



TEXAS TECH UNIVERSITY

Multidisciplinary Research in Transportation

Improvements of Full Depth Repair Practices for CRCP Distresses

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Texas Department of Transportation

Report #: 0-6611-1
www.techmrt.ttu.edu/reports.php

September 2013

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1. Report No. FHWA/TX-12-0-6611-1	2. Government Accession No.:	3. Recipient's Catalog No.:	
4. Title and Subtitle: Improvements of Full Depth Repair Practices for CRCP Distresses		5. Report Date: August 2012	
7. Author(s): Sungwoo Ryu, Pangil Choi, Wujun Zhou, Sureel Saraf, Moon C. Won,		8. Performing Organization Report No. 0-6611-1	
9. Performing Organization Name and Address: Texas Tech University College of Engineering Box 41023 Lubbock, Texas 79409-1023		10. Work Unit No.(TRAIS):	
		11. Contract or Grant No.: Project 0-6611	
12. Sponsoring Agency Name And Address Texas Department Of Transportation Research and Technology Implementation Office P.O. Box 5080 Austin, TX 78763-5080		13. Type of Report and Period Cover: Technical Report 09/2010-08/2012	
		14. Sponsoring Agency Code:	
15. Supplementary Notes: Project performed in cooperation with Texas Department of Transportation and the Federal Highway Administration			
16. Abstract: The Texas Department of Transportation (TxDOT) has by far the most continuously reinforced concrete pavement (CRCP) lane miles in the nation and sections as old as 50 years are still in service. Having served much longer than intended, some sections are showing distresses. FDR (full depth repair) is one of the methods to repair CRCP distresses in Texas. Over the years, various FDR methods have been used and the effectiveness of each method has varied. The most widely used FDR method – where a full-depth cut is made at 2 ft (30.48 cm) inside the transverse repair boundaries and partial-depth cut at repair boundaries with the concrete in between removed to expose longitudinal steel – has inherent disadvantages, with longer repair time required being the primary disadvantage. Full-depth cut FDR method – where a full-depth cut is made at repair boundaries with transverse and longitudinal tie bars epoxy grouted into the existing concrete – has advantages over other methods, one of which is the faster operation, minimizing the time of roadway closure. Since CRCP is normally utilized at high traffic volume areas, the maximum time allowed for the FDR operation in TxDOT is normally limited to nine hours, which makes the full-depth cut method the only acceptable repair method. Factors affecting the effectiveness of the full-depth cut method were investigated by laboratory testing and field evaluations. The way epoxy is injected into the holes was the most important variable affecting the performance of FDR. Based on the research findings, recommendations were made to revise specifications for FDR, and it is expected that the implementation will result in improved FDR performance of CRCP. Some distresses in CRCP are limited to the upper half of the slab depth, and for these distresses, partial depth repair (PDR) is the more effective method. A device called MIRA was evaluated for the detection of partial depth failures in CRCP. MIRA was able to detect not only delaminations, but voids and mud balls in concrete slab, reinforcement, and slab thickness. For the detection of partial depth distresses, MIRA can be a useful tool. Guidelines, special specifications, and design standards for PDR were developed.			
17. Key Words: FDR, PDR, FWD, Pullout test, Filed test		18. Distribution Statement No Restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161, www.ntis.gov	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. Of Pages 127	22. Price

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Project Report 0-6611-1

Project Number 0-6611

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ACKNOWLEDGMENTS

This research study was sponsored by the Texas Department of Transportation in cooperation with the Federal Highway Administration. The authors express their gratitude to Mr. Kit Black, project director, and PMC members, Quincy Allen, Dar Hao Chen, Hua Chen, Bernado Ferrel, Gerald Lankes, Chris Reed, and Paul Wong, and RTI Research Engineer, Dr. German Claros. In addition, the authors acknowledge the support provided by the contractors and TxDOT personnel during field testing.

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Chapter 1 Introduction

1.1 Background

Continuously reinforced concrete pavement (CRCP) is the most widely used rigid pavement type by the Texas Department of Transportation (TxDOT) primarily due to its excellent performance. As of 2010, TxDOT had 12,345 lane miles of CRCP, with some as old as 50 years still in service ⁽¹⁾. Having served much longer than intended, some sections are showing distresses. Over the years, various methods of full-depth repair (FDR) have been used to restore structural condition of the pavement. Until the early 1990s, the primary method used in Texas consisted of full-depth cuts at 2 ft away from transverse repair joints and partial-depth cuts at repair joints, with removal of the concrete in between by jack-hammering to expose 2 ft of longitudinal steel. Longitudinal steel was brought in and tied with 2 ft of exposed steel. A primary disadvantage of this FDR method was the time and manpower required to complete the repairs. Figure 1.1-(a) illustrates the manpower utilized in this method. Other disadvantages include the difficulty to rework the base below the exposed steel area and potential damage to existing concrete by jack-hammering below the longitudinal steel at transverse repair joints. The performance of FDR done this way varied, with some repairs showing good performance while some required additional repairs due to distresses at repair joints, as shown in Figure 1.1-(b).

In Texas, CRCP is selected for projects with heavy traffic. Over the years, traffic has steadily increased in metropolitan areas, and some districts of TxDOT, including Houston, Dallas and Fort Worth, have a strict policy regarding how long the pavement can be closed to traffic for repairs. Normally, FDR is allowed only at night from sometime between 8 and 9 pm to sometime between 5 to 6 am, which includes setting up a traffic control. The repair method described above became unreasonable due to the longer working time required. A new FDR method, called full-depth cut (FDC) method, which is more appropriate for short working time allowed, was introduced in TxDOT's 1994 standard specifications ⁽²⁾. This method consists of full-depth cut at repair boundaries, drilling holes at a length of 12 inches into the existing concrete and epoxy grouting tie bars, followed by placing transverse and longitudinal steel and concrete placement.

The effectiveness of this method also varied. A substantial portion of the repairs done with this method resulted in further distresses and needed further repairs within a few years. Figure 1.2-(a) shows the distress of the repairs done in this method. Figure 1.2-(b) shows a gap between Tie bars used for previous repairs and surrounding concrete. Since the full-depth cut face of concrete at repair boundaries is smooth, the load transfer efficiency (LTE) at repair joints is achieved primarily by Tie bars and base support. If bonding between Tie bar and surrounding concrete is not fully achieved, the Tie bar might become loose and shear mode failure or reduction in load transfer could occur due to repetitive traffic and environmental loading. Accordingly, it is imperative that Tie bars are fully bonded to existing concrete.

FDR is quite expensive, about \$170 to \$240 per square yard⁽³⁾. To identify the causes of ineffectiveness of the FDC method and to improve overall efficiency of TxDOT operations, TxDOT initiated this research study. The research study consisted of (1) evaluation of field performance of FDR by visual observations and non-destructive testing with falling weight deflectometer (FWD), (2) laboratory investigations in a test slab of various factors considered to be of a significant effect on FDR performance, (3) field evaluations of structural responses of repaired sections with various gages installed, including steel strain and concrete vibrating wire strain gages, (4) evaluations and revisions of FDR specifications, design standards, and Departmental Materials Specifications, and (6) identification and evaluation of a device that can detect delaminations in the concrete.



Figure 1.1- (a) Manpower for FDR



Figure 1.1- (b) Distress on existing CRCP at repair joint



Figure 1.2- (a) Distress at FDR



Figure 1.2- (b) Gap between tie bar and surrounding concrete

1.2 Objectives

As the 0-6611 project statement describes, there were three technical objectives in this project:

- (1) To identify causes for the ineffectiveness of current FDR practices and develop better repair methods in terms of improved design standards and materials/construction specifications and better quality control procedures.
- (2) To identify what distresses are best repaired by partial-depth repair (PDR) and what distresses are best repaired by FDR.
- (3) To identify or develop a test or procedure that allows the identification of partial depth failures that can be easily implemented in the field.

1.3 Report Organization

This research focused on evaluating the factors involved in the performance of FDR. This research report describes the work conducted during the course of research study, and discusses the findings. The report consists of five chapters as follows:

Chapter 2 provides literature reviews of other DOT's FDR practices. Through comparison between FDR of TxDOT and other DOTs, the characteristic of FDR in each state was reviewed.

Chapter 3 describes experimentations conducted in this study – laboratory testing with a test slab, field evaluations and testing using a falling weight deflectometer, field instrumentation of gages to evaluate the behavior of concrete and steel near the repair joints, and the review of TxDOT specifications for construction and materials.

Chapter 4 provides the information on the efforts made to identify a device that can measure distresses inside the concrete slab are presented. Discussions on partial-depth repairs are also presented.

Chapter 5 presents the conclusions of this study, along with recommendations to maximize the benefits of the findings in this study.

Chapter 2 FDR Practices in Other States

Even though TxDOT uses CRCP by far the most in the nation, other states have utilized CRCP, including Illinois, South Carolina, Virginia, Oregon, and Georgia. The repair practices of those states were reviewed and similarities and differences with TxDOT practices were evaluated.

2.1 Illinois⁽⁴⁾

Figure 2.1 shows the overview of Illinois DOT's (IDOT) FDR method, which is similar to what TxDOT did until the middle of 1990s. Full-depth cuts are made 2-ft away from the intended transverse repair joint locations. Partial depth saw cuts are made at the transverse repair joints. Since IDOT places steel closer to the surface than TxDOT does, normally 3.5-in from the surface, the depth of partial-depth saw cuts must be quite shallow. The concrete between partial- and full-depth cuts are removed by jack-hammering and manual labor. The highlights of IDOT's FDR specifications are as follows:

- 1) Partial-depth saw cuts at transverse repair joints should maintain a minimum distance of 18 in. from the end of the patch to the nearest transverse crack. The minimum distance may be reduced to 6 in. in the areas of close crack spacing when approved by the Engineer.

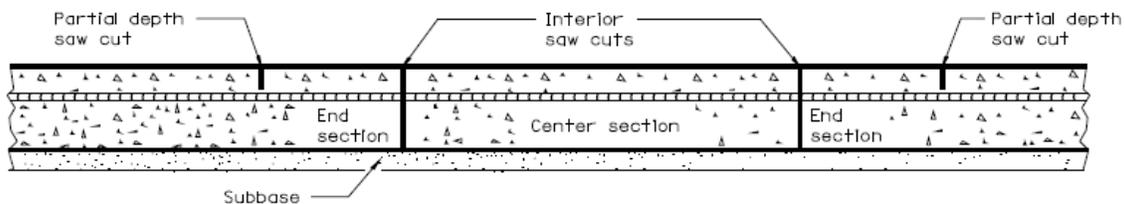
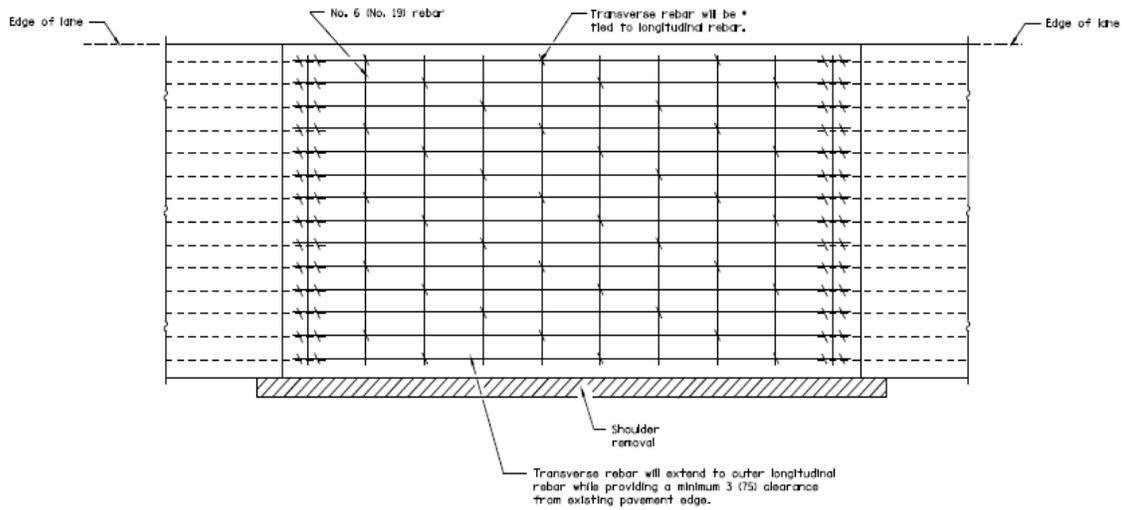


Figure 2. 1 IDOT Saw Cut Detail for CRCP Patch

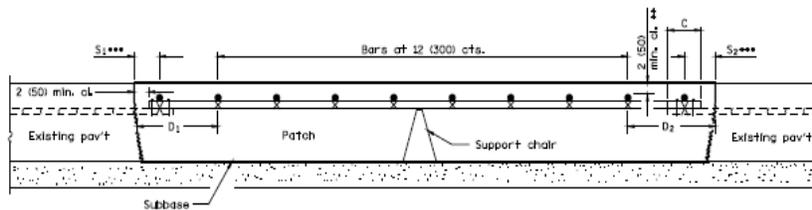
- 2) The pavement between the interior saw cuts shall be removed by lifting without subbase disturbance and pavement spalling. If the concrete has deteriorated and is not able to be lifted, breaking equipment that shall not transfer an impact energy greater than 3000 ft lb

(4000 J) will be used to break pavement into small pieces for removal.

- 3) The concrete in the 2-ft long splicing area, between the interior and outer saw cuts, shall be removed using handheld hammers and hand tools.
- 4) The reinforcing steel in the splicing area shall not be bent to aid in removal of the concrete. The patching details are shown in Figure 2.2.
- 5) Pavement support condition has a significant effect on the behavior and performance of pavement. The specifications suggest that any subbase or stabilized subbase material disturbed during pavement removal operations or determined unsuitable by the Engineer shall be removed and replaced with patch material.
- 6) The transverse rebar shall be tied to the bottom of the longitudinal rebar when the minimum clearance of 2 in. cannot be obtained with the transverse bar on the top.



PAVEMENT REINFORCEMENT DETAIL



PATCHING DETAIL

Figure 2. 2 IDOT Reinforcement for CRCP Patch

2.2 South Carolina⁽⁵⁾

South Carolina Department of Transportation (SCDOT) has a unique repair method for deteriorated CRCP. Unlike all other states in the U.S., SCDOT uses dowels at transverse repair joints, not tie bars. Communication with the SCDOT state pavement engineer revealed that SCDOT's experience with the normal FDR method (that was also used in Texas until the middle of 1990s) was not positive, and SCDOT decided to use a different method.

SCDOT's FDR method is similar to the current TxDOT FDR method, except that SCDOT uses dowels instead of tie bars at transverse repair joints, and does not use longitudinal or transverse reinforcement in the repair area. Figure 2.3 illustrates SCDOT's FDR. The key details of the SCDOT FDR method are as follows:

- 1) Repairs are full-lane width and in a single lane, typically in the outside lane of a two-lane, one-direction roadway.
- 2) The repair area perimeter cuts are made to the full depth.
- 3) Longitudinal steel continuity is not maintained in the repair area. In fact, longitudinal steel is not used in the repair area.
- 4) Similar to the conventional FDRs, tie bars are not used along the centerline longitudinal joint for patches less than 16 ft in length. For longer patches, longitudinal tie bars are spaced nominally at 30-in intervals, but the spacing may be varied to avoid existing cracking in the adjacent lane and to be at least 15 in away from the transverse joints at each end of the repair area.
- 5) Dowel bars are placed at mid-depth at a nominal spacing of 12 in starting and ending about 12 in from the corners of the repair area. Holes are drilled at a depth of 9 in, and one side of the dowels are epoxy grouted to the holes. The dowel bar spacing is adjusted to miss any longitudinal steel in the existing pavement.
- 6) Intermediate transverse joints are required for repair lengths greater than 16 ft. Dowel baskets are used at these intermediate joints, with dowels spaced at 12 in. The intermediate joints are sawed to a depth of one-third the depth of the repair area and

sealed.

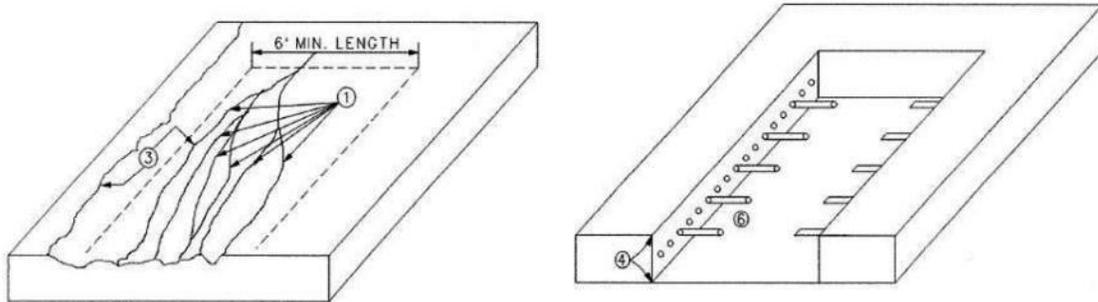


Figure 2.3 SCDOT CRCP Patches

Where the length of the FDR is between 6 ft and 12 ft, no tie bars are used at longitudinal repair joints, as shown in Figure 2.4. However, where the length of the FDR is more than 16 ft, tie bars are used at longitudinal repair joints as shown in Figure 2.5. In this case, dowel baskets are provided at a minimum of 8 ft or a maximum of 14 ft. SCDOT does not allow FDR with lengths between 12 ft and 16 ft. It appears that SCDOT wants to have dowels at a maximum of 14 ft spacing. At the locations of dowel baskets, saw cuts are made to the depth of $\frac{1}{3}$ slab thickness, and joints are sealed.

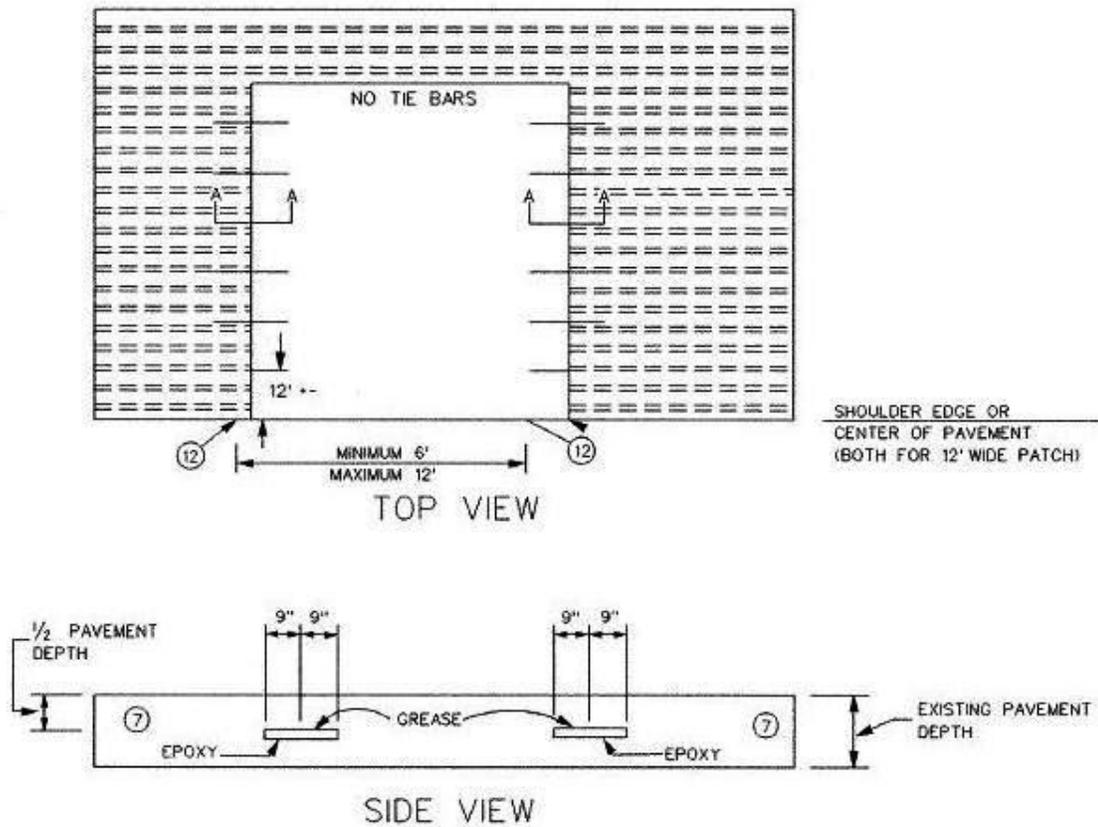


Figure 2.4 SCDOT Reinforcement for CRCP Patch with Length 6-12 ft.

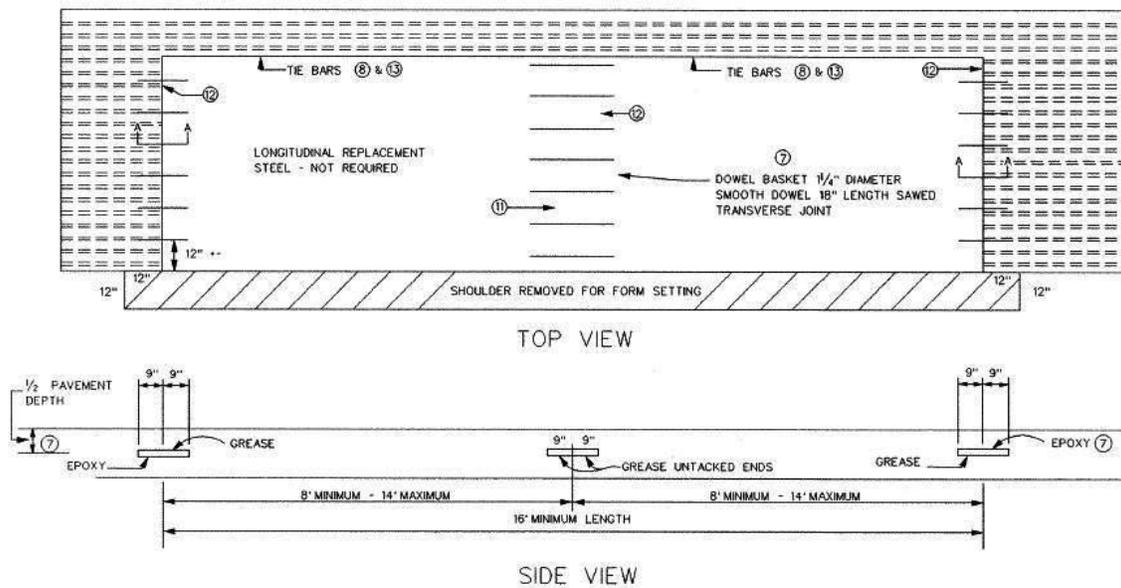


Figure 2.5 SCDOT Reinforcement for CRCP Patch with Minimum Length 16 ft.

2.3 Virginia⁽⁶⁾

Virginia DOT (VDOT) uses the same FDR method that TxDOT used to use until the middle of 1990s. VDOT's FDR standard is shown in Figure 2.6.

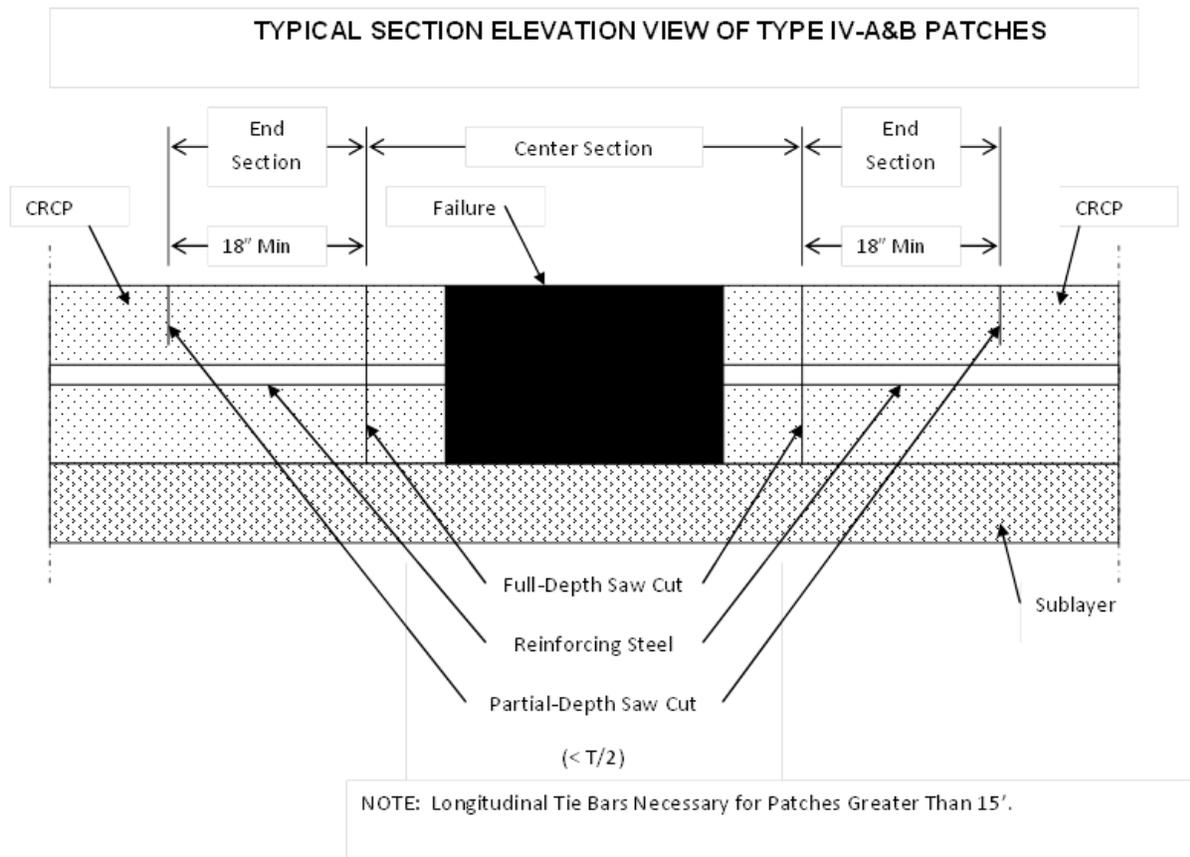


Figure 2. 6 VDOT's FDR drawing.

Since VDOT's FDR practice is quite similar to IDOT's, no further discussions are presented.

2.4 Oregon⁽⁷⁾

Oregon DOT's FDR practice is also similar to IDOT's and VDOT's, and no further discussions are provided. However, in Oregon DOT, partial depth repairs (PDR) are used for the repair of

CRCP. The major difference between Oregon DOT PDR specifications and special specifications developed for TxDOT in this study is the depth of the repairs. In Oregon DOT, the depth of PDR should be limited in depth to the top third of the slab and should not come in contact with dowel bars or reinforcing steel. If dowel bars or reinforcing steel are encountered, a full depth repair is required. The requirement of FDR when reinforcing steel is encountered during the removal of the distressed concrete for PDR can be problematic. When the repair projects are let, the owner should specify either FDR or PDR, since the unit bid price is quite different. If a repair method should be changed during the repair depending on the condition discovered during the repair, the management of the repair project could be quite difficult. In addition, from a technical standpoint, there is no reason to use FDR when longitudinal steel is exposed after all the distressed concretes are removed. As long as the concrete under the longitudinal steel is solid and not damaged, PDR should proceed.

2.5 Georgia⁽⁸⁾

Georgia DOT's FDR practice is also similar to IDOT's, VDOT's, and Oregon DOT's, and no further discussions are provided.

2.6 Summary of Other DOTs' FDR Practices

In general, there are three different FDR practices among the states investigated in this study. The most widely used method is the one TxDOT used until the mid-1990s, where full depth saw cuts are made about 18 in or 24 in from the transverse repair joints. Partial depth saw cuts, approximately 2 in deep, are made on each end of the CRCP repair for splicing of the longitudinal steel reinforcement. Jackhammers and hand chipping tools are used to chip away the existing PCC to expose the steel reinforcement to which the new steel is tied. After the base is repaired, if needed, and longitudinal and transverse steel is placed, repair concrete is placed. SCDOT is the only state where dowels, not tie bars, are used at transverse repair joints. FDR in SCDOT is quite similar to the FDR of CPCD in Texas. It appears that TxDOT is the only state where FDR with a full-depth cut method is used.

Chapter 3 Evaluation for FDR Performance

The performance of FDRs repaired with a full-depth cut (FDC) method was evaluated and factors relevant to the FDR performance were identified. The effect of each factor on the FDR performance was investigated in the laboratory.

3.1 Performance Evaluation of FDR

As a first step, visual observations of FDRs with the FDC method were made in a number of CRCP sections, with the objective of identifying common characteristics of good and poorly performing FDRs and factors relevant to FDR performance. In the rest of the report, FDR refers to the full-depth repairs by full-depth cut method. The findings included that even though the majority of the FDRs observed perform well, distresses at the transverse repair joints were the common characteristic of poorly performing FDRs, without almost no exception. The majority of the distresses were in the existing pavement side of the repair joints. Field evidence clearly indicated that the current practice of FDR with FDC has deficiencies in restoring the structural adequacy at transverse repair joints. The structural condition of transverse repair joints at good and poorly performing FDRs was evaluated with a falling weight deflectometer (FWD). FWD testing was conducted at various locations of FDR as illustrated in Figure 3.1-(a): two locations in the existing CRCP at 20 ft and 10 ft longitudinally from an approach repair joint, at upstream and downstream at an approach repair joint, a minimum of two locations in the repaired slab, at upstream and downstream at a leave repair joint, and two locations in the existing CRCP at 10 ft and 20 ft longitudinally from a leave repair joint. Deflections at all locations and load transfer efficiency (LTE) at transverse repair joints were evaluated. A total of 30 FDRs were selected for structural evaluations: 11 on the US 81, 9 on the US 59 in the Atlanta District, 9 on the US 287 in the Wichita Falls District, and one on the US 290 in the Houston District.

