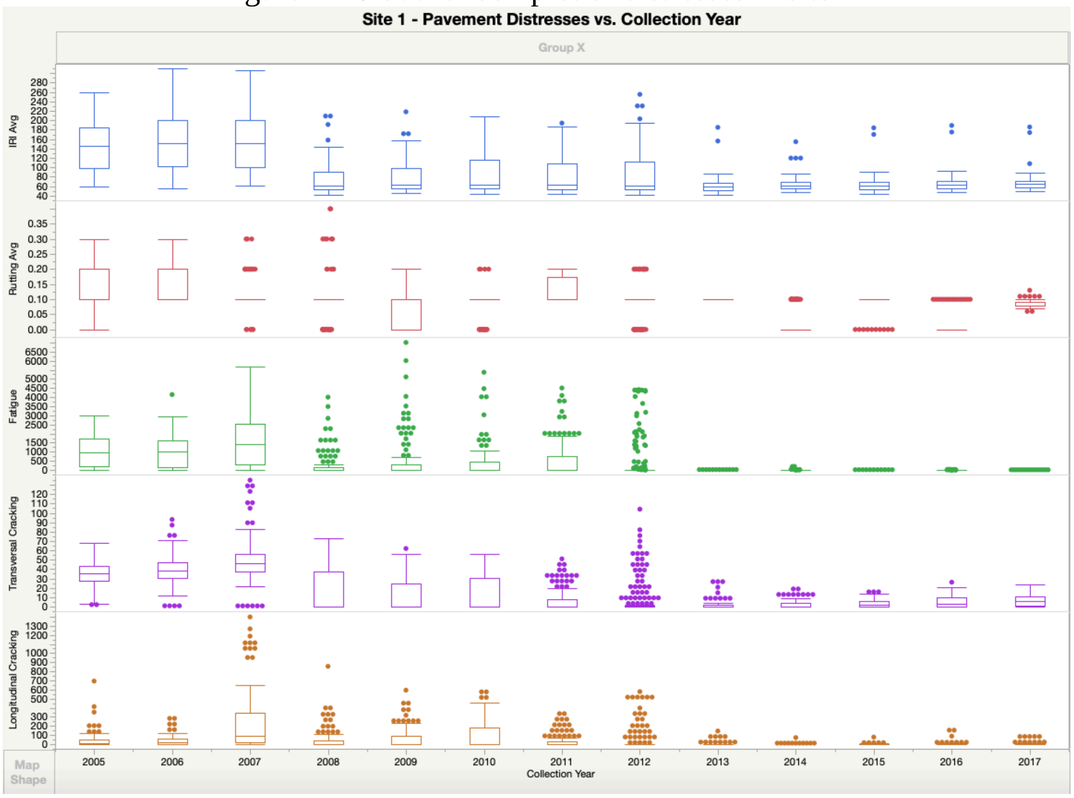


Figure A-2: Outlier box plot of distresses in site 2

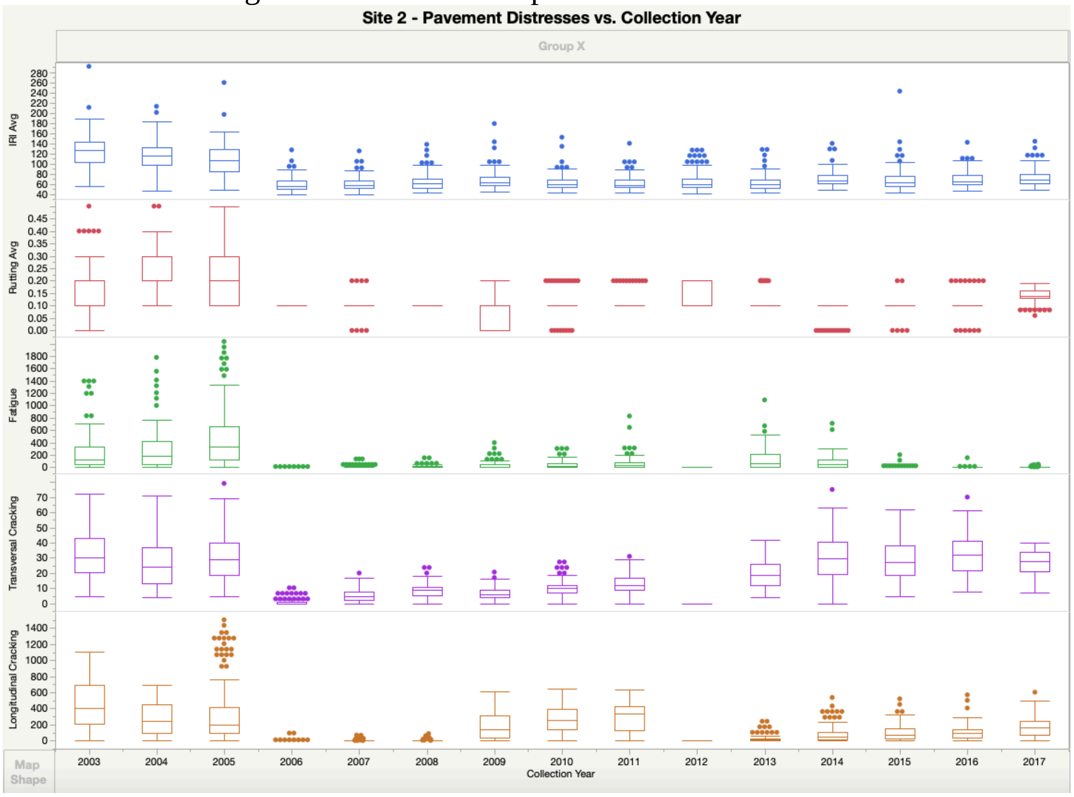


Figure A-3: Outlier box plot of distresses in site 3

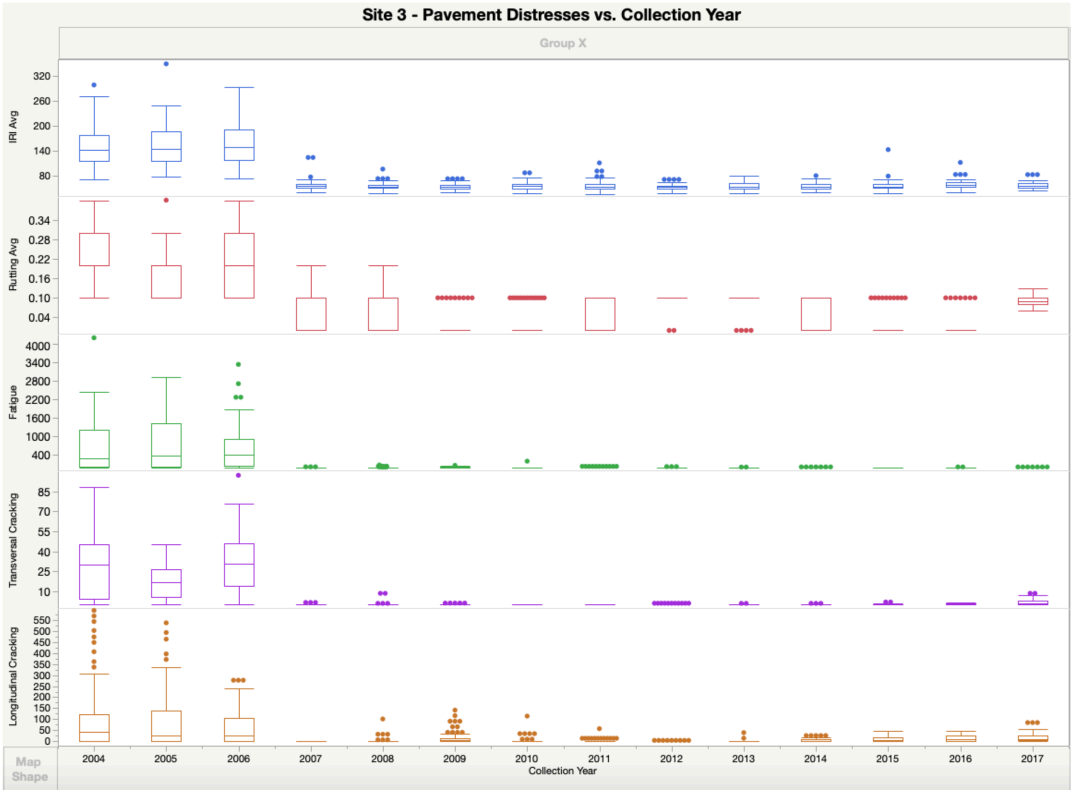


Figure A-4: Outlier box plot of distresses in site 4

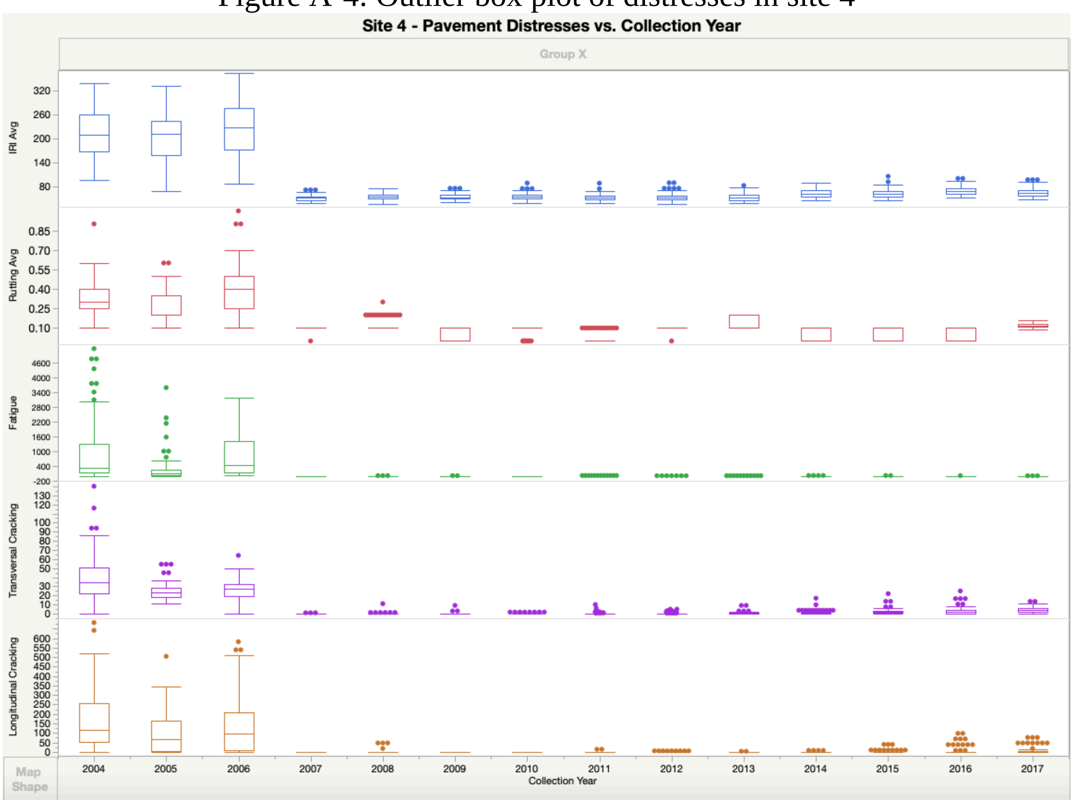


Figure A-5: Outlier box plot of distresses in site 5

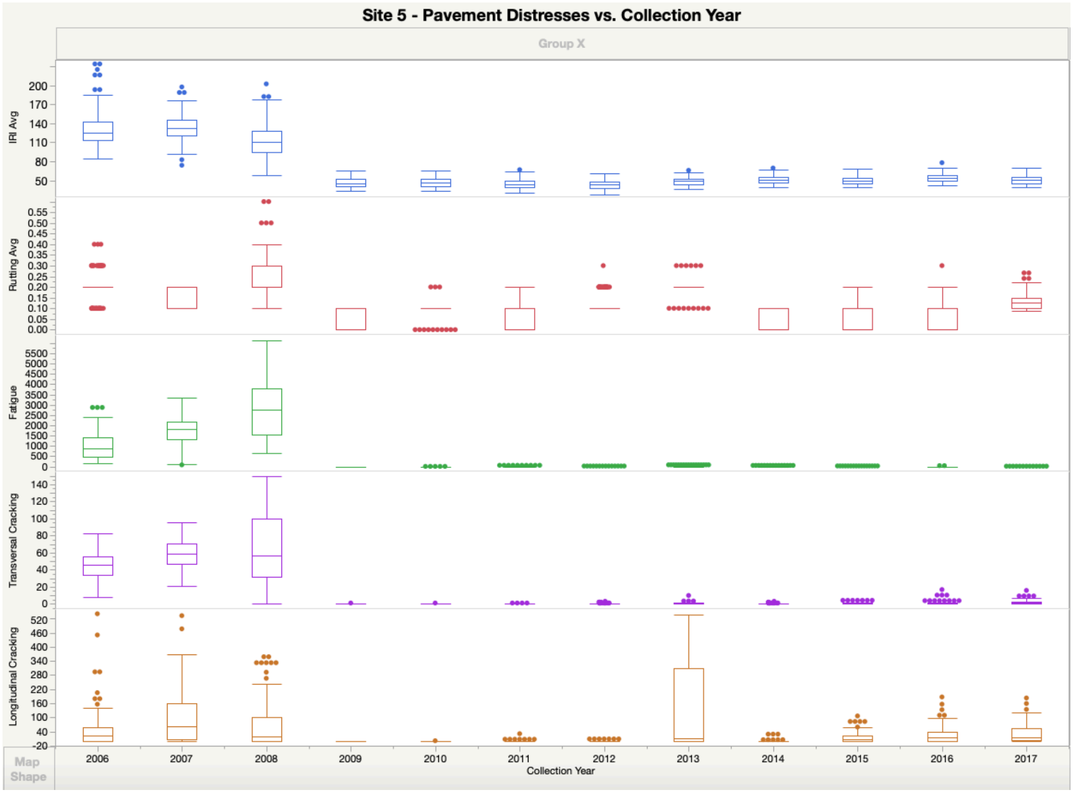


Figure A-6: Outlier box plot of distresses in site 6

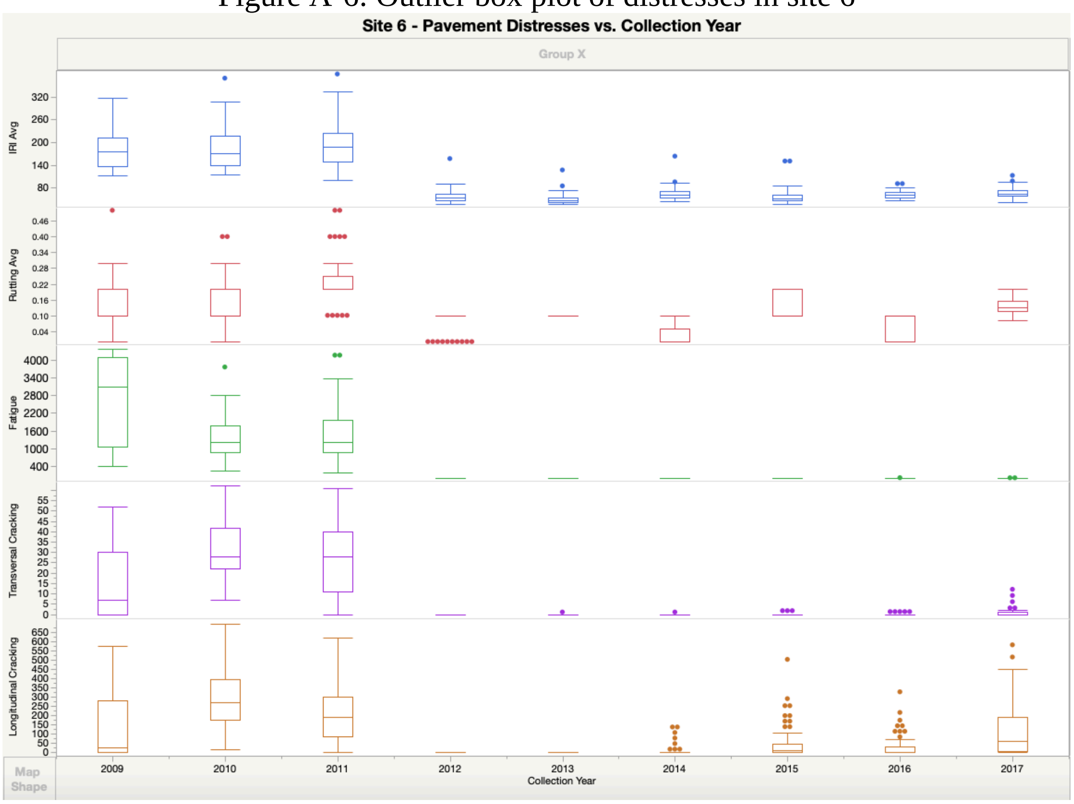


Figure A-7: Outlier box plot of distresses in site 7

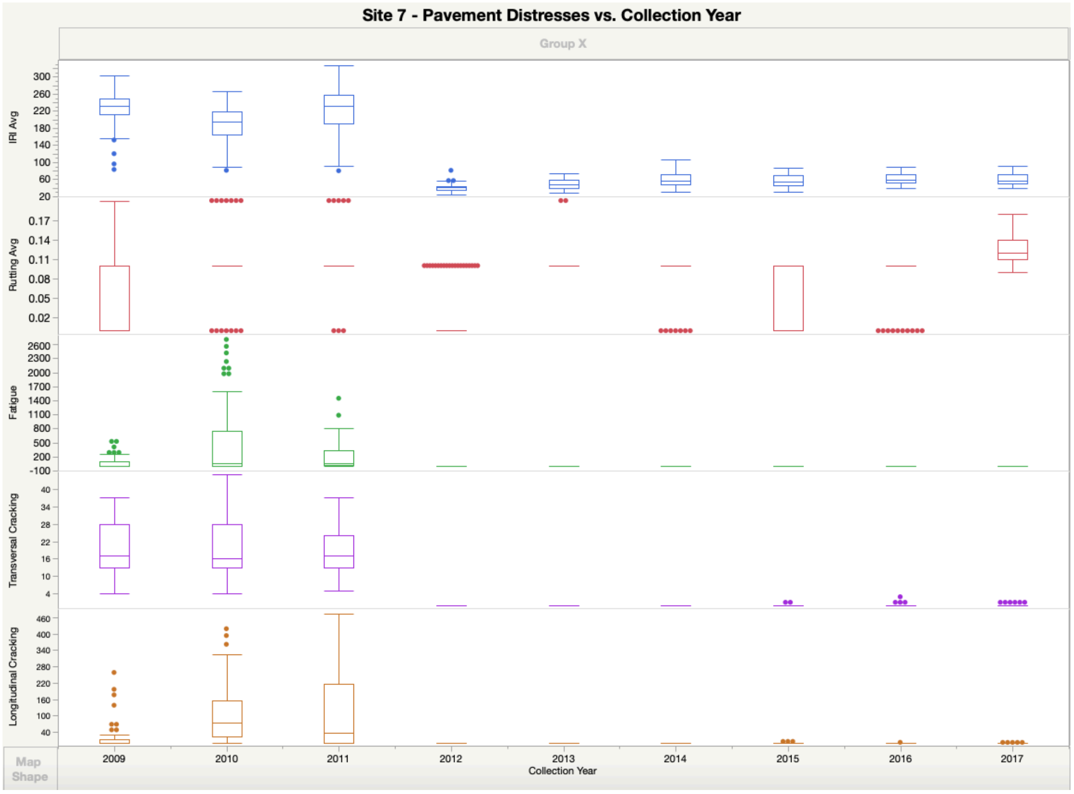


Figure A-8: Outlier box plot of distresses in site 8

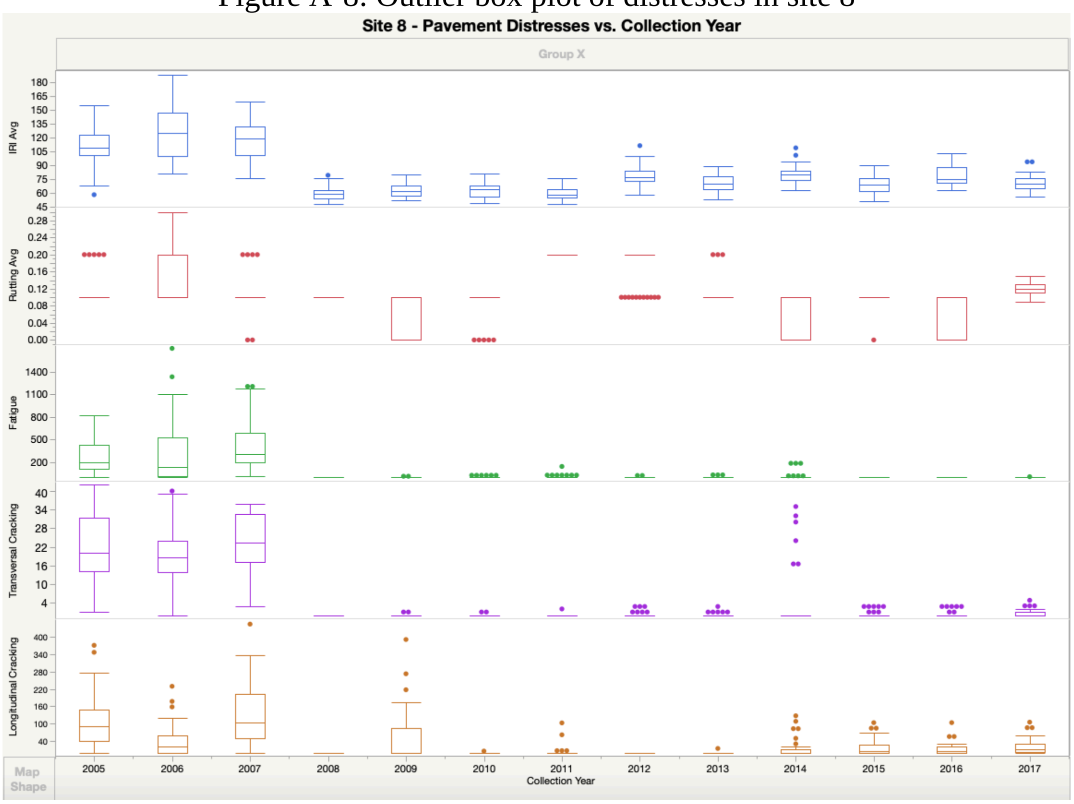


Figure A-9: Outlier box plot of distresses in site 9

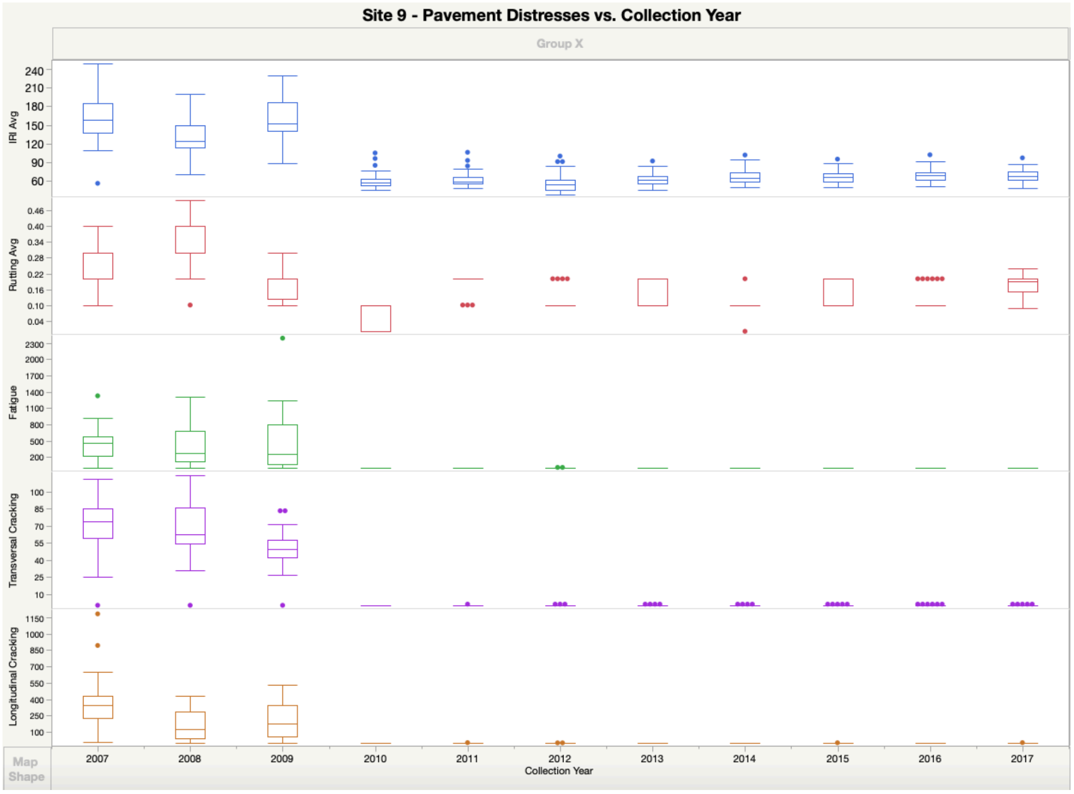


Figure A-10: Outlier box plot of distresses in site 10

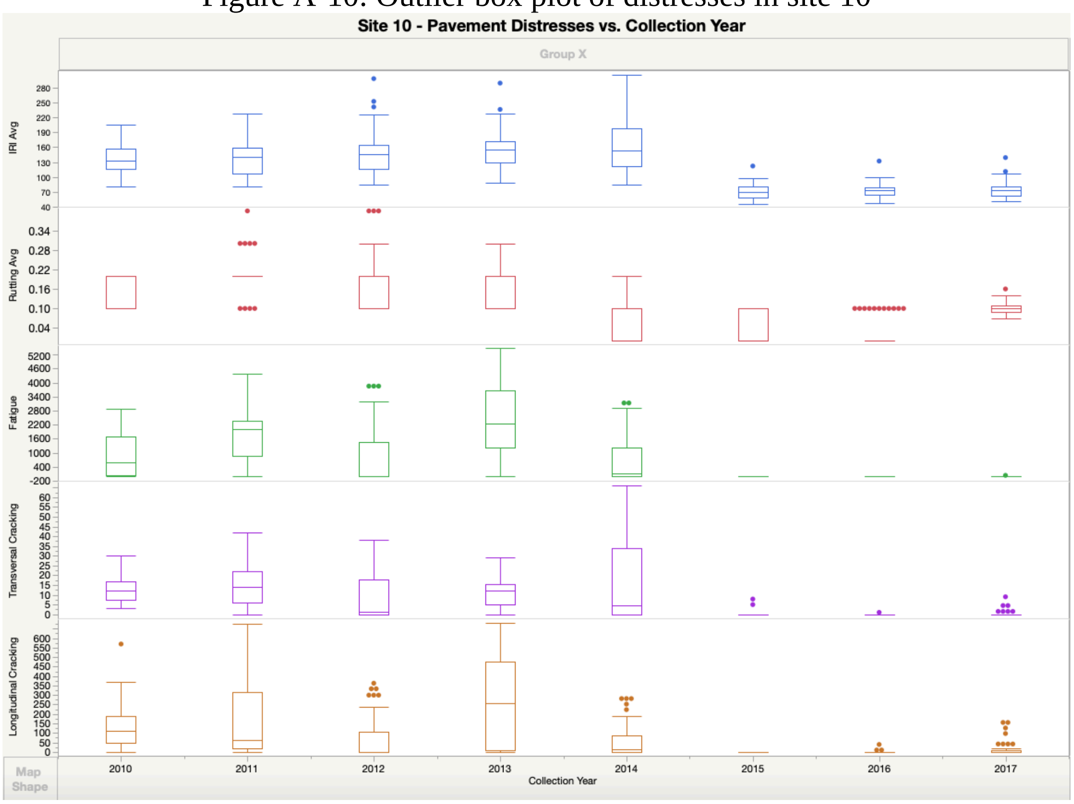
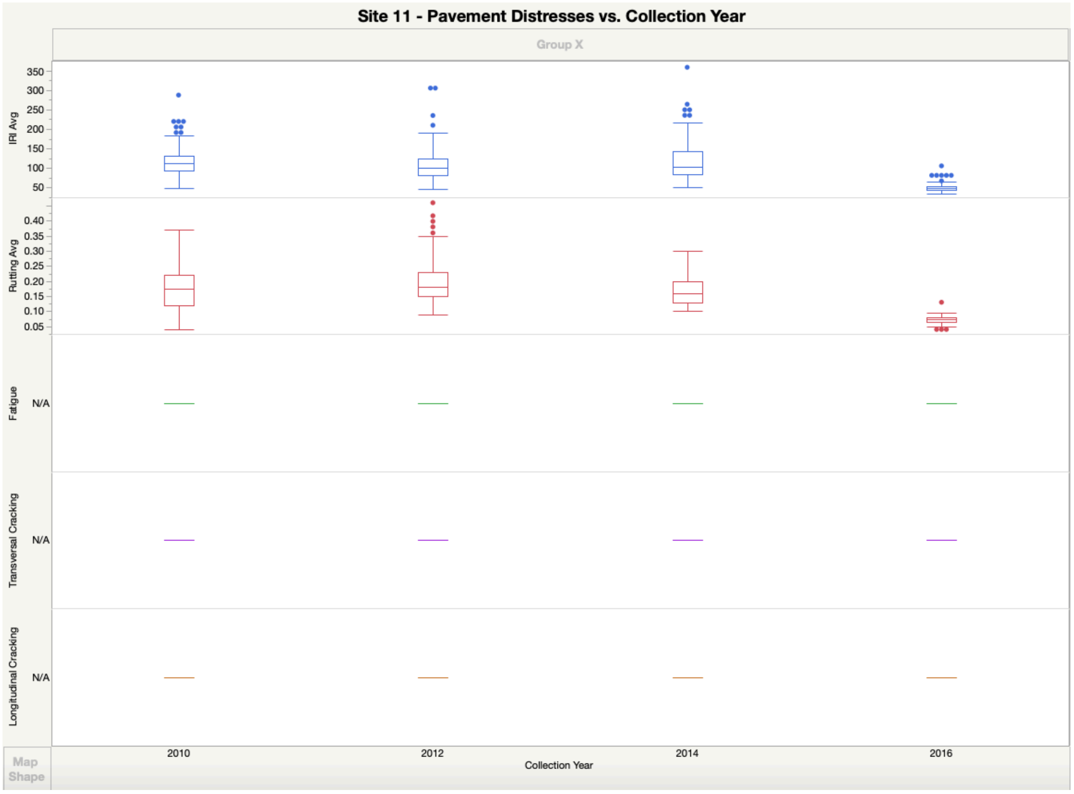


Figure A-11: Outlier box plot of distresses in site 11


Site 11 - Pavement Distresses vs. Collection Year
Group X
IRI Avg
350
300
250
200
150
100
50
Rutting Avg
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
Fatigue
N/A
Transversal Cracking
N/A
Longitudinal Cracking
N/A
Map Shape
2010
2012
2014
2016
Collection Year

Inconsistencies in the PMS Data Analysis

The data analysis has found seven general patterns showing data inconsistencies, particularly when the data is compared with expected trends of deteriorating road conditions that typically develop over time. These inconsistencies are not accounted for after including and evaluating Design-Construction-and-Performance (DCP) data, as provided by CDOT, such as maintenance performed on a given year.

In summary, the list of issues found is summarized next:

1. Distresses before "Ad Date + 1 year" show unexpected high values not consistent with new pavements performance
2. Distresses after "Ad Date + 1 year" continue to show unexpected high values for the next year or two, then values decline
3. Distresses unexpectedly improve with no records of maintenance (unexplained declining trends)
4. Random null distresses create disruptions on typically expected trends
5. Random high distresses create disruptions on typically expected trends
6. Rutting shows inconsistent trends throughout the years
7. Distress values after "Ad Date + 1 year" show null median values overtime

The following sub-sections will explain in detail each of the seven types of inconsistencies, relying on illustrative graph examples for better understanding. As each issue is described, possible interpretations for their occurrence are also offered. Finally, we propose recommendations and possible follow-ups to address each issue.

Issue #1— Distresses before "Ad Date + 1 year" show unexpected high values not consistent with new pavements performance

Essentially all sites show unexplained patterns of unexpected high distress values in their early years. These patterns are not consistent with the behavior associated with new pavements. Typically, new pavements show low levels of distresses in their early years. Nevertheless, based on the performance information provided by CDOT, all sites show unexpected high values for the five distresses. Figure A-12 shows a graph for ease of understanding, again only using site 1 as an illustration.

The research team's interpretation from this unexplained pavement behavior is that FDR was actually constructed on the "Ad Date"—as referred to in the DCP records—and not on the year when the earliest PMS data was made available. For example, referring to Figure 3, 2007 would be the year of FDR construction, not 2005. Nonetheless, considering data starting with the "Ad Date" still shows high unexpected distresses. The assumption for this is that PMS data may have been collected before FDR construction actually occurred on the same year. This would explain that such high distresses should not be accounted for in the data analysis.

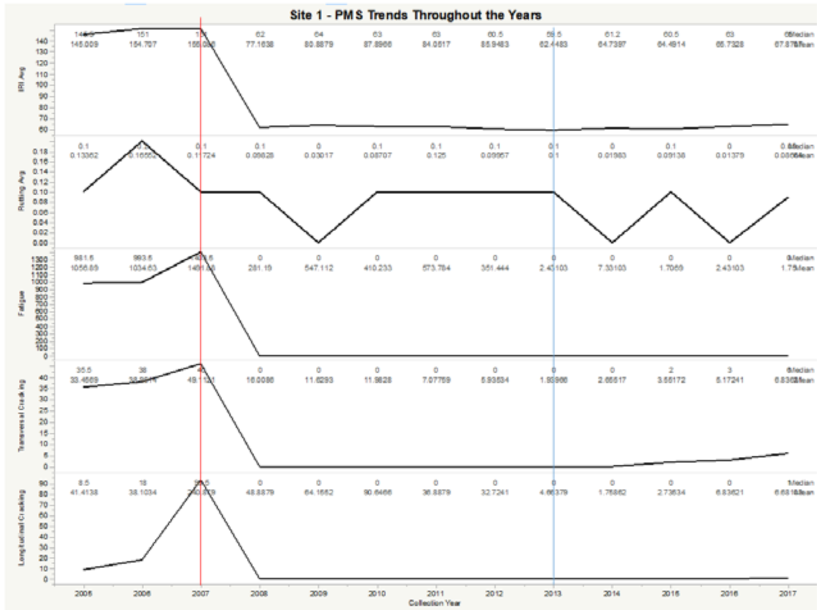


Figure A-12. Issue #1 described. Red line represents “Ad Date” from CDOT’s DCP records. Blue line indicates “Maintenance Date” from CDOT’s DCP records.

The recommendation to address this issue is to analyze distress data starting from “Ad Date” plus one year, effectively ignoring distress data from previous years, which always display random variability for all sites. Figure A-13 shows a visual representation of the proposed resolution.

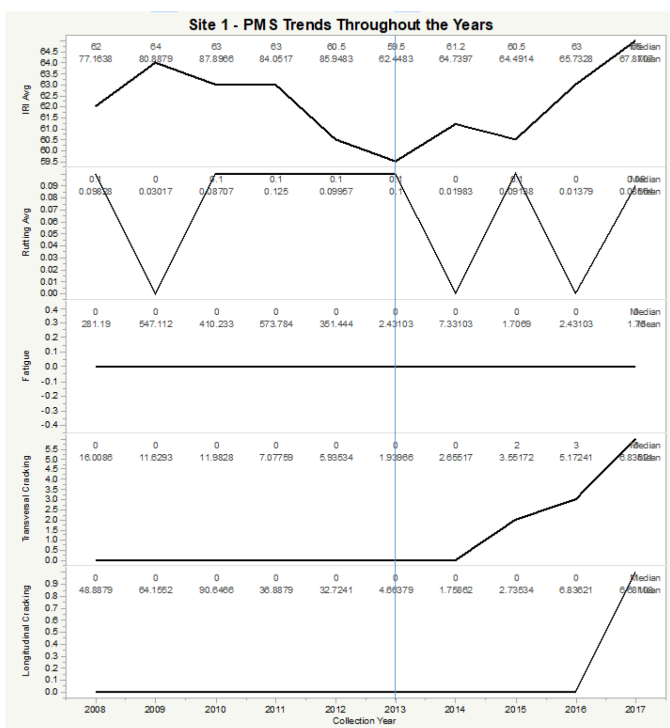


Figure A-13. Issue #1 resolved. PMS distresses throughout the years excluding data before “Ad Date“ plus one year. Blue line indicates “Maintenance Date” from CDOT’s DCP records.