Figure 3.1- (b) shows typical deflection variations along the FDR area. It is observed that deflections at repair joints are larger than those at the other locations. This trend holds true for almost all FDRs evaluated. The major difference between good and performing FDRs was the magnitude of the increase in deflections at repair joints. In good performing FDRs, such as

shown in Figure 3.1-(b), the increase was minimal, while in poorly performing FDRs, as shown in Figure 3.1-(c), the increase was substantial. In Figure 3.1-(c), the increase of deflections at a repair joint is about 6 times higher than those at other locations, which clearly indicates the potential for future deterioration at this repair joint. All FWD test and visual survey results are included in Appendix A.

The reason for the larger deflections at repair joints even in good performing FDRs is that, in the FDR method, the concrete surface after the cut is smooth, and no aggregate interlock is present at repair joints. All the load transfer is achieved by tie bars and base support. It appears that if tie bars are adequately installed and/or base condition is restored to an acceptable level during repairs, FDRs with FDC method can perform well. On the other hand, if tie bars are not properly installed and base condition is not adequately restored, the performance of FDR with FDC method could be poor. Performance evaluations of FDRs with FDC method clearly indicated the need for improving repair practices at transverse joint areas. Since loose base materials are removed during FDRs, as evidenced by deflections in the repair slabs comparable to those in the existing slabs, the primary cause for large deflections at repair joints is the inability of tie bars to provide adequate shear load transfer and slab continuity.

Based on the findings from the field performance evaluations of FDRs, in-depth investigations were conducted to identify variables that affect bond strength of the epoxy grouted tie bars. Even though both shear and bond strength of epoxy grouted tie bars will affect the performance of FDR, shear strength testing was not conducted, partly because shear strength testing is quite difficult and it is reasonable to assume a positive correlation between bond strength and shear strength.

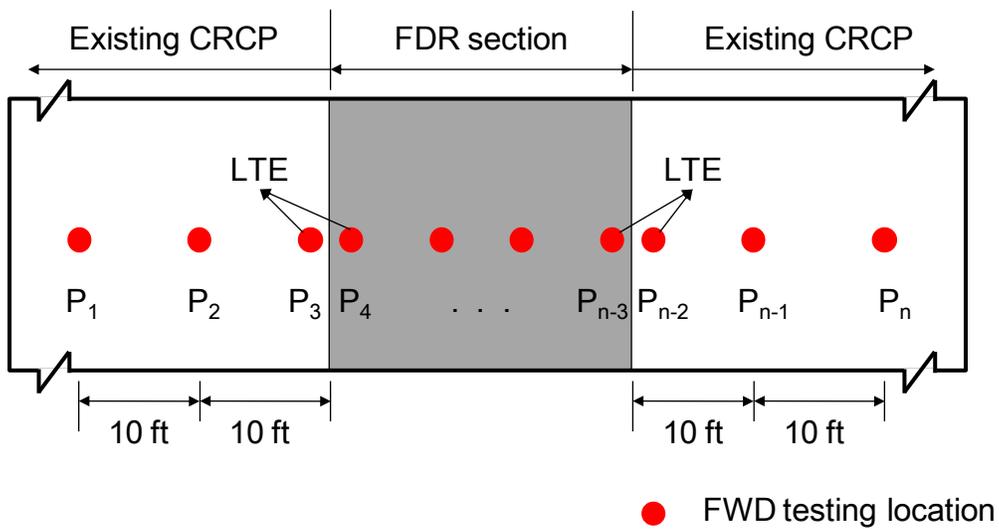


Figure 3.1- (a) FWD testing method

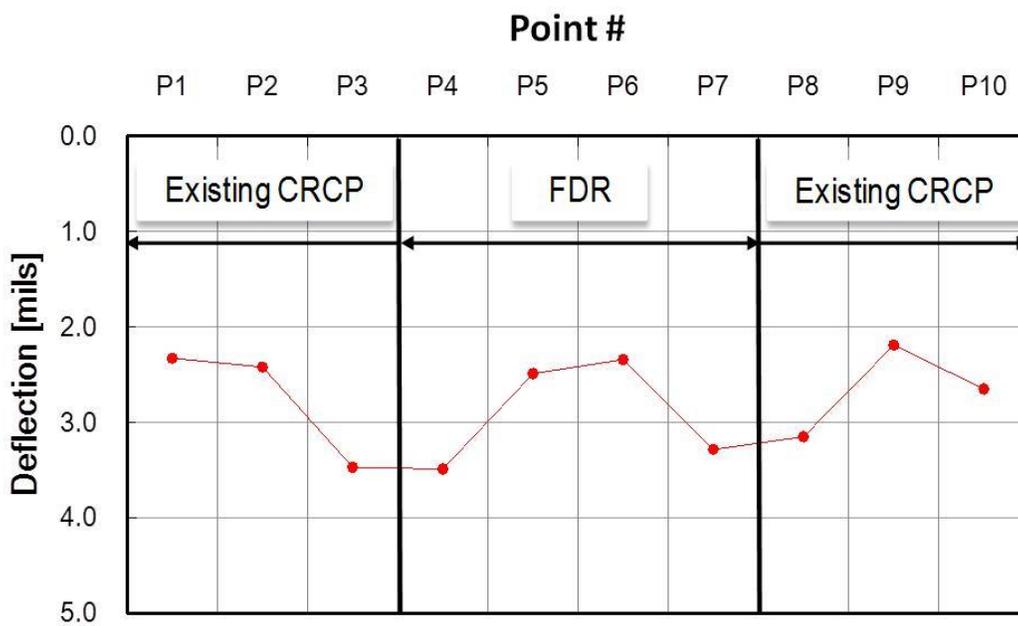


Figure 3.1- (b) FWD test result [good condition]

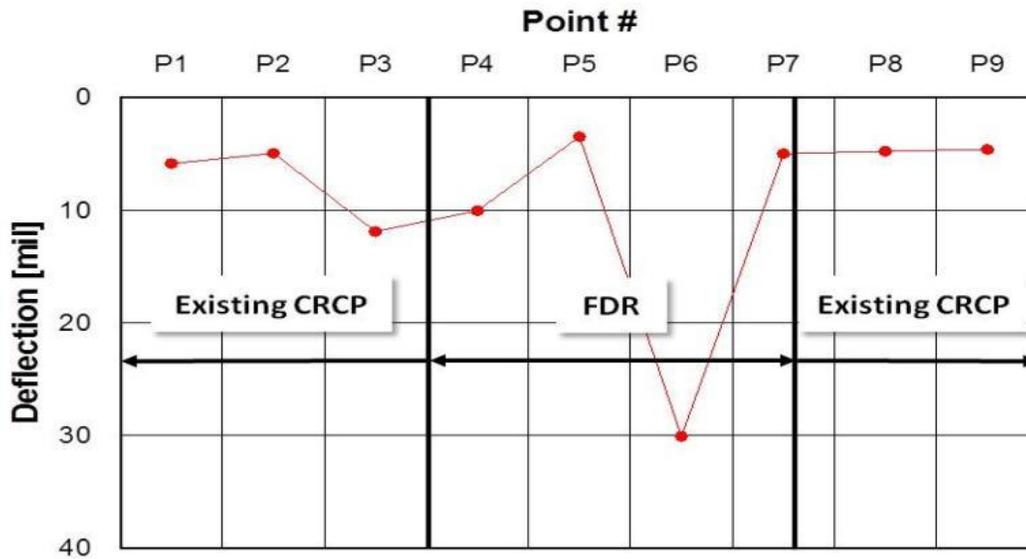


Figure 3.1- (c) FWD test result [poor condition]

3.2 Bond Strength Testing

As stated, the most common characteristic of poorly performing FDRs is large deflections at repair joints, and it appears that restoring structural adequacy at transverse repair joints is the key to good performance of FDRs. In this study, major effort was placed on restoring structural adequacy at repair joints, and the aspect of restoring adequate base support was not addressed. It was observed that the loss of bond between tie bars and surrounding concrete as shown in Figure 1.2-(b), and resulting large deflections at repair joints, were the primary cause for poor performance of FDR. Variables that could affect bond strength between tie bars and surrounding concrete were identified and testing was conducted in a test slab to evaluate the effect of each variable on bond strength development. The variables identified are (1) method of epoxy injection, (2) embedment length of tie bar, (3) drilling method, (4) epoxy curing time, (5) cleanness of drilled holes, (6) epoxy type, and (7) curing temperature. To evaluate the effect of each variable in a more efficient way, a factorial experiment was set up as shown in Table 3.1. Table 3.1 shows treatments in bold font, indicating the standard condition; when effects of one variable are investigated, the treatments of all the other variables were fixed at the standard condition. Accordingly, the experiment conducted in this study is not a full factorial experiment.

A test slab, 10 inches thick and 24 ft by 6 ft, was constructed in October 2010 at the Texas Tech research campus at 4th Street and Quaker in Lubbock, Texas. Typical TxDOT Class P concrete was used.

Table 3. 1 Variables of Pullout test

Experimental Variable	Treatment	Selection reason
Epoxy injection method	Compliance vs. non-compliance	Two epoxy application methods – one as required in the specifications and the other normally done in the field – are used.
Embedment length of tie bar	3 in., 6 in., 9 in. , 12 in. , and 15 in.	Embedment length effects bonding strength between a Tie bar and concrete.
Drilling method	Rotary vs. hammer drill	Hammer drilling and rotary drilling are the most commonly used method in FDR.
Epoxy curing time	30, 60 , 90 and 120 min.	Epoxy gains strength over time.
Cleanness of drilled holes	Dirty vs. clean	Remaining dust in a hole might reduce the bond strength between tie bar and surrounding concrete.
Epoxy type	Different 5 epoxies (Type III , class C)	Different epoxies have different properties.
Curing temperature	Morning vs. afternoon	Epoxy gains strength through chemical reactions, and the strength gain might depend on the temperature.

3.2.1 Test procedure

Figure 3.2 shows the test procedure adopted in this study for the pull-out test. The setup consisted of a steel plate, load cell, hydraulic pump and linear variable differential transformer (LVDT). As the hydraulic pump applied the tensile force to the tie bar, the reaction at the steel plate increased and a load cell measured the reaction, which is a pull-out force. The displacement of the rebar was measured with LVDTs. The procedure for the standard condition is:

- (a) Drill a hole with a rotary drill at the depth of 12 inches.
- (b) Clean hole with a brush, followed by compressed air.
- (c) Remove stain on rebar with steel brush for embedded part.
- (d) Fill the hole with epoxy by injecting epoxy into the hole, from the backside of the hole.
- (e) Insert rebar into the hole.
- (f) Place duct tape around the rebar at the hole to prevent the epoxy from leaking.
- (g) Place a steel plate through a tie bar. A load cell is installed in such a way that a tie bar is placed at the center of the load cell to minimize eccentricity and resulting bending moment. Connect the load cell wires to a data logger.
- (h) Install a hydraulic jack and a gripper.
- (i) Place a reference plate to install LVDTs for the measurements of tie bar displacements.
- (j) Install LVDTs to the reference plate. Connect the LVDT wires to a data logger to collect displacement data.
- (k) Apply pressure to hydraulic jack with a hydraulic pump.
- (l) Collect data for pull-out strength and tie bar displacements..



Figure 3.2- (a) Drill a hole



Figure 3.2- (b) Clean hole with brush



Figure 3.2- (c) Clean hole with air tube



Figure 3.2- (d) Remove stain on rebar



Figure 3.2- (e) Fill hole with epoxy



Figure 3.2- (f) Insert a rebar in the hole



Figure 3.2- (g) Prevent the epoxy leak



Figure 3.2- (h) Place a steel plate

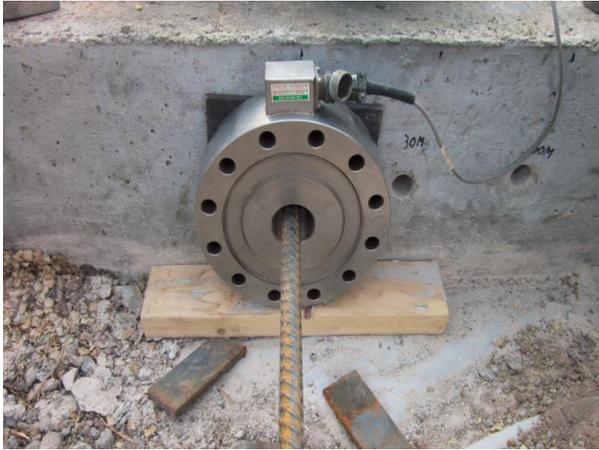


Figure 3.2- (i) Install a load cell



Figure 3.2- (j) Install hydraulic jack



Figure 3.2- (k)
Install a gripper and a reference plate



Figure 3.2- (l) Install LVDT



Figure 3.2- (m) Finish set-up



Figure 3.2- (n) Apply pressure



Figure 3.2- (o) The pulled out length

3.2.2 Discussion of Test Results

3.2.2.1 Effect of epoxy injection method

TxDOT specification Item 361 states: “Completely fill the tie bar hole with Type III Class A or Class C epoxy before inserting the tie bar into the hole.”⁽⁴⁾ However, field observations made during a number of FDR operations revealed that this requirement is not always properly adhered to. In many instances, repair crew dip the tie bars into a can of epoxy and insert the bars into the holes. The effect of this practice was evaluated.

Figure 3.3-(a) and (b) ~ (e) show a different epoxy injection method. Figure 3.3-(a) illustrates the epoxy injection method satisfying the Item 361 requirement. On the other hand, Figures 3.3-(b) through (e) simulate an epoxy injection method that does not comply with the Item 361 requirement, yet is a practice commonly observed in TxDOT repair projects. Figure 3.3 (f) illustrates a marked difference in bond strength for the different epoxy injection methods. In this figure, “Compliance” indicates the epoxy injection method that satisfies the Item 361 requirement as shown in Figure 3.3-(a), while “Non-compliance” refers to the epoxy injection method shown in Figures 3.3-(b) through (e), which is deviant from the Item 361 requirement. The large difference in bond strength between the two practices is somewhat unexpected because

even in the non-compliance method of epoxy injection, tie bars were coated with a large quantity of epoxy – enough to fill the voids between tie bars and concrete. Also, in every non-compliance method, it was observed that there was excess epoxy draining down from the holes. To further investigate the mechanism for the low bond strength of the “Non-compliance” method, experiments were conducted, where epoxy was injected into 12-in long and 7/8-in diameter transparent plastic tubes using both methods. Figure 3.3-(g) shows the simulated condition of the epoxy inside a hole when epoxy is injected by the method specified in Item 361. It shows that the hole is filled with epoxy. Figure 3.3-(h) shows the simulated condition of the epoxy inside a hole when epoxy is injected by the non-compliance method. The figure clearly shows voids inside the hole, which could explain the low bond strength shown in Figure 3.3-(f). It should be noted that, in this experiment, a tie bar was coated with more epoxy than needed to fill the voids between the tie bar and the tube wall. During the testing, it was noticed that a large amount of epoxy was flowing out of the tube, similar to what was observed in the field when epoxy was injected by the non-compliance method. The voids shown in Figure 3.3-(h) are not due to a deficient amount of epoxy coated in a tie bar; instead, it appears that air inside the tube was compressed while a tie bar coated with epoxy was being inserted, pushing the epoxy toward the outside of the tube. This mechanism might explain the large drain downs of epoxy in the field when epoxy is applied with the non-compliance method, as well as the low bond strength and poor performance of FDR. On the other hand, when epoxy is applied as specified in Item 361, a tie bar pushes the epoxy towards the entrance of the hole, pushing air out of the hole and filling the hole with epoxy with no void, as shown in Figure 3.3-(g), explaining a good bond strength and FDR performance.



Figure 3.3- (a) Inject epoxy with handgun – Compliance



Figure 3.3- (b) Put epoxy in the box



Figure 3.3- (c) Mix epoxy



Figure 3.3- (d) Put rebar into epoxy



Figure 3.3- (e) Insert the rebar into a hole

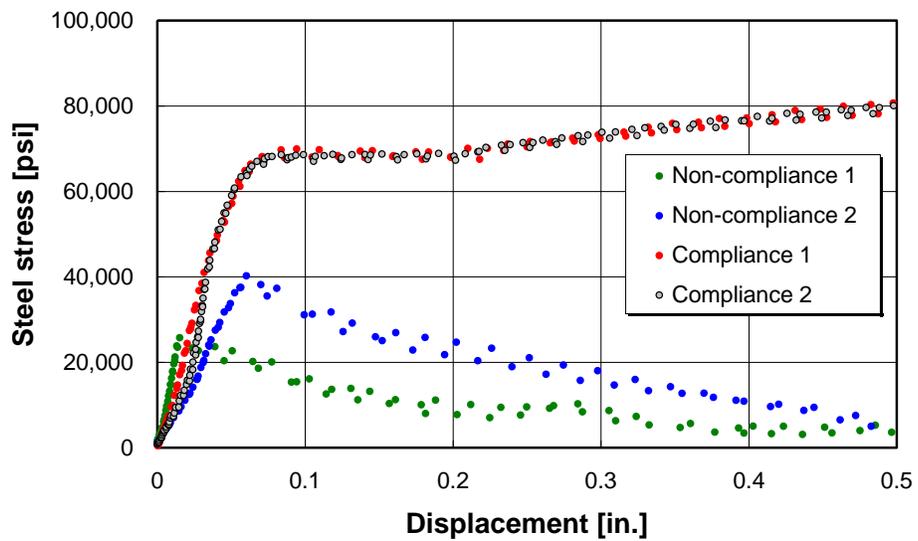


Figure 3.3- (f) Pullout test result for epoxy injecting method



Figure 3.3- (g) “simulated” epoxy injection by the method specified in Item 361



Figure 3.3- (h) “simulated” epoxy injection by “Non-compliance” method

3.2.2.2 Effect of embedment depth

Current TxDOT Item 361 requires the drilling depth of a minimum of 12 inches for repairs. Field evaluations revealed that the actual drilling hole depth is sometimes less than 12 inches, even though the frequency of this is low. To identify the effect of embedment length on the bond strength, 10 holes were drilled at five different depths – two each at 3, 6, 9, 12, and 15 inches as

shown Figure 3.4-(a). Epoxy was injected using the compliance method and bond strengths evaluated. Figure 3.4-(b) shows that adequate bond strengths were obtained in all depths except for the 3-in depth. Figure 3.4-(b) indicates that deficient embedment length might not be a primary cause for poor performance of FDR. Drilling holes takes time, especially in concrete containing hard coarse aggregates. The current TxDOT requirement of 12 inches is adequate.



Figure 3.4- (a) Tie bars of different embedment length

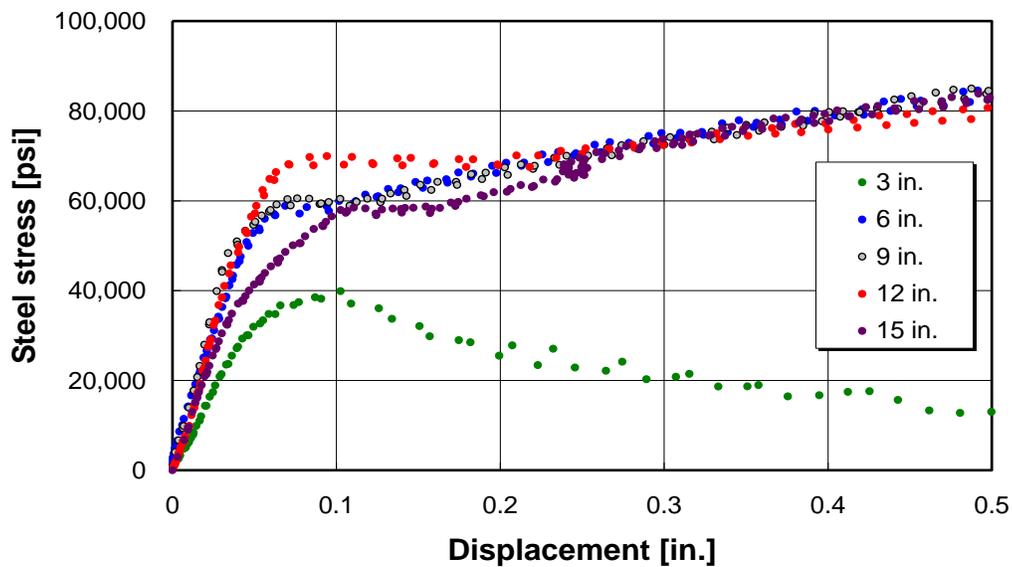


Figure 3.4- (b) Pullout test result for embedment length of tie bar

3.2.2.3 Effect of drilling method

Three drilling methods are currently in use in Texas – hammer drill, rotary drill, and core drill. From a production standpoint, the hammer drill method is most efficient, followed by rotary drill and core drill. Even though the core drill is preferred by some at TxDOT, this method is quite slow and leaves a large amount of water in the base. It has been stated by some engineers that the hammer drill method causes damage to concrete near drilled holes, and could result in poor performance of FDRs.

Figure 3.5-(a) and (b) show rotary and hammer drill methods, respectively. In the PCC pavement repair industry, the drilling method shown in Figure 3.5-(b) is also referred to as the rock drilling method, since this is the method primarily used for drilling rocks. To investigate whether drilling operations cause damage to concrete near the drilled-hole surfaces and the effect of the drilling method on bond strength, holes were drilled by both methods. This was done with a slab removed from an actual TxDOT FDR project, since it is important to use the actual drilling equipment used in the field. The core drill method was not included, since this method is not expected to be used in real repair projects. Once holes were drilled, three cores with 4-in diameter were taken vertically that contain the full length of a drilled hole. To evaluate damage to concrete near drilled-hole surfaces, petrographic analysis was conducted by a TxDOT geologist at the Materials and Pavements Section. Petrographic analysis shows evidence of some fractures of coarse aggregates in the hammer drilled holes; however, the damage was so minute that the geologist concluded it should not be a concern. Figure 3.5-(c) shows an example of the analysis. As illustrated by the images, even though some of the coarse aggregates were fractured adjacent to the hole, the fractures were confined to the aggregates adjacent to the hole and did not propagate any further into the paste. There was not any evidence of fractures propagating into the paste or debonding of the aggregate at interfaces between the paste and aggregate. In the rotary drilled holes, fractures of coarse aggregates were also observed, but they were minor.

Additional holes were made with both hammer and rotary drilling methods in the field and tie bars were installed using the compliance method. Figure 3.5-(d) shows the testing results, illustrating that both drilling methods produced almost identical bond strengths. Normally, it

takes about 15 seconds to drill a hole with a hammer drill, while it takes more than a minute using a rotary drill, with actual time varying depending on the hardness of coarse aggregate in concrete and overall strength of concrete. Based on the findings, the use of the hammer drill method should be allowed and encouraged.



Figure 3.5- (a) Rotary drill



Figure 3.5- (b) Hammer drill

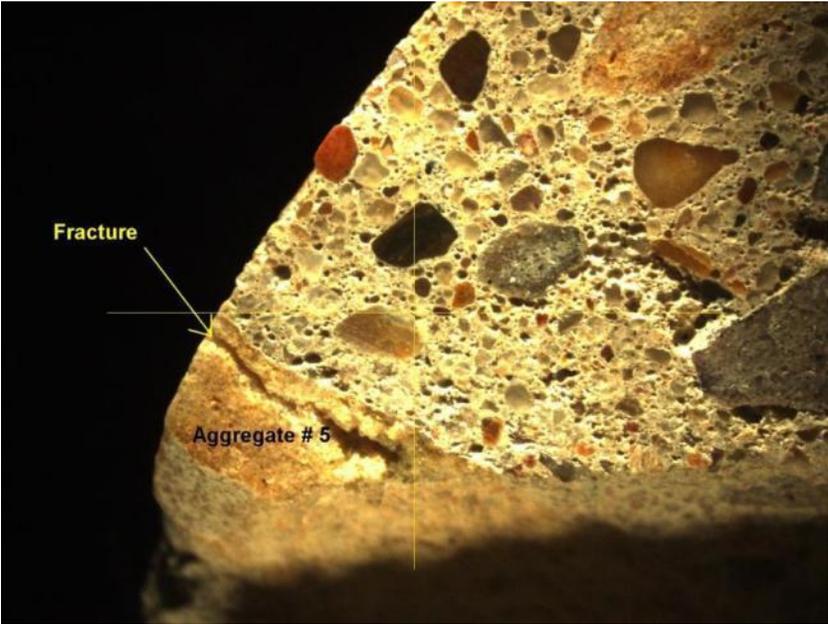


Figure 3.5- (c) Microscope picture for core of hammer drill

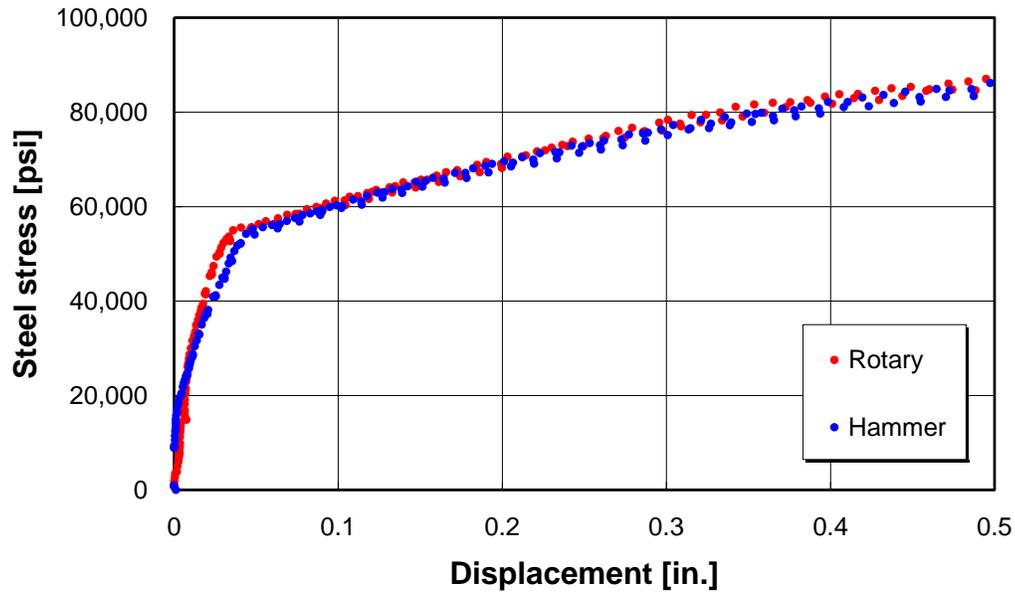


Figure 3.5- (d) Pullout test result for drilling method

3.2.2.4 Effect of epoxy curing time

As stated earlier, the time window of FDR in metropolitan areas in Texas is quite limited. It is a normal practice to place concrete once tie bars are inserted and steel placement is completed. The time between epoxy injection and concrete placement depends on the size of the repair and whether contractors pre-assemble steel mats, and could be quite short. It is important that epoxy gains early strength. Current TxDOT DMS (Departmental Materials Specification) 6100 requires a minimum gel time of 25 min and six min for Type III, Class A and Class C epoxies, respectively. Specification 6100 also requires tensile bond strength of 200 psi at six hours for both Type III Class A and Class C epoxies. Minimum gel time requirements are not relevant to FDR operations, since tie bars are inserted immediately after epoxy is injected. The requirement of bond strength is relevant. However, the current requirement of testing at six hours needs to be changed, because concrete placed at repair areas will go through expansion and contraction within a few hours and epoxy should be able to resist bond stresses resulting from concrete volume changes. Otherwise, bond failures and poor performance of FDR will result. Bond strengths were evaluated at 30, 60, 90, and 120 minutes after epoxy injection. Figure 3.6

illustrates the effect of epoxy curing time on bond strength, indicating that curing time as short as 30 minutes provides adequate bond strength in the temperature condition during this testing, which was between 50 °F and 75 °F. The testing results show that the epoxy used in this experiment will provide adequate resistance to concrete volume changes.

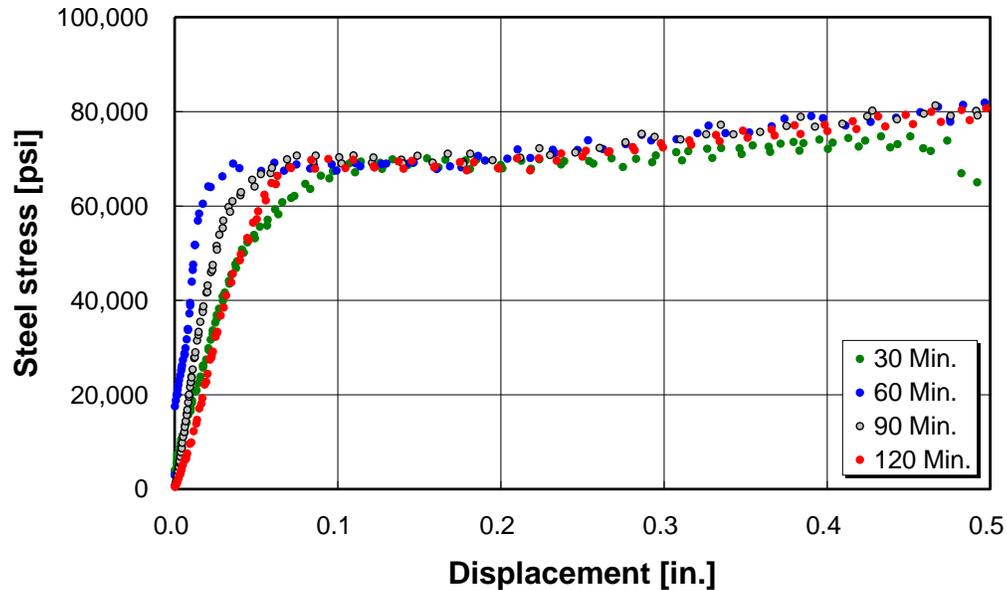


Figure 3. 6 Pullout test result over curing time

3.2.2.5 Effect of hole cleanness

Current Item 361 does not require hole cleaning before epoxy injection. Drilling operations generate large quantities of dust and, unless cleaned out, this dust will stay inside the holes and might reduce the bond strength. Figure 3.7-(a) shows the condition of drilled holes that were cleaned (left) and not cleaned (right) observed after pull-out testing. The effect of cleanness of the holes on the bond strength was evaluated, and Figure 3.7-(b) shows that, even though leaving dust in the holes before epoxy injection does not appear to adversely affect ultimate bond strength, the pull-out displacements are larger compared with those in clean holes due to larger shear strains at the interface between epoxy and dust. Epoxy injection without cleaning holes might result in widening of repair joint widths, reducing load transfer efficiency and leading to poor performance.



Figure 3.7- (a) Clean condition hole (left) and dirty condition hole (right)

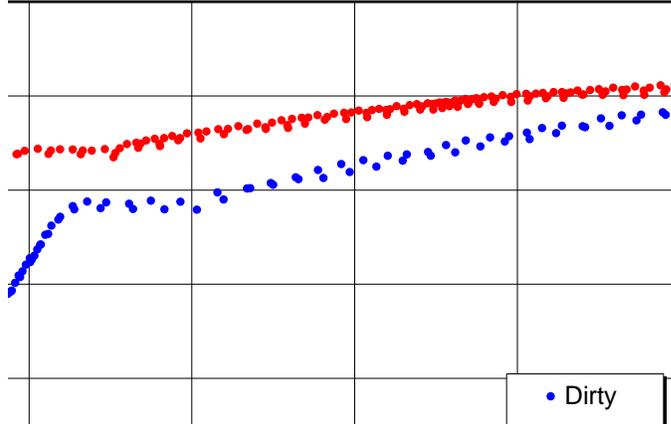


Figure 3.7- (b) Pullout test result for the cleanliness of drilled hole

3.2.2.6 Effectiveness of different epoxy brands

The materials requirements for Type III, Classes A and C appear to be inadequate in ensuring good performance of FDR, and materials that meet those requirements might not provide optimum FDR performance. Five epoxies were randomly selected from the list of TxDOT's approved list for Type III, Class C epoxy, and bond strength was evaluated ⁽¹⁰⁾. Figure 3.8-(a) shows five epoxies selected for the test. Figure 3.8-(b) illustrates that all five epoxies produced similar bond strength performance. This does not necessarily indicate the adequacy of the current requirements for Type III, Classes A and C epoxies in DMS 6100.



Figure 3.8- (a) Five epoxies tested

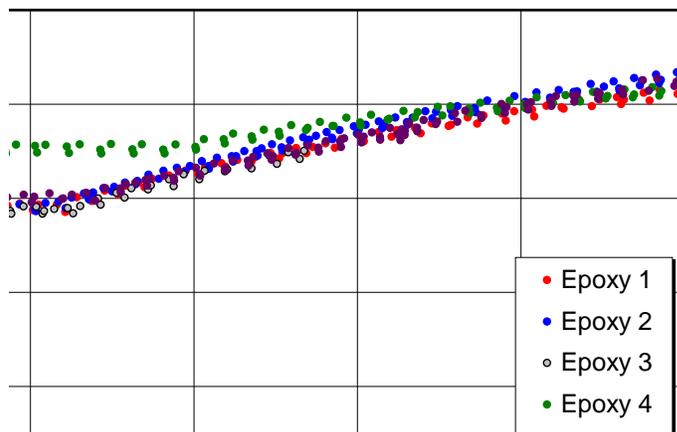


Figure 3.8- (b) Pullout test result for different epoxies

3.2.2.7 Effectiveness of epoxy curing temperature

Since the chemical reactions between two parts of the epoxy are responsible for strength gain, the rate of strength gain of epoxy should depend on the curing temperature. Curing temperature effect was evaluated by injecting epoxy in the morning when the air temperature was about 50 °F, and in the afternoon when the air temperature was about 75 °F. Figure 3.9 shows testing results. A difference observed in the bond strength should be due to the different yield strengths of the tie bars used in the testing program. From a practical standpoint, there was no difference in bond strength or pull-out displacement. As long as air temperature during FDR is above 50 °F, the temperature effect should be minimal.

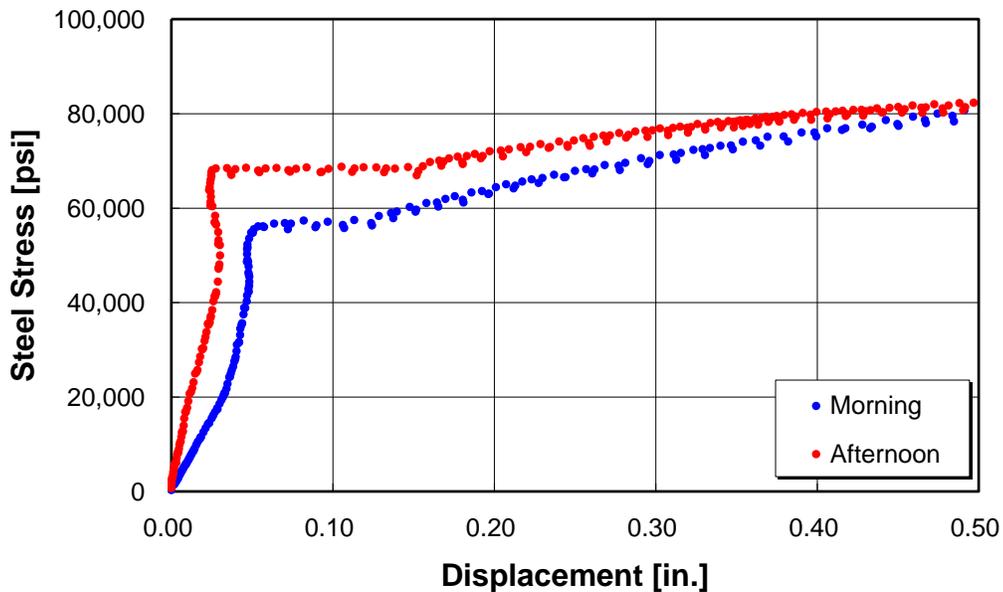


Figure 3. 9 Pullout test result for curing temperature

3.3. Field Testing

To evaluate the structural responses of full-depth repaired CRCP system as well as to identify causes of poor performance of FDR, field testing was conducted in two FDR projects – one in the Wichita Falls District and the other in the Amarillo District. Steel strain and concrete strain

gages were installed near repair joints to investigate concrete and steel strains and their implications on FDR performance.

3.3.1 Wichita Falls Section

A test section in the Wichita Falls District was located in the inside lane of eastbound frontage road of US 82 at the southwest corner of US 82 and Southwest Parkway in Wichita Falls, starting at 92.5 ft west from the stop sign at Southwest Parkway. The repair section was 92.5 ft long and 9.2 ft wide. Figure 3.10 shows the test section location. Existing pavement was 7-in thick CRCP with a two-inch asphalt overlay, and the distress type included severe cracks in the asphalt surface and faulting at a longitudinal joint. The repair was conducted on January 19, 2012. Holes 12-in deep were made by rotary drills. Dust was removed by compressed air with an air nozzle. Epoxy was injected by a handgun type applicator; however, some holes were not completely filled with epoxy, as required by Item 361.

Figure 3.11-(a) shows base condition before the concrete placement. It shows loose base as well as water in the base from the saw cutting operations of repair joints. The original CRCP tie bars were not installed at the same spacing as that of longitudinal bars, which is not compliant with Item 361 requirements. Figure 3.12 shows the layout of the rebars at a transverse repair joint and three steel strain gages (SSGs) installed on tie bars. As discussed, tie bar spacing was double the longitudinal bar spacing.

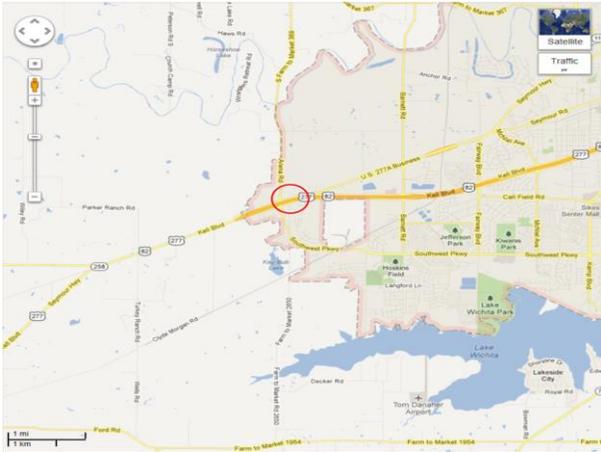


Figure 3.10-(a) Wichita Falls Section



Figure 3.10-(b) Detailed location



Figure 3.11-(a) Condition of base



Figure 3.11-(b) Insufficient tie bars

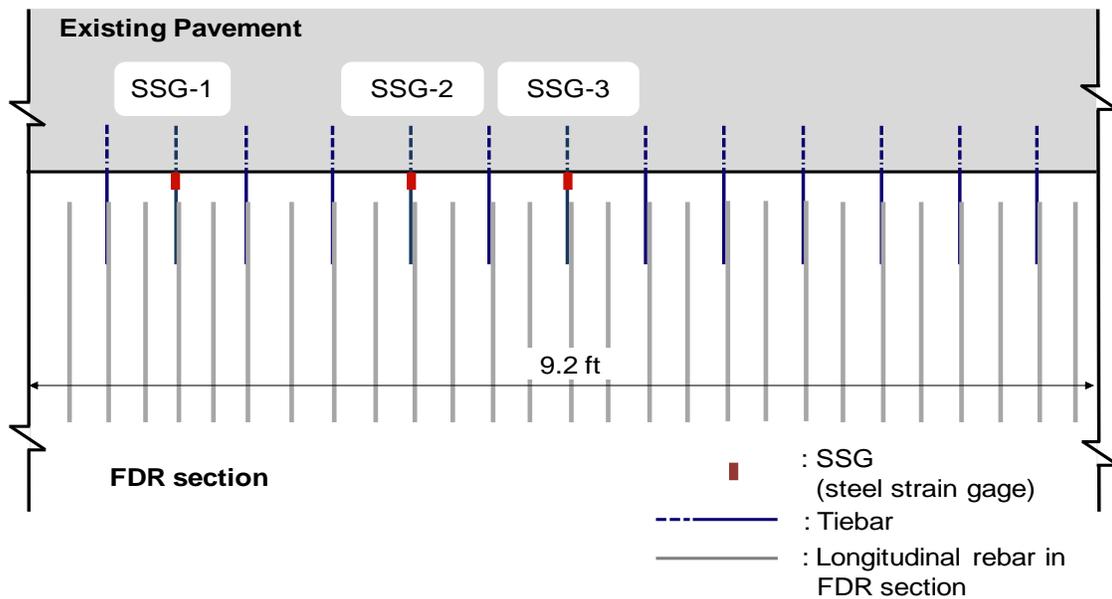


Figure 3.12 Layout of gages installation and steel

Figure 3.13-(a) shows the variations in steel strains for 2.5 months. Temperature was measured by thermocouples in concrete installed at the tie bar depth, which is the mid-depth of the 7-in concrete slab and 5.5-in from the new pavement surface. Little variations are observed in steel strains in SSG-1 and SSG-3. It appears that the holes were not completely filled with epoxy, and there was a debonding between tie bars and concrete. On the other hand, large variations are observed in strain gage SSG-2, which indicates that in this hole, epoxy was properly applied and adequate bonding achieved between tie bar and surrounding concrete. It is observed that the steel strain exceeded a yield point when the concrete temperature dropped in the middle of February. This high strain and stress in the tie bar #2 could be the result of a reduced number of tie bars and some tie bars not functioning properly due to poor bond between tie bars and concrete in the holes. Low subbase friction between concrete and natural subgrade could have contributed to the high strain in the tie bar as well. Later on, as concrete temperature went up, tensile strain in the tie bar decreased. Eventually, as concrete temperature further increased, steel strain became in compression. It is noted that from March 10 through March 14, 2012, concrete temperature dropped substantially. It is expected that tensile strain in the tie bar will increase due to the decrease in concrete temperature. However, tensile strain in the tie bar during that time period actually decreased substantially, resulting in a compression state due to a one-inch rain and

subsequent swelling of concrete on March 10, 2012. Figure 3.13(a) also shows that concrete temperature increased from March 15, 2012; however, strains in the tie bar moved in the tensile direction, which is contrary to what is expected. This is probably due to the shrinkage of concrete due to continued drying after the rain. After March 23, 2012, concrete temperature went up and so did the strains in the tie bar.

Figure 3.13-(b) shows steel strain versus temperature from the 2nd to 4th of March, 2012. The rate of steel strain variations over temperature of SSG-2 was about five or seven times larger than those of SSG-1 and SSG-3, as expected. Poor bonding condition of SSG-1 and SSG-3 resulted in smaller variations of steel strain, compared with those of SSG-2.

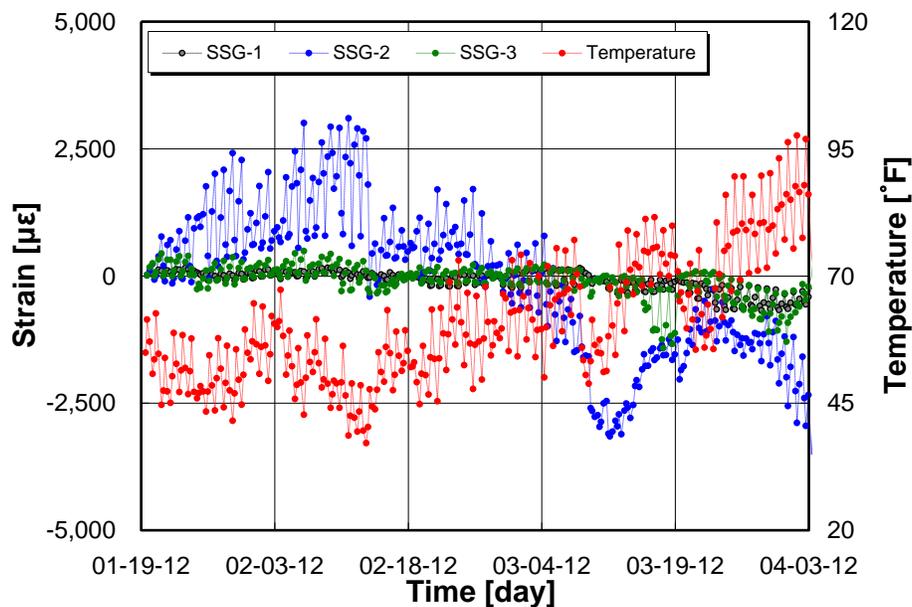


Figure 3.13- (a) Behavior of steel strain gages on tie bars

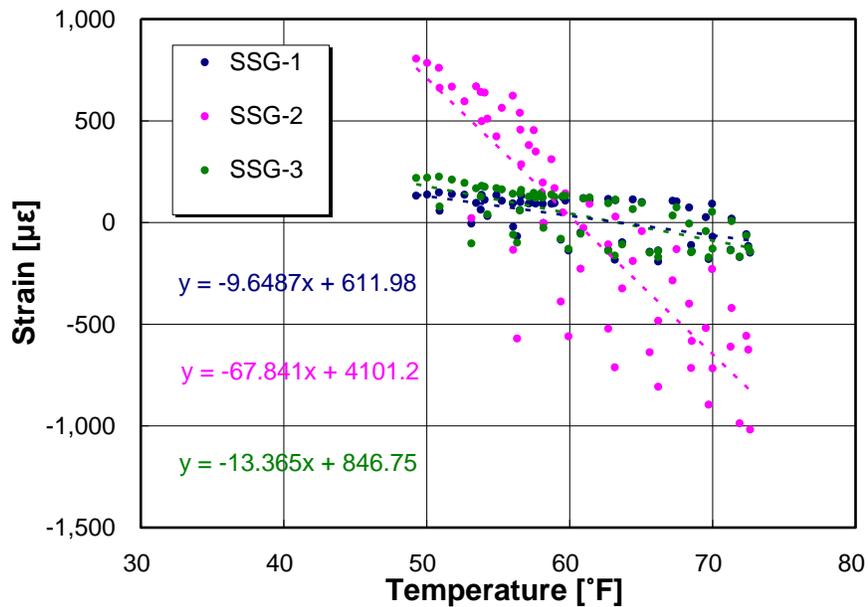


Figure 3.13- (b) Strain vs. temperature

3.3.2 Amarillo Section

The repair section in the Amarillo District was located on the eastbound main lane of IH 40 in the town of Groom. Figure 3.14 shows the location of the repair section. Existing pavement was 9-in CRCP on top of 4-in asphalt stabilized base, completed in 1979. Major distresses included widened and faulted longitudinal joints, and some spalling near transverse cracks. Prior to the repair operations, a training session was conducted in the Amarillo District headquarters on April 16, 2012 as a part of pre-construction meeting. Both TxDOT project staff and contractor personnel attended. The findings made in this research study up to that point were presented and discussed, with dos and don'ts and the emphasis placed on the filling the holes with epoxy before inserting tie bars.

The repair slab was 26.4 ft long and 12.6 ft wide and the repair was done in May, 2012. Holes were made with hammer drill, and the repair operations went smoothly. Figure 3.15-(a) shows the condition of the asphalt base after the slab was removed. The asphalt base in some areas was quite loose, indicating potential moisture damage and resulting CRCP distress. Figure 3.15-(b)

shows the operations of loose base material removal in accordance with the Item 361 requirement.

Since loose materials in the base course were observed in the distressed areas, further investigation was conducted to evaluate the base condition of the CRCP section in this area. A CRCP section in good condition was selected, and dynamic cone penetrometer (DCP) testing was conducted in the FDR area as well as in the CRCP section in good condition. Figure 3.16-(a) shows the DCP testing locations in the FDR area and CRCP section in good condition. Figure 3.17 shows resilient modulus values in each layer at both the FDR and CRCP section in good condition. In general, there was no significant difference in modulus values between two sections, and the modulus values were quite low. During the DCP test, it was confirmed that soil was saturated at both the FDR and CRCP section in good condition as shown in Figure 3.16-(c). It appears that moisture in the soil degraded soil stiffness and strength, and potentially caused stripping of asphalt base material, which resulted in larger deflections and subsequent distresses.

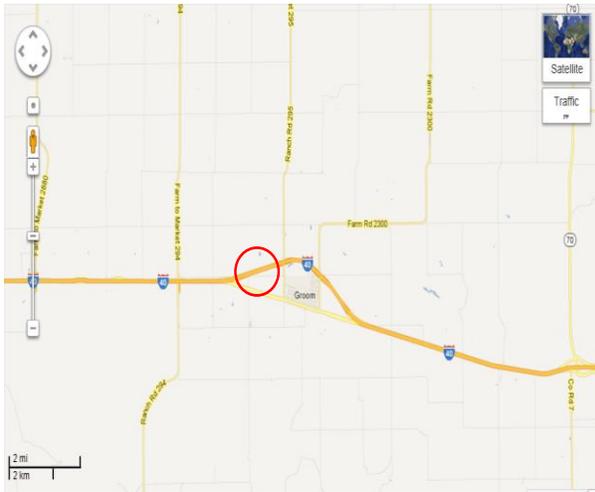


Figure 3.14-(a) Amarillo Section



Figure 3.14-(b) Detailed location



Figure 3.15- (a) Loose asphalt base



Figure 3.15- (b) Removal of loose base

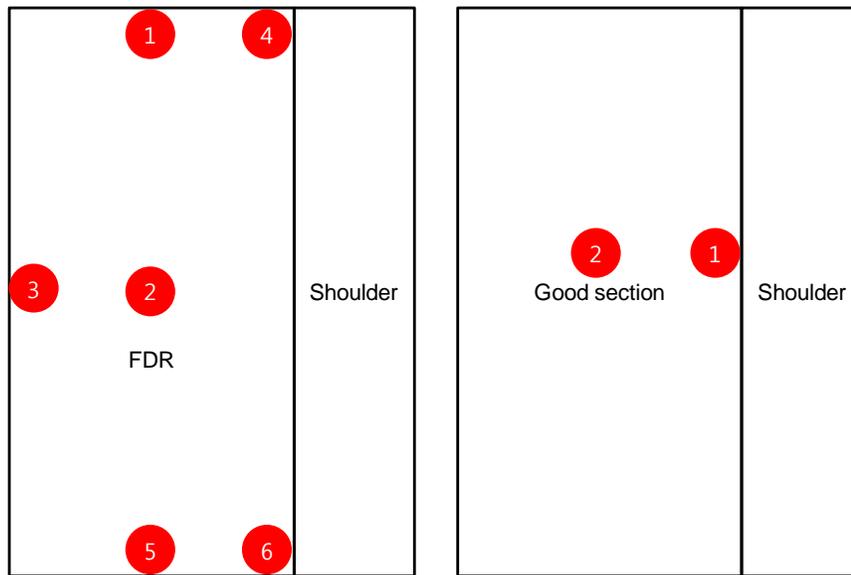


Figure 3.16- (a) DCP testing locations at FDR (left) and good section (right)



Figure 3.16- (b) DCP test at FDR



Figure 3.16- (c) DCP test in good condition

Figure 3.18 shows the gage installation layout. A total of six gages were installed: four SSGs on tie bars, one vibrating wire strain gage (VWSG) at 1.75 inches left side of the SSG-4 and one electrical concrete gage at 1.75 inches on the right side of the SSG-4. VWSG was installed at the FDR side of the repair joint at a distance of 3/8-in from the repair joint. Half of the electrical concrete gage was installed in the existing concrete at the repair joint by drilling a hole into the concrete and epoxy-grouting half of the electrical concrete strain gage. The other half of the gage was in the repair slab side. The objective of the electrical concrete strain gage in this manner was

to evaluate relative displacements between old and new concrete at the transverse repair joint. The repair operations went smoothly, all in accordance with Item 361.

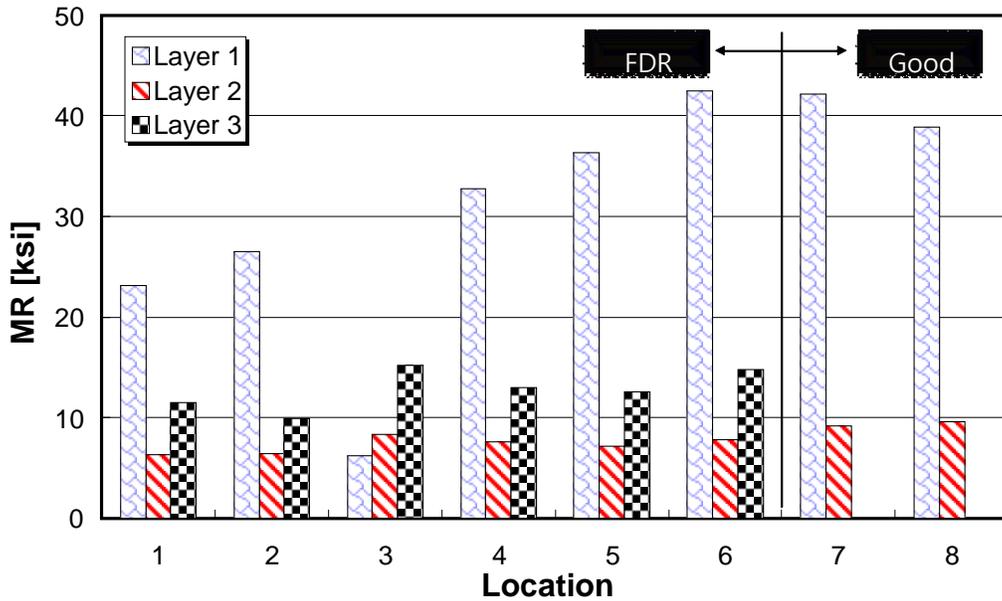


Figure 3.17 Result of DCP test at all location

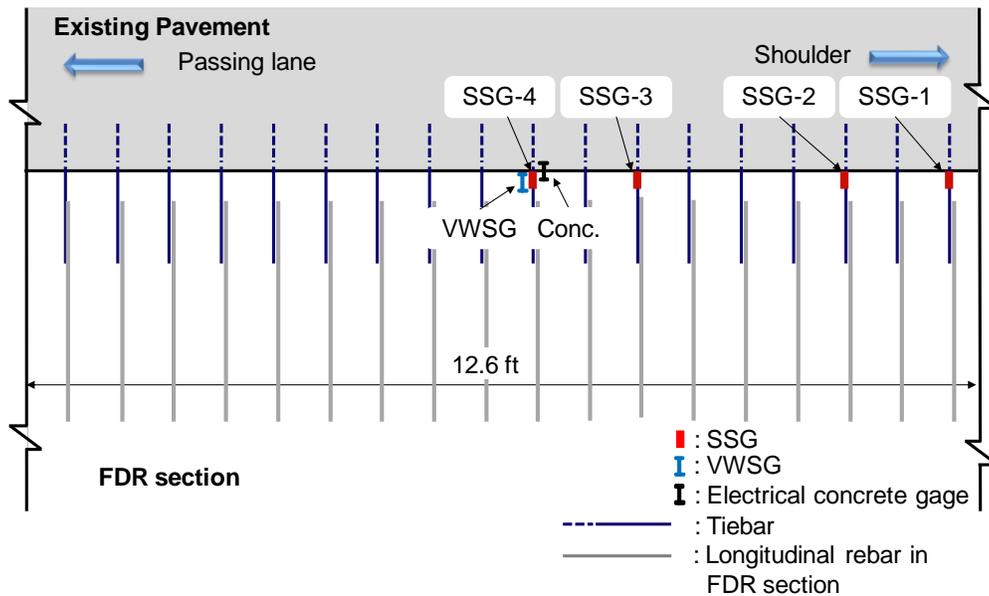


Figure 3.18 Layout of gages installation and steel

Figure 3.19-(a) shows steel strain rate vs. temperature measured from the 24th to 29th day of May, 2012. The steel strain rates ranged from -25 to -40 $\mu\epsilon/^\circ\text{F}$. Variations in the steel strain rates per temperature among the four tie bars in the Amarillo section were much smaller than those in the Wichita Falls section, which had the maximum and minimum rates of -68 $\mu\epsilon/^\circ\text{F}$ and -10 $\mu\epsilon/^\circ\text{F}$, respectively. It is presumed that the adequate bonding condition achieved in the Amarillo section by the proper method of epoxy application resulted in decreased variability in the steel strain rates per temperature. Also, the steel strains and stresses in the Amarillo section were much lower than those of the tie bar properly installed at the Wichita Falls section. Three factors – quality epoxy injection, adequate amount of tie bars, and reasonable base friction in the Amarillo section – apparently contributed to the lower values of strains and stresses as well as rate of steel strain rate per temperature in tie bars in the Amarillo section.

Figure 3.19-(b) illustrates the rate of strain changes of various gages with respect to temperature variations at the repair joint during the 24th to 29th day of May, 2012. Here, “Conc” represents electrical concrete strain gage. Figure 3.19-(b) shows that the slope of the electrical concrete strain gage data is the largest and negative, which implies that there was no full-bond between old and new concrete. It should be noted that one half of the electrical concrete strain gage was embedded into the existing concrete and the other half in the repaired concrete. Figure 3.19-(b) also indicates a positive yet quite small slope of the VWSG data, which indicates that the repaired concrete is effectively restrained by tie bars. When the concrete temperature goes up, both repaired concrete and existing concrete expands, resulting in tensile direction of concrete strain in the VWSGs and compressive direction of strain in the electrical concrete strain gage. This dataset provides valuable information on the behavior of concrete and steel at a repair joint, which can be summarized by this statement:

“Full-bond between existing concrete and repair concrete at repair joints is not likely. Most of the load transfer will be achieved by tie bars and base support.”

The above summary signifies the importance of providing adequate bond strength between tie bars and concrete and of restoring good base support. Without both, the performance of FDR will be significantly compromised.