Issue #2— Distresses after "Ad Date + 1 year" continue to show unexpected high values for the next year or two, then values decline

Some sites continue to display unexplained patterns of high distress values for the next year or two even after the criteria to solve issue #1 has been applied (i.e.: exclude data before “Ad Date” plus one year). Figure A-14 shows an illustrative graph, using site 5 as an example, where excluding distress data *before* “Ad Date” plus one year does not resolve data inconsistencies. Instances such as this were observed not only for site 5, but also for sites 6, 9 and 10. The research team’s interpretation is that the "Ad Date" may have been mistakenly documented.

The ensuing recommendation is to analyze distress data from the year that all distresses start increasing from the lowest values. Figure A-15 shows a visual representation of the proposed resolution applied to the same site for ease of understanding.

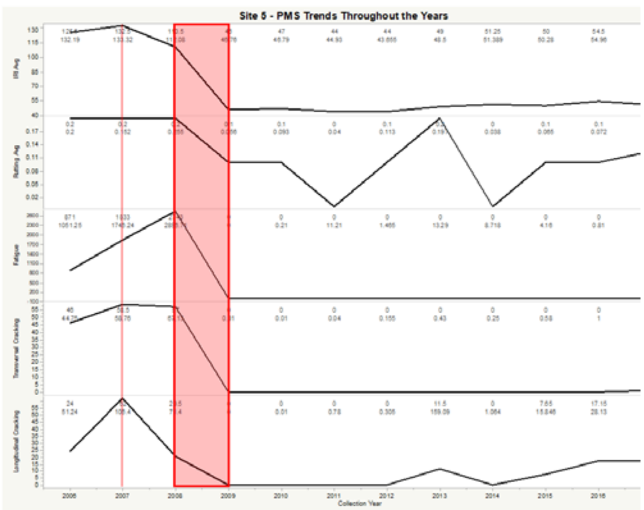


Figure A-14. Issue #2 visually described. Red-colored area includes distress data with unexpected high values after Ad Date + 1 year. Red line shows actual “Ad Date”.

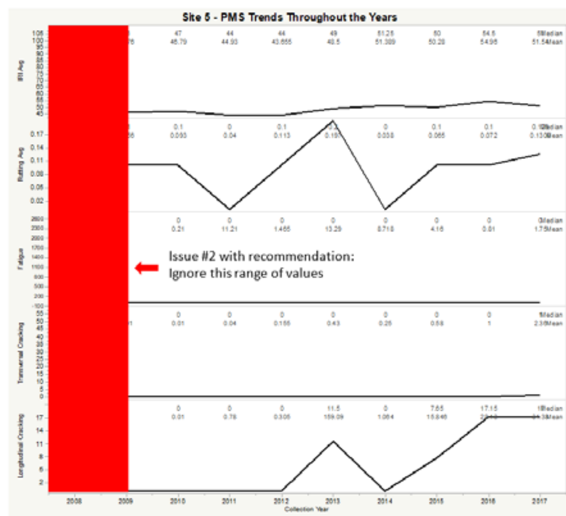


Figure A-15. Issue #2 resolved. PMS distresses throughout the years excluding noise.

Issue #3— Distresses unexpectedly improve with no records of maintenance (unexplained declining trends)

Only site 2 displays unexpected improvements on distress values with no existing correlations to available maintenance records, based on CDOT’s PMS archives. Figure A-16 shows this graphically, where significant improvements on pavement conditions as measured by transversal and longitudinal cracking can be observed in 2012. Furthermore, an unexpected improvement only on fatigue can be observed in 2015 without any correlation to documented maintenance that year. This inconsistency is particularly puzzling considering that the other distresses remain unaffected.

One possible interpretation is that some maintenance may have been performed but perhaps not documented on the PMS records. This is unlikely, because a more thorough evaluation of the 2012 dataset reveals that the perceived improvements may be related to no collection of PMS data on fatigue, transversal and longitudinal cracking in 2012. In light of this, an alternate interpretation is proposed as issue #4 is explained.

Particularly, site 2 presents additional unexplained inconsistencies, specifically on distresses such as fatigue in 2015 and longitudinal cracking in 2012. The research team reckons that resolving these inconsistencies may require foregoing analysis of both fatigue and longitudinal cracking for site 2 altogether, as these behaviors are not coherent in the absence of additional, contextual information. Figure A-17 shows a visual representation of the unresolved issues, which applies to site 2 only.

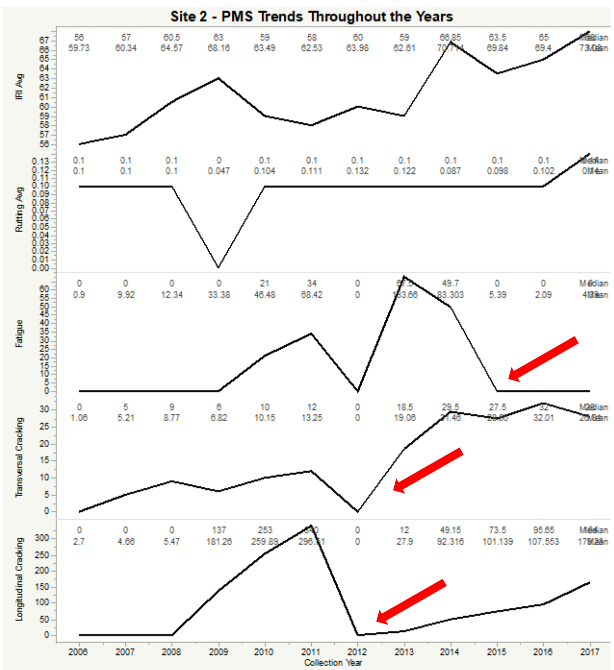


Figure A-16. Issue #3 visually described.

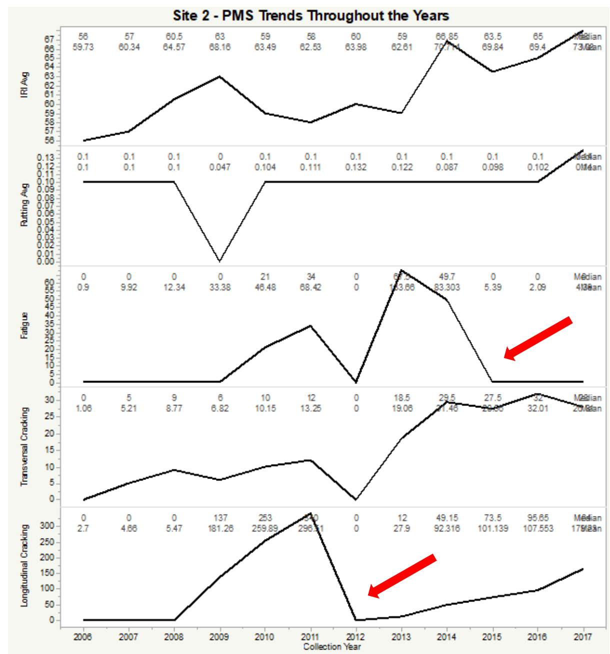


Figure A-17. Issue #3 not resolved. Refer to Issue #4 for an alternate resolution.

Issue #4— Random null distresses create disruptions on typically expected trends

Only site 2 displays random null values for distresses such as fatigue, transversal and longitudinal cracking for the year 2012 (refer to Figure A-18 for a graphical illustration).

In years when null values can be noted, there are two possible interpretations. One interpretation, albeit unlikely, was put forth for issue #3 (i.e.: maintenance was performed but not documented). Another interpretation is that PMS data may not have been collected in 2012 for certain distresses, in this case for fatigue, transversal and longitudinal cracking. This second interpretation is more likely because the 2012 dataset reveals that every 0.1-mile interval contains null values on fatigue, transversal and longitudinal cracking, thereby strongly suggesting that no PMS data on these three distresses was collected that year whatsoever.

The recommendation associated with this issue is to ignore null data and extrapolate the trend as it would be expected. Figure A-19 shows a visual representation of the proposed resolution applied to site 2. However, as it can be observed, this recommendation does not resolve the inconsistencies associated with fatigue in 2015 and longitudinal cracking in 2012. The research team reckons that resolving this inconsistency may require foregoing analysis of both fatigue and longitudinal cracking for site 2, as their associated behaviors are not coherent in the absence of additional, contextual information.