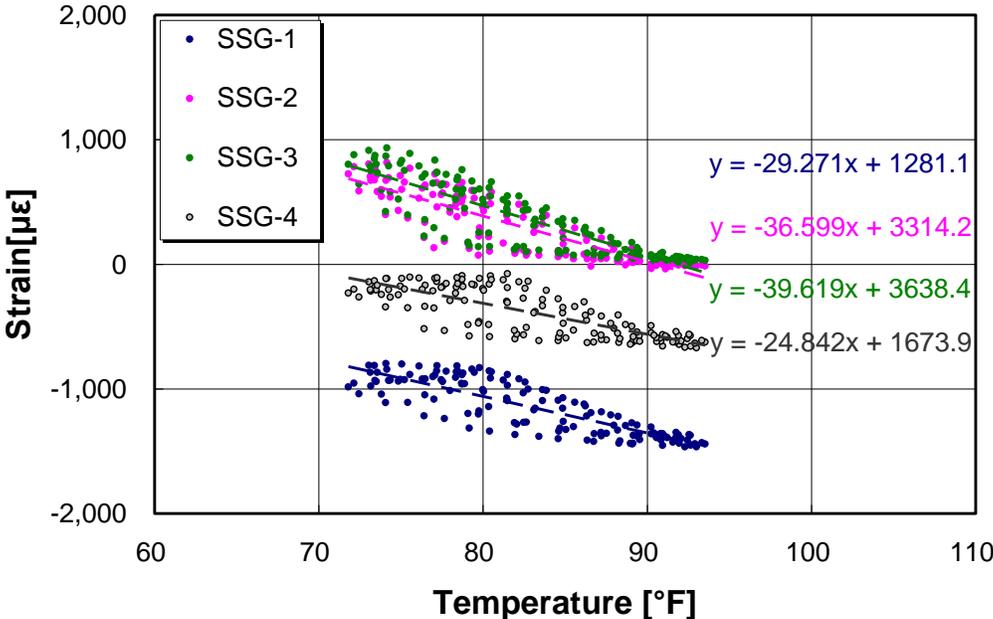


Figure 3.19- (a) Behavior of steel strain gages on tie bars

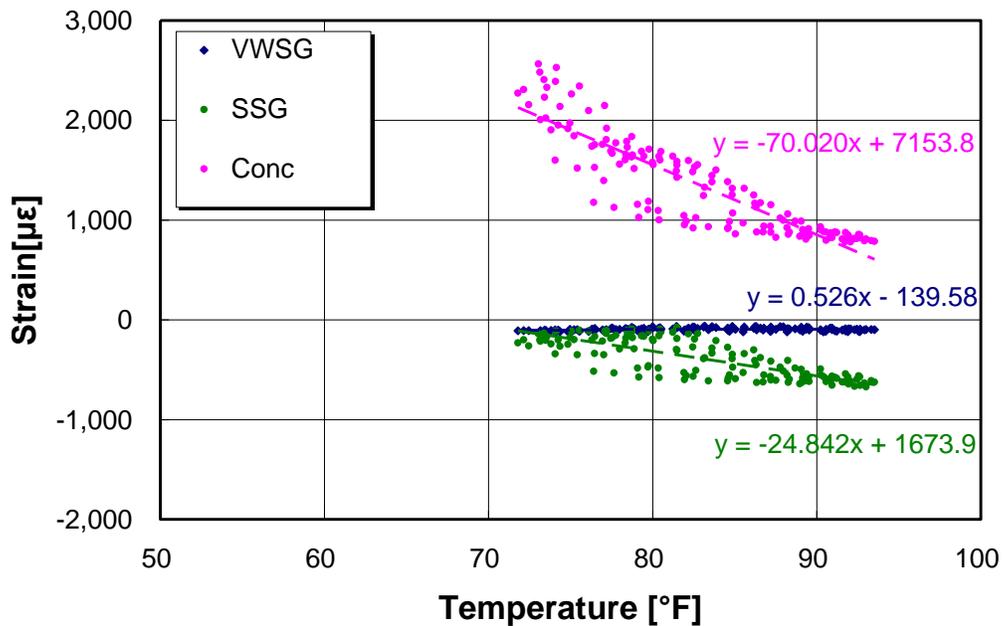


Figure 3.19- (b) Strain vs. temperature of each gage at SSG 4 location

3.4. Improvements of TxDOT Construction and Materials Specifications

The requirements of the current TxDOT Specification Item 361 and DMS 6100 were reviewed. Based on the information obtained from field observations, laboratory evaluations, and field testing, changes to the current TxDOT Construction and Materials Specifications are recommended. The recommended revisions to Item 361 are as follows:

1. The first sentence in the second paragraph of **361.3. Construction**: “Remove or repair loose or damaged base material, and replace or repair it with approved base material to the original top of base grade.”

This requirement does not provide clear and detailed guidance on what to do. The condition of the base is not known until the slab is removed. The condition of the base will partly depend on how to remove the slab as well. There will not be enough time to evaluate the base condition, determine what the best base repair material is, how much will be needed, and replace with the material, unless a decision is made to use a specific

base material such as cold mix asphalt, and sufficient quantity is prepared and available on site. Also, it is not known where the list of the “approved base material” in the specification is available. In actual projects, it is quite rare to observe contractors follow this requirement. A more practical, simple, and less time-consuming approach would be if all the loose materials were removed and, after the compaction of the existing base after the removal of loose materials, replaced with repair concrete.

2. The second sentence in the second paragraph of **361.3. Construction**: “Place a polyethylene sheet at least 4 mils thick as a bond breaker at the interface of the base and new pavement.”

This requirement is appropriate for CPCD repairs, but not for CRCP repairs. If CRCP repair is done in accordance with this requirement, steel stresses in tie bars will be higher and the joint widths at the repair joints will be larger, resulting in poor performance of FDR. This requirement should be only for CPCD repairs.

3. The third sentence in the second paragraph of **361.3. Construction**: “Allow concrete used as base material to attain sufficient strength to prevent displacement when placing pavement concrete.”

This requirement is neither practical nor needed, and will delay the FDR operation substantially. There is no need to place concrete to the original top of the base layer and wait until the concrete develops sufficient strength before repair concrete is placed. As discussed in the above, once base material is compacted, repair concrete should be placed to the top of the slab in one operation.

4. The first sentence in the third paragraph of **361.3. Construction**: “Use only drilling operations that do not damage the surrounding operations.”

This requirement is vague and does not provide clear directions on which drilling method is acceptable and which is not. As discussed in Section 3.2.2.3 of this chapter, neither

rotary drill nor hammer drill method causes significant damage to the surrounding concrete. This requirement needs to be removed.

5. The fifth sentence in the third paragraph of **361.3. Construction**: “Completely fill the tie bar hole with Type III, Class A or Class C epoxy before inserting the tie bar into the hole.”

The major difference between Type III, Type A and Type C epoxies is that, in addition to a minor difference in materials requirement in gel time, machine application is not required for Type A epoxy. As discussed previously, in order to ensure good performance of FDR, machine application or the use of a cartridge for the injection of epoxy is required. Accordingly, only Class C epoxy should be included in the specifications. Also, it is not necessary to fill holes completely with epoxy. About half of the epoxy will be pushed out during the insertion of tie bars. Only the back half of the hole needs to be filled with epoxy.

6. The sixth sentence in the third paragraph of **361.3. Construction**: “Provide grout retention disks for all tie bar holes.”

If epoxy is injected in accordance with specification requirements, the small amount of drained epoxy will not adversely affect bond strength. Field evaluations of the effectiveness of retention disks reveal that retention disks are not effective in keeping epoxy from draining down. The value of retention disks is highly questionable, and it is recommended that this requirement is removed.

7. The eighth sentence in the third paragraph of **361.3. Construction**: “Demonstrate, through simulated job conditions, that the bond strength of the epoxy-grouted tie bars meets a pull-out strength of at least $\frac{3}{4}$ of the yield strength of the tie bar when tested in accordance with ASTM E 488 within 18 hr. after grouting.”

As discussed previously, bond strength is highly dependent on how much of the void between tie bars and surrounding concrete in the drilled holes is filled with epoxy. The

bond strength is not very dependent on the embedded depth, air temperature, or epoxy type, within the limitations of the testing program. In other words, the bond strength depends on the quality of the repair operations, especially the way epoxy is injected, and not on the epoxy material itself. A contractor can inject epoxy in the right way during a demonstration, resulting in sufficient bond strength, but may produce poor quality FDR by not injecting epoxy properly during actual repairs. This requirement, therefore, does not add any real value.

8. The ninth and tenth sentences in the third paragraph of **361.3. Construction**: “Increase embedment depth and retest when necessary to meet testing requirements. Perform tie bar testing before starting repair work.

As shown in the laboratory testing, increasing the embedded depth beyond 6-in does not improve bond strength substantially. Lower bond strength cannot be improved by increasing the embedded depth beyond 12-in. There are other reasons for lower bond strength, and it will be most likely due to not filling the voids with epoxy. Accordingly, the requirement of increased embedment depth is not needed. It is recommended that this requirement is removed.

9. As discussed previously, cleaning the holes before epoxy injection will reduce shear displacements, resulting in tighter joint widths. However, the requirement of cleaning holes is not included in the current Item 361. It is recommended that the cleaning requirement is included in Item 361.

Based on the discussions above, new special specification for FDR was developed, which is included in the Appendix D.

The recommended revisions to DMS 6100 are as follows:

1. In Table 4, a minimum wet pullout strength is 4,500 lbf. The comment in the table for

wet pullout strength states, “The wet pullout test determines the strength of the adhesive bond between a steel anchor and the surface of a hole in concrete or masonry units.” The test method for the wet pullout strength is not specified. The pullout strength requirement for Grade 60 steel in Item 361 is 45,000 psi, and for #6 bar, it is about 19,800 lbf. There is a discrepancy between the strength requirements in Table 4 and those in Item 361. Either “wet” pullout strength needs to be well defined, or the strength requirement needs to be adjusted.

2. Minimum tensile bond strength at six hours is 200 psi for all classes of epoxy. It is not known whether this tensile bond strength is adequate to provide 1,590 psi shear bond strength for #6 tie bars ($45,000 \text{ psi} / 28.3 \text{ square inches} = 1,590 \text{ psi}$).

Chapter 4 Partial Depth Repairs

Distresses in CRCP are broadly classified as spalling or punchout. Spalling is a surface defect, and not considered as a structural failure. On the other hand, punchout is a structural failure, caused by deficiencies in the structural capacity of CRCP. Spalling is considered a minor distress and usually repaired with appropriate patching materials in accordance with Item 720 “REPAIR OF SPALLING IN CONCRETE PAVEMENT.” On the other hand, punchouts are repaired, in most cases, in accordance with Item 361, “FULL-DEPTH REPAIR OF CONCRETE PAVEMENT.” Extensive evaluations of punchouts in Texas conducted under TxDOT’s rigid pavement research project indicate a large portion of the so-called punchouts are not full-depth distress; rather, the distresses are confined to the concrete above the longitudinal steel, with the concrete below the longitudinal steel solid and in good condition. For those distresses, FDR may not be the most cost-effective repair method. Partial-depth repair (PDR) might be the optimum repair method for those distresses. However, the challenge is to properly identify the extent of the distress. Because of the challenge of differentiating full-depth and partial-depth punchouts in the field, it appears that FDR was used to repair both full-depth and partial-depth punchouts.

In this study, efforts were made to identify a simple and easy method or device that can detect the extent of the distress. A device called MIRA was identified as the one with the most potential. One unit was purchased and extensive evaluations were made of the MIRA’s capability and limitations in detecting the extent of concrete distress.

4.1 MIRA

MIRA is an instrument for creating a three-dimensional (3-D) representation (tomogram) of internal defects that may be present in a concrete element. Figure 4.1-(a) shows MIRA. MIRA is based on the ultrasonic pulse-echo method using transmitting and receiving transducers in a "pitch-catch" configuration as shown in Figure 4.1-(b). One transducer sends out a stress-wave pulse and a second transducer receives the reflected pulse.



Figure 4.1-(a) MIRA

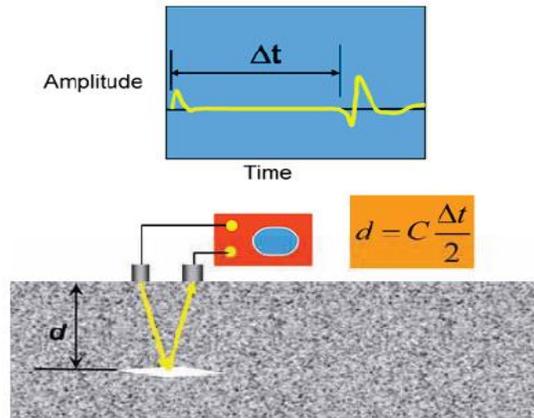


Figure 4.1-(b) Principal of MIRA system

The time from the start of the pulse to the arrival of the echo is measured. The wave speed is computed and the depth of the reflecting interface is estimated. If there are distresses at the depth of the longitudinal steel in the form of horizontal cracking, which is usually the case in partial-depth punchouts in CRCP, MIRA will detect the existence of the distress, as graphically shown in Figure 4.1-(b). The transducer array is under computer control and the recorded data are transferred wirelessly to a host computer in real time. The computer takes the raw data and creates a 3-D image of the reflecting interfaces within the element.

4.2 Application of MIRA to Detect Horizontal Cracking

The MIRA system was utilized to detect the extent of the distresses in a CRCP section on IH 35 in the Laredo District. The section is located on IH 35, northbound from mileposts 51 to 52. Two main lanes and outside and inside tied concrete shoulder with 9-in thick slab were placed on top of existing asphalt pavement in 2002 as a test section to demonstrate the viability of concrete overlay on existing asphalt pavement. Initial performance of this one-mile section was reported as excellent; however, distresses in the form of Y-cracks and punchouts were observed as early as 2009. Figure 4.2-(a) shows pavement condition observed in April, 2009. The transverse crack on the left is a normal crack in CRCP, caused by temperature and moisture variations (environmental loading). However, the crack on the right is not a crack normally observed in

CRCP. First, this crack is much straighter than normal cracks in CRCP that occur at early ages due to environmental loading. Second, this crack is quite close to the existing transverse crack. Normal transverse cracks caused by environmental loading will occur somewhere in the middle of two adjacent transverse cracks, because concrete stresses due to environmental loading are zero at transverse cracks, and increase with the distance from transverse cracks. Accordingly, the crack on the right in Figure 4.2-(a) was not caused by environmental loading alone. Figure 4.2-(b) shows cracks with similar characteristics. This section is located in southbound main lanes on IH 35 in the Waco District between mileposts 362 and 363. This 14-in CRCP was completed in 1999, and horizontal cracking was observed at the depth of longitudinal steel in November 1999, before this section was open to traffic. This picture was taken in August, 2009, and the pavement was 10 years old when this picture was taken. The crack on the left is a normal crack due to environmental loading that occurred at early ages. On the other hand, the crack on the right has the same characteristics as the one on the right in Figure 4.2-(a). Based on the similarities between the two projects, it is concluded that the crack on IH 35 in the Laredo District was caused by a horizontal crack at the depth of the longitudinal steel.



Figure 4.2-(a) Cracks on IH 35 in Laredo



Figure 4.2-(b) Cracks on IH 35 in Waco

The existence of horizontal cracks at the depth of longitudinal steel and resulting distresses were discovered first in Texas on IH 35 project in the Waco District. It appears that CRCP distresses due to horizontal cracking at the depth of steel were quite common, but were not recognized until 1999. Figure 4.3-(a) shows a distress on IH 35 in the Waco District observed in 2009. It is

observed that the crack on the left is a normal early-age crack, while the one on the right has the same characteristics of the cracks on the right in Figures 4.2-(a) and 4.2-(b). The slab segment between two close transverse cracks deteriorated primarily due to wheel loading applications, and it appears that multiple applications of various materials were made to repair this distress. It took 10 years for horizontal cracks at the depth of longitudinal steel to develop into a distress. Distress shown in Figure 4.3-(b) is on IH 35 in the Laredo District. Three transverse cracks occurred right next to each other, and distresses developed near wheel paths. There are similarities between the distresses shown in Figures 4.3-(a) and 4.3-(b). These distresses are not full-depth punchouts. In the past, these were misdiagnosed as full-depth punchouts, and researchers tried to hypothesize their mechanisms, theorizing that slab segments with short crack spacing behave like a cantilever beam and, with wheel load stress in the transverse direction becoming large due to wheel loading applications, longitudinal cracks will develop, resulting in punchouts. Obviously, this theory is not technically correct. TxDOT recognized the deficiency in this theory and initiated a research study on horizontal cracking, 0-5549. Efforts made in that study identified the mechanism of horizontal cracking and subsequent distress.



Figure 4.3-(a) Distress due to horizontal cracking on IH 35 in Waco



Figure 4.3-(b) Distress due to horizontal cracking on IH 35 in Laredo

To evaluate the distress mechanisms on IH 35 in the Laredo District and the capability of MIRA in correctly detecting horizontal cracking in CRCP, field testing was conducted on May 15, 2012, which included coring, MIRA testing, and FWD testing. Locations with potential horizontal

cracking were identified and selected for testing. Figure 4.4-(a) shows two cracks – the one on the top is a natural CRCP crack, while the one in the middle is due to horizontal cracking. Figure 4.4-(b) shows the MIRA analysis and image, illustrating the existence of horizontal cracking. The red color illustrates voids or gap in the concrete. To confirm MIRA analysis results, coring was made and horizontal cracking was observed, as shown in Figure 4.4-(c) and (d).



Figure 4.4-(a) Condition of section 1

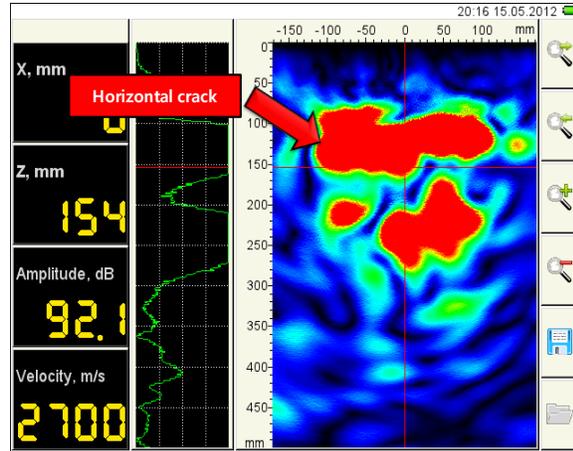


Figure 4.4-(b) MIRA result at section 1



Figure 4.4-(c) Coring



Figure 4.4-(d) Horizontal crack

Another location was selected for MIRA testing. Figure 4.5-(a) shows CRCP condition at this location. There are three transverse cracks close to each other. The one on the right is a normal transverse crack, and the other two on the left are due to horizontal cracking. The condition of

concrete inside the pavement was evaluated with MIRA. Figure 4.5-(b) illustrates 3-D analysis result from MIRA testing. Horizontal crack was detected at mid-depth, along with longitudinal steel. Figure 4.5-(c) shows the plan view of the MIRA analysis results. The extent of horizontal cracking and longitudinal steel is illustrated.



Figure 4.5-(a) Condition of section 2

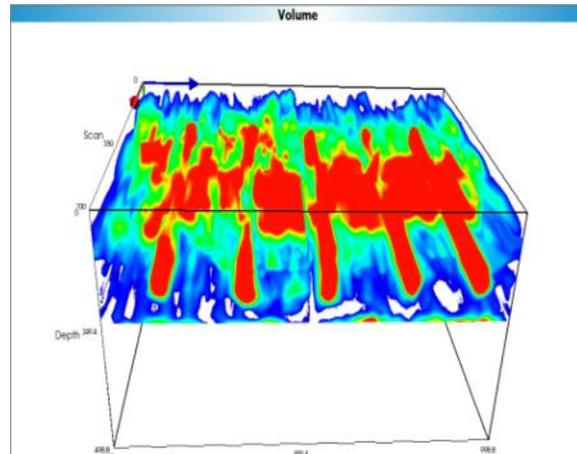


Figure 4.5-(b) 3D result by MIRA

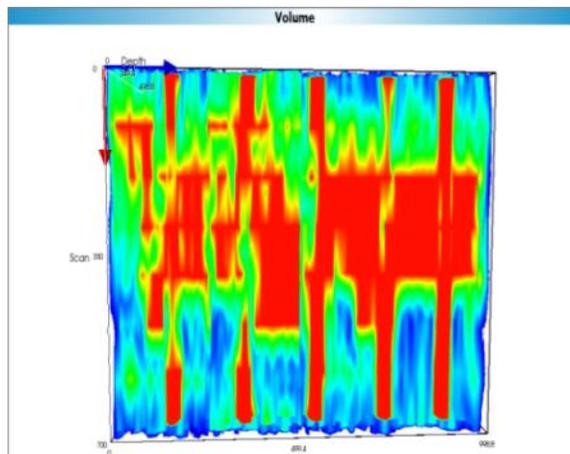


Figure 4.5-(c) Plan view by MIRA

Figure 4.6-(a) shows another CRCP location, where the evidence of horizontal cracking does not exist. This section was evaluated with MIRA. Figure 4.6-(b) illustrates the 3-D image analysis result with MIRA. MIRA detected rebars at mid-depth, but there was no horizontal crack at mid-depth. Figure 4.6-(c) shows the plan view of MIRA analysis result, illustrating that there was no

horizontal cracking at the depth of longitudinal steel, with only longitudinal steel shown. MIRA was also used for the detection of longitudinal steel locations, so that cores could be obtained without cutting longitudinal steel. A total of five cores were taken from locations with no horizontal cracks by MIRA, and all five cores were taken without hitting steel and horizontal cracks.

The field testing confirmed the capability of MIRA to detect horizontal cracking and locations of longitudinal steel in CRCP as well as the slab thickness quite accurately. MIRA also successfully detected mud balls in concrete in a non-TxDOT paving project. It appears that MIRA can detect discontinuities in concrete, whether they are horizontal cracking, voids, or mud balls. MIRA can be used to determine the extent of horizontal cracking and the area of needed repairs. However, MIRA is expensive. At the writing of this report, its cost is about \$60,000.



Figure 4.6-(a) Condition of section 2

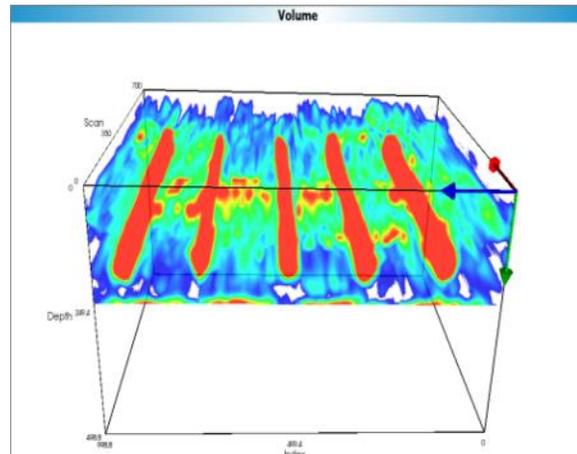


Figure 4.6-(b) 3D result by MIRA

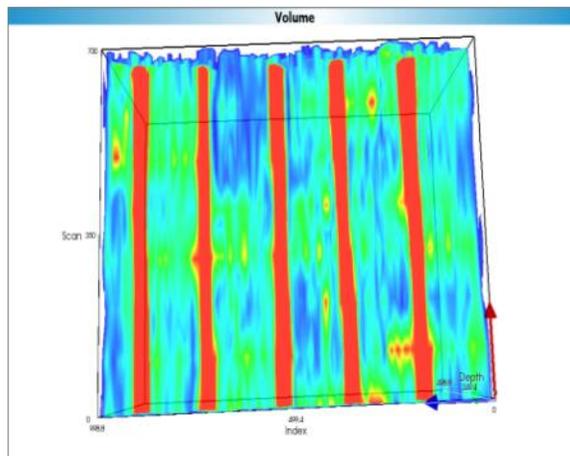


Figure 4.6-(c) Plan view by MIRA

4.3 Use of MIRA to Determine Correct Repair Method and Extent of Repair

The capability of MIRA to detect horizontal cracking or delaminations in CRCP was confirmed. It takes less than one minute to conduct MIRA testing for one spot. The data analysis is conducted in near real-time, and whether horizontal cracking exists is determined in real time. One major characteristics of horizontal cracking at the depth of longitudinal steel in CRCP is the unique nature of the horizontal cracking limits. Numerous field evaluations conducted in TxDOT research project 0-5549 positively indicate that horizontal cracking is limited to the area between a normal transverse crack and a transverse crack caused by a horizontal crack. Accordingly, the most practical way to determine whether FDR or PDR is needed is to evaluate the distressed area using MIRA for horizontal cracking. If horizontal cracking exists and bottom concrete is solid, PDR is the more appropriate repair method. Otherwise, FDR needs to be made. Once a decision is made to conduct PDR, the boundaries should include the distressed area between one normal transverse crack and one transverse crack caused by horizontal cracking.

More detailed guidelines for PDR are included in Appendix F.

Chapter 5 Conclusions and Recommendations

Various full depth repair methods have been used for the repair of distresses in CRCP, and their performance has varied. CRCP is normally used in highways with heavy traffic, and the window of repair in metropolitan areas in Texas is quite limited, normally not to exceed nine hours, including setting up traffic control. The most practical repair method for CRCP in metropolitan areas is the so called full-depth cut (FDC) method, where full-depth cuts are made along repair boundaries and tie bars are epoxy-grouted into the existing concrete. The primary advantage of this method is that it is quick and relatively simple. However, the performance has been less than optimal. Investigations were made to identify the causes of poor performance of FDR with this method and areas of needed improvements, which could include specifications, design standards, and FDR operations. In TxDOT, it was observed that some partial-depth failures were restored with a full-depth repair method. Partial-depth repairs could be a better rehabilitation tool for partial-depth failures. One of the objectives of this study was to identify a device that can detect partial-depth failures. To achieve the objectives of this study, the following tasks were conducted:

- (1) field testing at repair areas using a non-destructive testing device,
- (2) experiments in a test slab,
- (3) field experimentation at repair areas using various gages,
- (4) field evaluation of a device that has a potential for the detection of delamination, and
- (5) review of specification Item 361 and DMS 6100.

A vast amount of information was obtained, and the findings can be summarized as follows:

- 1) The primary cause for poor performance of FDR is the failure to restore structural continuity at transverse repair joints. Common characteristics of poorly performing FDRs were large deflections at transverse repair joints. Poor bond between tie bars and the surrounding concrete at repair joints appears to contribute to these large deflections and poor performance.
- 2) Experiments in a test slab revealed:

- a) Keeping the epoxy full in the voids between tie bars and the surrounding concrete in the holes is key to good bond strength between the tie bars and concrete and performance. The requirement of Item 361 that epoxy should be applied in the holes first, followed by the insertion of tie bars, should be positively enforced during FDR.
 - b) Neither rotary drill nor hammer drill caused significant damage to concrete, and resulted in practically the same bond strength. The hammer drill is much faster than the rotary drill, and will expedite the FDR process.
 - c) The drilling depth is not a critical element for bond strength, as long as it is greater than six inches.
 - d) Keeping the hole clean positively affects bond strength and shear displacement.
 - e) Curing time of the epoxy does not appear to affect bond strength, as long as it is greater than 30 minutes.
 - f) Curing temperature of the epoxy does not affect bond strength as long as the air temperature is above 50 °F.
- 3) Field experimentation with steel and concrete strain gages to evaluate stress levels in tie bars and concrete at repair joints indicated:
- a) Physical separation occurred between existing and new concretes at transverse repair joints, which implies that shear stiffness at a repair joint is primarily achieved by tie bars and base support.
 - b) When tie bars were installed in accordance with specifications and design standards, stresses in tie bars were maintained below the limiting value, which resulted in good load transfer and reduced deflections. Reduced deflections will enhance FDR performance.
- 4) Efforts were made to identify a device that can detect horizontal cracking in CRCP. A device called MIRA was selected and field evaluations were conducted. The findings were:
- a) MIRA can detect horizontal cracks, mud balls, voids, and reinforcing steel in the

- concrete with accuracy.
- b) Slab thickness can be accurately estimated by MIRA.
 - c) 3-D analysis image from MIRA shows a view of the inside condition of concrete.
- 5) A review of TxDOT Specification Item 361 and DMS 6100 revealed that there are deficiencies in both and revisions need to be made. Detailed descriptions and recommendations are provided in Chapter 3, and special specifications were developed and included in Appendix D.

In order to enhance the operational efficiency of CRCP repairs, specifications and design standards as well as guidelines for partial-depth repairs were developed. They are included in Appendix E, C, and G, respectively.

Optimum performance of repairs of CRCP distresses can be achieved only when repair operations are conducted strictly in accordance with the requirements in specifications and design standards. Any deviations could result in less than optimum performance, potentially requiring subsequent repairs of the already-repaired areas. To enhance the operational efficiency of TxDOT operations on CRCP repairs, periodic training for TxDOT project staff, including maintenance engineers and inspectors, through webinar or other means, on the findings of this study is recommended.

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Appendix A: FWD test and condition survey

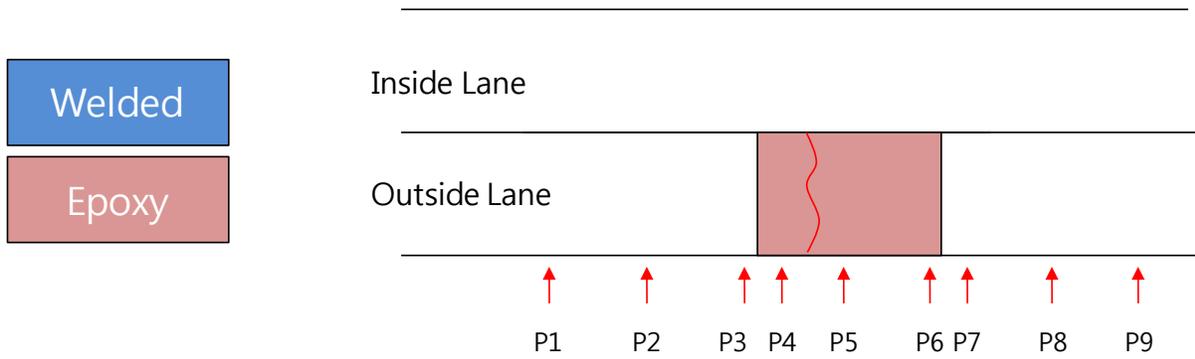
1. Bowie –Atlanta District (US 59) – 02/21~23/2011

1.1 Section 1

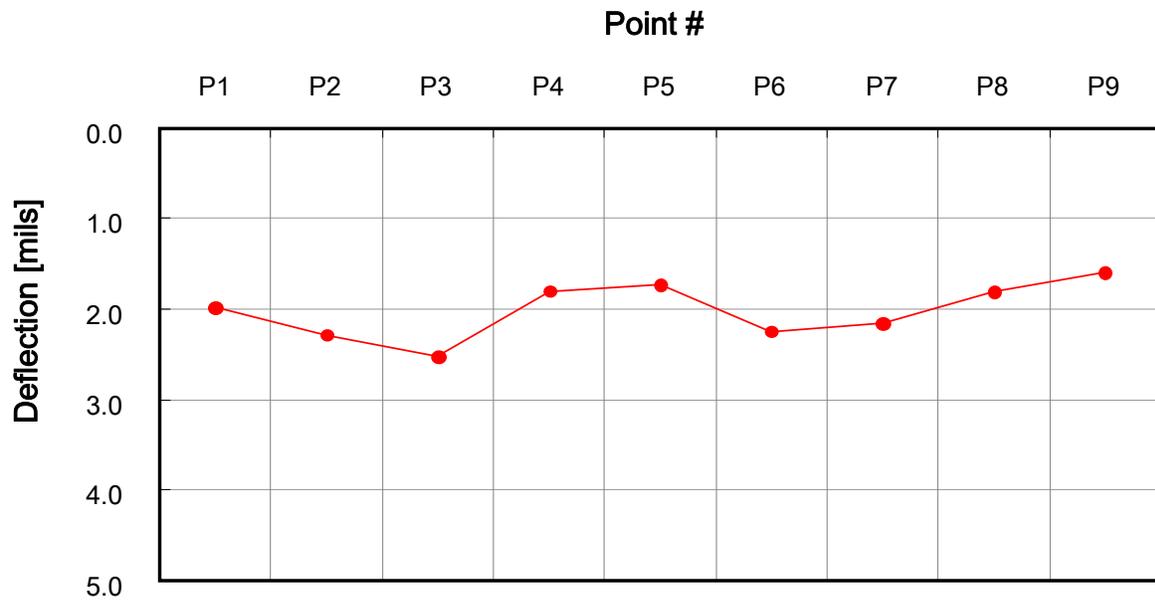
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(2) Dimension and FWD testing points



(3) FWD result

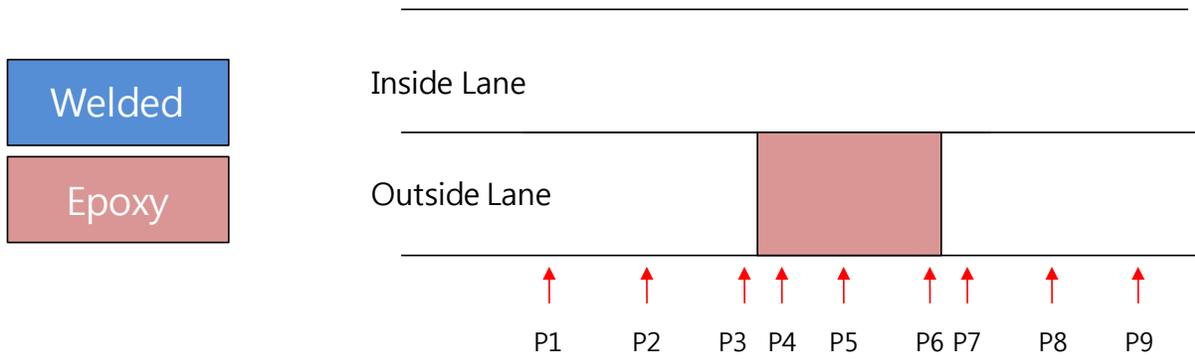


1.2 Section 2

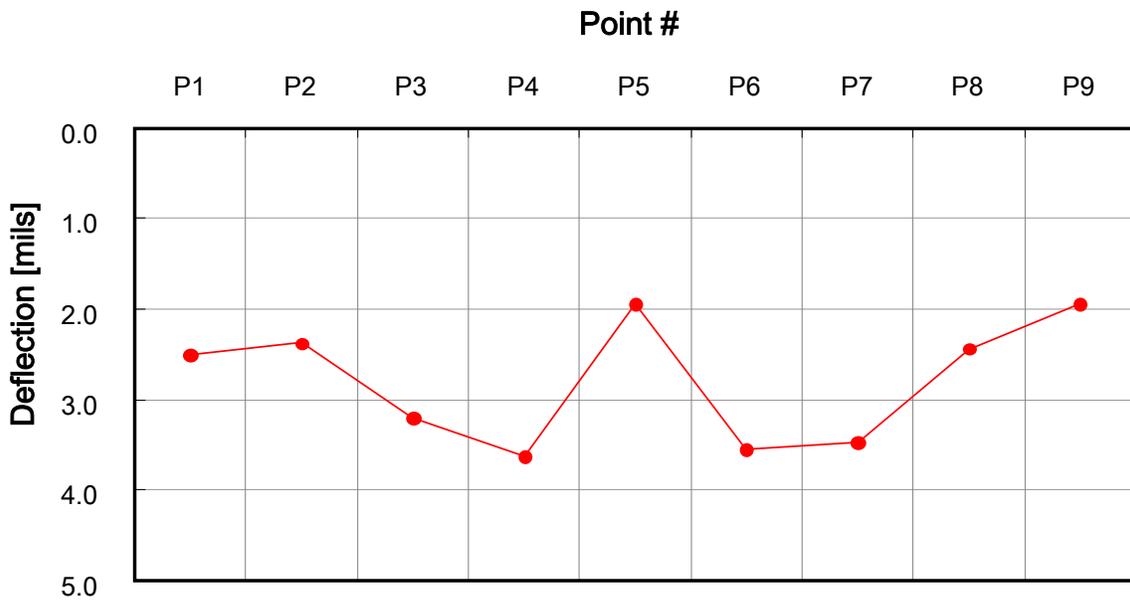
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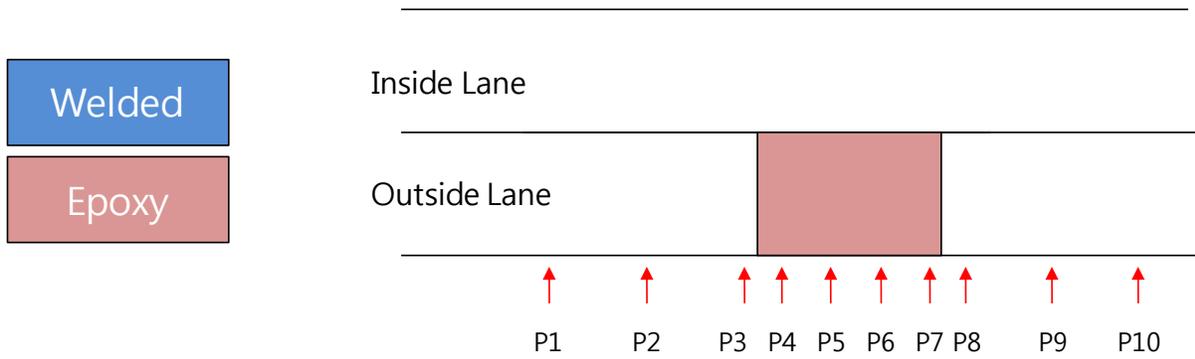


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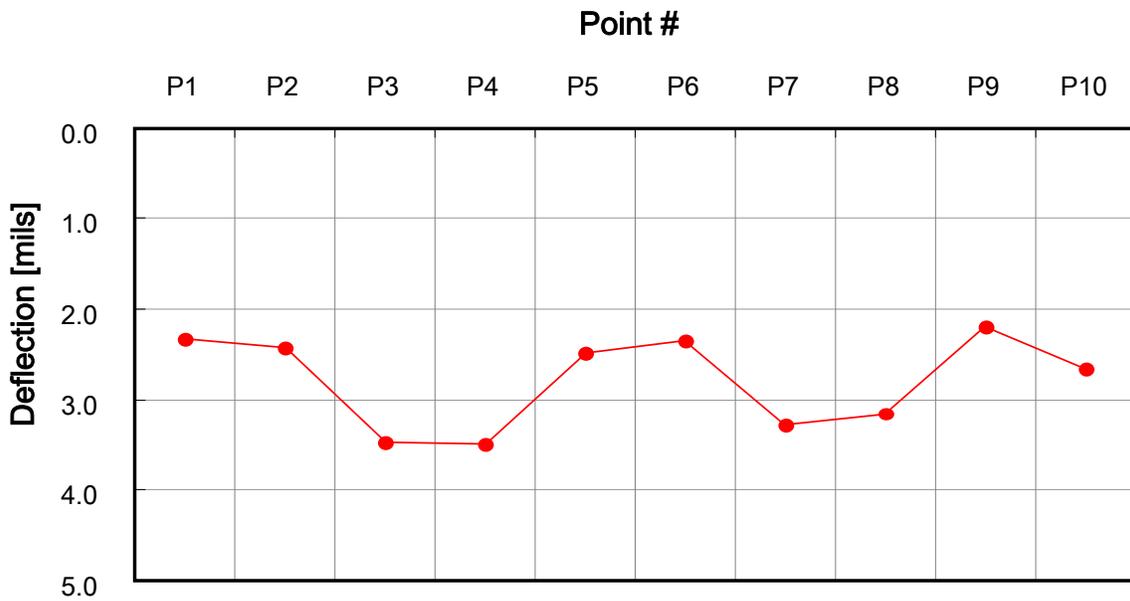
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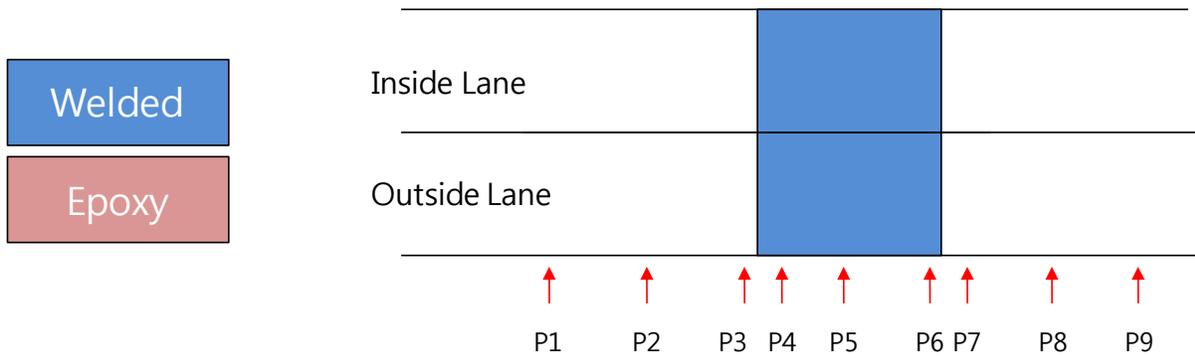


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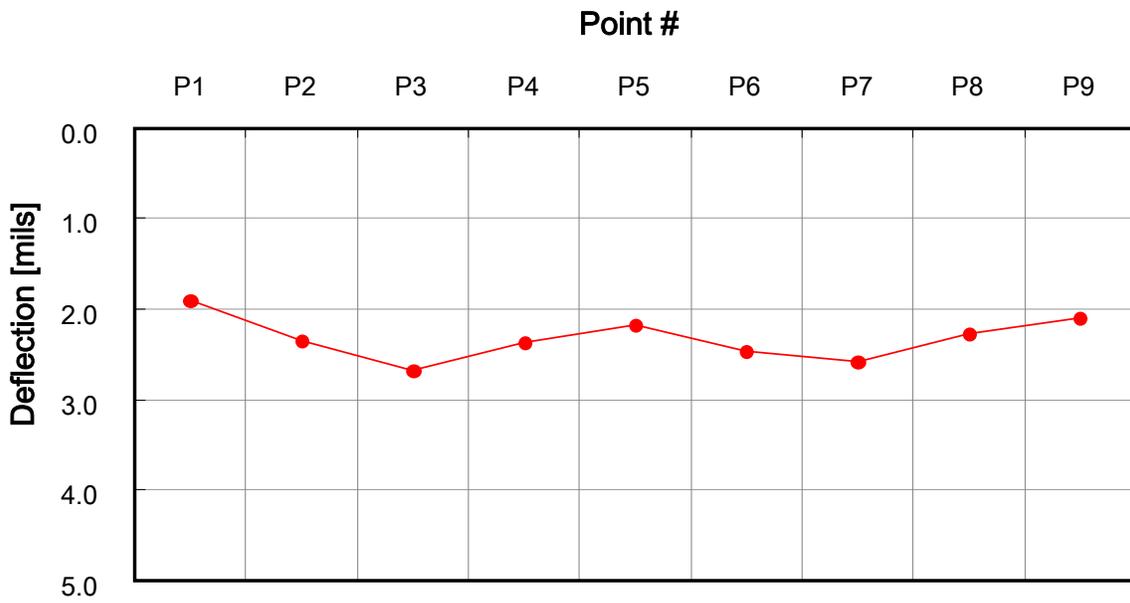
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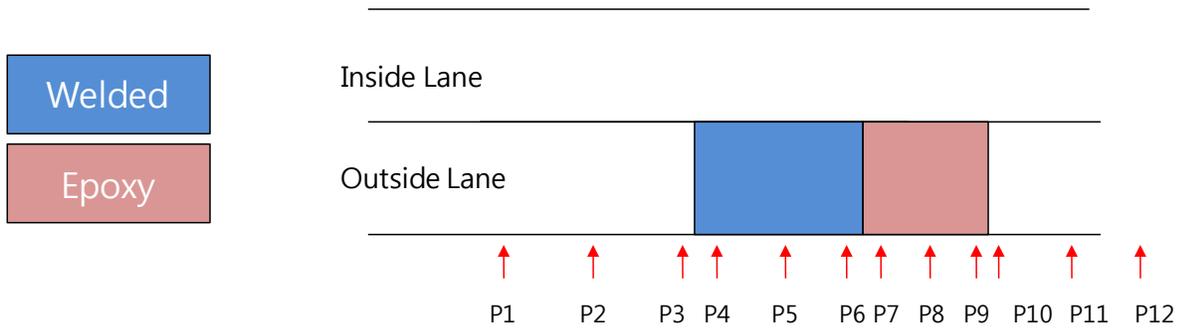


1.5 Section 5

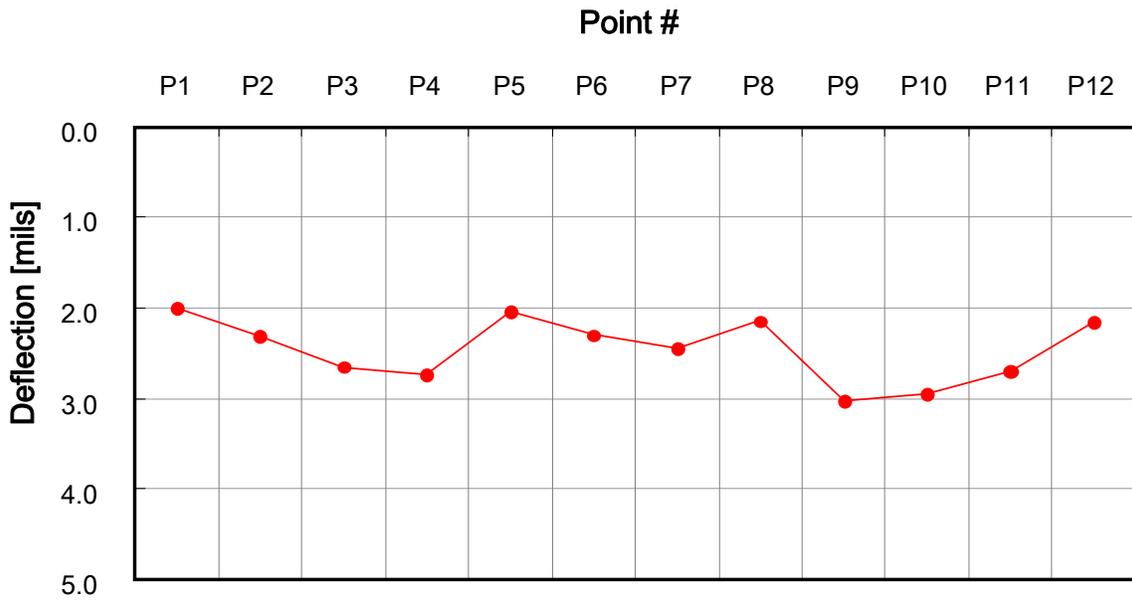
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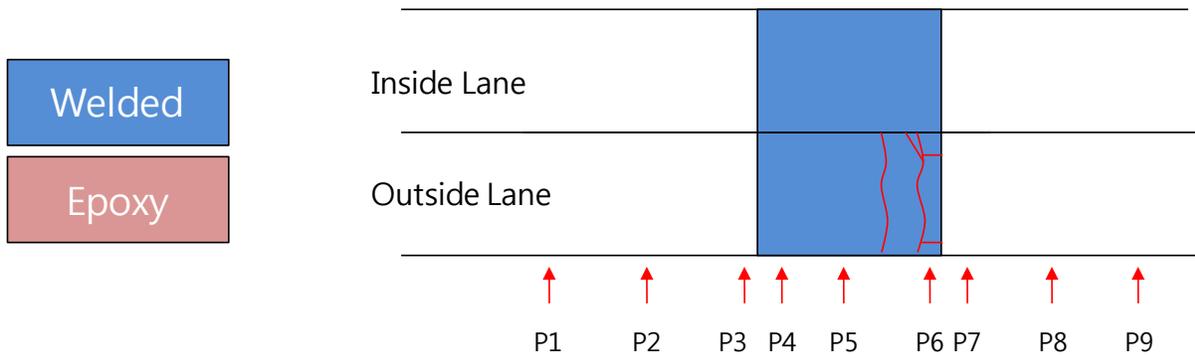


1.6 Section 6

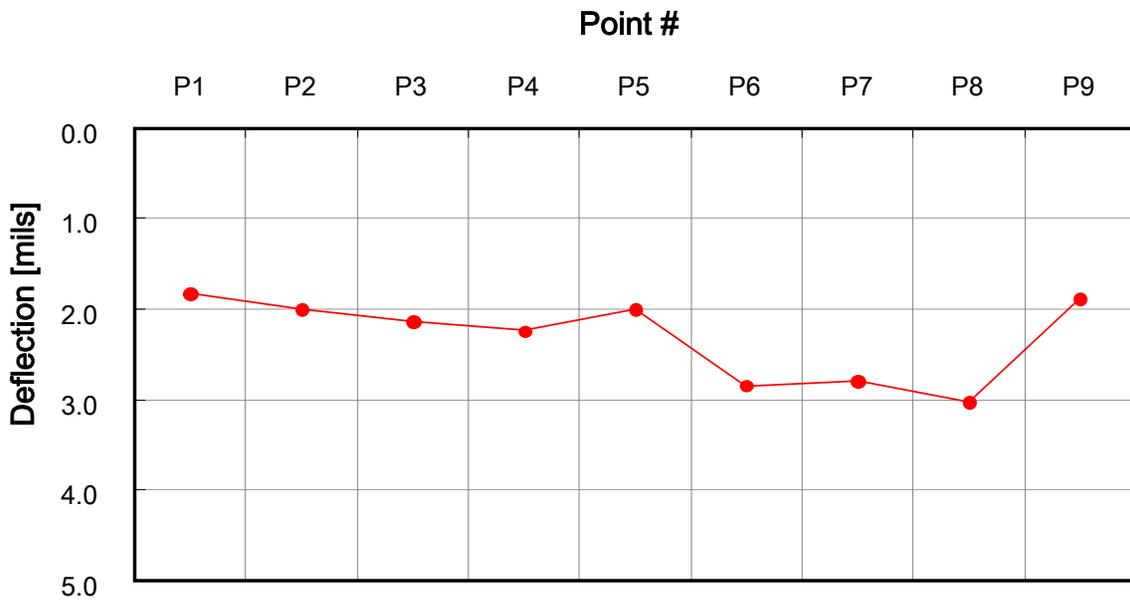
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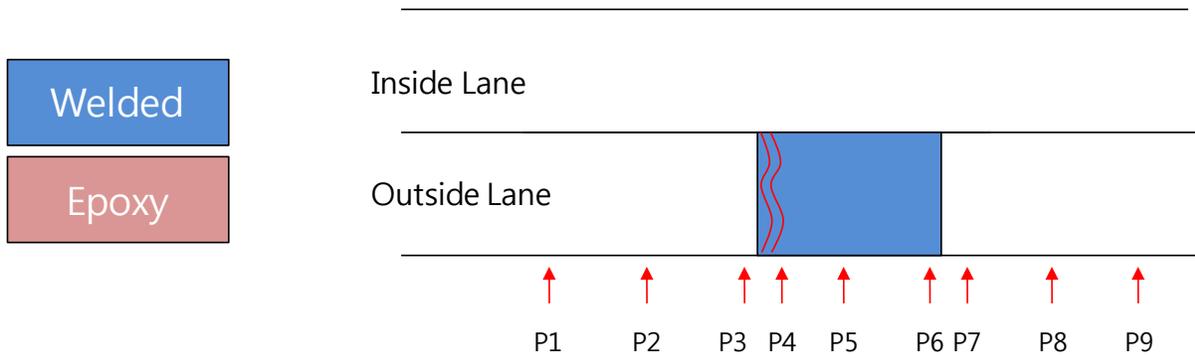


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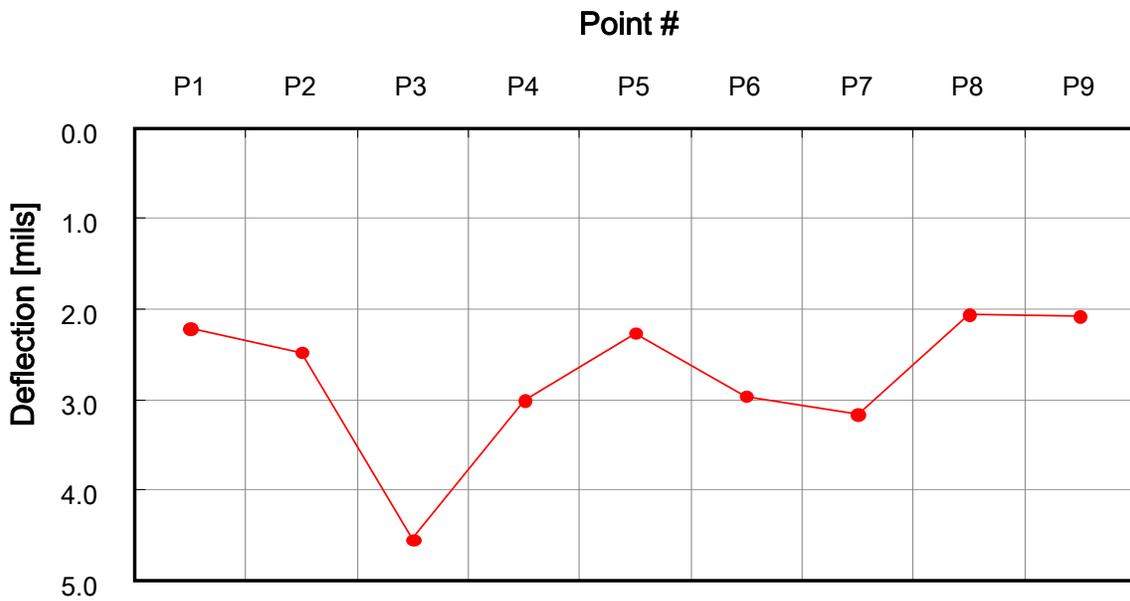
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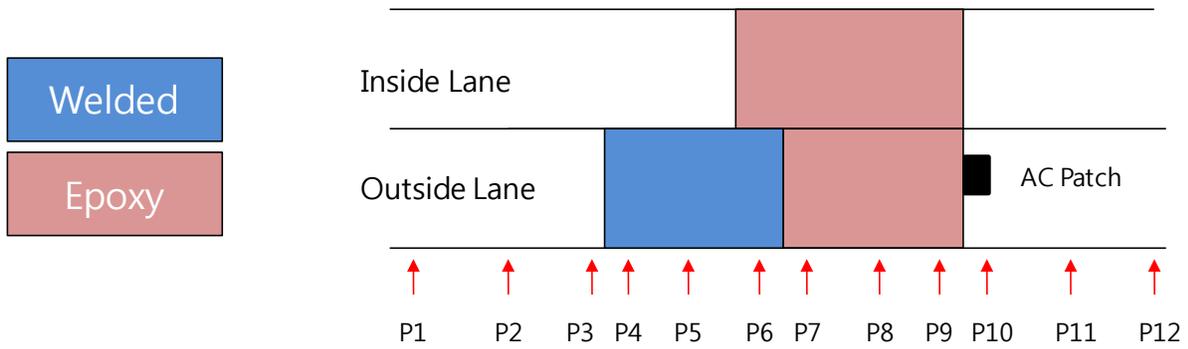
2. Cass –Atlanta District (US 59) – 02/21~23/2011

2.1 Section 1

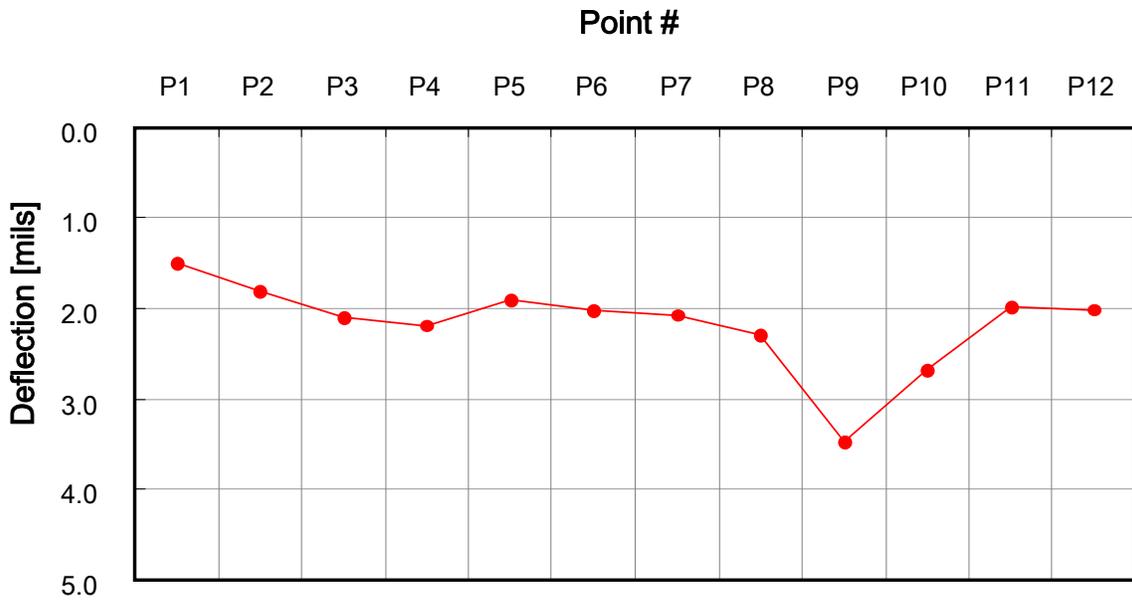
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(3) FWD result

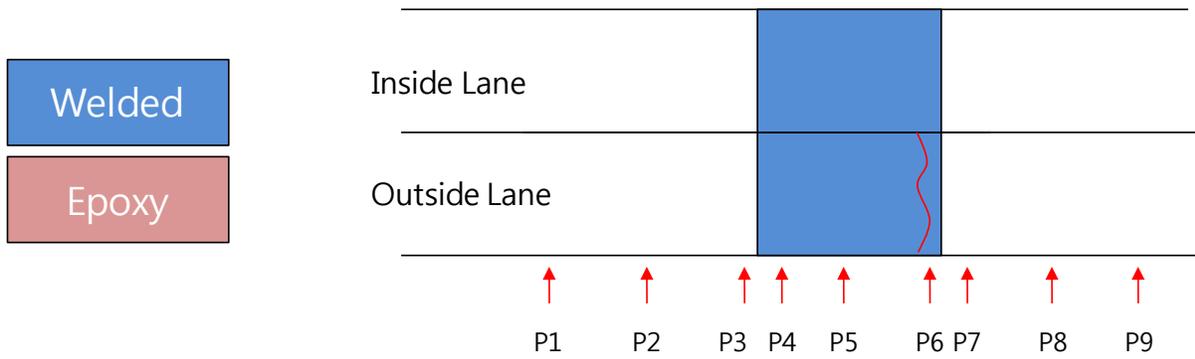


2.2 Section 2

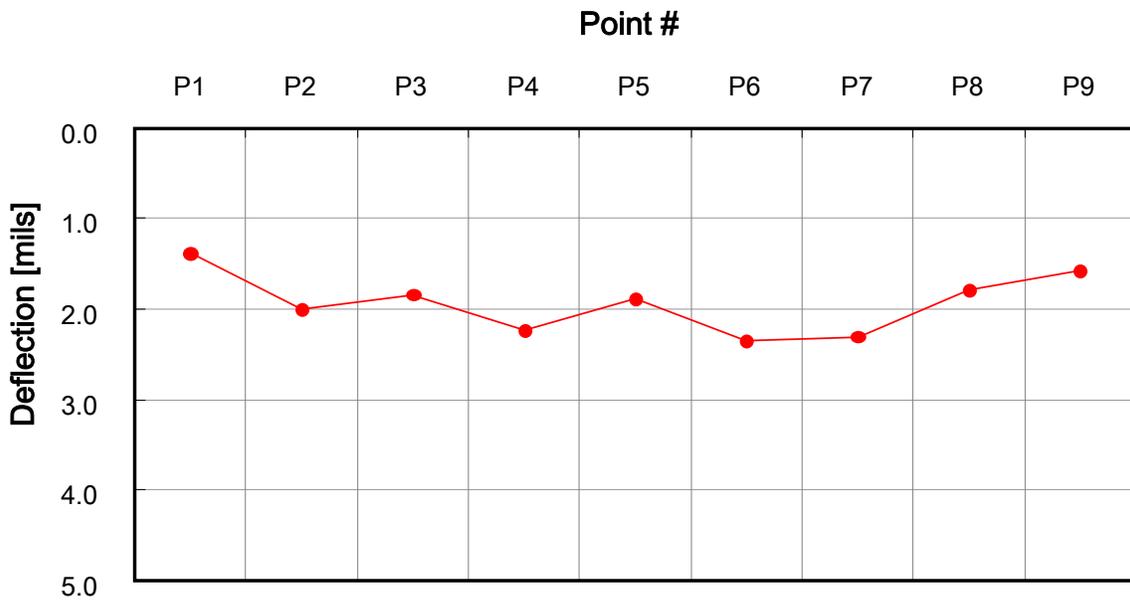
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(2) Dimension and FWD testing points