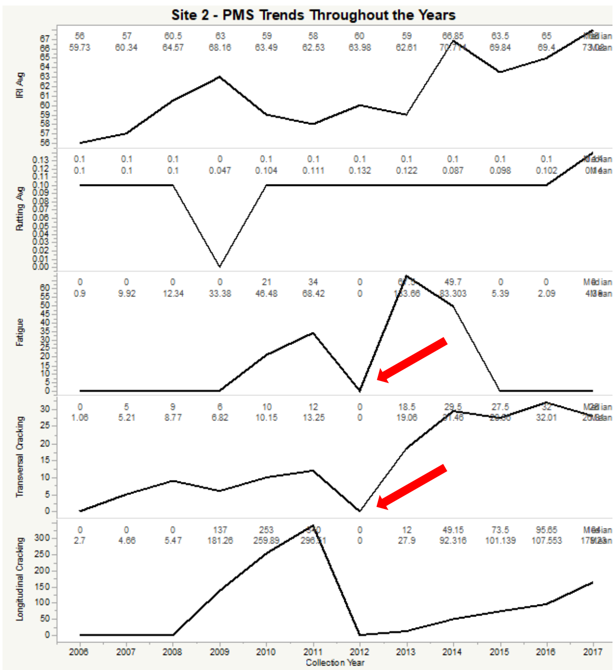


Figure A-18. Issue #4 visually described.

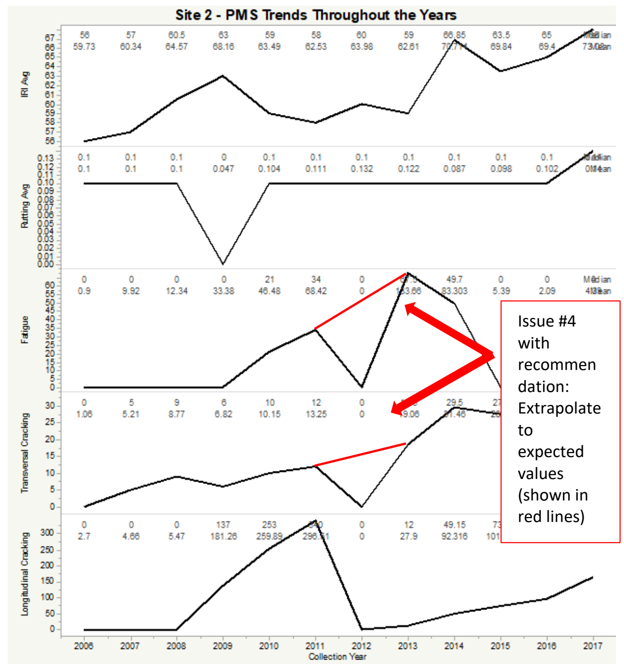


Figure A-19. Issue #4 resolved. Red lines indicate possible "extrapolated values".

Issue #5— Random high distresses create disruptions on typically expected trends

Only site 3 and 5 display random high distresses which create disruptions on typically expected pavement behavior trends. This can be observed on fatigue and longitudinal cracking at site 3 in 2009 and on longitudinal cracking at site 5 in 2013 (refer to Figure A-20 for a graphical illustration using site 3).

The associated interpretation is that instrument error affected the collection of fatigue and longitudinal cracking in 2009. Correspondingly, ignoring this isolated data and extrapolating the trend as it would be expected is recommended. Figure A-21 shows a visual representation of the proposed resolution applied to the same site.

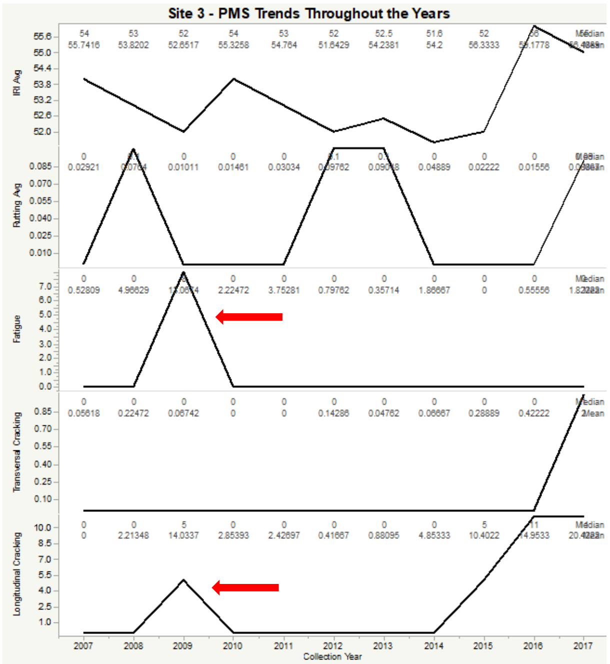


Figure A-20. Issue #5 visually described.

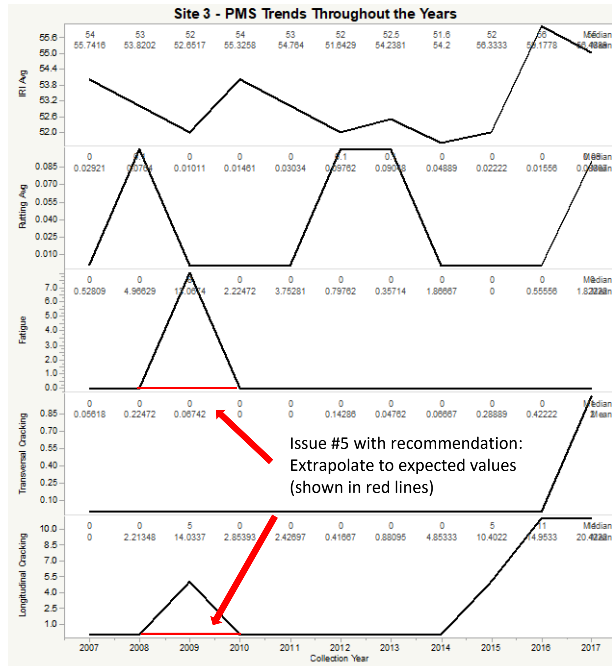


Figure A-21. Issue #5 resolved. Red lines indicate possible "extrapolated values".

Issue #6— Rutting shows inconsistent trends throughout the years

Some sites display inconsistent rutting trends over the years. Referring to Figure A-22, using site 1 as an example, it can be observed that rutting values vary randomly without any correlation to maintenance performed. Instances such as this were observed not only for site 1, but also for every site except sites 7 and 11.

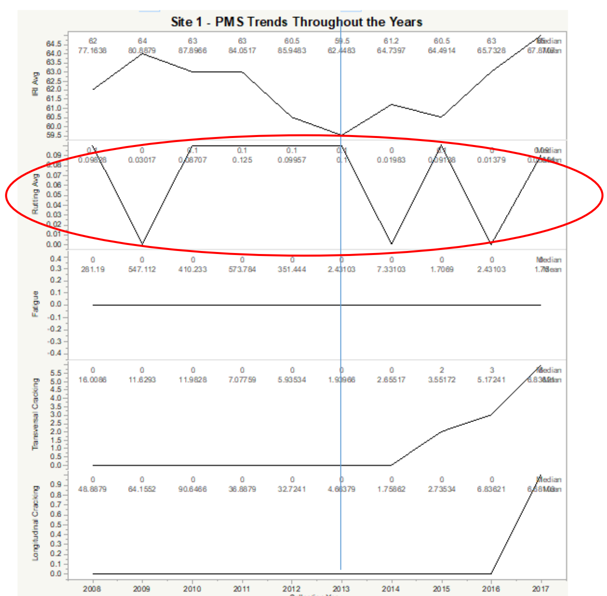


Figure A-22. Issue #6 visually described. Rutting shows inconsistent trends throughout the years. Blue line indicates "Maintenance Date" from CDOT's DCP records.

The interpretation is that rutting scale may be too tight, resulting in small variations seemingly displaying large discrepancies. To address this issue, the research team would like to request information on the type(s) of equipment(s) used to measure rutting as well as request instrument tolerances. Obtaining this information would be helpful for further interpretations and analysis on rutting, especially since these random patterns are visible on most sites.

Issue #7— Distress values after "Ad Date + 1 year" show null median values overtime

Some sites display null distress median values for all years since FDR construction to until 2017 (after excluding data before "Ad Date" plus one year). This issue is observed only on three distresses: fatigue, transversal and longitudinal cracking. Figure A-23 shows site 7 as an illustrative example where this pattern is evident for all three aforementioned distresses; however, some sites may display null values only for one of the three. Specifically, null fatigue values were observed only on sites 1, 2, 3 and 10. In turn, null transversal cracking values were observed on sites 7, 5, 6, 8 and 9. Lastly, null longitudinal cracking values were observed only on sites 7, 4 and 9. It is worth noting that some outliers did not affect the median computation; that is, values other than zero were collected over the years, but these values did not seem to have driven a change in the median trend.

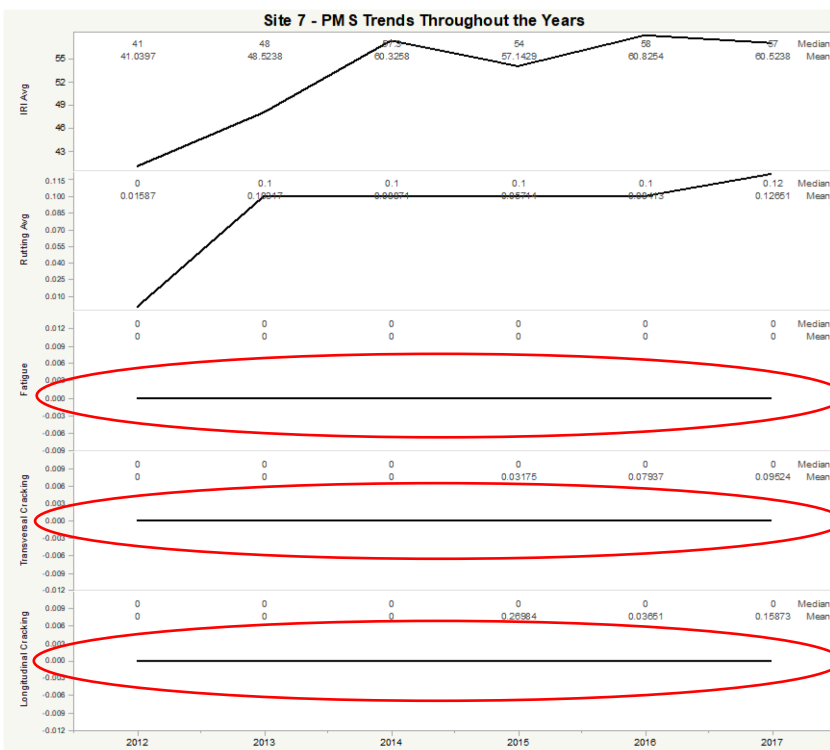


Figure A-23. Issue #7 visually described. Null median values for fatigue, transversal and longitudinal cracking over the years.

The interpretation is that distresses such as these—fatigue, transversal and longitudinal cracking—may not have developed significantly overtime, thus analyzing the data as is would be recommended.

Summary of Issues Found Aggregated by Sites

Table A-5 summarizes all the issues found in the data analysis along with their associated possible interpretation and recommendation to address them.

Table A-5. Summary of interpretations and recommendations.

ID	Issues Found	Interpretations	Recommendations
1	Distresses <u>before</u> "Ad Date + 1 year" show unexpected high values	FDR was actually constructed on the "Ad Date", as specified in the DCP records	Analyzing data starting from "Ad Date +1" only; ignore previous data, which creates random variability
2	Distresses <u>after</u> "Ad Date + 1 year" continue to show unexpected high values for next one or two years, then values decline	"Ad Date" was mistakenly documented	Analyze data from the year that all distresses start increasing from the lowest values
3	Distresses unexpectedly improve with no records of maintenance; unexplained declining trends	Maintenance performed but perhaps not documented on the PMS records	Assume minor maintenance was performed yet not documented
4	Random null distresses create disruptions on typically expected trends	Data may not have been collected	Ignore this data and extrapolate trend as it would be expected
5	Random high distresses create disruptions on typically expected trends	Instrument error when measuring at site	Ignore this data and extrapolate trend as it would be expected
6	Rutting shows inconsistent trends throughout the years	Rutting scale is too tight, so small variations seemingly display large discrepancies	Request information on type(s) of equipment(s) used and check instrument tolerances
7	Distress values after "Ad Date + 1 year" show null values overtime	Distresses did not significantly develop overtime	Analyze data as is (distress may not have developed)

It is worth noting that not all issues occur on every site. That is, different sites may show different data inconsistencies (Table A-6).

Table A-6. Summary of inconsistencies encountered by site.

ID	Issues Found	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6	Site 7	Site 8	Site 9	Site 10	Site 11
1	Distresses <u>before</u> "Ad Date + 1 year" show unexpected high values	x	x	x	x	x	x	x	x	x	x	x
2	Distresses <u>after</u> "Ad Date + 1 year" continue to show unexpected high values for next one or two years, then values decline					x	x			x	x	
3	Distresses unexpectedly improve with no records of maintenance; unexplained declining trends		x									
4	Random null distresses create disruptions on expected trends		x									
5	Random high distresses create disruptions on expected trends			x		x						
6	Rutting shows inconsistent trends throughout the years	x	x	x	x	x	x		x	x	x	
7	Distress values after "Ad Date + 1 year" show null values overtime	x			x	x	x	x	x	x		

Proposed Performance Curves

In view of the data inconsistencies analyzed in the previous section, and after including the corresponding recommendations, this section proposes next the performance curves that will be analyzed from now on in the analysis of the sections long-term performance. For most of the sites, the proposed curves are not materially different than their original counterparts. In fact, simply excluding noise around "Ad Date" resolves the majority of the observed inconsistencies. Indeed, after both resolving issues #1 and #2 and observing issues #6 and #7, inconsistencies are effectively addressed for sites 1, 4, 6, 7, 8, 9 and 10 (Table A-7).

Because recommendations to address inconsistencies for sites 2, 3 and 5 do alter proposed curves when these are directly compared with original data, a direct comparison between proposed and original curves will be specifically highlighted in this section for these three sites.

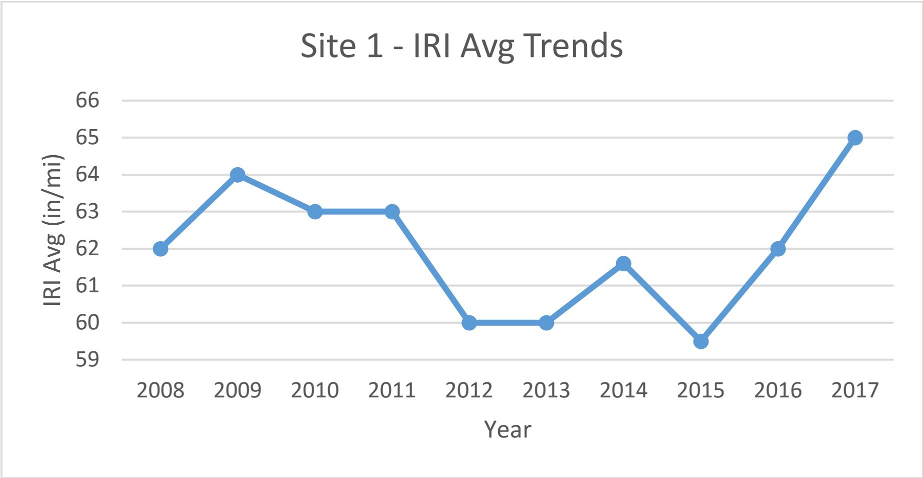
Site 1 — Proposed Median Distress Values Trending Over the Years

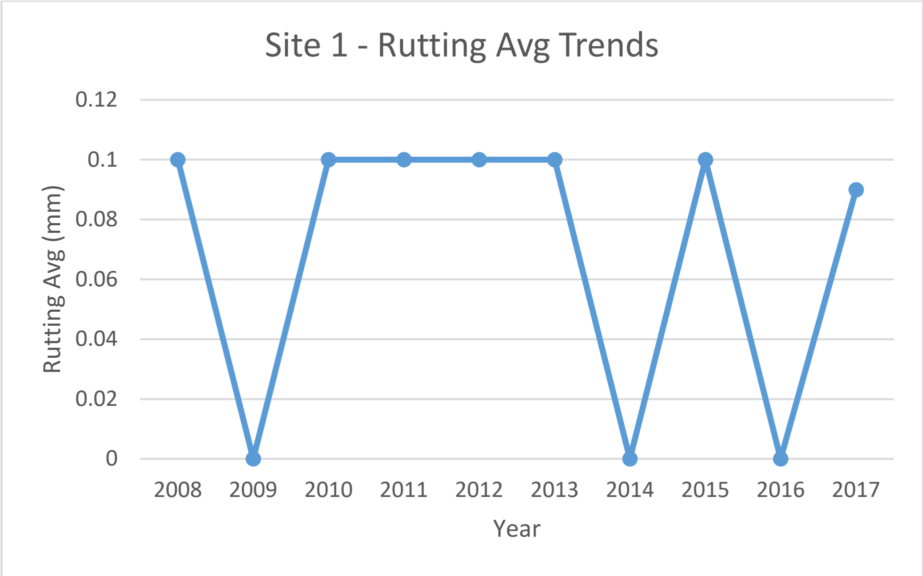
Background Information Based on PMS Records

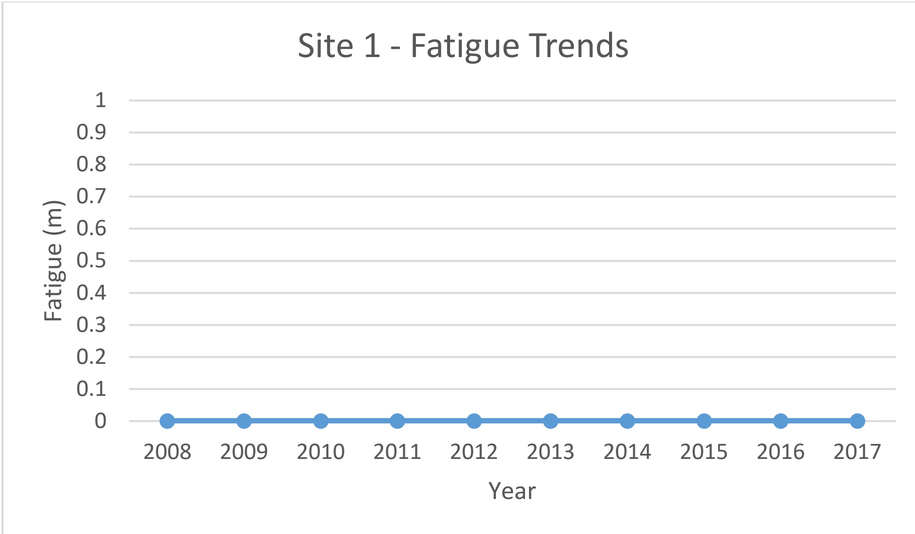
- Ad Date: MAY 17 2007
- Maintenance: Overlaid in 2013

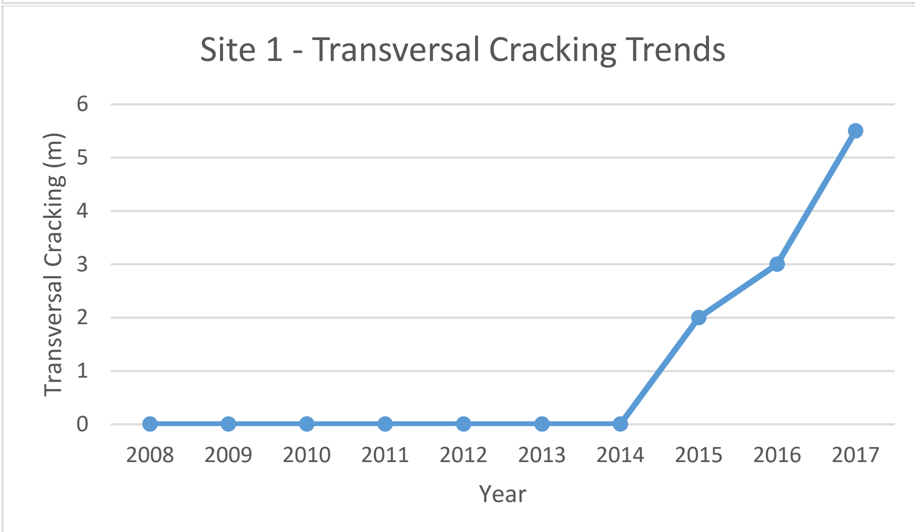
Recommendations

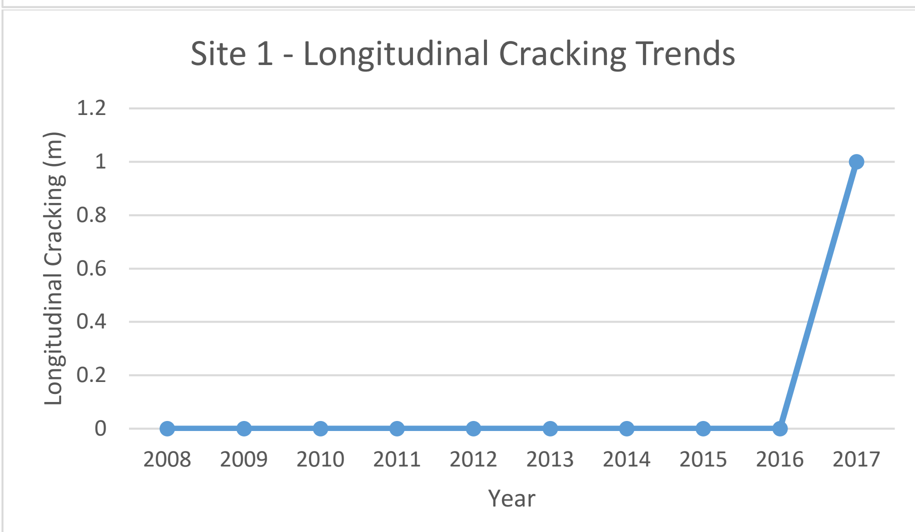
- Effective FDR Date: 2008






Site 1 - Fatigue Trends
Fatigue (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 1 - Transversal Cracking Trends
Transversal Cracking (m)
6
5
4
3
2
1
0
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 1 - Longitudinal Cracking Trends
Longitudinal Cracking (m)
1.2
1
0.8
0.6
0.4
0.2
0
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year

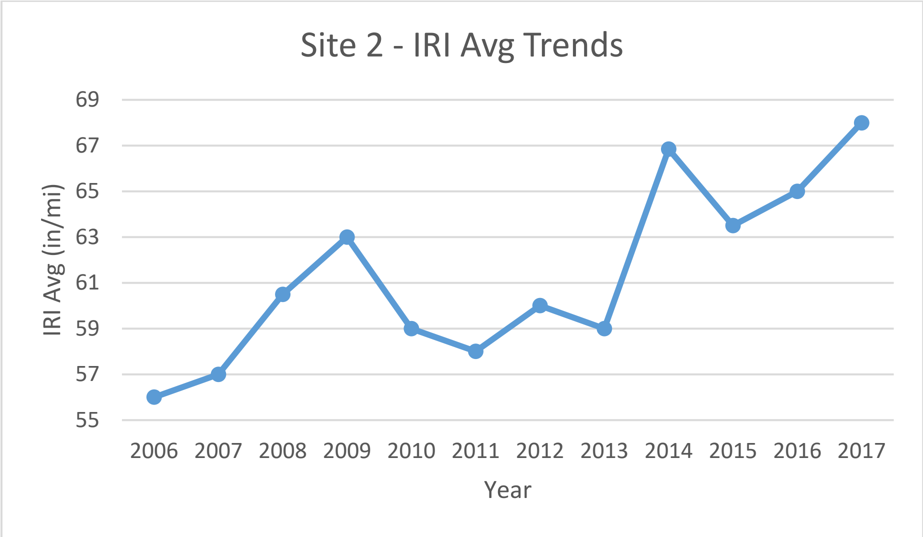
Site 2 — Proposed Median Distress Values Trending Over the Years

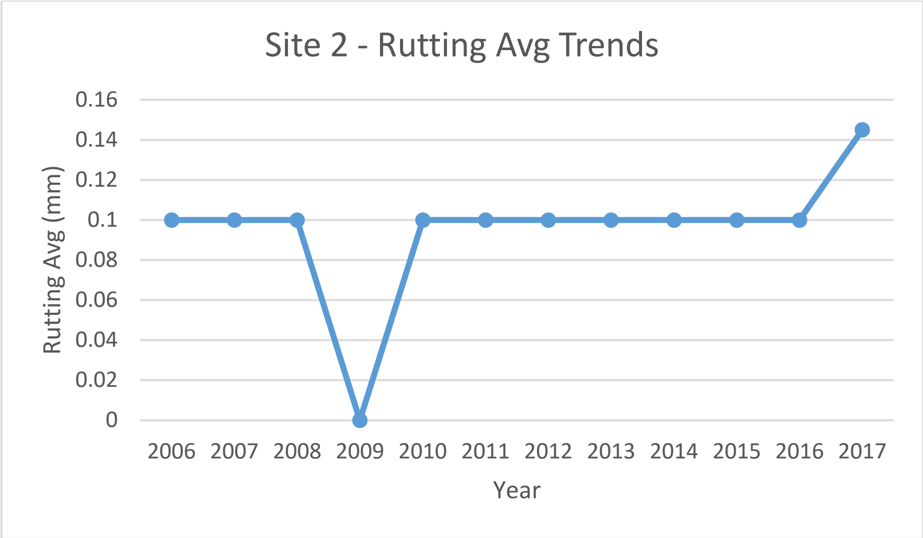
Background Information Based on PMS Records

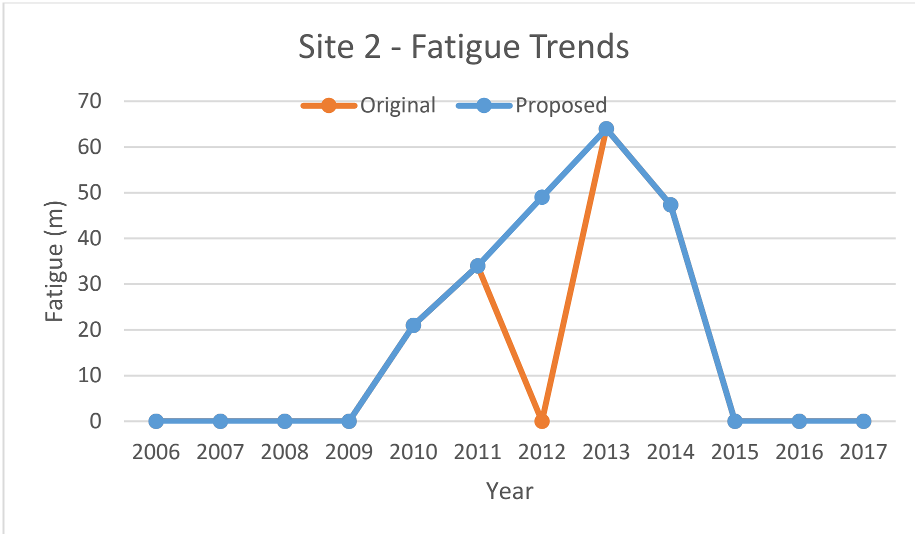
- Ad Date: MAR 03 2005
- Maintenance: None

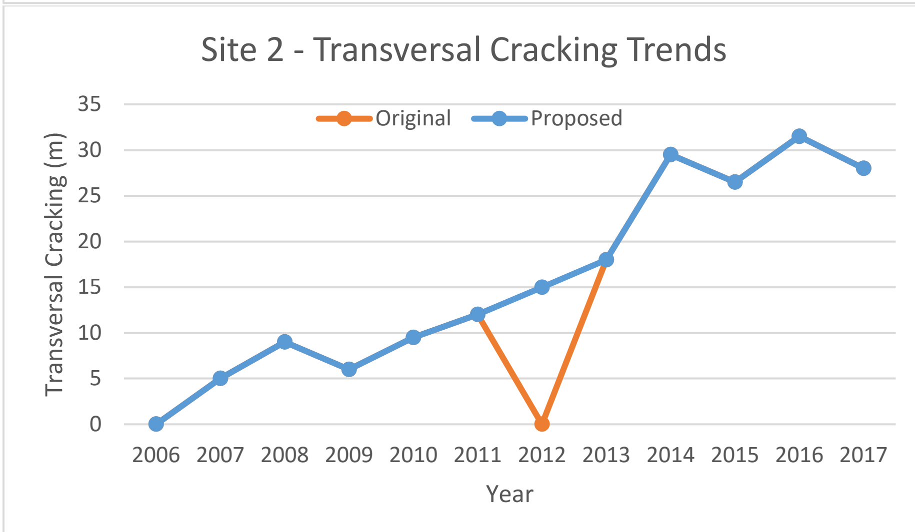
Recommendations

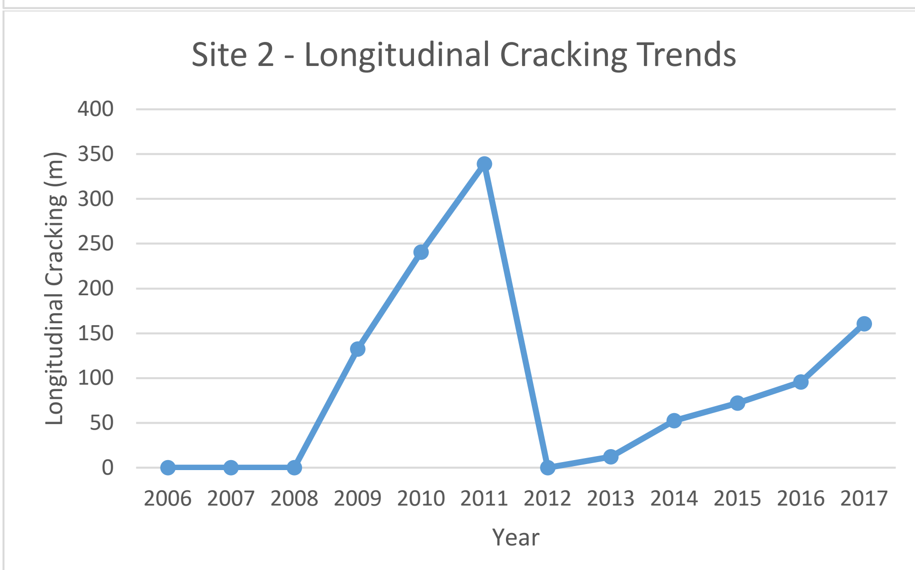
- Effective FDR Date: 2006
- Proposed distress curves are blue
- Original distress values shown in orange as applicable
- Note: Site 2 still presents some unexplained inconsistencies, specifically on distresses such as fatigue in 2015 and longitudinal cracking in 2012. The research team reckons that resolving these inconsistencies may require foregoing analysis of both fatigue and longitudinal cracking for site 2 altogether, as their associated behaviors are not coherent in the absence of additional, contextual information.






Site 2 - Fatigue Trends
Original
Proposed
Fatigue (m)
70
60
50
40
30
20
10
0
2006
2007
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 2 - Transversal Cracking Trends
Original
Proposed
Transversal Cracking (m)
35
30
25
20
15
10
5
0
2006
2007
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 2 - Longitudinal Cracking Trends
Longitudinal Cracking (m)
400
350
300
250
200
150
100
50
0
2006
2007
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year

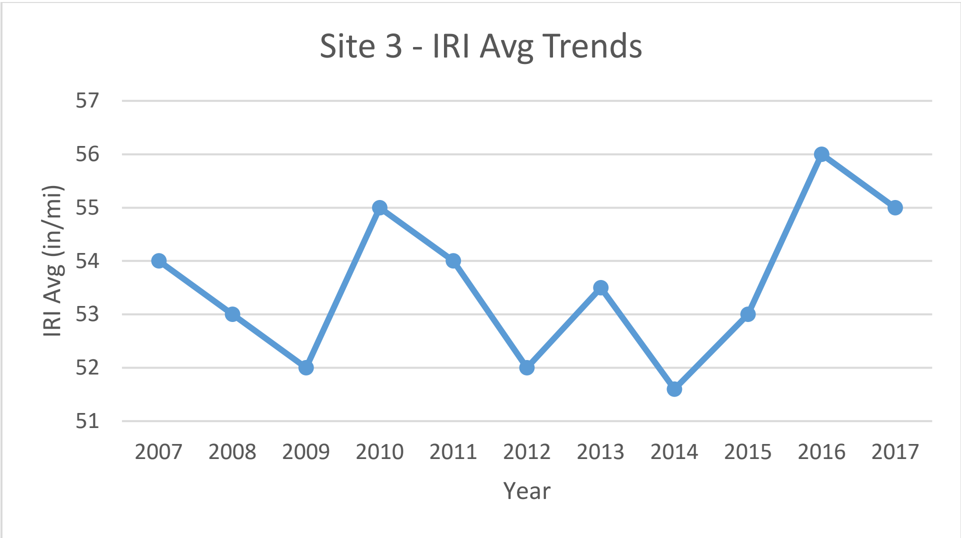
Site 3 — Proposed Median Distress Values Trending Over the Years

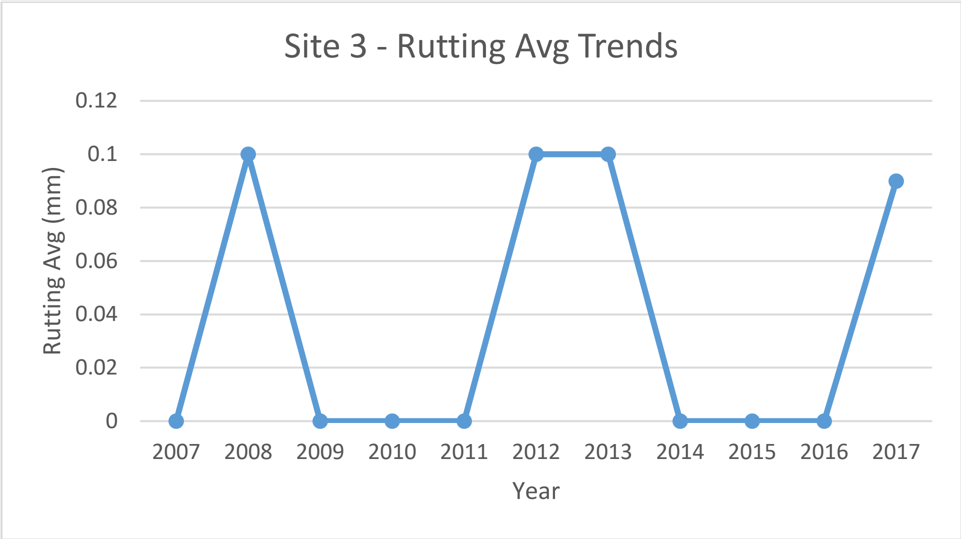
Background Information Based on PMS Records

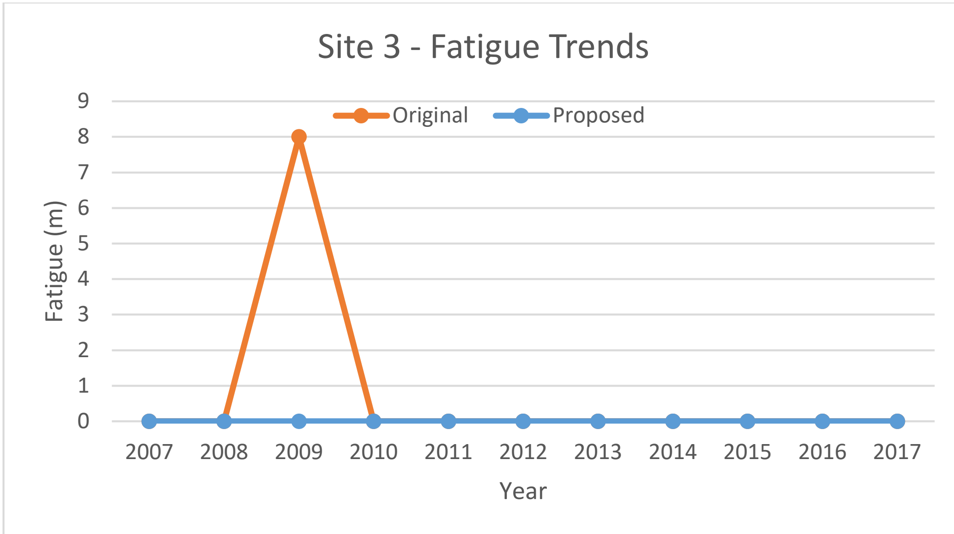
- Ad Date: JAN 12 2006
- Maintenance: None

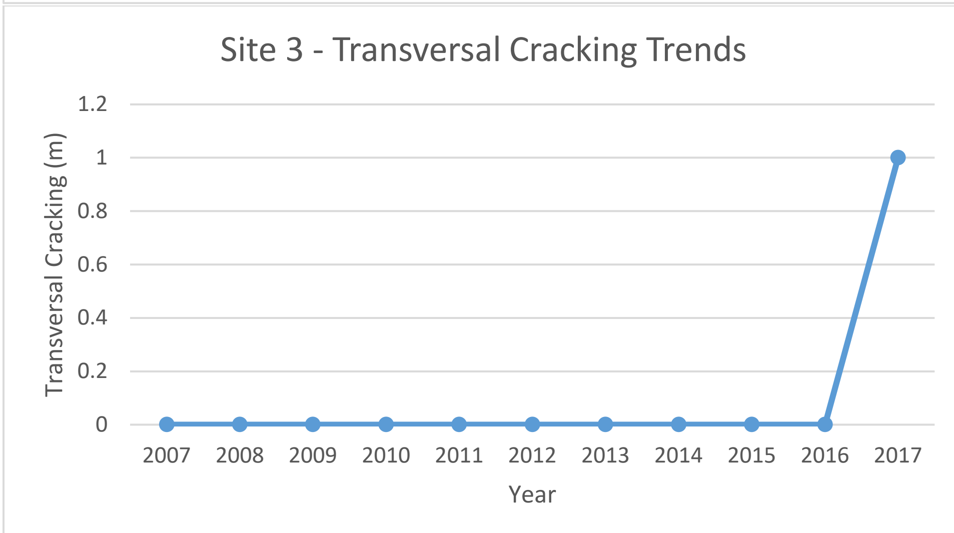
Recommendations

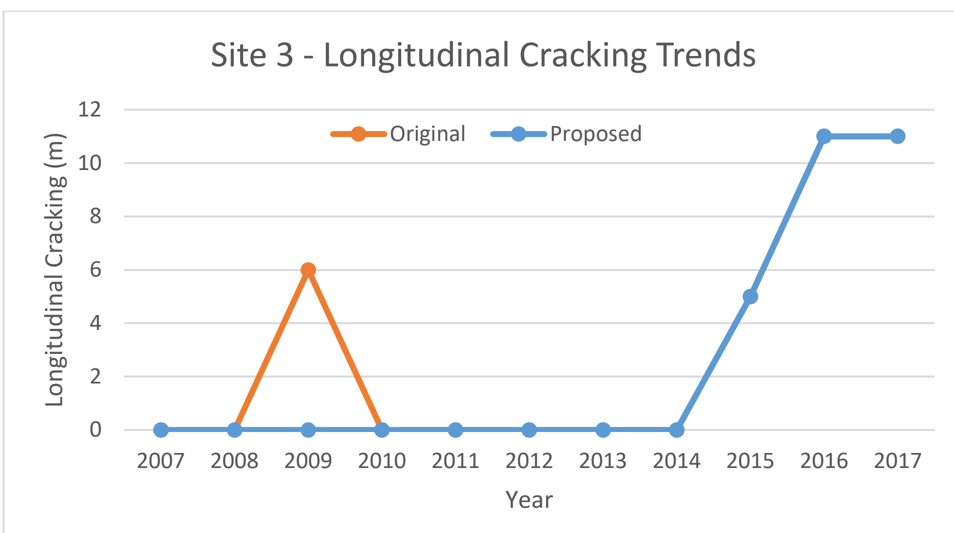
- Effective FDR Date: 2007
- Proposed distress curves are blue
- Original distress values shown in orange as applicable






Site 3 - Fatigue Trends
Original
Proposed
Fatigue (m)
9
8
7
6
5
4
3
2
1
0
2007
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 3 - Transversal Cracking Trends
Transversal Cracking (m)
1.2
1
0.8
0.6
0.4
0.2
0
2007
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 3 - Longitudinal Cracking Trends
Original
Proposed
Longitudinal Cracking (m)
12
10
8
6
4
2
0
2007
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year

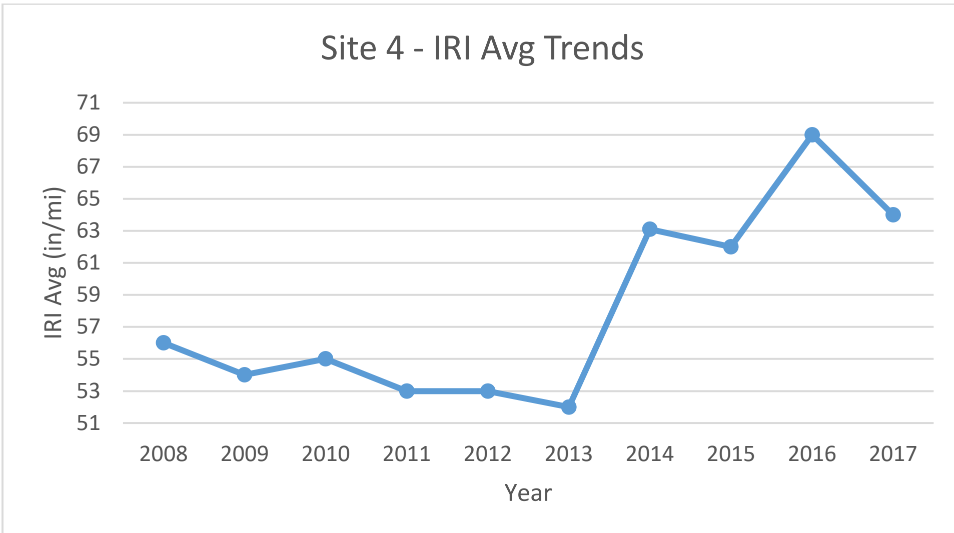
Site 4 — Proposed Median Distress Values Trending Over the Years

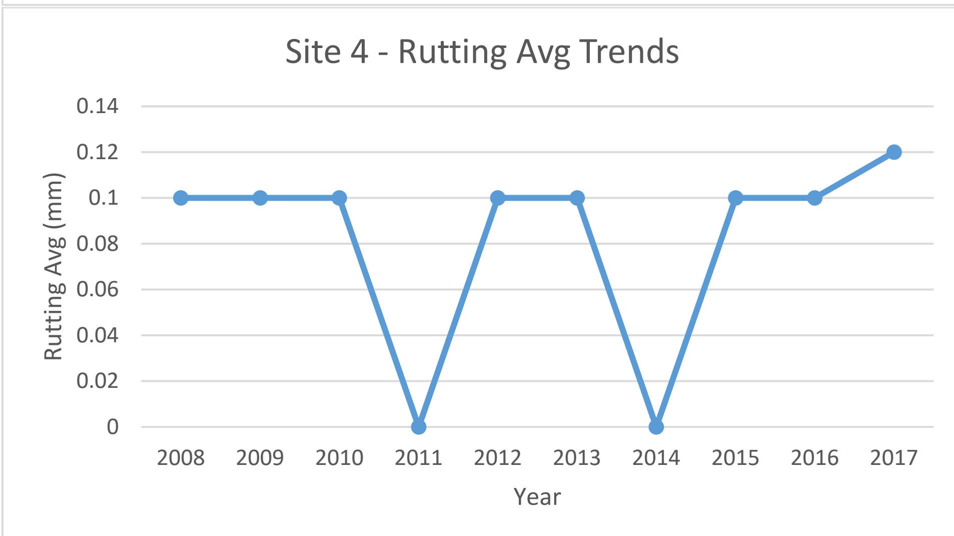
Background Information Based on PMS Records