(3) FWD result

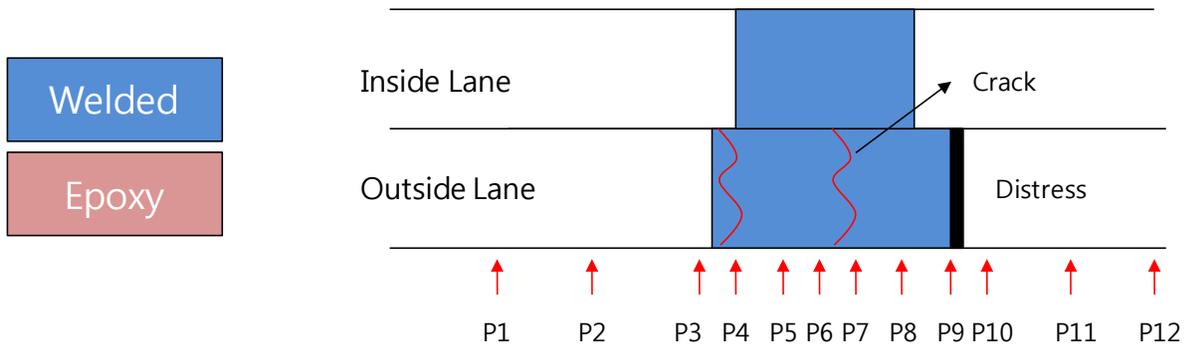


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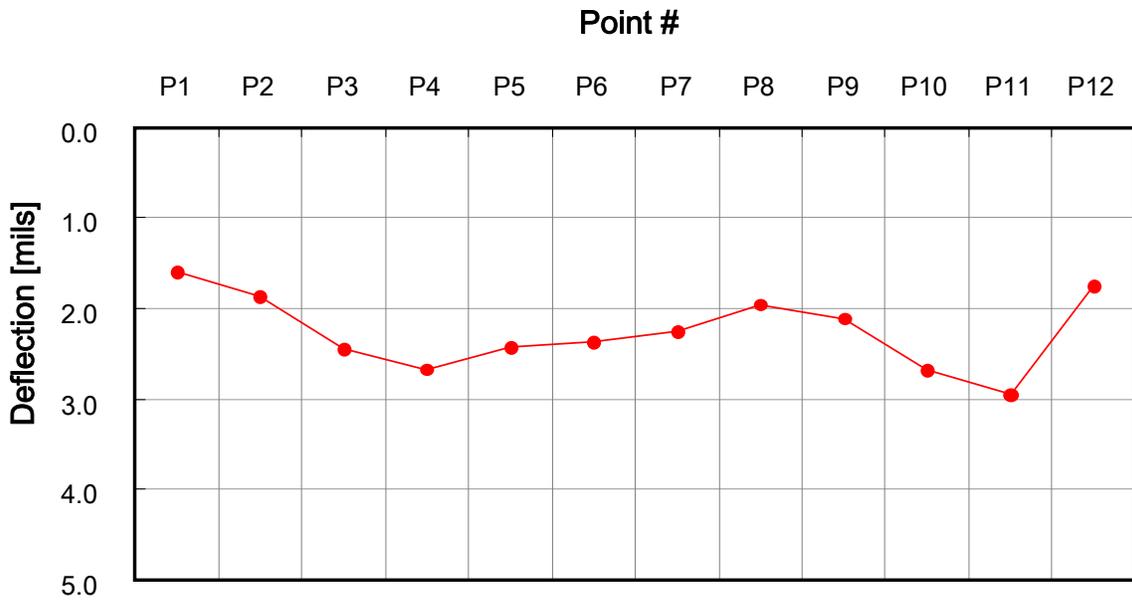
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(2) Dimension and FWD testing points



(3) FWD result

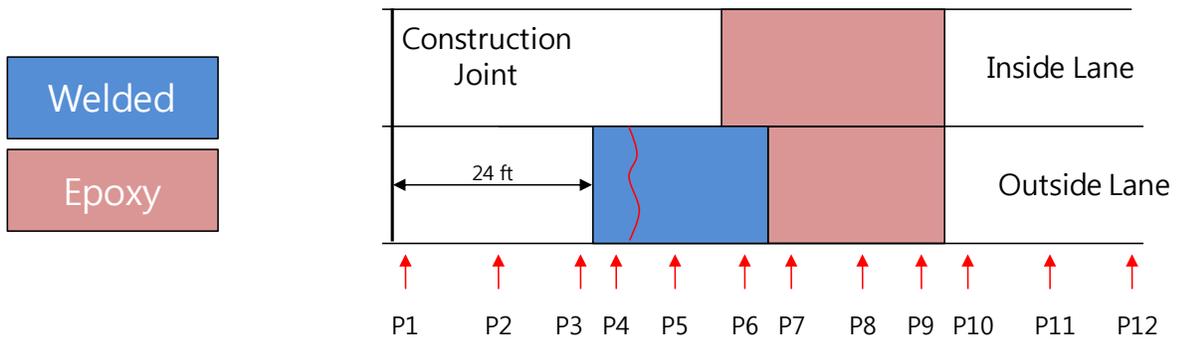


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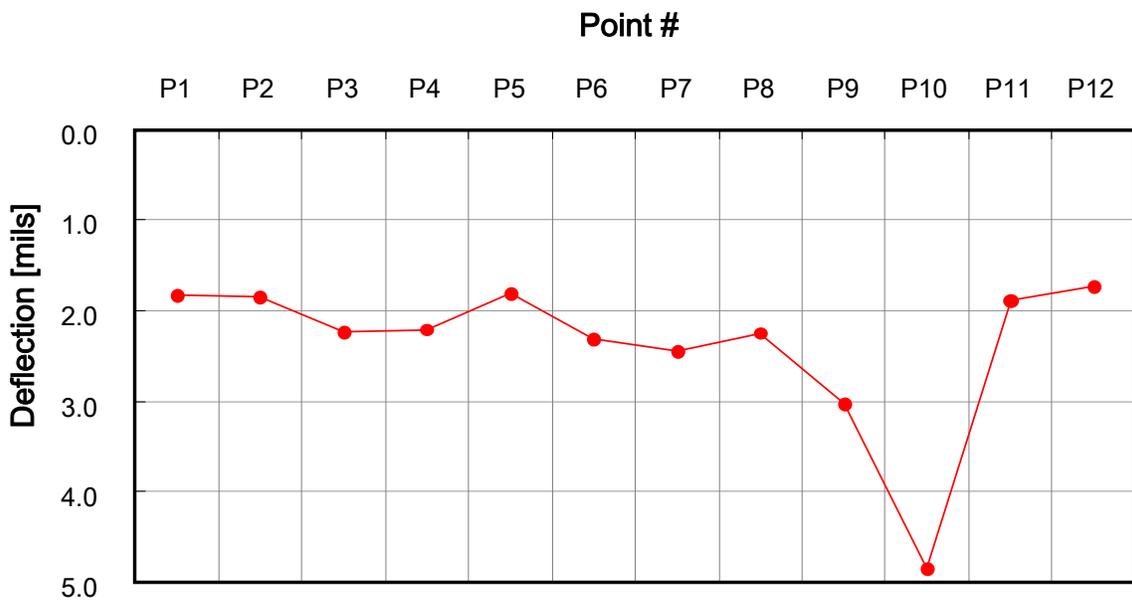
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(2) Dimension and FWD testing points



(3) FWD result

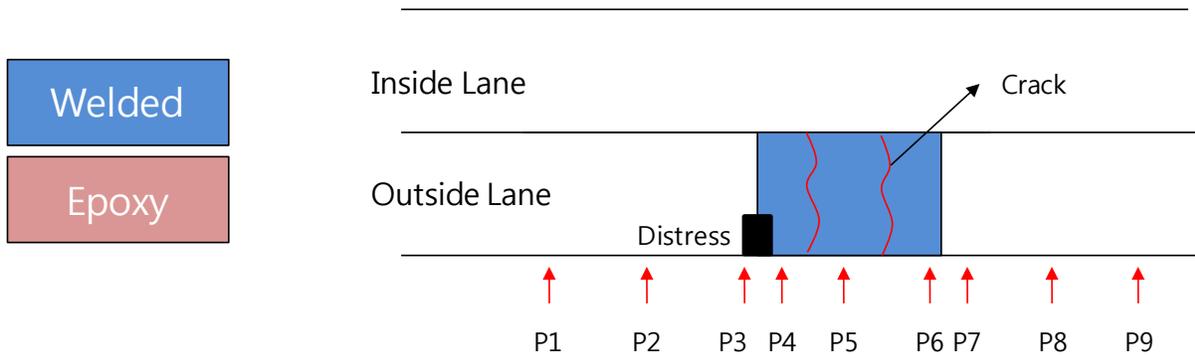


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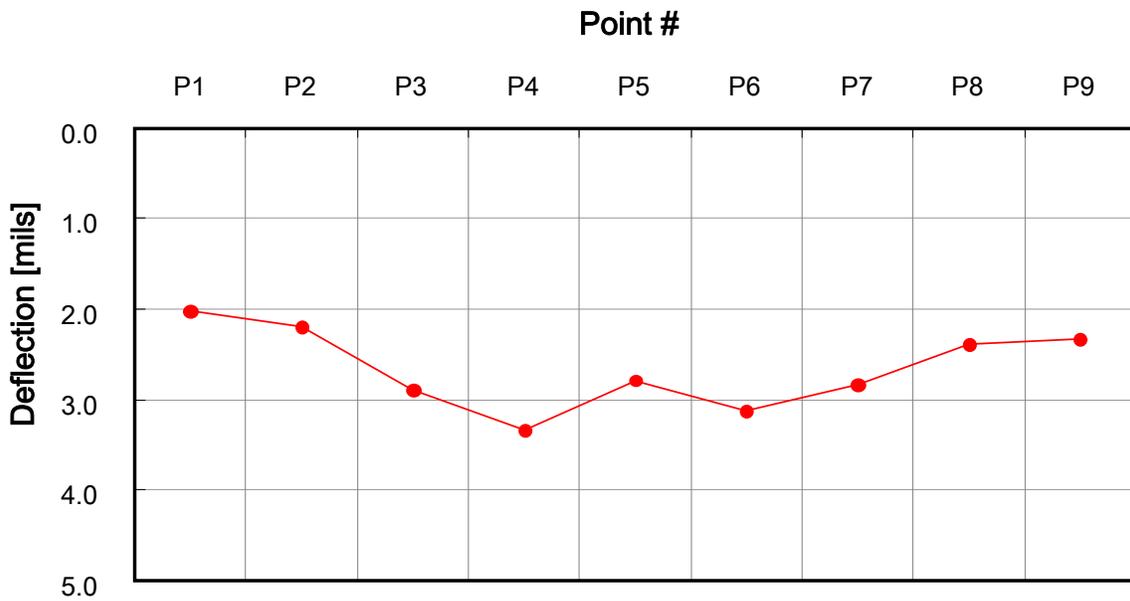
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(2) Dimension and FWD testing points



(3) FWD result

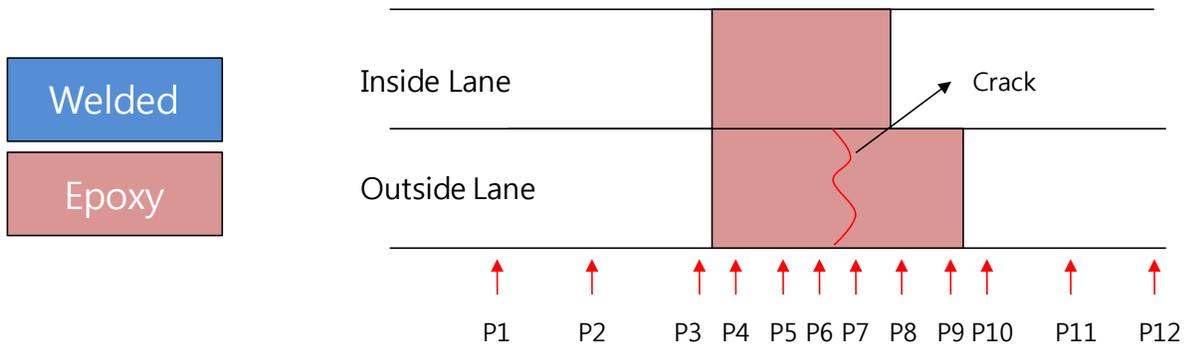


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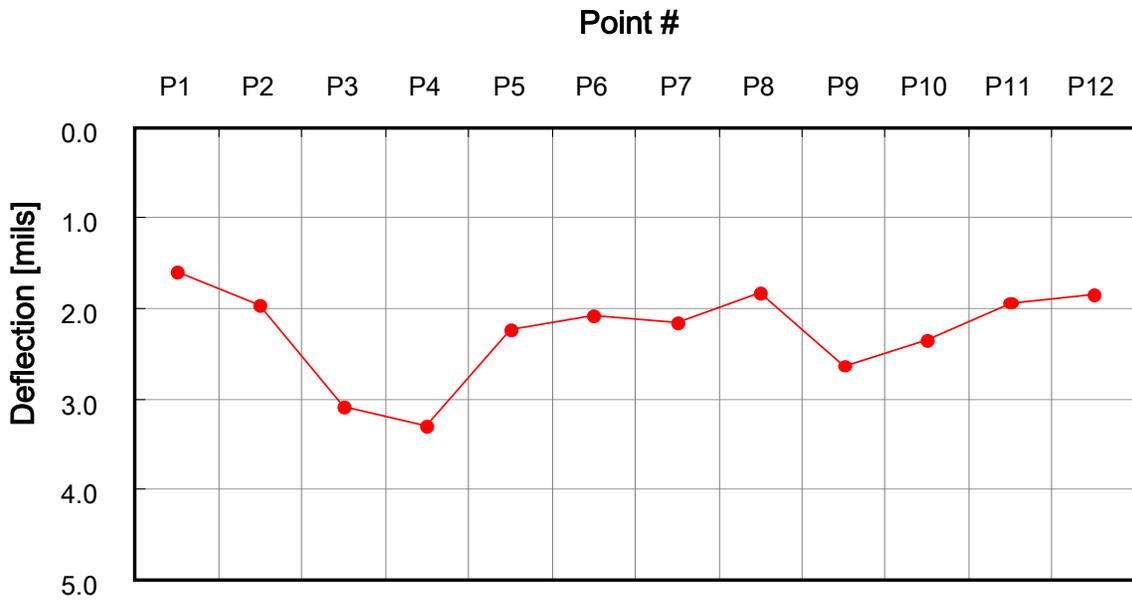
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(2) Dimension and FWD testing points



(3) FWD result



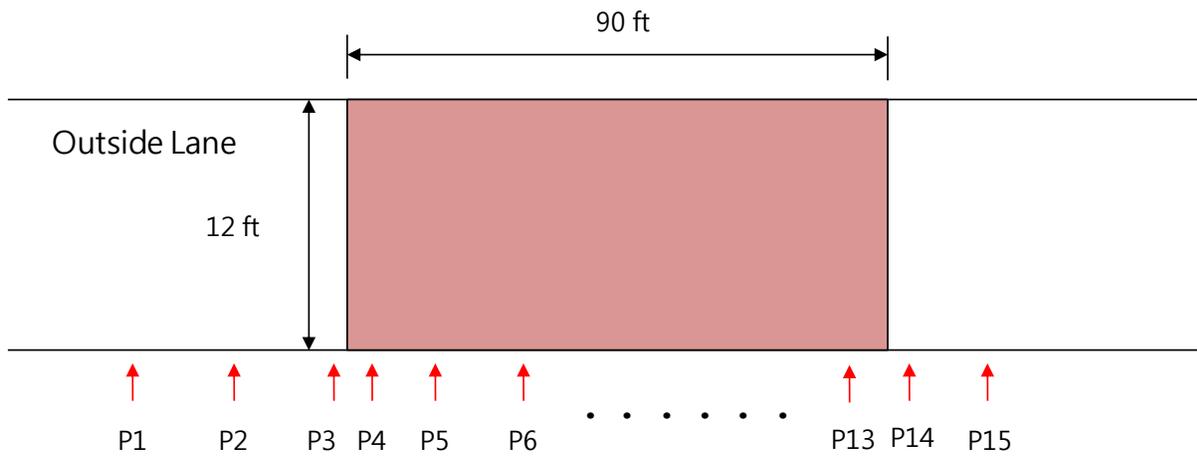
3. Bowie – Wichita Falls District (US 287) – 01/12/2012

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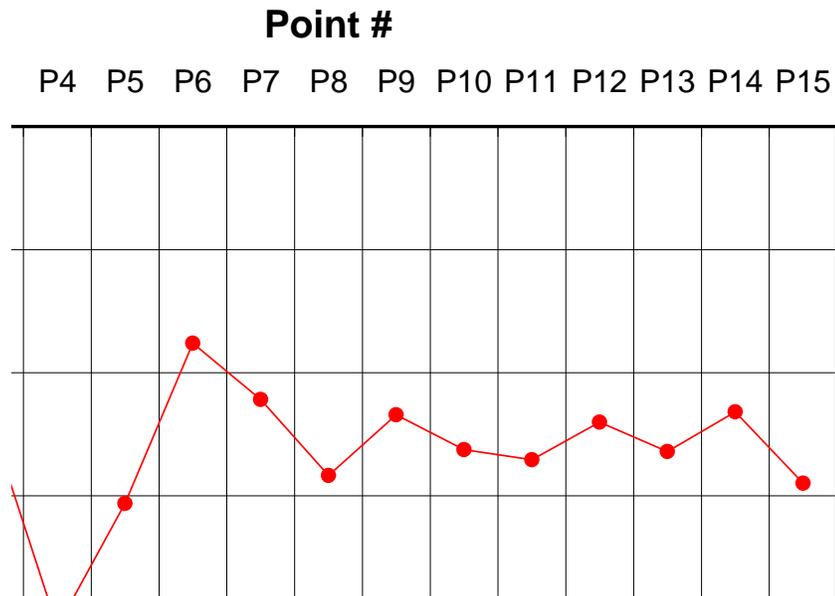
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(2) Dimension and FWD testing points



(3) FWD result



3.2 Section 2 – NB

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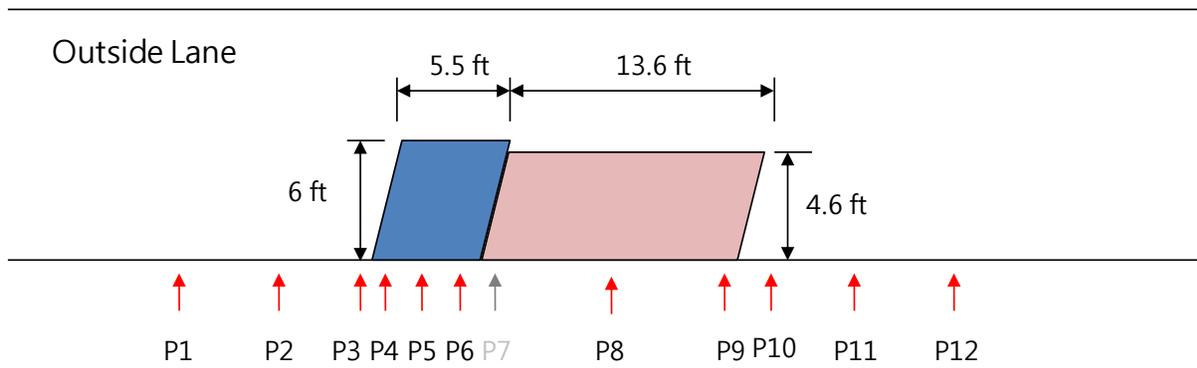


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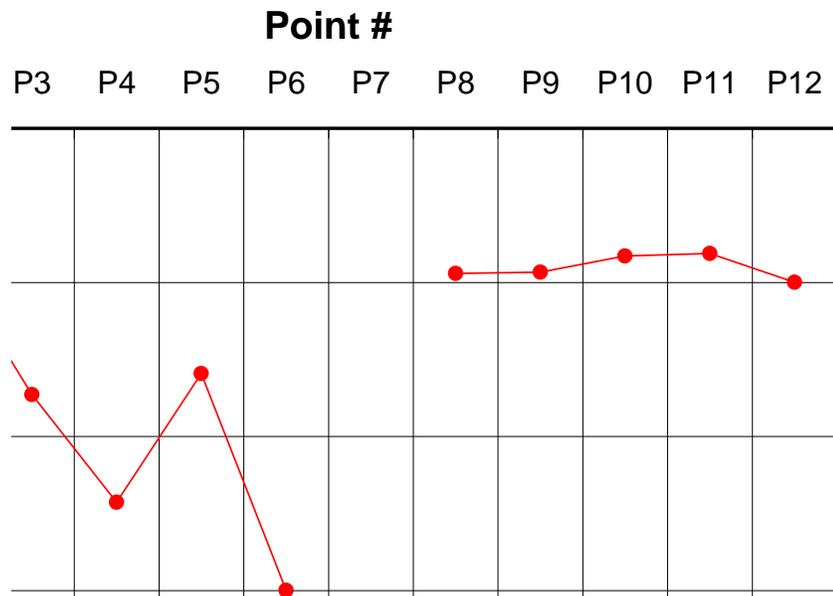
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(2) Dimension and FWD testing points



(3) FWD result

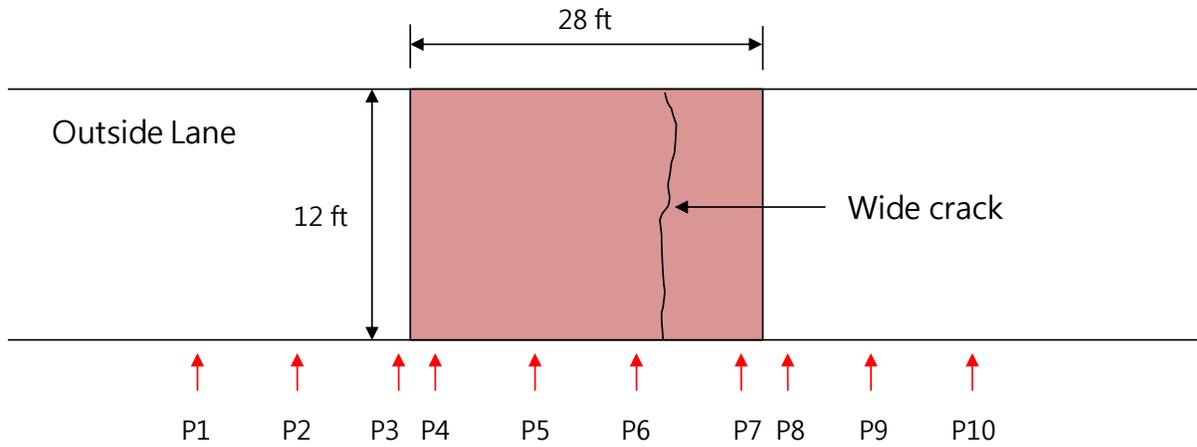


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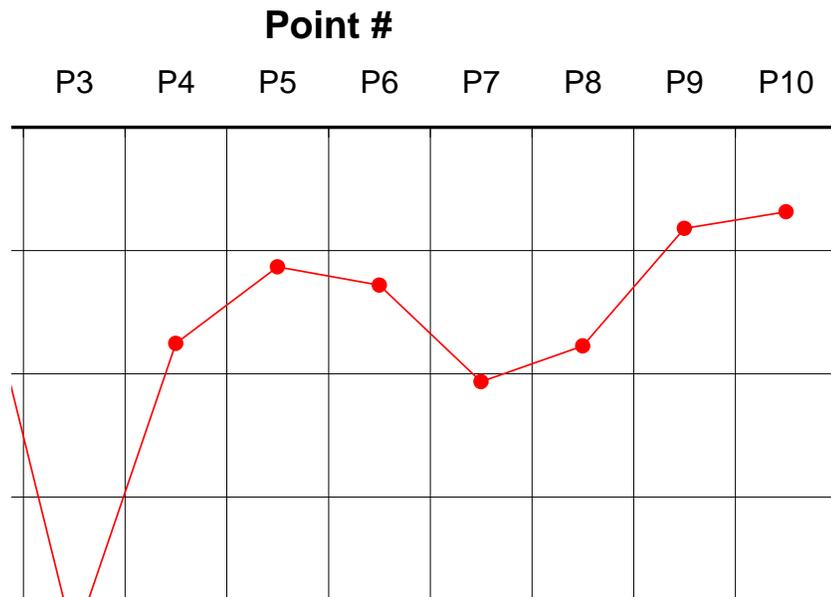
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(2) Dimension and FWD testing points



(3) FWD result

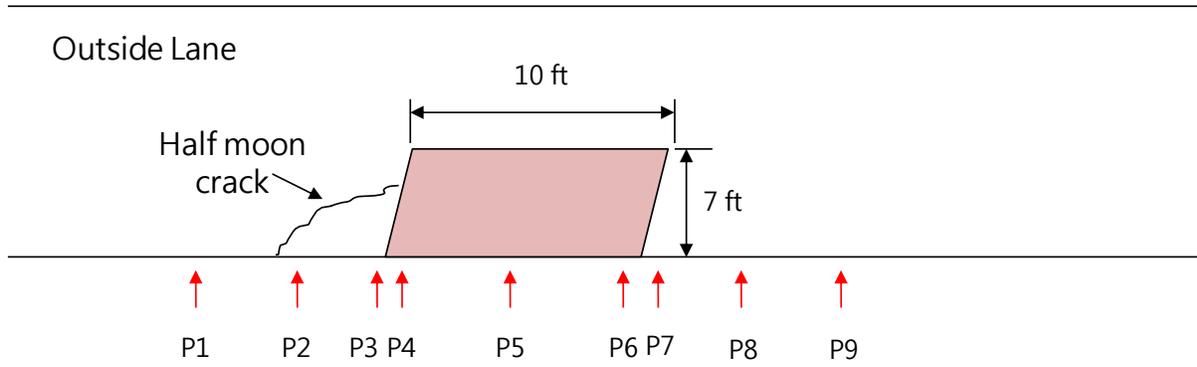


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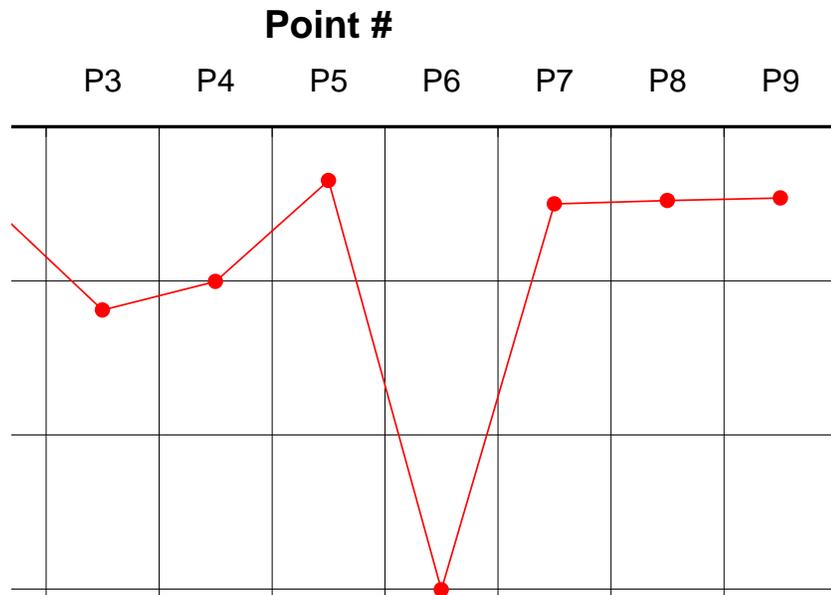
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(2) Dimension and FWD testing points