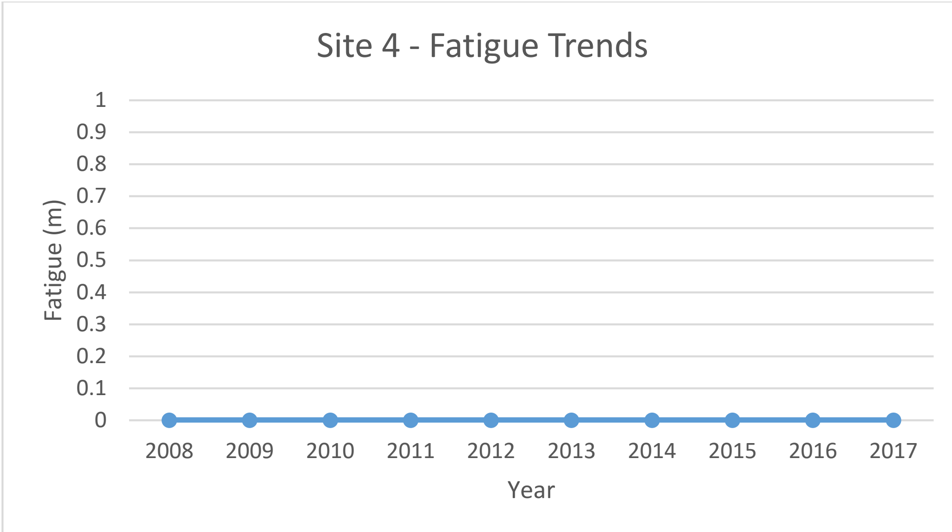
- Ad Date: MAY 31 2007
- Maintenance: None

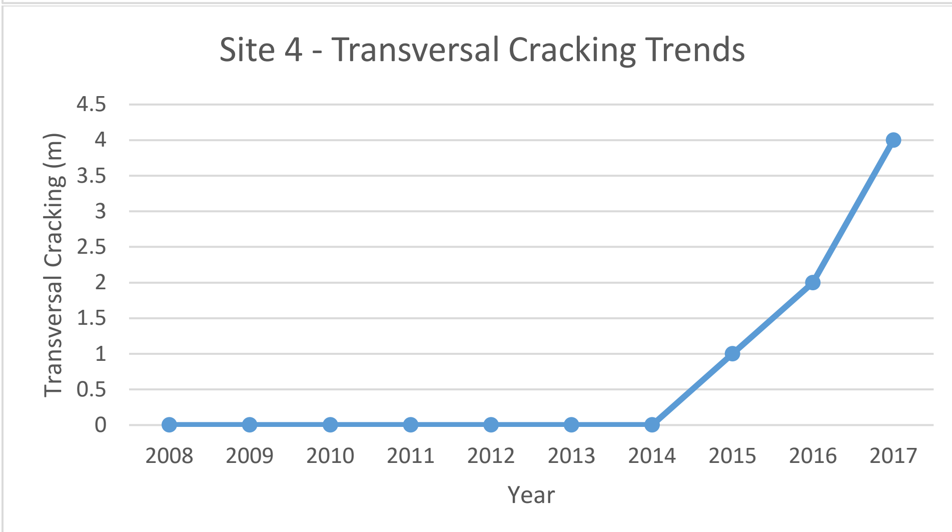
Recommendations

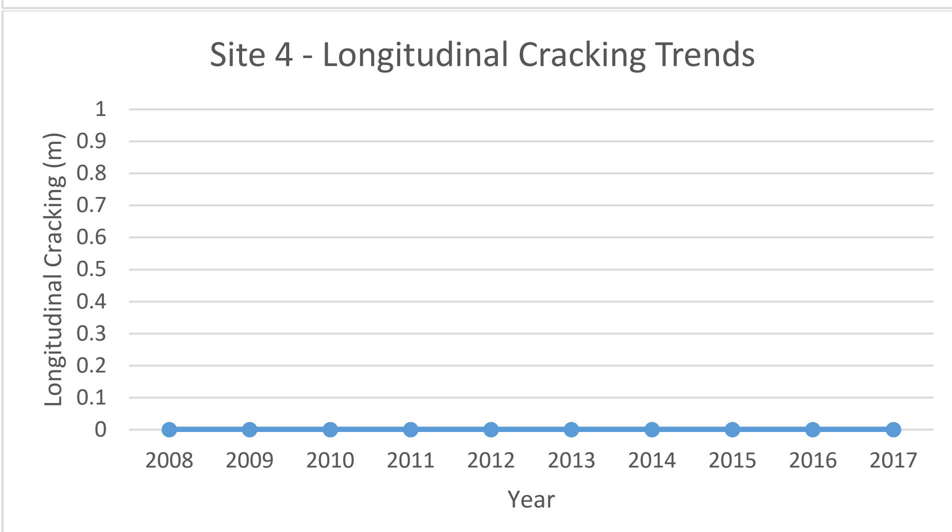
- Effective FDR Date: 2008
- Proposed distress curves are blue






Site 4 - Fatigue Trends
Fatigue (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 4 - Transversal Cracking Trends
Transversal Cracking (m)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 4 - Longitudinal Cracking Trends
Longitudinal Cracking (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year

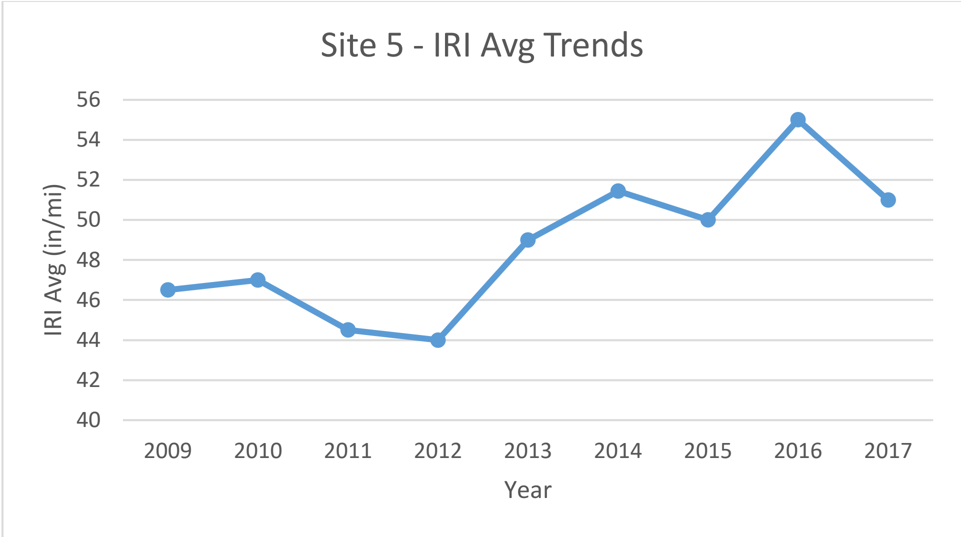
Site 5 — Proposed Median Distress Values Trending Over the Years

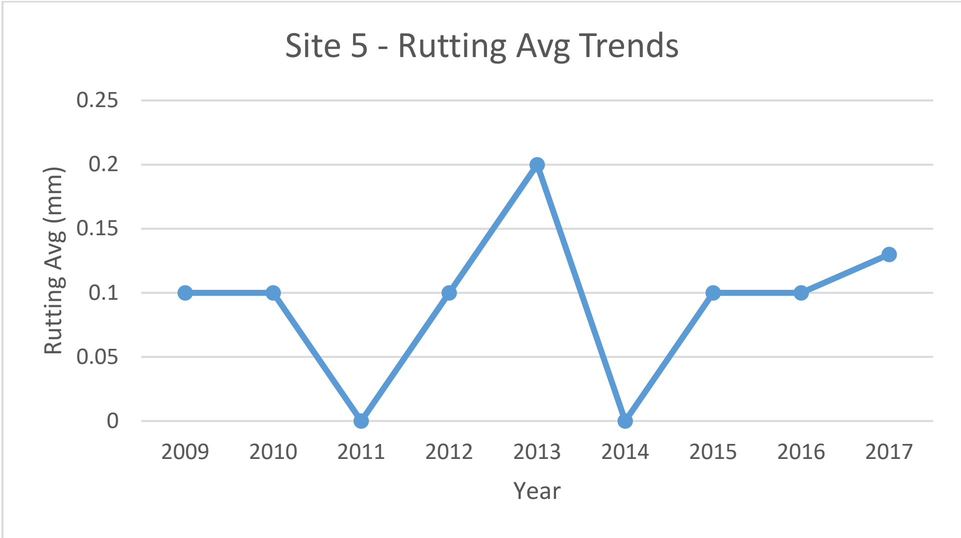
Background Information Based on PMS Records

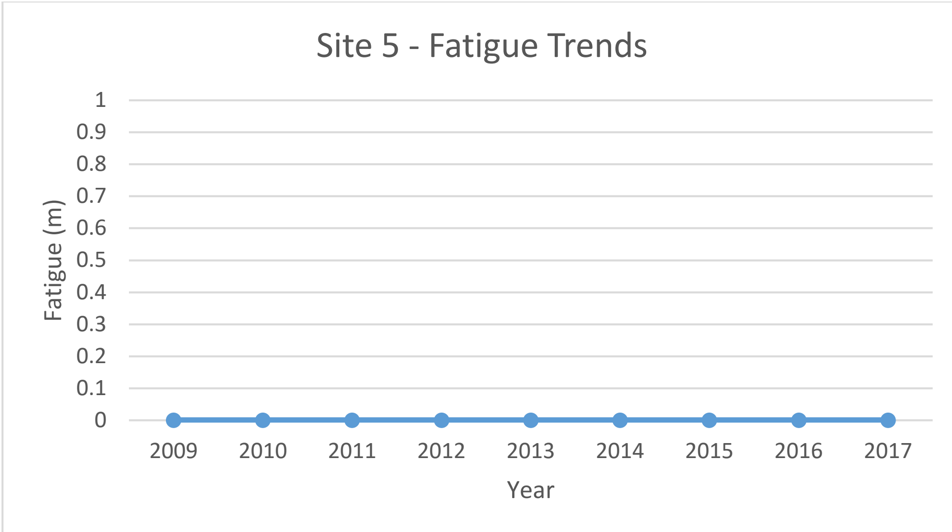
- Ad Date: MAY 31 2007
- Maintenance: None

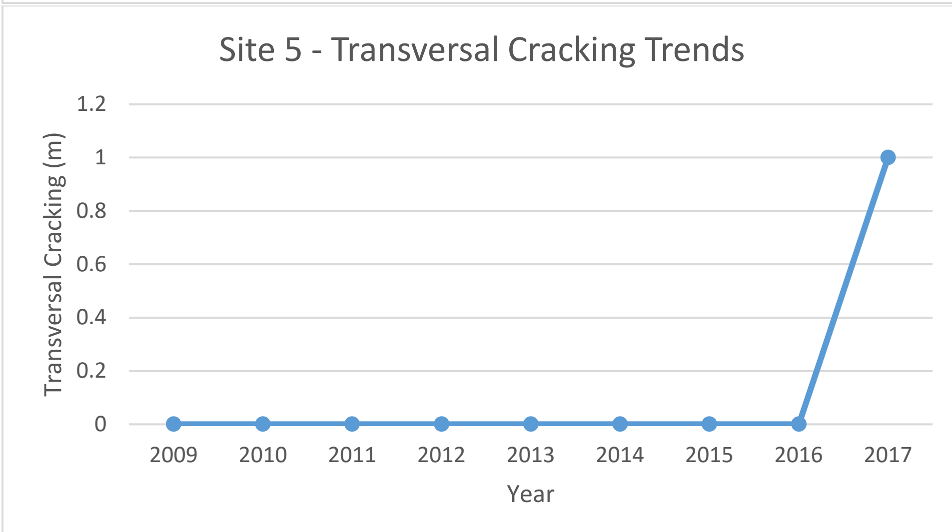
Recommendations

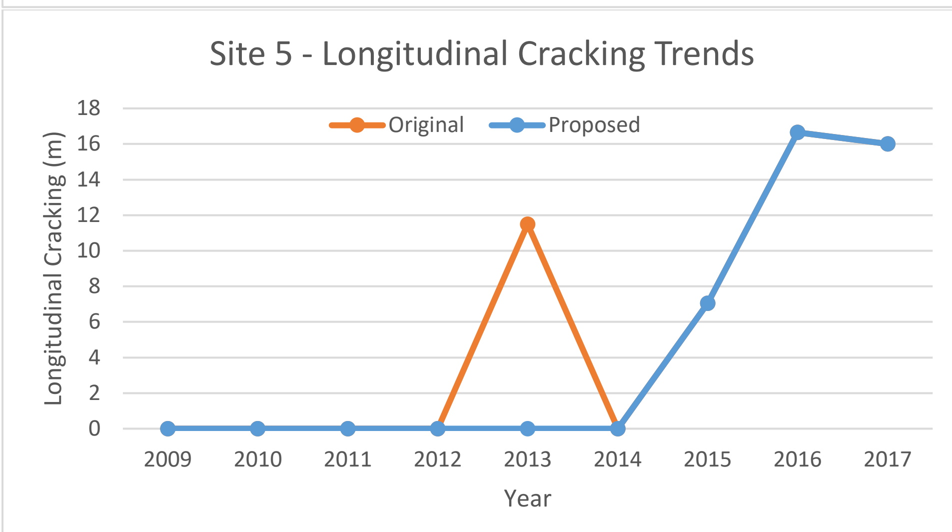
- Effective FDR Date: 2009
- Proposed distress curves are blue
- Original distress values shown in orange as applicable






Site 5 - Fatigue Trends
Fatigue (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 5 - Transversal Cracking Trends
Transversal Cracking (m)
1.2
1
0.8
0.6
0.4
0.2
0
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 5 - Longitudinal Cracking Trends
Original
Proposed
Longitudinal Cracking (m)
18
16
14
12
10
8
6
4
2
0
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year

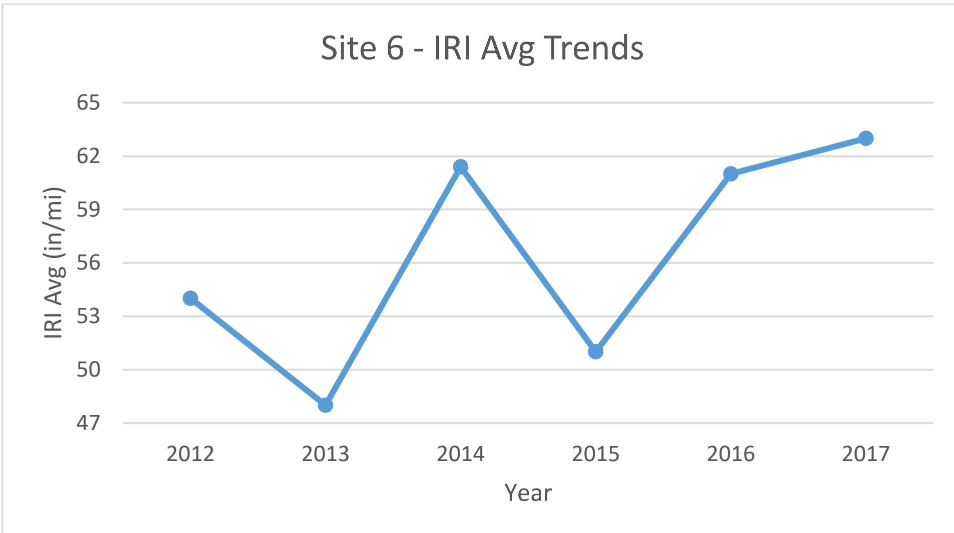
Site 6 — Proposed Median Distress Values Trending Over the Years

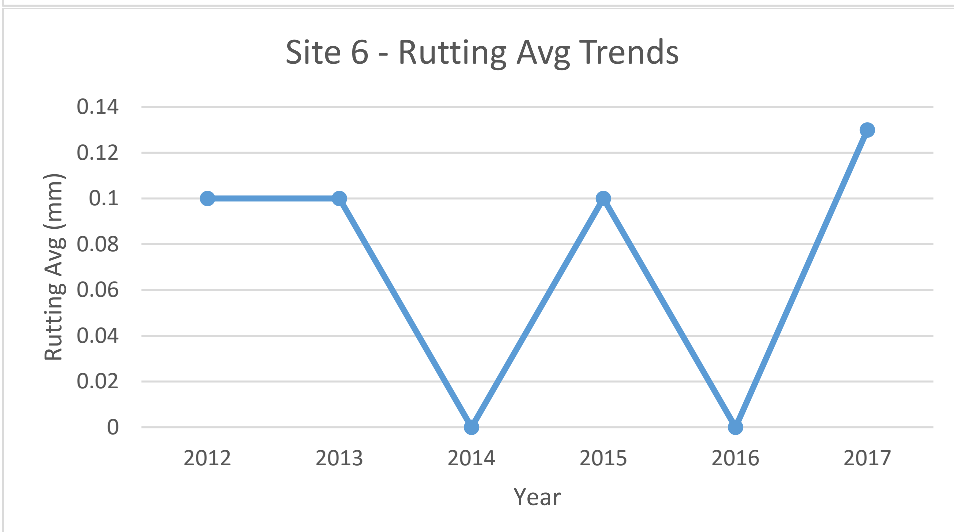
Background Information Based on PMS Records

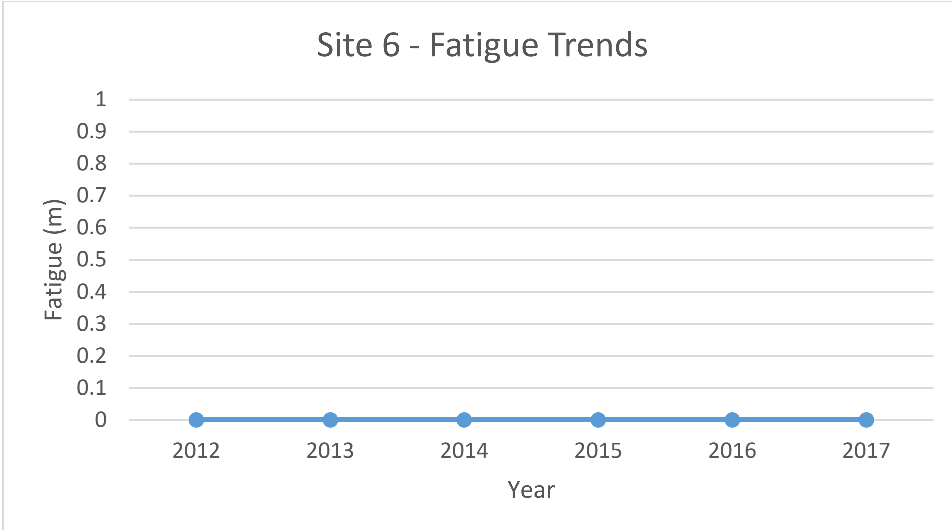
- Ad Date: SEP 09 2010
- Maintenance: None

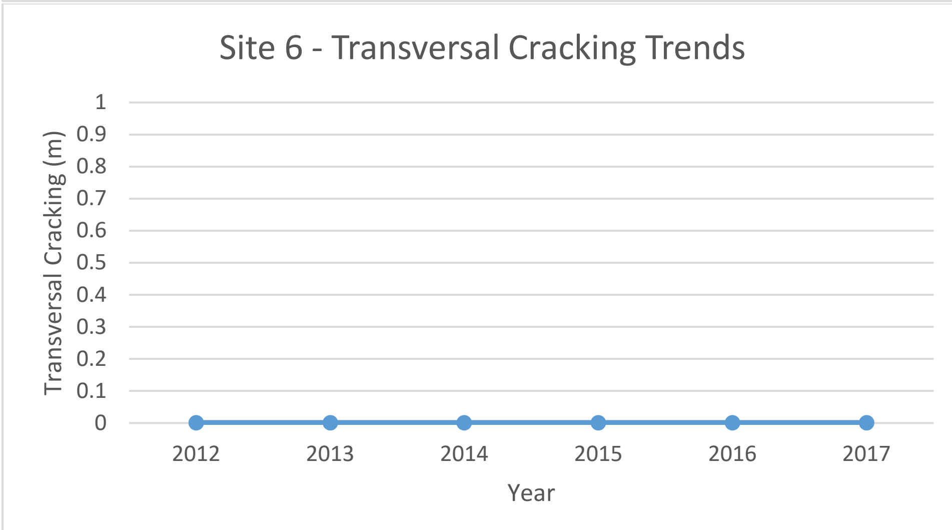
Recommendations

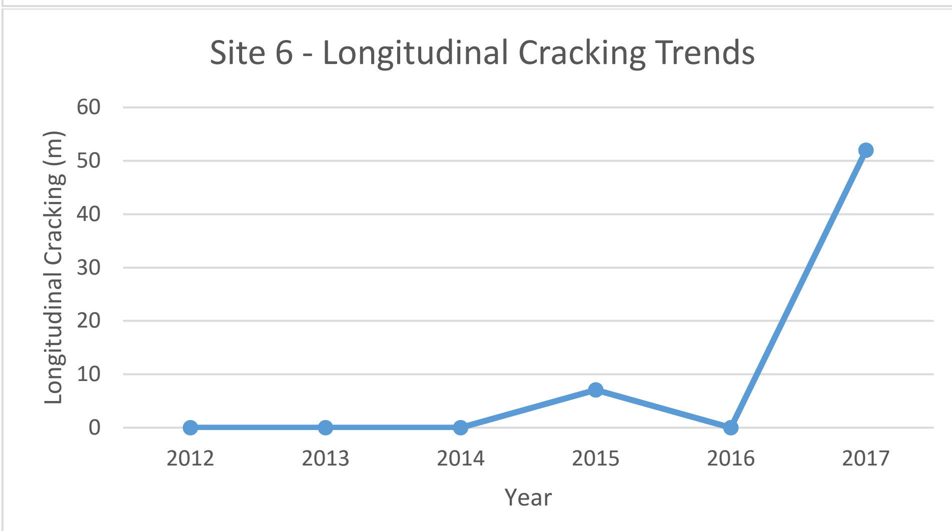
- Effective FDR Date: 2012
- Proposed distress curves are blue






Site 6 - Fatigue Trends
Fatigue (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2012
2013
2014
2015
2016
2017
Year


Site 6 - Transversal Cracking Trends
Transversal Cracking (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2012
2013
2014
2015
2016
2017
Year


Site 6 - Longitudinal Cracking Trends
Longitudinal Cracking (m)
60
50
40
30
20
10
0
2012
2013
2014
2015
2016
2017
Year

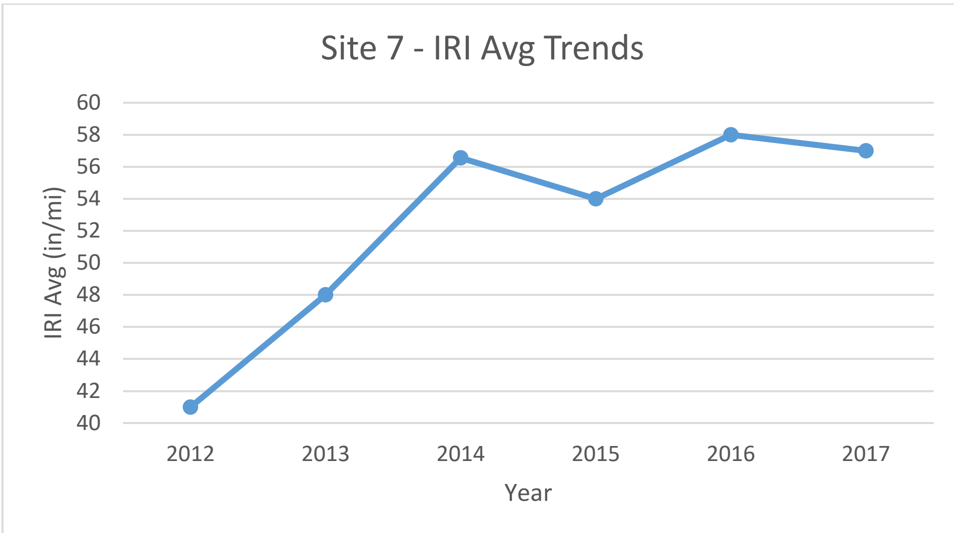
__Site 7 — Proposed Median Distress Values Trending Over the Years__

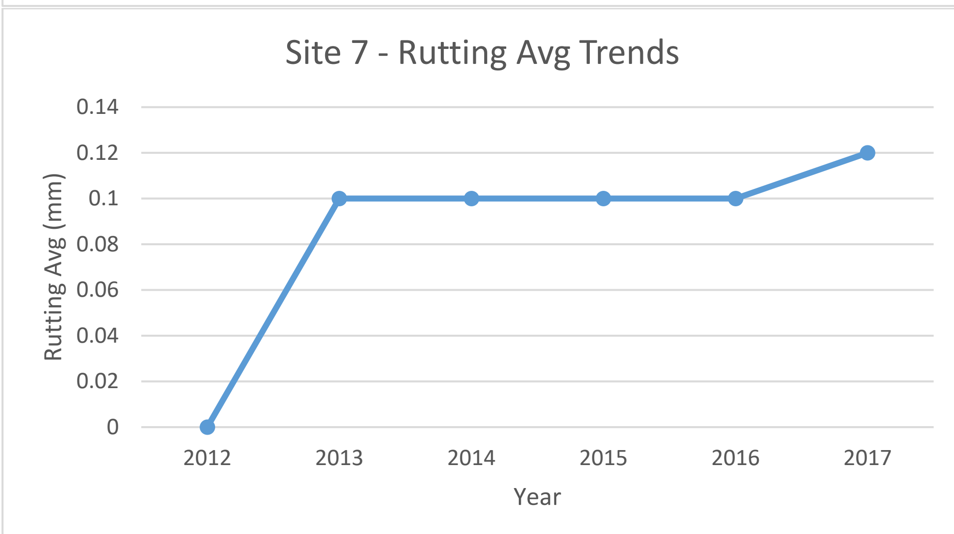
Background Information Based on PMS Records

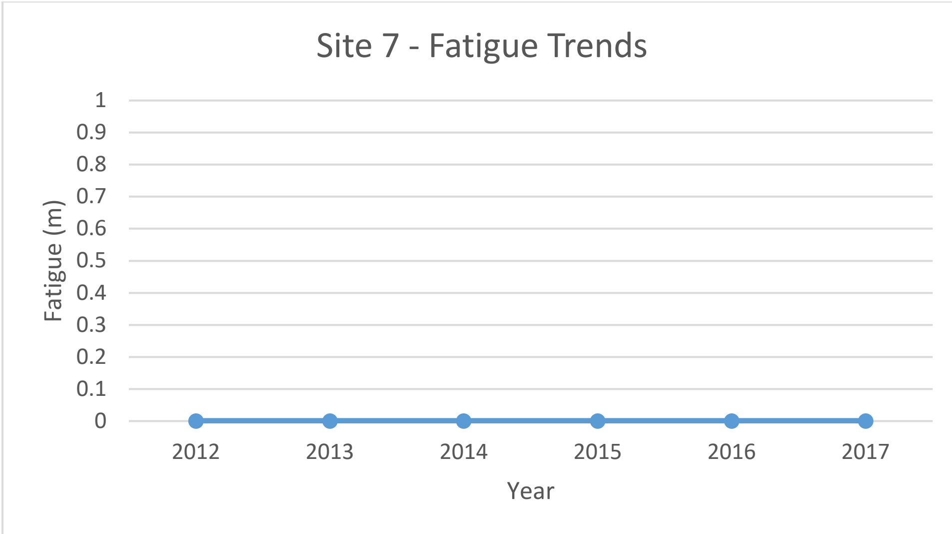
- Ad Date: SEP 01 2011
- Maintenance: Chip sealed in 2012

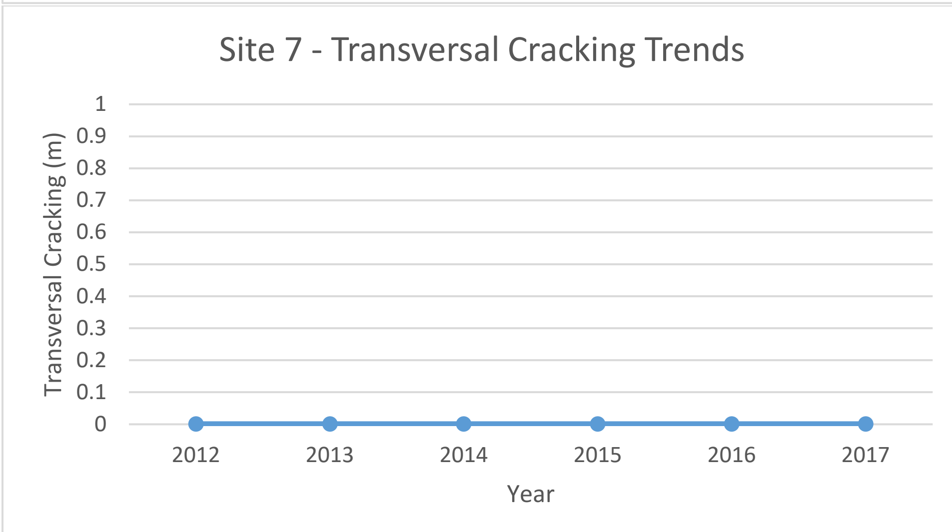
Recommendations

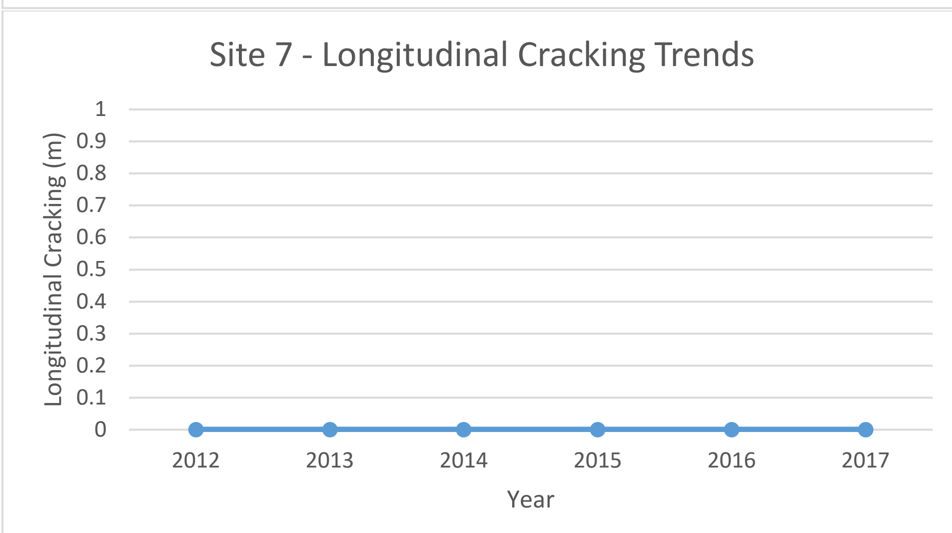
- Effective FDR Date: 2012
- Proposed distress curves are blue






Site 7 - Fatigue Trends
Fatigue (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2012
2013
2014
2015
2016
2017
Year


Site 7 - Transversal Cracking Trends
Transversal Cracking (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2012
2013
2014
2015
2016
2017
Year


Site 7 - Longitudinal Cracking Trends
Longitudinal Cracking (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2012
2013
2014
2015
2016
2017
Year

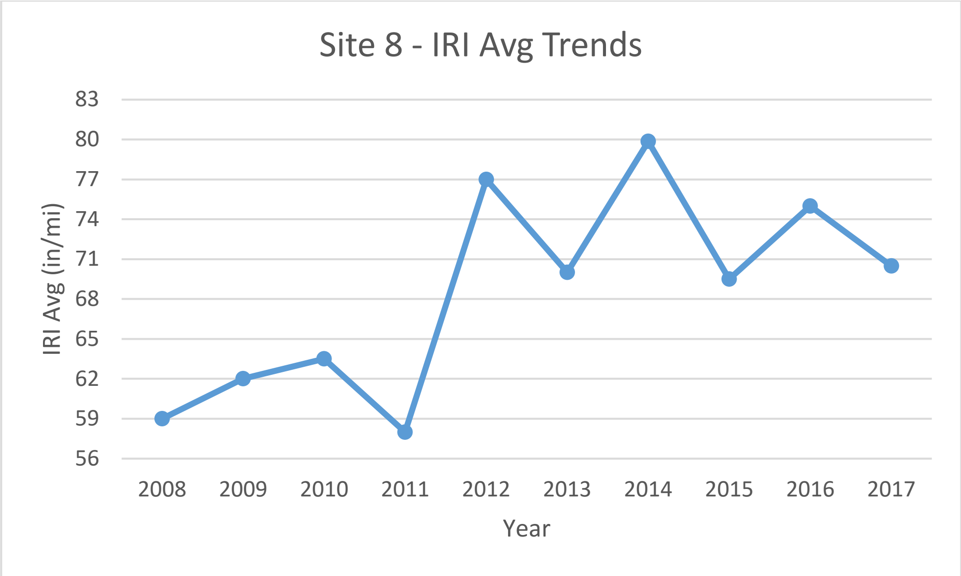
Site 8 — Proposed Median Distress Values Trending Over the Years

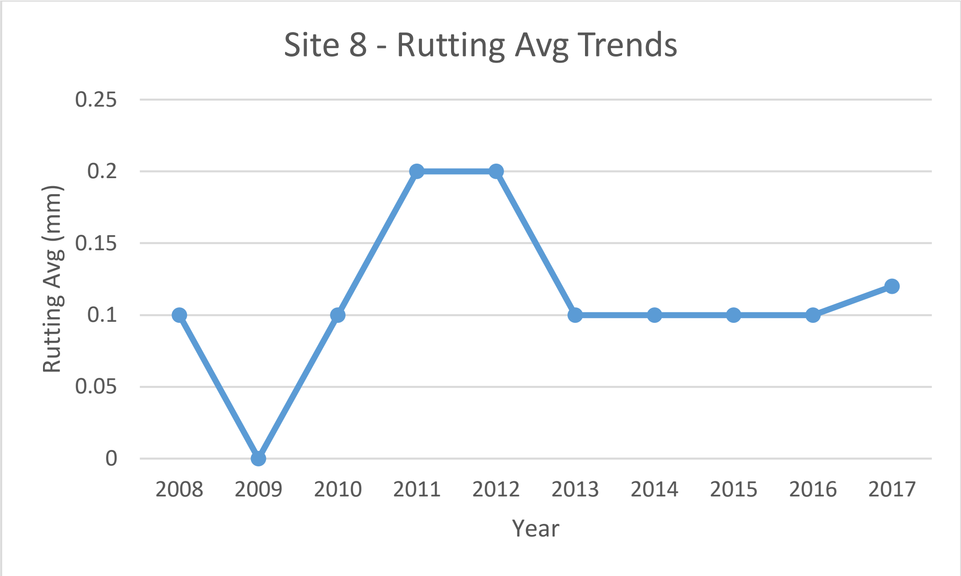
Background Information Based on PMS Records

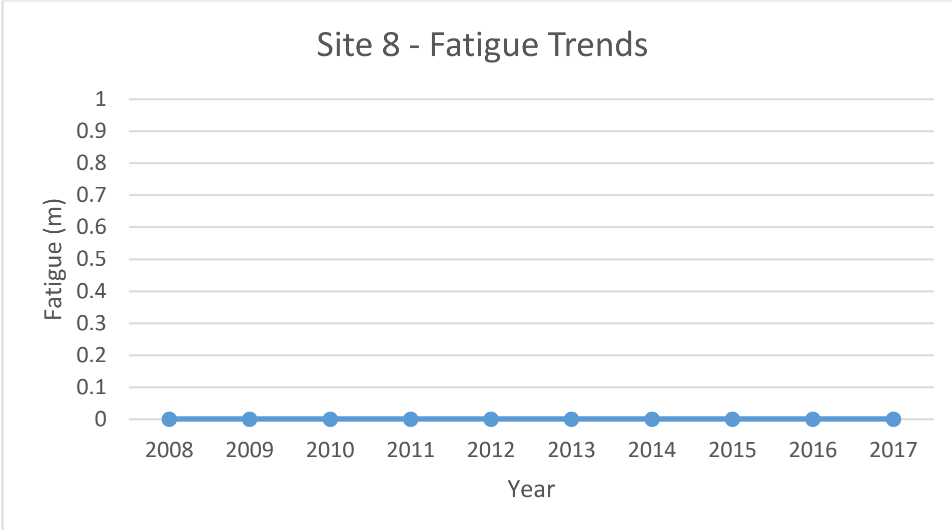
- Ad Date: APR 26 2007
- Maintenance: Chip sealed in 2011

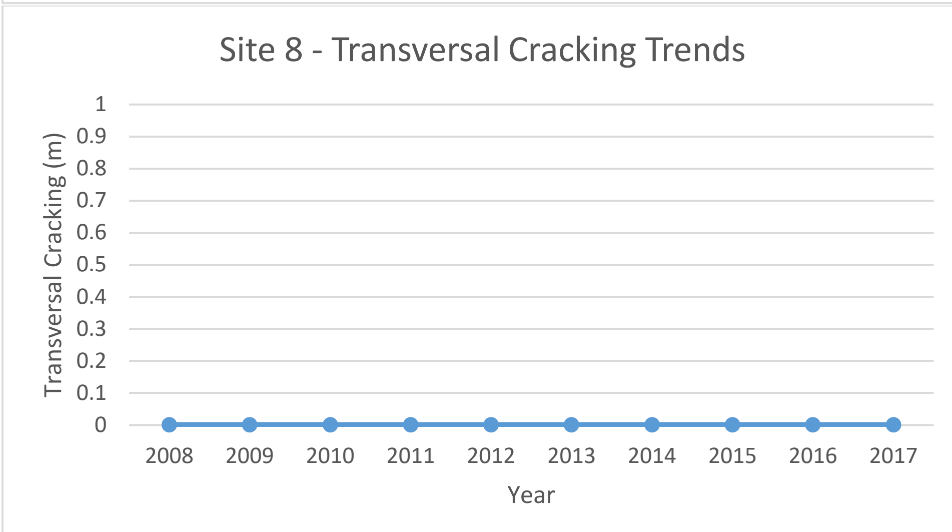
Recommendations

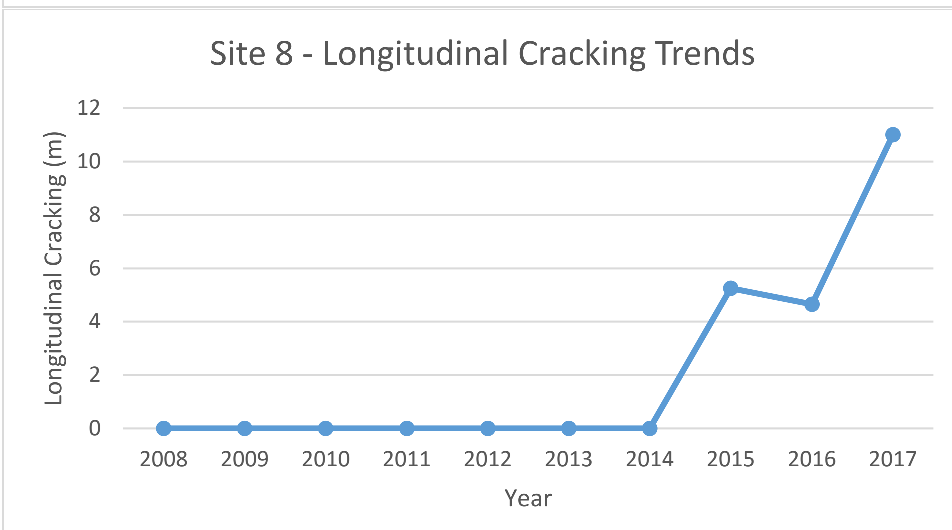
- Effective FDR Date: 2008
- Proposed distress curves are blue






Site 8 - Fatigue Trends
Fatigue (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 8 - Transversal Cracking Trends
Transversal Cracking (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 8 - Longitudinal Cracking Trends
Longitudinal Cracking (m)
12
10
8
6
4
2
0
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
Year

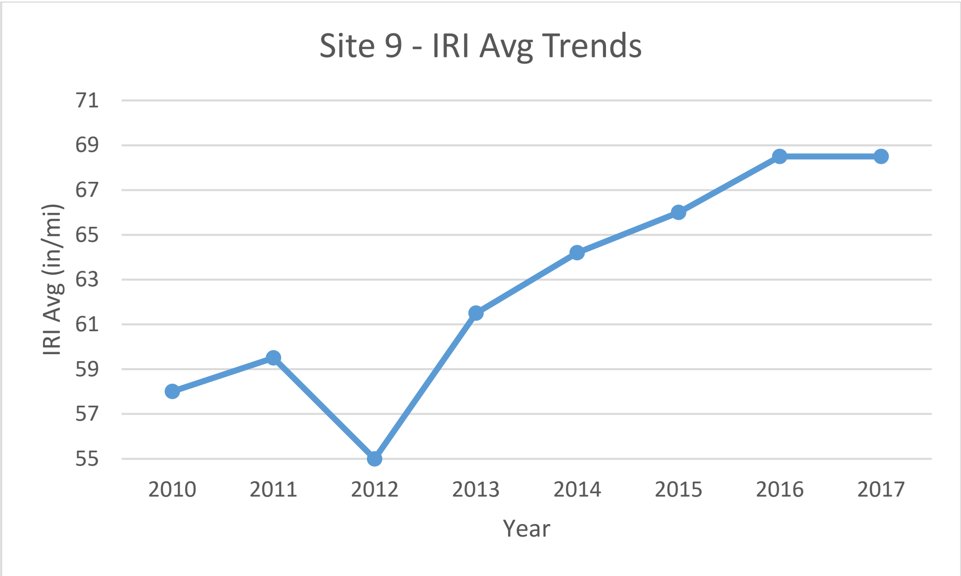
Site 9 — Proposed Median Distress Values Trending Over the Years

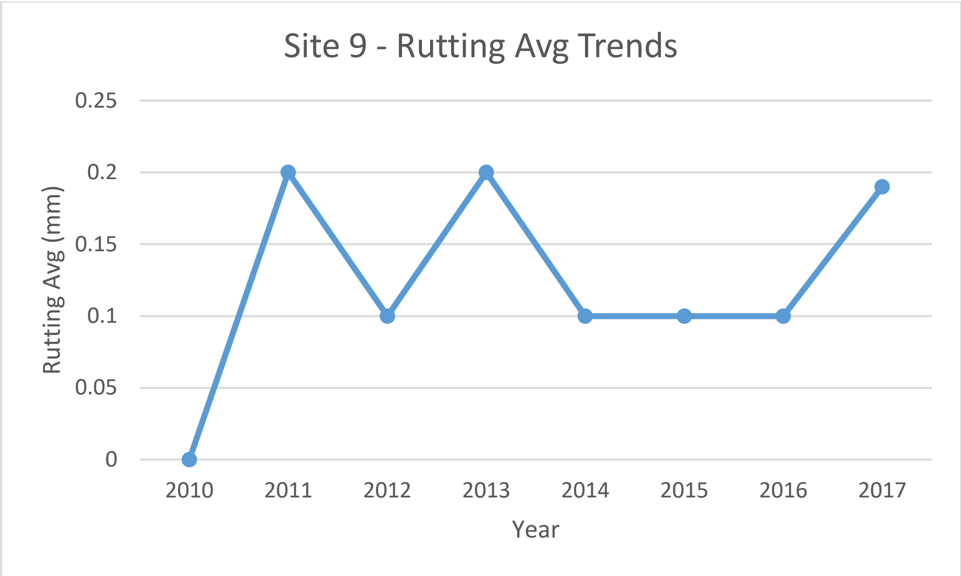
Background Information Based on PMS Records

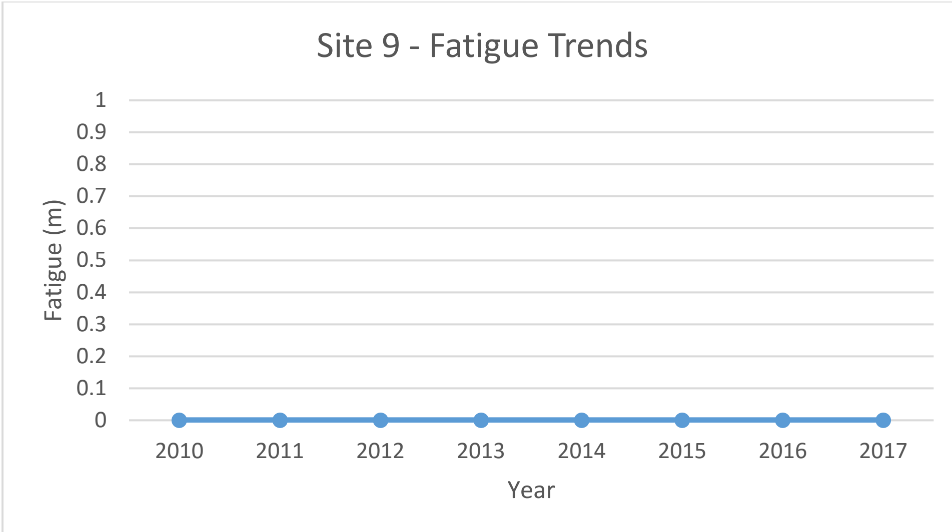
- Ad Date: JAN 10 2008
- Maintenance: None