(3) FWD result

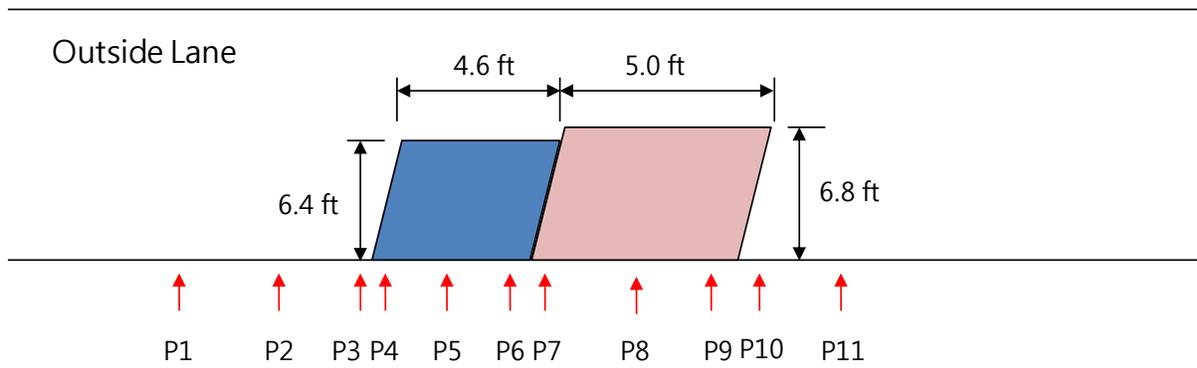


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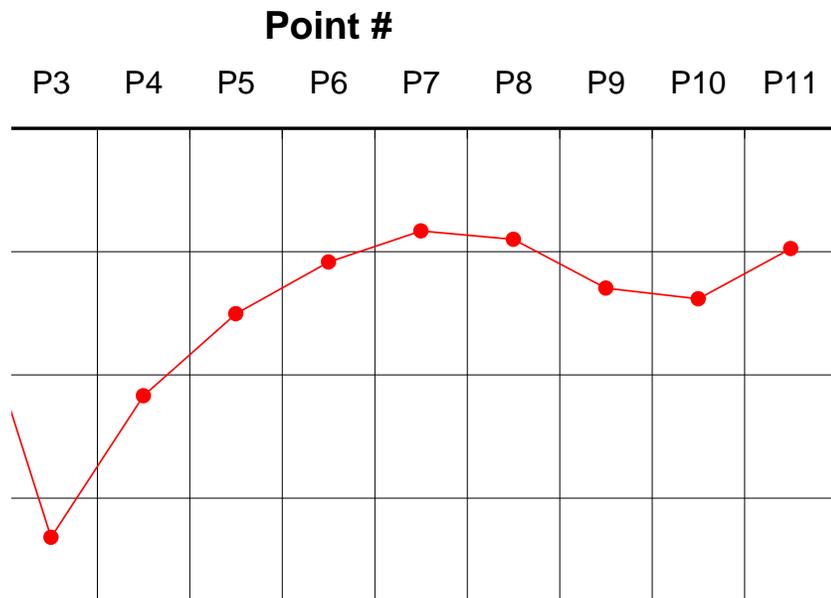
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(2) Dimension and FWD testing points



(3) FWD result

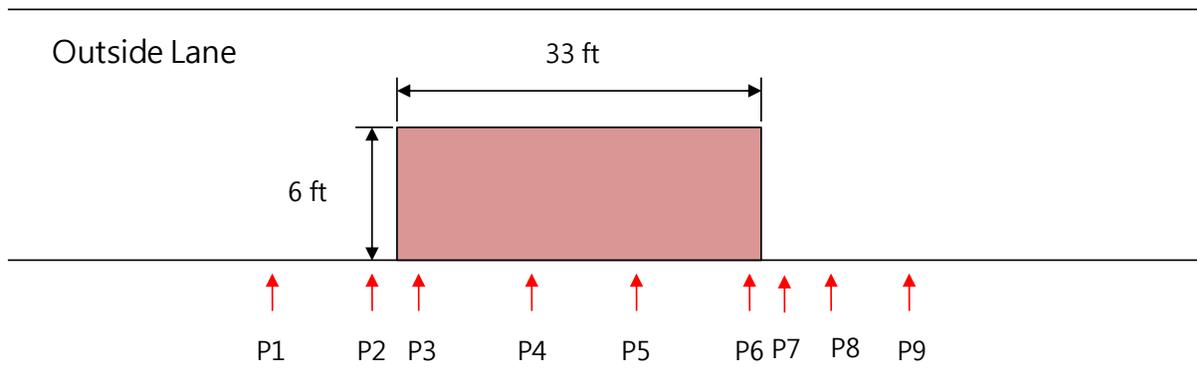


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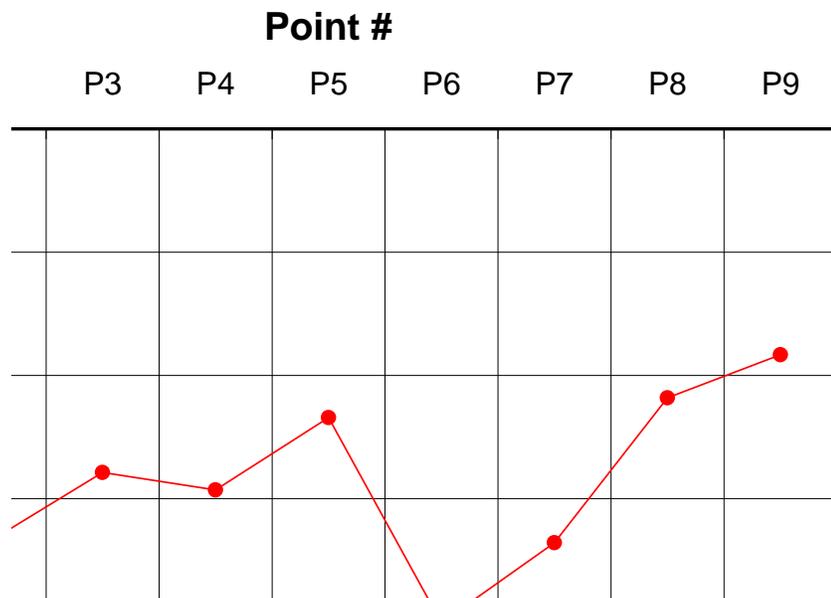
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(2) Dimension and FWD testing points



(3) FWD result

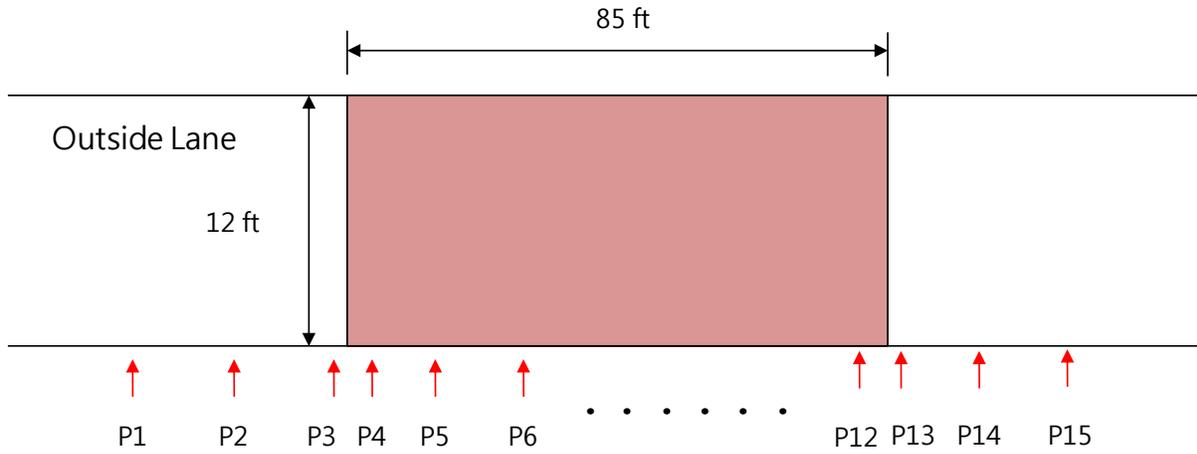


3.8 Section 4 - SB

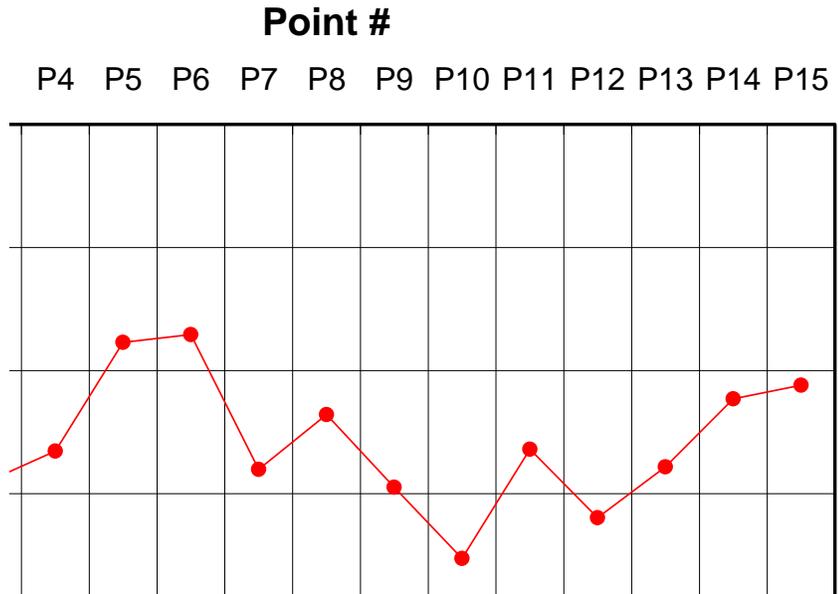
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(2) Dimension and FWD testing points



(3) FWD result

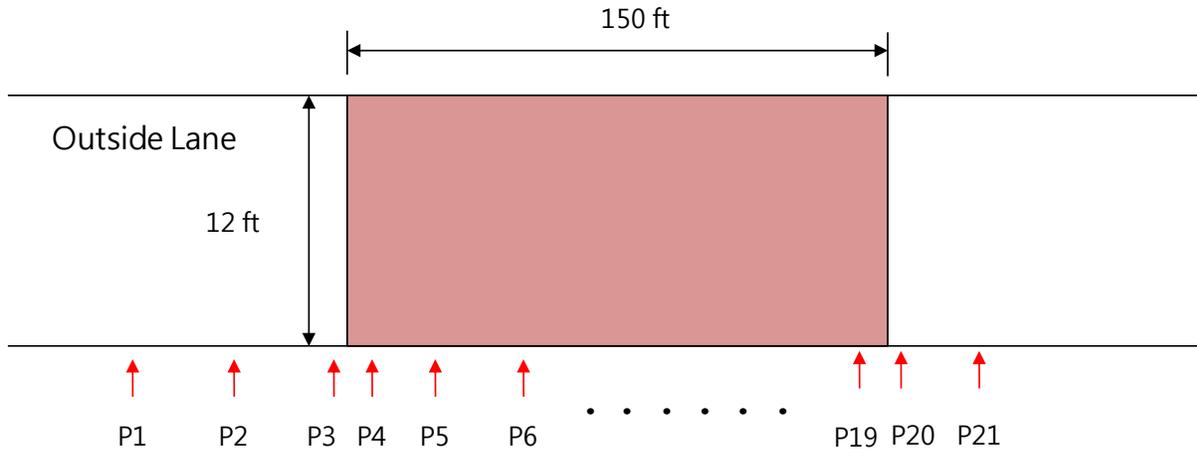


3.9 Section 5 - SB

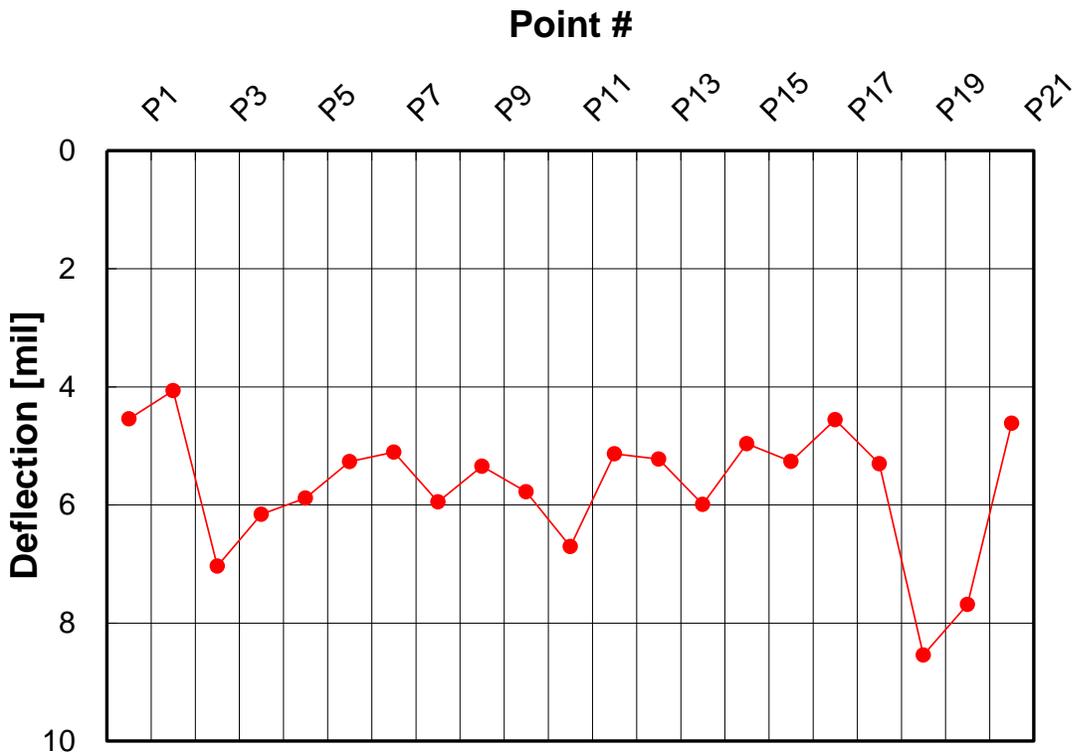
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(2) Dimension and FWD testing points



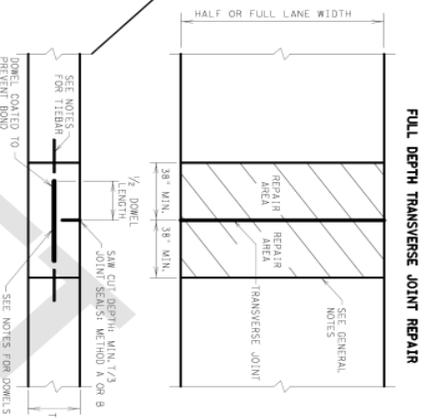
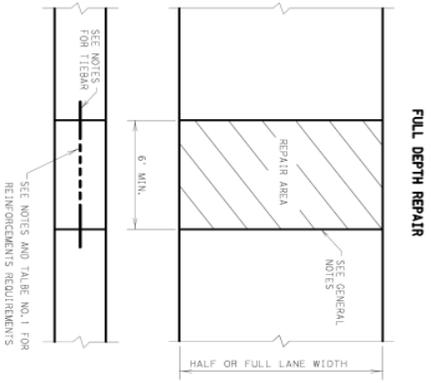
(3) FWD result



Appendix B: Design Standards for FDR

DISCLAIMER: The use of this standard is governed by the "Texas Engineering Practice Act". No warranty of any kind is made by TxDOT for any purpose whatsoever. TxDOT assumes no responsibility for the conversion of this standard to other formats or for incorrect results or damages resulting from its use.

DATE: _____
FILE: _____



TRANSVERSE CONTRACTION JOINT

- GENERAL NOTES:**
- ITEM 367 FULL-DEPTH REPAIR OF CONCRETE PAVEMENT, SHALL BE PERFORMED IN ACCORDANCE WITH THE FOLLOWING SPECIFICATIONS ARE REFERENCED IN ITEM 367 ONLY.
 - * ITEM 366, CONCRETE PAVEMENT
 - * ITEM 368, CLEANING AND SEALING JOINTS AND CRACKS
 - * ITEM 438, CLEANING AND SEALING JOINTS AND CRACKS
 - * ITEM 440, FRIED PAVEMENT AND BRIDGE DECKS
 - * DMS-4650, HYDRAULIC CEMENT CONCRETE CURING MATERIALS AND EVAPORATION RETARDANTS
 - * DMS-6180, SLOW SET AND FAST SET
 - * DMS-8310, CONCRETE SEALERS AND FILERS
 - FULL DEPTH SAW CUTS SHALL BE MADE AROUND THE PERIMETER OF THE AREA TO BE REPAIRED. THE CUT SHALL BE MADE AT A RIGHT ANGLE TO THE PAVEMENT EDGE AND TO THE CENTER LINE OF THE PAVEMENT.
 - DEPTH SAW CUTS SHALL BE AT EXISTING LONGITUDINAL JOINTS.
 - ADDITIONAL SAW CUTS MAY BE REQUIRED WITHIN THE AREA OF THE REPAIR TO PREVENT SPALLS FROM FORMING OUTSIDE THE REPAIR AREA.
 - THE SAW CUTS WHICH EXTEND OUTSIDE THE AREA OF THE REPAIR WILL BE CLEANED AND FILLED WITH A GROUT APPROVED BY THE ENGINEER.
 - EXISTING LONGITUDINAL AND TRANSVERSE JOINTS AFFECTED DUE TO REPAIR OPERATION SHOULD BE RESTORED IN ACCORDANCE WITH STANDARD SHEET "CONCRETE PAVING DETAILS, JOINT SEALS."

- REINFORCEMENTS REQUIREMENTS**
- REINFORCING STEEL SHALL BE #6 DEFORMED STEEL BARS CONFORMING TO ASTM A 615 (GRADE 60) OR ASTM A 996 (GRADE 60).
- R1. THE STEEL SPACING FOR CONTINUOUSLY REINFORCED CONCRETE PAVEMENT (CRCP) AND JOINTED REINFORCED CONCRETE PAVEMENT (LRCP) SHALL BE REINFORCED AS SHOWN IN TABLE NO.1.
- R2. REINFORCING BARS SHALL BE PLACED IN ONE LAYER AND SHALL BE TIED TO THE TIEBARS.
- R3. THE LENGTH OF THE REINFORCING BAR SHALL BE THE LENGTH OR WIDTH OF THE REPAIR AREA MINUS 2 INCHES. THE END OF THE BAR SHALL BE PLACED WITHIN 1 INCH FROM THE REPAIR EDGE.
- TIEBARS FOR REPAIR AREAS**
- T1. TIEBARS SHALL BE PLACED AT APPROXIMATELY THE MID-DEPTH OF SLABS. THE BOTTOM OF THE HOLE DRILLED FOR THE LONGITUDINAL BARS SHALL BE AT MID-DEPTH AND THE TOP OF THE HOLES DRILLED FOR THE TIEBARS SHALL BE AT THE TOP OF THE SLAB. THE DEPTH OF THE HOLES SHALL BE ESTABLISHED BY MEASURING FROM THE TOP OF THE SLAB DOWN. THE THICKNESS OF THE CONCRETE SLAB WILL BE DEFINED BY THE PLANS OR THE EVIDENCE.
- T2. THE BAR SIZE AND SPACING OF TIEBARS ARE SHOWN IN TABLE NO. 1.
- T3. THE MINIMUM LENGTH OF TIEBARS EXTENDED INTO THE REPAIR AREA SHOULD BE 29 INCHES FOR A #6 BAR.
- T4. THE TIEBAR SHALL BE GROUTED INTO THE EXISTING CONCRETE. A MINIMUM OF 2 INCHES BEFORE REPAIR WORK BEGINS ON THE REPAIR AREA. THE TIEBAR SHALL BE PLACED IN ACCORDANCE WITH THE REQUIREMENTS OF PULL-OUT TEST SPECIFIED IN ITEM 367.
- T5. MULTIPLE PIECE TIEBARS SHALL BE USED WHEN THE REPAIR AREA MUST BE PLACED IN TWO SPACES DUE TO SPACING OR CONSTRUCTION.
- DOMELS FOR TRANSVERSE JOINT REPAIRS**
- D1. SMOOTH DOMEL BARS SHALL BE DELIVERED TO THE JOBS SITE IN PRE-FABRICATED DOMEL ASSEMBLIES. THE ENTIRE DOMEL BAR SHALL BE PLACED IN A MATERIAL WHICH WILL PREVENT BONDING TO THE CONCRETE.
- D2. THE SIZE AND SPACING OF DOMEL BARS SHALL BE AS SHOWN IN TABLE NO.2.
- D3. PLACEMENT OF TIEBARS AND OTHER REINFORCING STEEL SHALL BE STAGED APPROXIMATELY 4" FROM THE DOMEL BAR ASSEMBLY.
- D4. DOMEL BAR PLACEMENT SHALL MEET THE REQUIREMENTS OF ITEM 360, "CONCRETE PAVEMENT."

TYPE OF REINFORCEMENTS	TYPE OF PAVEMENT	PAVEMENT THICKNESS (INCHES)	TIEBARS		REGULAR REBARS		ALL BARS TEST & TEST SPACING AT END OR SIDE (INCHES)
			SIZE (BAR NO.)	BAR SPACING (INCHES)	SIZE (BAR NO.)	SPACING (INCHES)	
TRANSVERSE BARS	CRCP	ALL	#6	24	#6	48	12
		5.0	#5	7.5	#5	7.5	12
		6.5	#5	7.0	#5	7.0	12
		7.0	#5	6.5	#5	6.5	12
		7.5	#5	6.0	#5	6.0	12
		8.0	#6	9.0	#6	9.0	12
		8.5	#6	8.5	#6	8.5	12
		9.0	#6	8.0	#6	8.0	12
		9.5	#6	7.5	#6	7.5	12
		10.0	#6	7.0	#6	7.0	12
		10.5	#6	6.75	#6	6.75	12
		11.0	#6	6.5	#6	6.5	12
LONGITUDINAL BARS	CRCP	10.0	#6	7.0	#6	7.0	12
		10.5	#6	6.75	#6	6.75	12
		11.0	#6	6.5	#6	6.5	12
		11.5	#6	6.25	#6	6.25	12
		12.0	#6	6.0	#6	6.0	12
		12.5	#6	5.75	#6	5.75	12
		13.0	#6	5.5	#6	5.5	12
		13.5	#6	5.25	#6	5.25	12
		14.0	#6	5.0	#6	5.0	12
		14.5	#6	4.75	#6	4.75	12
		15.0	#6	4.5	#6	4.5	12
		15.5	#6	4.25	#6	4.25	12

PAVEMENT THICKNESS (INCHES)	SIZE AND DIA. (INCHES)	LENGTH (INCHES)	SPACING (INCHES)
8	1 IN.	18	12
9	1 1/4 IN.	18	12
≥ 10	1 3/4 IN.	18	12


Texas Department of Transportation
Design Division Standard

FULL DEPTH REPAIR FOR CONCRETE PAVEMENT
FDR (CP) - 12

TITLE: FDR(12)cp
 DATE: November 2012
 VERSION:

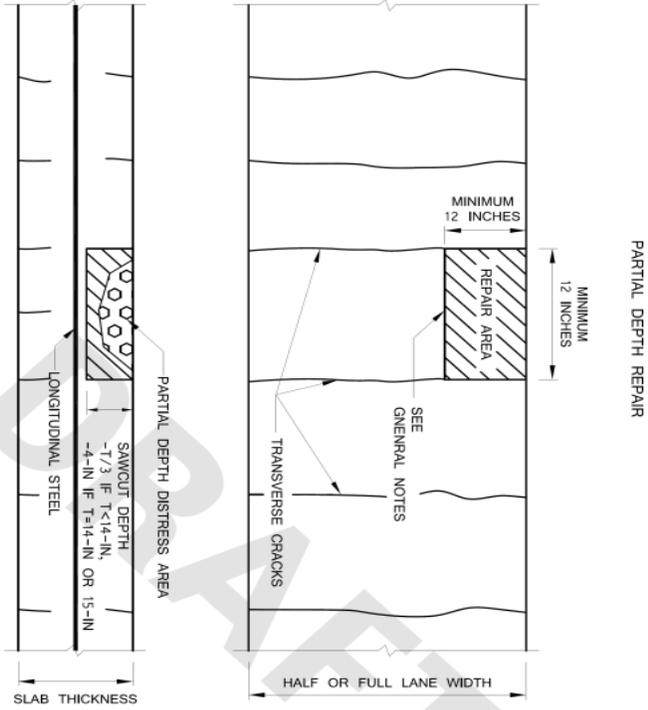
DATE	BY	CHECK	DATE

SHEET NO. _____
 OF _____

Appendix C: Design Standards for PDR

DATE:
FILE:

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GENERAL NOTES

1. MINIMUM SIZE OF PARTIAL DEPTH REPAIR AREA SHOULD BE 12-IN IN BOTH LONGITUDINAL AND TRANSVERSE DIRECTIONS.
2. REPAIR BOUNDARIES SHOULD ENCOMPASS ALL DISTRESSED AREAS. TRANSVERSE REPAIR BOUNDARIES SHOULD BE 2 TO 3 INCHES AWAY FROM ANY DISTRESSED AREA. LONGITUDINAL REPAIR BOUNDARY SHOULD BE 2-IN BEYOND THE EXTENT OF DISTRESS AREA.
3. SAW-CUT AT REPAIR BOUNDARIES SHOULD BE STRAIGHT AND VERTICAL TO PROVIDE A VERTICAL SURFACE AND SQUARE CORNERS.
4. SAW-CUT DEPTH SHOULD BE ONE-THIRD OF SLAB THICKNESS FOR SLABS WITH 13-IN THICK OR LESS, OR 4-IN FOR SLABS WITH 14-IN AND 15-IN SLAB THICKNESS. SAW-CUT OPERATION SHOULD NEVER CUT THE LONGITUDINAL STEEL.



Design Division Standard

PARTIAL DEPTH REPAIR
FOR
CONTINUOUSLY REINFORCED CONCRETE
PAVEMENT (CRCP)
PDR (CRCP) -12

FILE:	DATE:	BY:	CHK:
PROJECT:	NOVEMBER 2012	CONTRACT:	NOVEMBER
NOVEMBER	NOVEMBER	COUNTY:	SHEET NO.

Appendix D: Special Specification for FDR

2004 Specifications

SPECIAL SPECIFICATION 3XXX FULL-DEPTH REPAIR OF CONCRETE PAVEMENT

361.1. Description. Repair concrete pavement to full depth.

361.2. Materials. Provide materials that meet the pertinent requirements of the following:

- Item 360, “Concrete Pavement”
- Item 421, “Hydraulic Cement Concrete”
- Item 438, “Cleaning and Sealing Joints and Cracks”
- Item 440, “Reinforcing Steel”
- DMS 6100, “Epoxies and Adhesives”

A. Hydraulic Cement Concrete for Pavement. If the time frame designated for opening to traffic is less than 72 hr. after concrete placement, provide Class HES concrete designed to attain a minimum average flexural strength of 255 psi or a minimum average compressive strength of 1,800 psi within the designated time frame. Otherwise provide Class P concrete conforming to Item 360, “Concrete Pavement.” Type III cement is permitted for Class HES concrete.

B. Base Material. Unless otherwise shown on the plans or permitted, furnish pavement concrete or cold mix asphalt materials for replacement base material when required. The Engineer may waive quality control tests for base material.

C. Asphalt Concrete. Furnish asphalt concrete material for overlay and asphalt shoulder repair in accordance with Item 340, “Dense-Graded Hot-Mix Asphalt (Method),” as shown on the plans. The Engineer may waive quality control tests for this material.

361.3. Construction. Repair areas identified by the Engineer. Make repair areas rectangular, at least 6 ft. long and at least ½ a full lane in width unless otherwise shown on the plans. Unless otherwise shown on the plans, accept ownership of all removed material, and dispose of it in accordance with federal, state, and local regulations. Saw-cut and remove existing asphalt concrete overlay over the repair area. Saw-cut full depth through the concrete around the perimeter of the repair area before removal. Do not spall or fracture concrete adjacent to the

repair area. Schedule work so that concrete placement follows full-depth saw cutting by no more than seven days unless otherwise shown on the plans or approved.

Remove loose or damaged base material completely, leaving no loose base materials. Compact base materials to the satisfaction of the Engineer. For CPCD repairs, place a polyethylene sheet at least four mils thick as a bond breaker at the interface of the base and new pavement.

For CRCP repairs, drill holes into the existing concrete at least 12 inches deep. Use a drill bit with a diameter that is 1/8 inch greater than that of tie bars. Clean the holes with compressed air to remove all the dust. Insert the tip of the epoxy cartridge or the tip of the machine applicator to the end of the tie bar hole, and inject Type III, Class C epoxy to fill the back $\frac{3}{4}$ of the hole. Insert tie bars. Place new deformed reinforcing steel bars of the same size and spacing as shown on the plans. Lap all reinforcing steel splices in accordance with Item 440, "Reinforcing Steel." Provide and place approved supports to firmly hold the new reinforcing steel and tie bars in place.

For CPCD repairs, drill holes into the existing concrete at least 12 inches deep. Use a drill bit with a diameter that 1/8 inch greater than that of tie bars. Clean the holes with compressed air to remove all the dust. Insert the tip of the epoxy cartridge or the tip of the machine applicator to the end of the tie bar hole, and inject Type III, Class C epoxy to fill the back $\frac{3}{4}$ of the hole. Insert tie bars. Place dowel bars as shown on the plans. Provide and place approved supports to firmly hold the dowel bars in place.