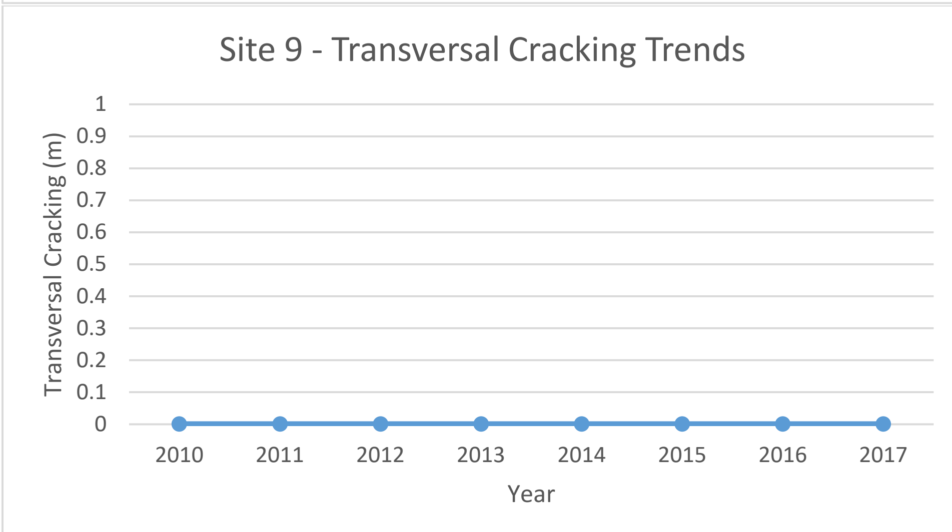
Recommendations

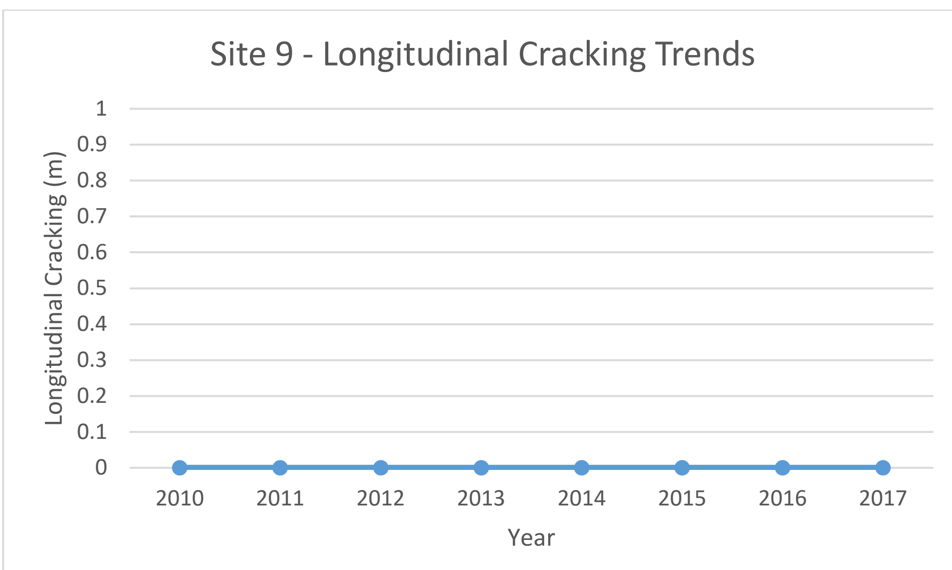
- Effective FDR Date: 2010
- Proposed distress curves are blue






Site 9 - Fatigue Trends
Fatigue (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 9 - Transversal Cracking Trends
Transversal Cracking (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2010
2011
2012
2013
2014
2015
2016
2017
Year


Site 9 - Longitudinal Cracking Trends
Longitudinal Cracking (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2010
2011
2012
2013
2014
2015
2016
2017
Year

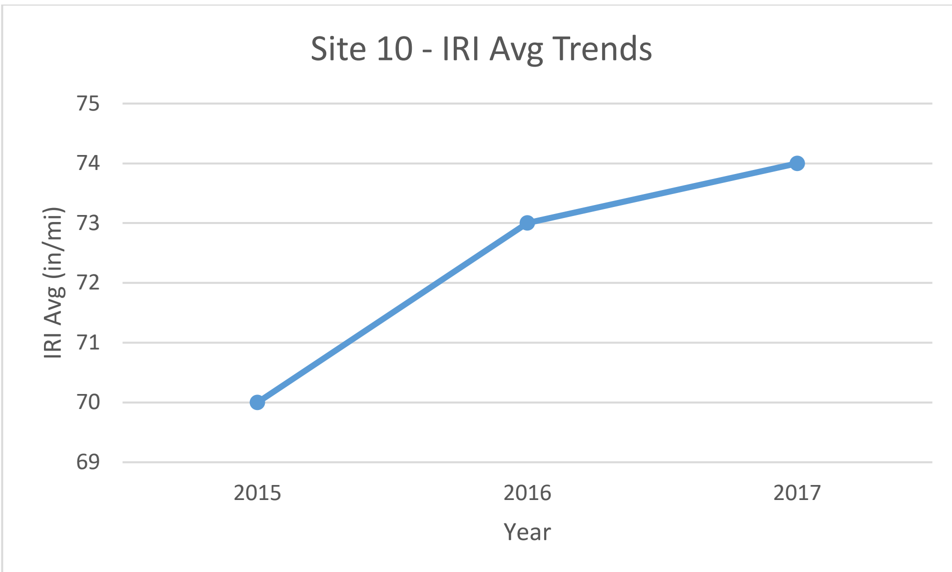
Site 10 — Proposed Median Distress Values Trending Over the Years

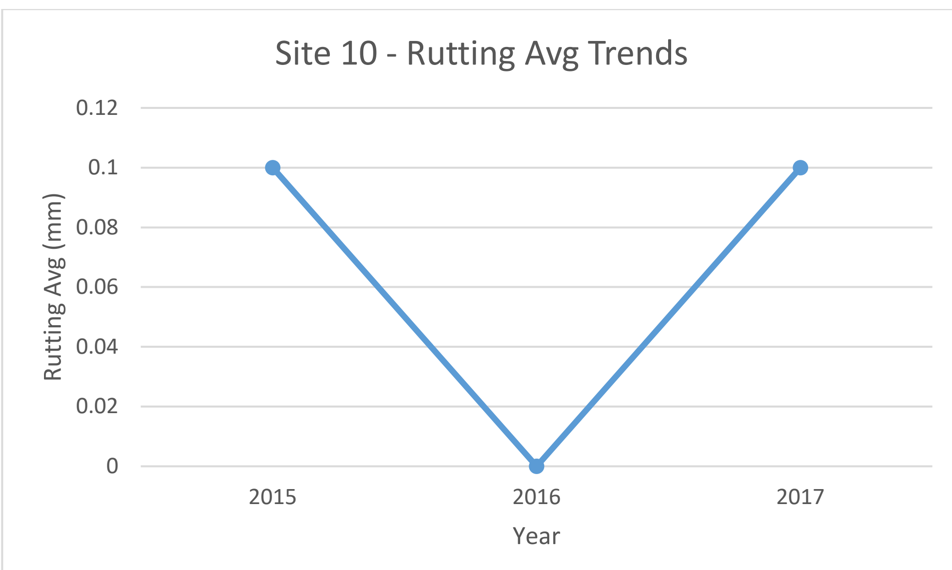
Background Information Based on PMS Records

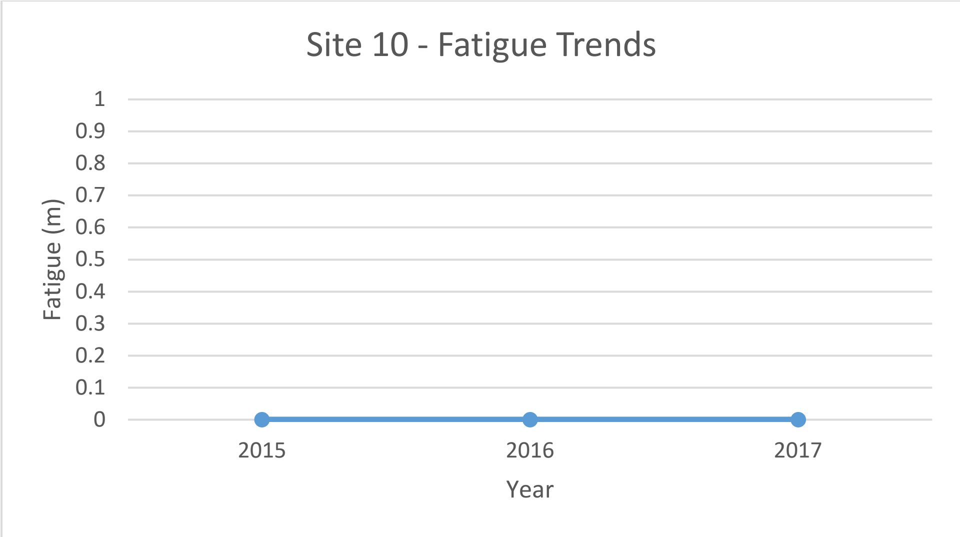
- Ad Date: NOV 08 2012
- Maintenance: None

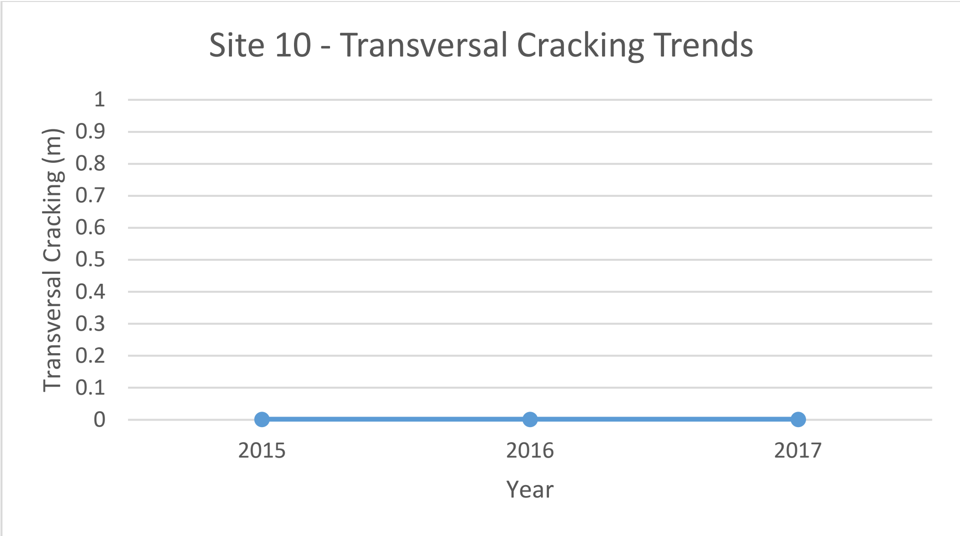
Recommendations

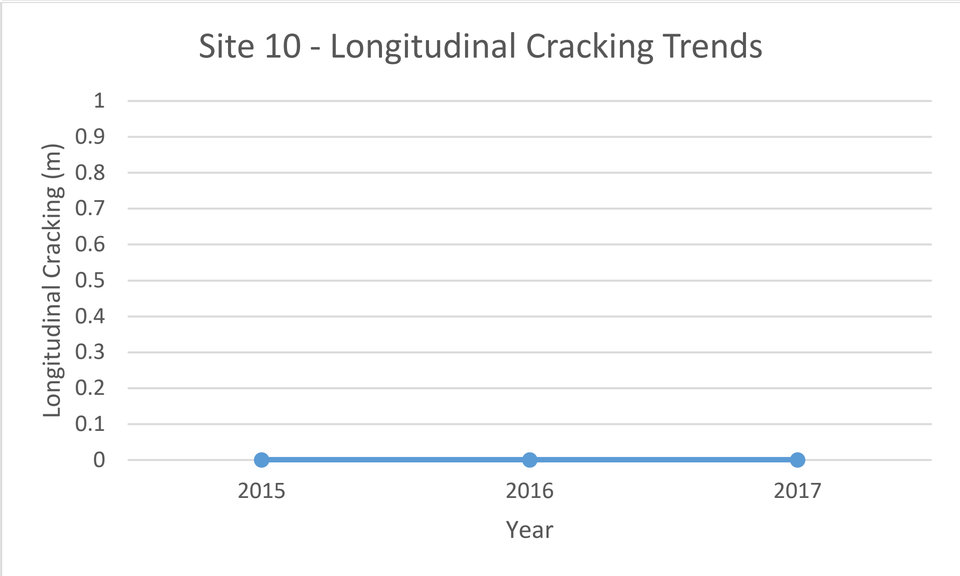
- Effective FDR Date: 2015
- Proposed distress curves are blue






Site 10 - Fatigue Trends
Fatigue (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2015
2016
2017
Year


Site 10 - Transversal Cracking Trends
Transversal Cracking (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2015
2016
2017
Year


Site 10 - Longitudinal Cracking Trends
Longitudinal Cracking (m)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2015
2016
2017
Year

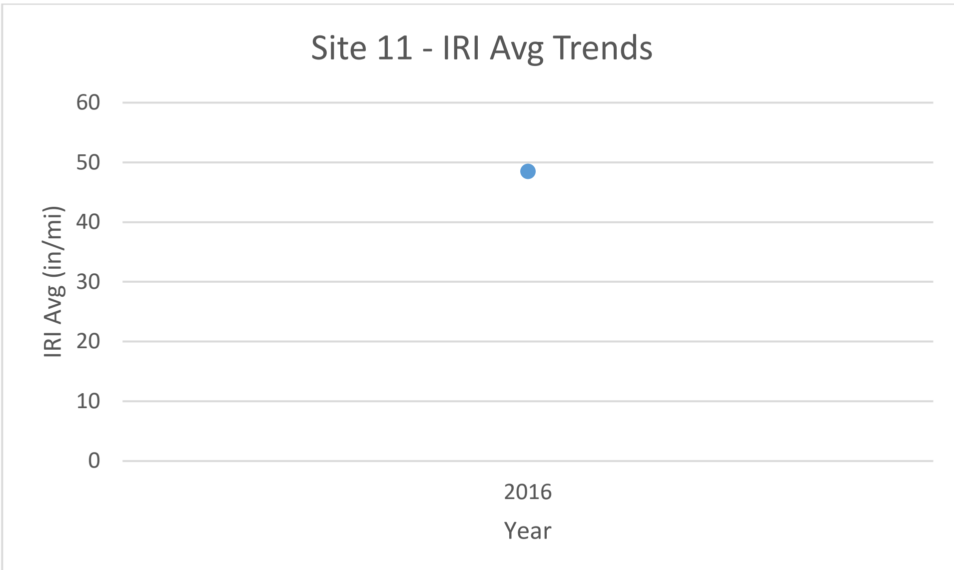
Site 11 — Proposed Median Distress Values Trending Over the Years

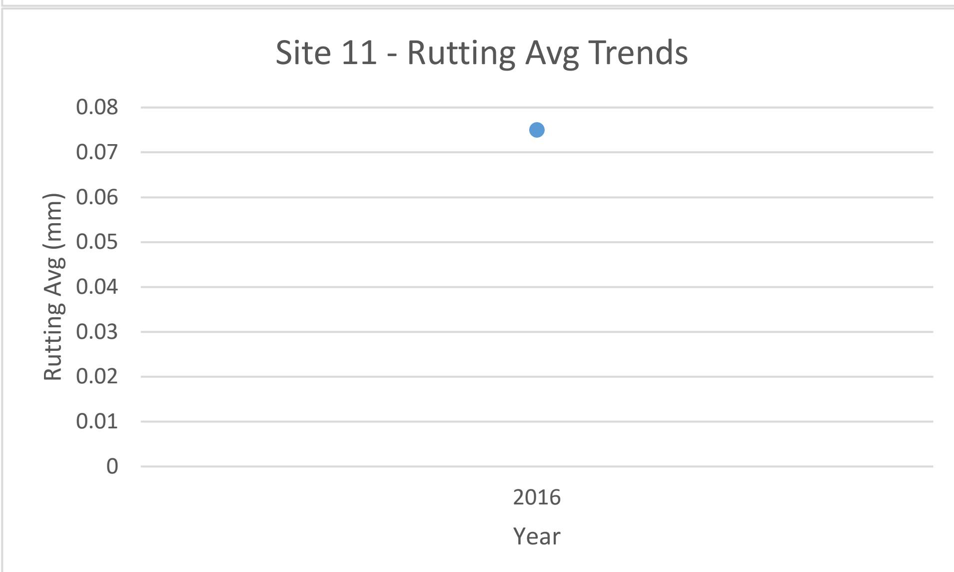
Background Information Based on PMS Records

- Ad Date: 2014
- Maintenance: Yes (no PMS information on year)
- No distress data on fatigue, transversal and longitudinal cracking for site 11 was collected and provided to the research team.

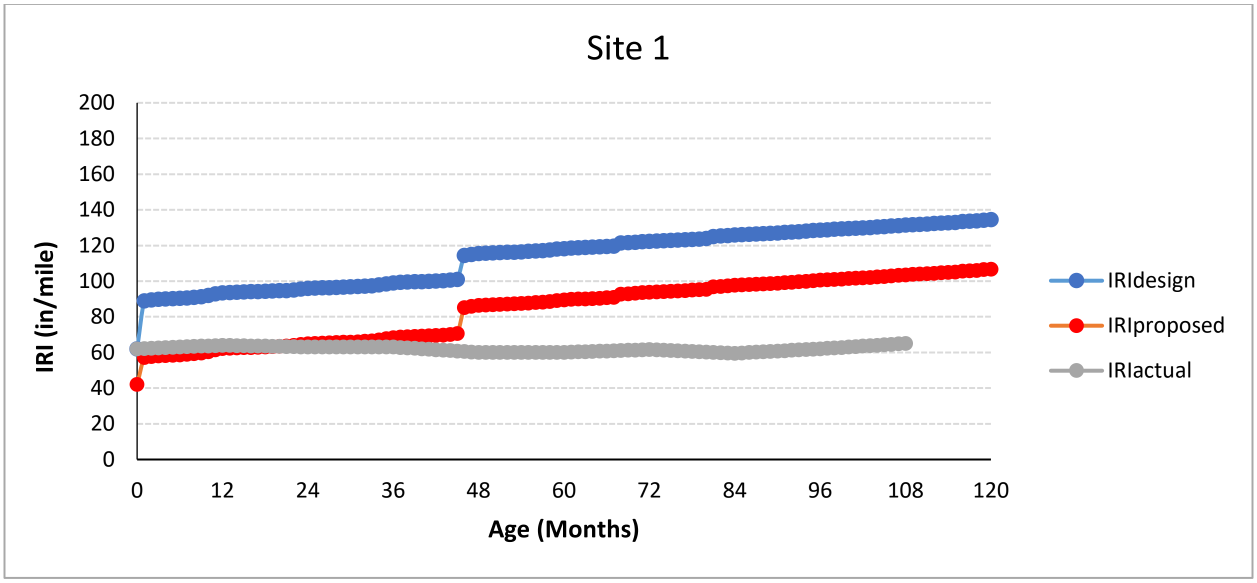
Recommendations

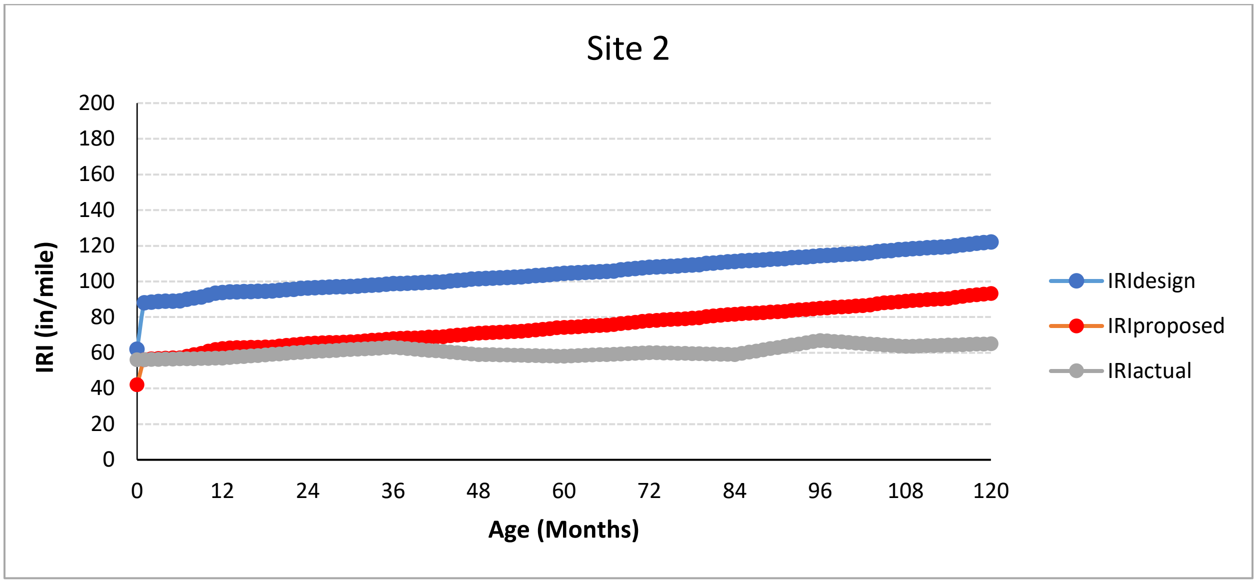
- Effective FDR Date: 2015
- Assumed Maintenance: 2014 (same as effective FDR rehabilitation date)
- Proposed distress curves are blue

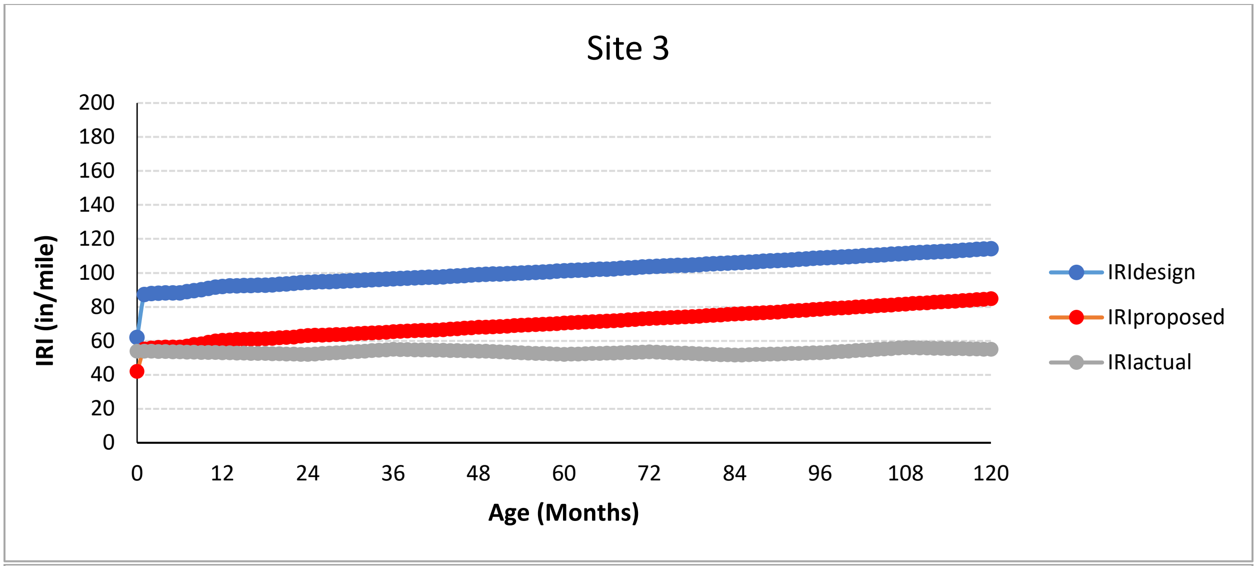


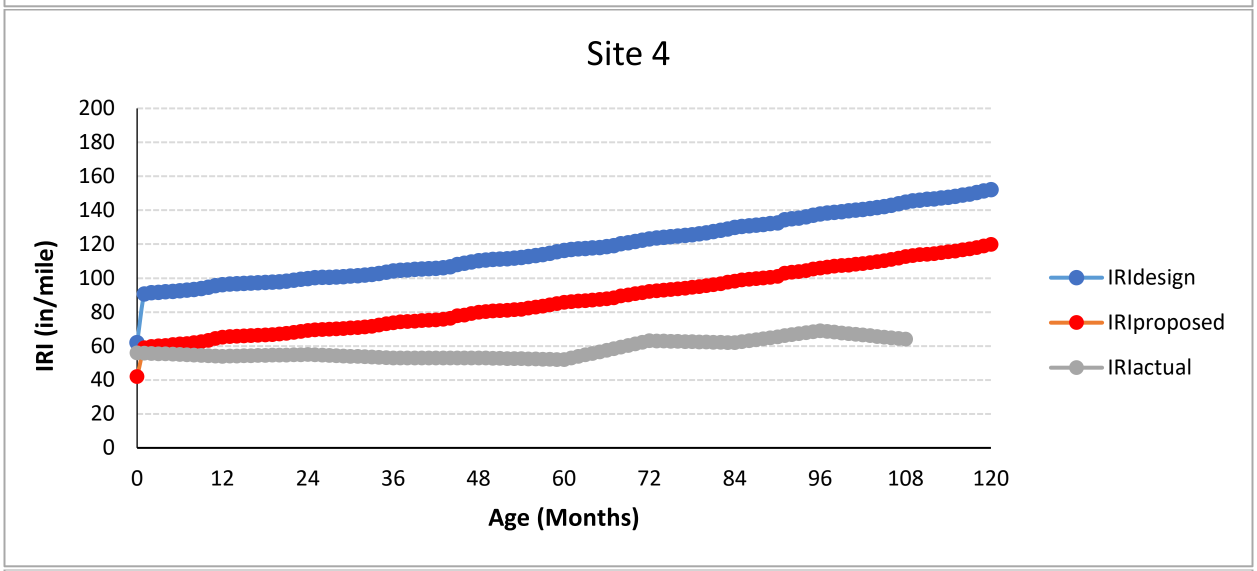


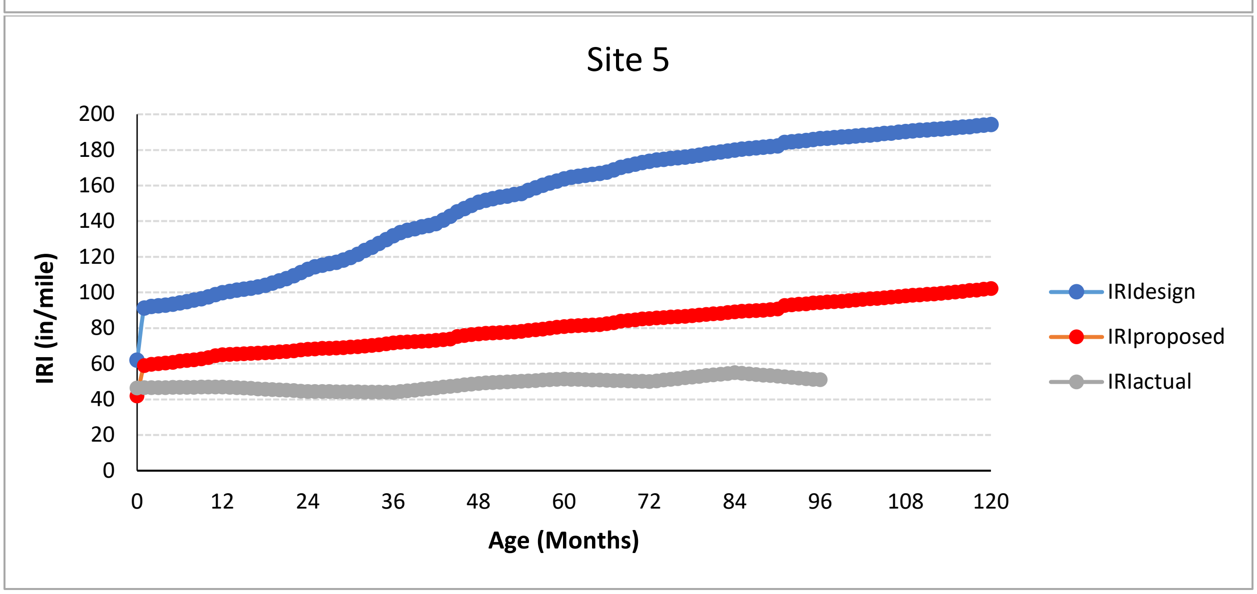
Appendix B: Comparison of actual and ME predicted performance based on current design and proposed input parameters

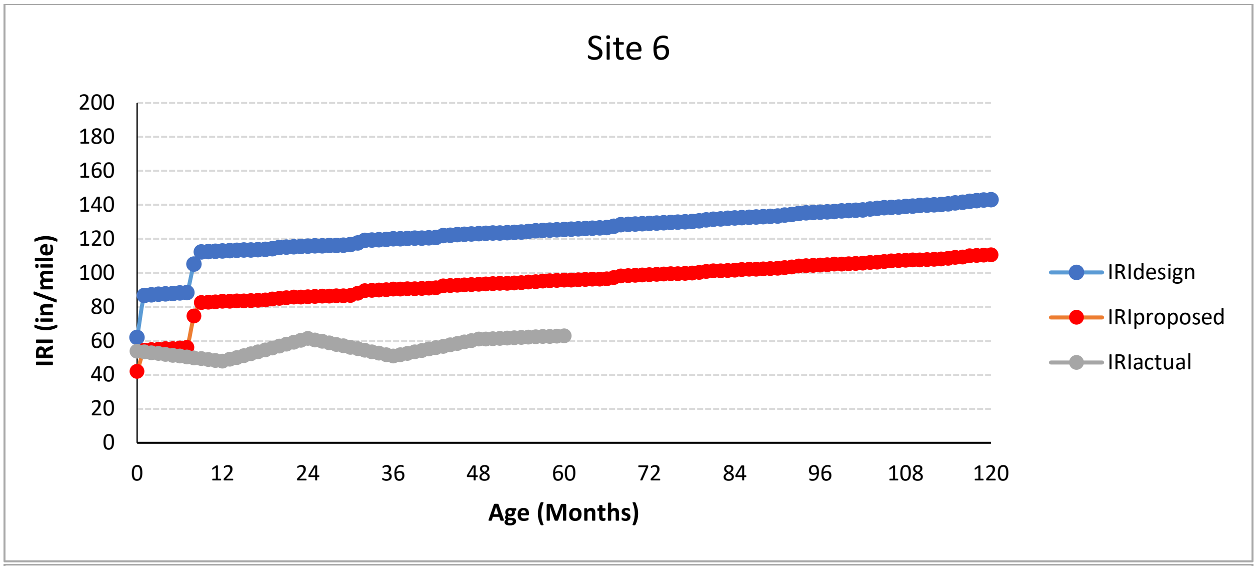


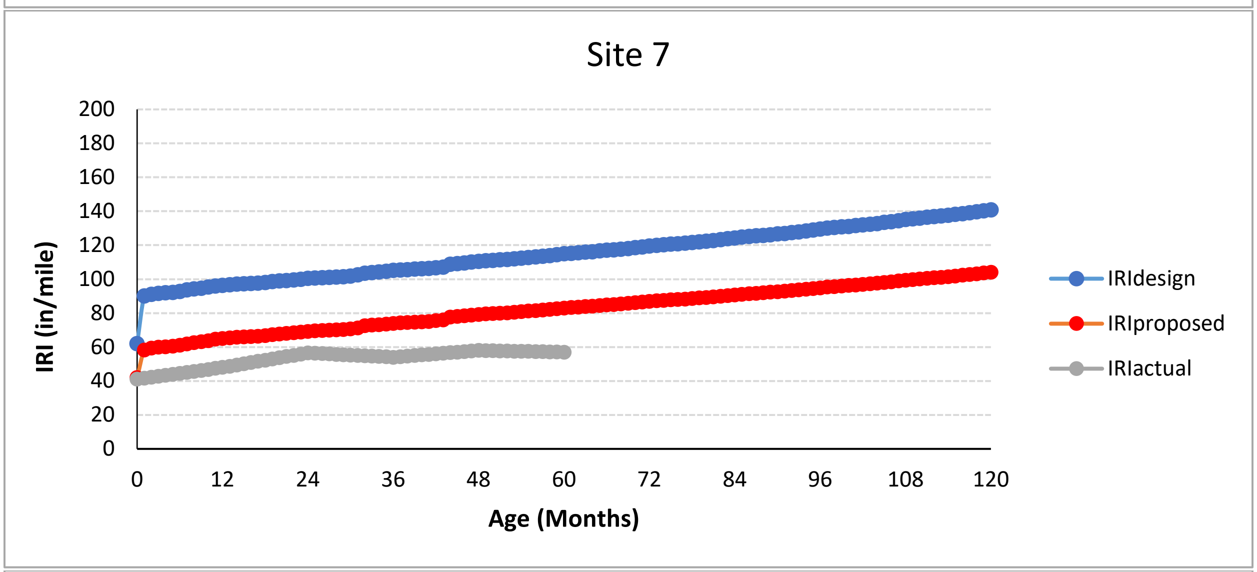


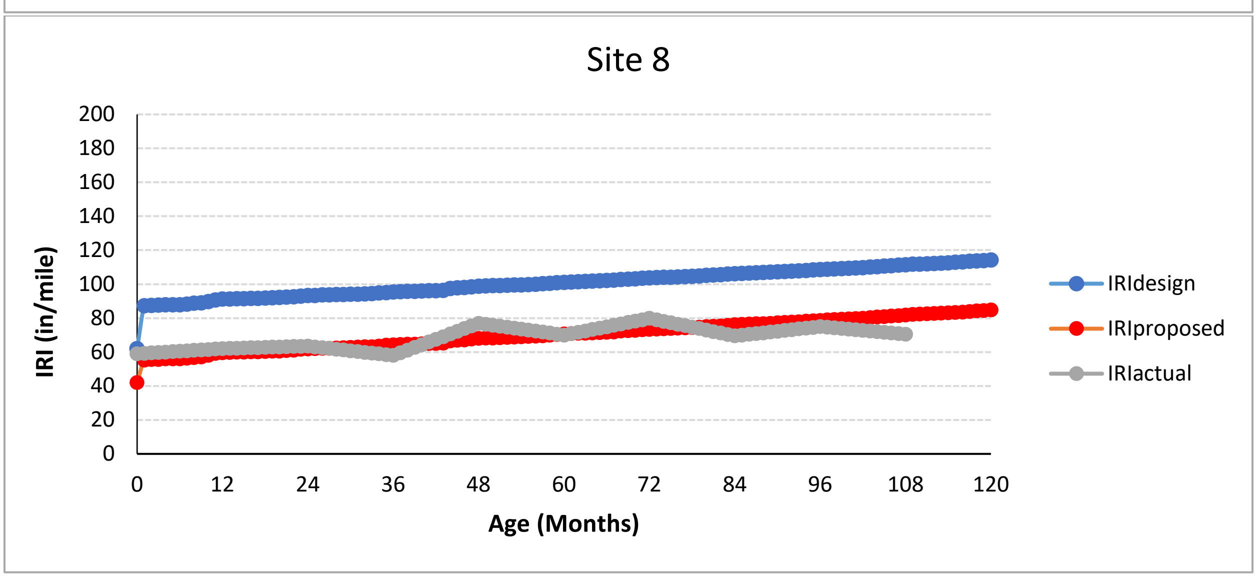

Site 3
IRI (in/mile)
200
180
160
140
120
100
80
60
40
20
0
0
12
24
36
48
60
72
84
96
108
120
Age (Months)
IRIdesign
IRIproposed
IRIactual

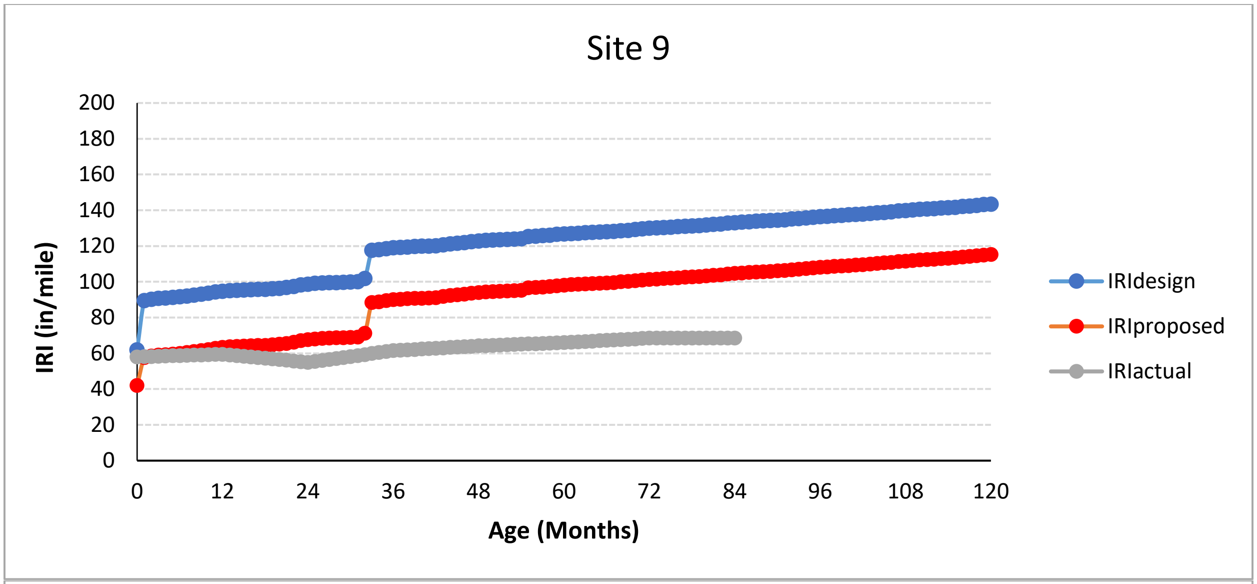

Site 4
IRI (in/mile)
200
180
160
140
120
100
80
60
40
20
0
0
12
24
36
48
60
72
84
96
108
120
Age (Months)
IRIdesign
IRIproposed
IRIactual


Site 5
IRI (in/mile)
200
180
160
140
120
100
80
60
40
20
0
0
12
24
36
48
60
72
84
96
108
120
Age (Months)
IRIdesign
IRIproposed
IRIactual

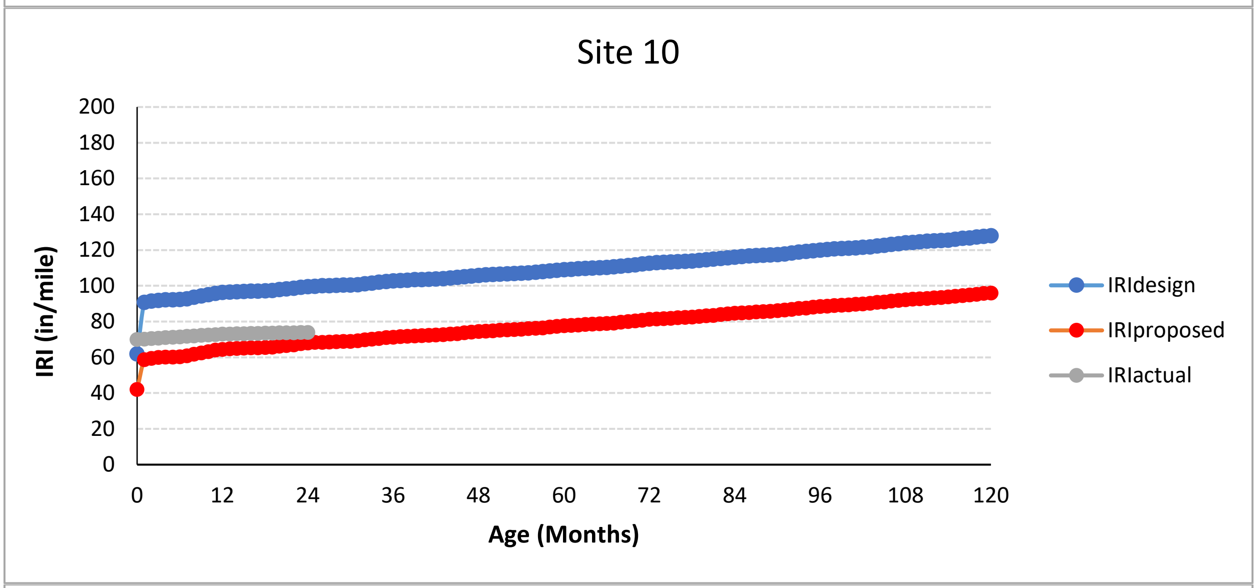

Site 6
IRI (in/mile)
200
180
160
140
120
100
80
60
40
20
0
0
12
24
36
48
60
72
84
96
108
120
Age (Months)
IRIdesign
IRIproposed
IRIactual


Site 7
IRI (in/mile)
200
180
160
140
120
100
80
60
40
20
0
0
12
24
36
48
60
72
84
96
108
120
Age (Months)
IRIdesign
IRIproposed
IRIactual

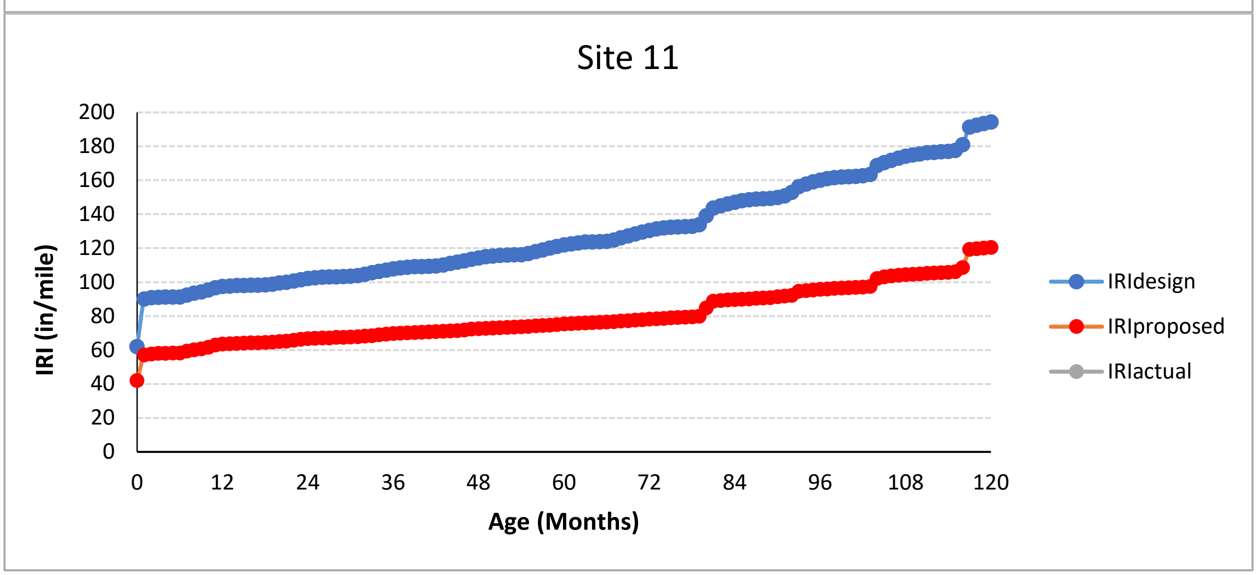

Site 8
IRI (in/mile)
200
180
160
140
120
100
80
60
40
20
0
0
12
24
36
48
60
72
84
96
108
120
Age (Months)
IRIdesign
IRIproposed
IRIactual



Site 9
200
180
160
140
120
100
80
60
40
20
0
IRI (in/mile)
0
12
24
36
48
60
72
84
96
108
120
Age (Months)
IRIdesign
IRIproposed
IRIactual



Site 10
200
180
160
140
120
100
80
60
40
20
0
IRI (in/mile)
0
12
24
36
48
60
72
84
96
108
120
Age (Months)
IRIdesign
IRIproposed
IRIactual



Site 11
200
180
160
140
120
100
80
60
40
20
0
IRI (in/mile)
0
12
24
36
48
60
72
84
96
108
120
Age (Months)
IRIdesign
IRIproposed
IRIactual