Mix, place, cure, and test concrete to the requirements of Item 360, "Concrete Pavement," and Item 421, "Hydraulic Cement Concrete." Broom-finish the concrete surface unless otherwise shown on the plans. Match the grade and alignment of existing concrete pavement. After concrete strength requirements have been met, replace any asphalt overlay and shoulder material removed with new asphalt concrete material in accordance with Item 340, "Dense-Graded Hot-Mix Asphalt (Method)."

For repair areas to be opened to traffic before 72 hr., use curing mats to maintain a minimum concrete surface temperature of 70°F. Cure repaired area for at least 72 hr. or until overlaid with asphalt concrete, if required, or until the area is opened to traffic. Saw and seal construction joints in the repair area in accordance with Item 360, "Concrete Pavement." Remove repair area debris from the right of way each day.

361.4. Measurement. This Item will be measured by the square yard of concrete surface area repaired. No measurement will be made for areas damaged because of Contractor negligence.

361.5. Payment. The work performed and the materials furnished in accordance with this Item

and measured as specified under “Measurement” will be paid for at the unit price bid for “Full-Depth Repair” of the type and depth specified. This price is full compensation for removal, stockpiling, and disposal of waste material and for equipment, materials, labor, tools, and incidentals. Asphalt concrete, base material, and curbing will not be paid for directly but will be considered subsidiary to this Item.

Appendix E: Special Specification for PDR

2004 Specifications

SPECIAL SPECIFICATION 3XXX Partial Depth Repair of Concrete Pavement

1. **Description.** Repair concrete pavement to partial depth in accordance with the details shown on the plans and the requirements of this Item.
2. **Materials.** Provide materials that meet the pertinent requirements of the following:
 - Item 360, “Concrete Pavement”
 - Item 421, “Hydraulic Cement Concrete”
 - Item 440, “Reinforcing Steel”
 - DMS 6100, “Epoxies and Adhesives.”

If material in Item 421 does not meet the strength requirement, provide material that meets the requirements in DMS-4655, “Rapid-Hardening Cementing Materials for Concrete Repair”

3. **Equipment.** Provide tools and equipment necessary for proper execution of the work that meet the pertinent requirements of the following:
 - Item 360, “Concrete Pavement”
 - Item 429, “Concrete Structure Repair”

In addition, provide following equipment:

- A. Drill. Use a maximum 40 lb. hydraulic drill with tungsten carbide bits.
- B. Air Compressor. Provide compressor capable of delivering air at 120 cu. ft. per minute and with a minimum 90 psi nozzle pressure.

4. **Construction.** Obtain approval for all materials and methods of application at least 2 weeks before beginning any repair work. Repair locations will be indicated on the plans or by the Engineer.

- A. Saw Cut Repair Boundaries.** Saw cut repair boundaries to the depth shown on the plans. Do not saw cut deep enough to cut longitudinal or transverse steel.
 - B. Remove Concrete.** Use jackhammer and other equipment to remove concrete from repair area designated by the Engineer. Start at the center of the repair area. Use caution not to damage the sound concrete during this operation. Make sure that all loose concrete materials are removed.
 - C. Clean Repair Surfaces.** Clean the area to be repaired by abrasive blasting or other approved methods. Remove all loose particles, dirt, deteriorated concrete, or other substances that would impair the bond of the repair material. Follow this with a high-pressure air blast for final cleaning.
 - D. Repair Material Application.** Before the application of the repair materials, keep the repair surface areas damp. Mix, place, cure, and test concrete to the requirements of Item 360, "Concrete Pavement," and Item 421, "Hydraulic Cement Concrete." Broom-finish the concrete surface unless otherwise shown on the plans. Match the grade and alignment of existing concrete pavement. For repair areas to be opened to traffic before 72 hr., use curing mats to maintain a minimum concrete surface temperature of 70°F when air temperature is less than 70°F.
 - E. Repairs.** Repair damages to concrete pavement caused by Contractor's operation without any additional compensation. Perform repairs as directed.
- 5. Measurement.** This Item will be measured by the square foot, in place, as measured on the surface of the completed repair.
- 6. Payment.** The work performed and materials furnished in accordance with this Item and measured as provided under "Measurement" will be paid for at the unit price bid for "Partial Depth Repair of Concrete Pavement." This price shall be full compensation for furnishing all materials, tools, equipment, labor, and incidentals necessary to complete the work. No payment will be made for extra work required to repair damage to the adjacent pavement that occurred during concrete removing or cleaning operations.

Appendix F: Guidelines for FDR

Guidelines for Full Depth Repair in Continuously Reinforced Concrete Pavement

Many miles of CRCP in Texas already reached or exceeded the intended design traffic. At the same time, the base support system utilized in the early CRCP projects in Texas was not durable from today's traffic loading standpoint. There are other design features that should have been utilized such as tied-concrete shoulders that were not incorporated in old CRCP projects. With these deficiencies in the CRCP projects constructed in the 1960s and 1970s, along with ever-increasing truck traffic, full-depth failures take place in old CRCP sections. These distresses should be repaired with full-depth cut (FDC) method. The guidelines provide steps to be followed for full-depth repair (FDR) in CRCP.

1. Evaluate CRCP Distress.

CRCP distresses that require FDR are full-depth failures caused by large deflections and have the following unique features.

1. Distresses are located at the edge of the outside lane with asphalt shoulder.
2. Distresses are not necessarily confined to adjacent two transverse cracks whose spacing is small, as some documents on CRCP describe. In general, distresses take the shape of a half-moon, as shown in Figure 1, where evidence of minor pumping and depression of asphalt material at the pavement edge exist.
3. Near the distressed area, evidence of pumping, either significant or minor, is normally observed, as shown in Figure 2. Punchouts in this area were repaired.



Figure 1 Typical full-depth punchout



Figure 2 Evidence of severe pumping

On the other hand, distresses that occurred near longitudinal joints, either construction or warping joints, where tied-concrete shoulder was used, are probably not full depth failures. Also, if distresses are under wheel path, and two or more transverse cracks are close to each other with quite small spacing, chances are that these distresses are not full depth failures. Figures 3 through 8 illustrate examples of distresses that are not full-depth failures caused by structural deficiencies. These distresses are confined to the top half of the slab and should be restored with partial-depth repair (PDR) method. For the detailed information on PDR, refer to the guidelines in the Appendix # of this report.



Figure 3 – Partial depth distress near longitudinal construction joint



Figure 4 – Partial depth distress under wheel path



Figure 5 – Partial depth distress under wheel path with fractured concrete



Figure 6 – Partial depth distress under wheel path



Figure 7 – Partial depth distress under wheel path



Figure 8 – Partial depth distress under wheel path

2. Determine Repair Boundaries.

The determination of repair boundaries for FDR is quite straightforward. The repair area should include all the distressed areas. When determining the locations of transverse repair boundaries, they should not be located close to existing transverse cracks. At least 2-ft distance is recommended between transverse repair joints and the nearest transverse cracks in the existing CRCP. Figure 9 shows the distress caused by a transverse repair joint too close to a transverse crack in the existing CRCP.



Figure 9 – Repair joint too close to existing transverse crack

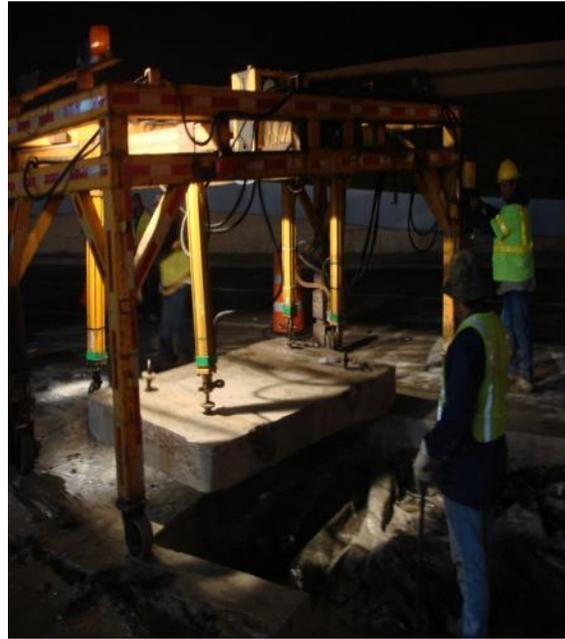


Figure 10 – Removal of slab

3. Saw Cut and Remove Concrete.

Repair boundaries are saw cut full-depth. Once saw cut is completed, the concrete should be removed within 7 days. There are several different ways to remove the concrete slab. Slab removal operations should not cause damage to the base. Figure 10 illustrates one of the most desirable ways of removing the slab. This method causes the least damage to the base during the slab removal.

4. Repair Base.

After the removal of the concrete slab, the condition of the base should be closely inspected for any damage that could have occurred during the concrete slab removal operation or that already existed. It is quite possible that base has been damaged, not due to concrete slab removal operations, but due to cumulative applications of trucks and/or moisture intrusion. FDR performance depends to a great extent on the condition of the base layer prepared during the FDR operations. As a minimum, all the loose materials need to be completely removed and replaced with repair concrete. The rest of the base material should be fully compacted. The practice of placing concrete to the top of the existing base layer to fill the space vacated by the removal of the loose base material is not recommended. The space vacated by the removal of the loose base material should be filled with repair concrete during the concrete slab placement.

5. Drill Holes.

Once the concrete removal is completed, the surfaces of the repair area, both vertical faces and the bottom area, should be thoroughly cleaned. Due to the nature of the distress and concrete removal operations, there will be a large amount of fine materials within the PDD area that were generated by the abrasion actions of concrete pieces under traffic wheel loading as shown in Figure 10. These fine materials must be completely removed to provide a good bonding between the repair materials and existing concrete. Experience shows that good bonding between repair materials and existing concrete surface is essential to the good performance of PDR. If a good bonding is not achieved, the effectiveness of PDR will be limited and further distress will result.

First, apply compressed oil-free air to remove fine materials. Then, apply sandblasting to remove remaining fine materials and fractured pieces on the repair surface. In many cases, reinforcing steel will be exposed as shown in Figure 10. Use sandblasting to remove any cement paste from reinforcing steel. Finally, clean the repair area and exposed reinforcing steel using compressed air.

6. Drill Holes and Epoxy Grout Tie Bars.

Normally, drilling hole operations are conducted concurrently with base repair and preparation. Drill holes to the depth of 12 inches. Use either rotary or hammer drilling method. The spacing between holes should be 2 ft in longitudinal repair joint, and the spacing specified in the FDR Standards. Use a drill bit that is 1/8-in larger than the size of the tie bar to be used. Apply compressed oil-free air in the back of the holes to clean the holes. Inject Type III, Class C epoxy

into the holes. DMS 6100 Type III, Class C is either a cartridge dispensed material or a bulk material for machine application only. When using bulk material, use a pneumatic dispenser or a static mixing nozzle. Never coat tie bars and insert them into the holes. The practice shown in Figure 11 should never be allowed. When epoxy is injected as shown in Figure 11, air inside a hole is compressed as a tie bar is inserted, which will push epoxy towards outside of a hole, creating voids between tie bar and concrete surface in a hole. The void space thus created will reduce bond strength substantially, resulting in a poor FDR performance. Figure 12 shows the condition of “simulated” inside holes when epoxy was injected in different ways. Epoxy was injected in accordance with Item 361 for the two tubes in the left. The space between tie bars and tube walls was completely filled with epoxy. In this case, epoxy was injected to the back end of the tubes, and tie bars were inserted. As tie bars were inserted, epoxy was pushing air, filling the space between tie bars and tube wall. On the other hand, in three tubes on the right, epoxy was injected the same way as shown in Figure 11. In this case, air inside the tubes was compressed as tie bars were inserted and pushed epoxy out of the tube. The voids shown in the three tubes in the right appear to be the primary cause for a poor performance of FDR.



Figure 11 – Incorrect way of injecting epoxy into the holes



Figure 12 – Condition of epoxy inside holes

7. Place Reinforcing Steel.

Once epoxy injection and installations of tie bars are completed, transverse and longitudinal steel is placed. Make sure that steel is placed in the right elevation and stable by the use of adequate number of chairs.

8. Place Concrete.

Once concrete is placed in the repair area, consolidate using internal vibrators. It is important to remove entrapped air and consolidate the concrete as well as possible without over-vibrating, which might cause segregation problems. Also, take a precaution not to hit steel during vibration. If a maturity curve was developed during the mix design stage, there are two options to estimate the maturity of the concrete for the determination of the time for opening to traffic. One is to embed thermo-couple wire in the concrete. In this case, place one end of the thermo-couple in the center portion of PDR and connect the other end to the maturity meter. To minimize the interference with finishing operations, embedding the thermo-couple can be done just after screeding and finishing, and before the application of curing compounds. The other option is to make a cylinder and place a thermo-couple in the center position of the cylinder and attach the other end of the thermo-couple to a maturity meter. Place the cylinder near the repair site.

If a maturity curve was not developed during the mix design stage, make a minimum of 3 4 by 8 cylinders and cure them at the job site.

9. Provide optimum curing.

It is important to provide quality curing for FDR. It is because poor curing will increase plastic and drying shrinkage, which will induce larger bond stress in the epoxy. Large shear strains in the epoxy will reduce the structural continuity at repair joints, a potential cause for a poor FDR performance. In general, concrete used in FDR has high cement content to achieve high early strength, which increases drying shrinkage potential and heat of hydration, and subsequent thermal contraction. To reduce the potential cracking problems due to the use of high cement content, it is important to provide quality curing.

10. Open to traffic.

When the strength of the concrete meets the opening to traffic strength requirement, which is 2,600 psi compressive strength, the FDR section can be open to traffic. Saw and seal repair joints in accordance with Item 360 prior to opening to traffic.

Appendix G: Guidelines for PDR

Guidelines for Partial Depth Repair in Continuously Reinforced Concrete Pavement

Over the years, the Texas Department of Transportation (TxDOT) has improved design and construction practices of CRCP, which has resulted in good overall performance of CRCP in Texas. At the same time, the improved design and construction practices changed the type of distresses in CRCP. Typical punchout distresses that were once prevalent and required full-depth repairs, have become rare; rather, a substantial portion of distresses observed in CRCP built with the improved design and construction practices are partial depth distress (PDD). In PDD, the distress is confined to the top half of the slab, above longitudinal steel. The concrete below the longitudinal steel, or the approximate bottom half of the slab, remains in sound condition.

In CRCP, deflections are maintained relatively small, by proper slab thickness and adequate base support. Deflections at transverse cracks are also maintained small, primarily due to adequate amount of longitudinal steel and good base support. The small deflections in CRCP are responsible for excellent performance of CRCP in Texas. To maintain good performance of CRCP, any repairs of CRCP distresses should restore structural continuities at repair joints to keep the deflections at the level comparable to those in non-distressed areas. In repairing PDD, it is strongly advisable to keep the continuity of the longitudinal steel so that a high level of load transfer is maintained at the transverse repair joints. Full-depth repair (FDR) requires cutting of longitudinal steel at the transverse repair joints and it's difficult to restore the continuity of longitudinal steel and load transfer capability at transverse repair joints. Accordingly, partial-depth repair (PDR) should be used for the repair of PDD, because it will keep the continuity of longitudinal steel, provides a high level of load transfer at transverse repair joints, and is more cost-effective than FDR. This document provides guidelines for the evaluation of CRCP distress to determine whether it is PDD and for the proper repair procedures including repair material selection. This document does not address the repair of CPCD (Concrete Pavement Contraction Design).

1. Evaluate CRCP Distress.

Typically there are two distress types in CRCP. One is spalling and the other punchout. The primary difference between two distress types is that spalling is a functional distress that does not negatively affect structural capacity of CRCP. On the other hand, punchout is a structural distress and if not repaired, it will further deteriorate the pavement condition and ride quality. TxDOT has Specification Item 720 for the repair of spalling. For the full-depth repair of CRCP,

TxDOT has Specification Item 361 and design standards “Full-Depth Repair for Concrete Pavement (FDR (CP)-05).” However, TxDOT does not have specification item or design standards for PDR.

The first step to determine whether PDR is the optimum repair strategy is to evaluate the nature of the distress. More specifically, the depth of the distress has to be determined. At the writing of this document, no non-destructive device is available that is easy to use and that can provide positive and accurate information on the extent of the distress in terms of the depth. Until such devices are developed and become available, sounding testing using a solid steel rod or a ball peen hammer might be the only feasible method to use. Areas yielding a dull sound are most probably areas experiencing PDD and should be considered as a candidate for PDR. This testing method has limitations in that it might not be applicable to thicker slabs.

Fortunately, CRCP distresses that extend to the depth of longitudinal steel and need PDR have unique appearances, and it could be possible to determine whether the distress needs PDR or not based on its appearances.

The unique features of PDD are:

1. Multiple longitudinal cracks with small spacing occurred between two closely spaced transverse cracks,
2. The distress occurred near tied longitudinal construction or longitudinal contraction joint, or under wheel path, and
3. There is no faulting at longitudinal joints.

If one of the features above is missing, the probability of the distress being PDD becomes smaller. Figures 1 through 6 below illustrate typical partial depth distress. Note that the distresses meet all four features described above.



Figure 1 – Partial depth distress near longitudinal construction joint



Figure 2 – Partial depth distress under wheel path

In Figure 1, the distress is confined within two closely spaced transverse cracks. Note that there is no faulting at the longitudinal construction joint. The concrete within the distress fractured into smaller pieces and asphalt patching material was applied. In Figure 2, two longitudinal cracks occurred closely next to each other. Note that the two longitudinal cracks occurred within three closely spaced transverse cracks. No faulting is observed at the longitudinal construction joint.



Figure 3 – Partial depth distress under wheel path with fractured concrete



Figure 4 – Partial depth distress under wheel path

In Figure 3, it is shown that the distress occurred under the wheel path and the surface of the concrete has been fractured into smaller pieces. Note that there is no faulting at the longitudinal construction joint. Figure 4 shows similar traits, with fractured pieces and no faulting at the longitudinal warping joint.



Figure 5 – Partial depth distress under wheel path



Figure 6 – Partial depth distress under wheel path

Figure 5 meets all four features described above. Note whitish material at the second transverse

crack from the right. This material is calcium carbonate and it is observed at PDD quite often. Figure 6 also shows no faulting at longitudinal warping joint and several longitudinal cracks.

TxDOT initiated a research project in September 2010 to further investigate effective repair of concrete pavement, and this document will be modified as the findings from the study become available.

2. Determine Repair Boundaries.

Once the distress has been determined as a PDD, the boundary of the repair area has to be established. Unlike full depth distress, PDD is normally confined longitudinally by transverse cracks. Transversely, the boundary should contain all longitudinal cracks or fractured concrete. Accordingly, about 2 to 3 inches outside of the transverse cracks that confine the distress area can be the limit of PDR in the longitudinal direction. Repair limits for transverse direction has to be beyond the distress area. Taking 1-in diameter cores can provide more definite information on the repair limits in transverse direction. In addition, repair areas shall not be closer than 6-in to any other transverse crack or joint. The minimum size of the repair area should be 1-ft in both longitudinal and transverse directions. The boundaries thus determined are clearly marked for saw cutting operation as shown in Figures 6 and 7. The repair boundaries should be straight for the ease of saw cutting.



Figure 7 – Boundary of PDR straight and clearly marked

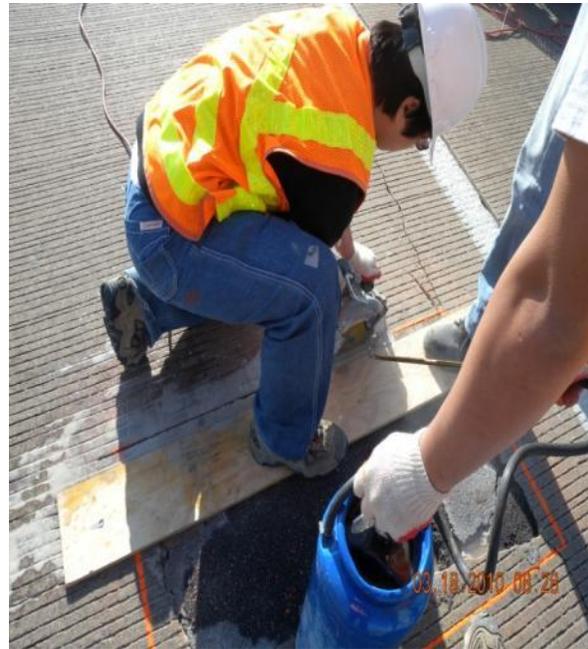


Figure 8 – Saw cut operation

3. Saw Cut and Remove Concrete.

Repair boundaries are saw cut to a maximum one-third of the slab depth if the slab thickness is less than 14 inches. In Texas, longitudinal steel is placed approximately at mid-depth for slabs not greater than 13 inch thick, and the saw cut should never be close to the mid-depth since it increases the potential for cutting longitudinal steel. For slabs 14 to 15 inch thick, the saw cut depth should be a maximum of 4 inches. Saw cut depth should be deep enough to avoid feathered shape in the existing concrete, but not too deep to cut longitudinal steel.

Once saw-cut is completed, remove the concrete using jackhammers with a maximum weight of 37.5 lbs. Start the removal of concrete at the center portion of the repair area and proceed toward the repair boundaries as shown in Figure 9. If the removal of concrete is difficult, additional saw cutting the distressed concrete into smaller pieces can facilitate the removal operation. In addition to using jackhammers, small hammers with spade bits can be useful for removing concrete near repair boundaries. Experience shows that careful removal of the concrete pieces using jackhammers with a maximum weight of 37.5 lbs and small hammers with spade bits won't damage the surrounding concrete. After the removal of the concrete, inspect the repair boundaries to ensure that the repair boundaries include all the distressed concrete. If delaminations are observed beyond the repair boundaries, conduct sound testing to determine new repair boundaries, and saw cut the new boundaries. If the distress is extended below the mid-depth or the longitudinal steel depth, remove the distressed concrete using small jack hammers.

After the removal of the concrete, the condition of the remaining concrete should be closely inspected for any damages that could have occurred during the concrete removal operation. Any damaged concrete should be removed with care not to cause further damage to existing concrete.



Figure 9 – Removing concrete using a jackhammer



Figure 10 – Large amount of fine materials at the bottom of the PDD

4. Hook-bar Installation

If each length of the repair boundaries is equal to or greater than 3 ft, install hook bars in 1 ft spacing in both longitudinal and transverse directions. The hook bars shall be #5 bars with the length equal to a half slab thickness, meeting the requirements of Item 440, “Reinforcing Steel.” Drill vertical holes at 2-in depth for hook bars. Clean the holes with oil-free compressed air. Inject Type III, Class A or Class C epoxy material to the holes and insert hook bars. The amount of the epoxy should be just enough to fill the holes after hook bars are inserted, but not too excessive.

5. Clean Repair Area.

Once the concrete removal is completed, the surfaces of the repair area, both vertical faces and the bottom area, should be thoroughly cleaned. Due to the nature of the distress and concrete removal operations, there will be a large amount of fine materials within the PDD area that were generated by the abrasion actions of concrete pieces under traffic wheel loading as shown in Figure 10. These fine materials must be completely removed to provide a good bonding between the repair materials and existing concrete. Experience shows that good bonding between repair materials and existing concrete surface is essential to the good performance of PDR. If a good bonding is not achieved, the effectiveness of PDR will be limited and further distress will result.

First, apply compressed oil-free air to remove fine materials. Then, apply sandblasting to remove

remaining fine materials and fractured pieces on the repair surface. In many cases, reinforcing steel will be exposed as shown in Figure 10. Use sandblasting to remove any cement paste from reinforcing steel. Finally, clean the repair area and exposed reinforcing steel using compressed air.

6. Prepare Repair Materials.

The selection of repair materials should be based on (1) how soon the repaired section should be open to traffic and (2) ambient temperature condition during repairs. If the ambient temperature is low, such as lower than 60 °F, PDR should be postponed until ambient temperature becomes more favorable (higher than 60 °F). If PDR must be done when the ambient temperature is much lower than 60 °F, temporary patching with asphalt materials should be considered.

If the repaired area should be open to traffic as soon as possible, for example within 3 to 4 hours after repair material placement, using proprietary materials with the capability of achieving high early strength should be considered. There are several different proprietary material types available. They include cementitious and polymeric materials. In general, polymer-based materials are more expensive than proprietary or general cementitious materials, even though polymer-based materials provide higher early strength than cementitious materials. Also, polymer-based materials have a larger coefficient of thermal expansion (CoTE) than Portland cement concrete (PCC) materials, which is not desirable in achieving good bond between repair materials and existing concrete. There are a number of different types of proprietary cementitious materials available for PDR; contact Rigid Pavement and Concrete Materials Branch of CSTMP for the selection of optimum repair materials. The volume change potential of PDR materials due to temperature variations has substantial effects on PDR performance. The use of coarse aggregate that provides lower CoTE and modulus of elasticity will be ideal for PDR. If at all possible, crushed limestone should be used, as it will give low CoTE and modulus of elasticity regardless of the coarse aggregate type used in the existing pavement. Compatibility in material properties between repair materials and existing concrete should not be a concern.

Once the material is selected, a mix design has to be developed. For Class HES concrete, refer to Item 421 for the mix design development. For other proprietary materials, consult the manufacturer's instructions. During the trial mix design, a maturity curve can be developed in accordance with Test Method Tex-426-A, "Estimating Concrete Strength by the Maturity Method." The maturity curve can be used to determine when PDR is ready to open to traffic without performing strength testing.

The frequency of PDD is usually sporadic and, compared with full-depth distress, the size of PDD is small. Accordingly, the quantity of the repair materials needed for PDR is relatively small. Ready mix trucks or other large mixing equipment cannot mix small quantity materials efficiently. Also, the setting time of the repair materials is shorter than normal PCC materials. Accordingly, it will be more efficient if the materials are mixed at the job site in a small drum or paddle-type mixer. This will require the preparation of the components of repair materials in advance in separate containers, bringing and mixing them at the job site. Figure 11 shows the mixing of the previously prepared repair material components at the job site in a small mixer.

7. Place Repair Material.

Before placing repair materials, wet the surfaces of the repair area without leaving excess water at the bottom of the repair area. Start placing repair materials in the middle of the repair areas and pushing toward the edges. Place repair materials in the repair area a little bit higher than the pavement surface to allow for the reduction of volume during consolidation. Consolidate the repair materials using an internal vibrator. It is important to remove entrapped air and consolidate the repair materials as well as possible without over-vibrating, which might cause segregation problems.

If a maturity curve was developed during the mix design stage, there are two options to estimate the maturity of repair materials for the determination of the time for opening to traffic. One is to embed thermo-couple wire in the repair material. In this case, place one end of the thermo-couple in the center portion of PDR and connect the other end to the maturity meter. To minimize the interference with finishing operations, embedding the thermo-couple can be done just after screeding and finishing, and before the application of curing compounds. The other option is to make a cylinder and place a thermo-couple in the center position of the cylinder and attach the other end of the thermo-couple to a maturity meter. Place the cylinder near the repair site.

If a maturity curve was not developed during the mix design stage, make a minimum of 6 4 by 8 cylinders and cure them at the job site.

The surface area of PDR is relatively small, and the usual requirements for surface finish in normal concrete pavement are not applicable. The use of 2 by 4 is appropriate for screeding and finishing. Start at the center of the repair and move toward the boundaries of the repair to enhance the bonding between repair materials and the repair surface of the existing concrete.



Figure 11 – Mixing of prepared repair materials at the job site



Figure 12 – Application of sufficient curing compound

8. Provide optimum curing.

Repair materials used for PDR have relatively lower water-cement ratio than normal PCC materials. Keeping the water in the repair materials is of utmost important by providing optimum curing. Non-optimum curing practice in PDR will result in poor performance of PDR. It is because (1) PDR has a larger surface to volume ratio than normal PCC pavement and thus is prone to moisture loss resulting in shrinkage, (2) volume changes due to moisture loss at the early ages will degrade the bond between repair materials and the repair surface of existing pavement and (3) water-cement ratio of repair materials is low and any loss will reduce the potential for full hydration of cementitious materials. Optimum curing is achieved by the application of curing compounds as soon as tining operation is completed. Since the PDR is a small area, tining may not be needed, or if tining is provided, it can be done as soon as the surface of the repair area is screeded and finished. Since there is almost no bleeding in the repair materials, there is no need to wait until bleeding stops. Provide curing compounds in a much greater amount than required in Item 360 and apply uniformly. Figure 12 shows the surface of the repair area after sufficient curing compounds were applied. Once the curing compounds are dried and not sticky anymore, place a blanket that will completely cover the repair area and apply water to keep the blanket wet continuously until the PDR area is open to traffic.

9. Open to traffic.

When the strength of the repair material meets the opening to traffic strength requirement, which is 2,600 psi compressive strength, the PDR section can be open to traffic. If the maturity curve was developed during the mix design stage and thermo-couple was placed in the PDR or in a cylinder, read the maturity readings to estimate the compressive strength of in-situ repair material. When the maturity reading indicates the in-situ strength achieved is greater than 2,600 psi, PDR is ready to open to traffic.

If a maturity curve was not developed, break the cylinders at appropriate intervals to estimate the in-situ strength of the repair material. When the estimated strength is greater than 2,600 psi, PDR is ready to open to traffic.

When the estimated strength of repair material is greater than 2,600 psi, remove the wet-mat and the section is ready for opening to traffic.



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