Technical Report Documentation Page

1. Report No. FHWA/TX-05/0-4517-2	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle USING ROLLING DEFLECTOMETER AND GROUND PENETRATING RADAR TECHNOLOGIES FOR FULL COVERAGE TESTING OF JOINTED CONCRETE PAVEMENTS		5. Report Date July 2005 Published: April 2006
		6. Performing Organization Code
7. Author(s) Tom Scullion		8. Performing Organization Report No. Report 0-4517-2
9. Performing Organization Name and Address Texas Transportation Institute The Texas A&M University System College Station, Texas 77843-3135		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. Project 0-4517
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P. O. Box 5080 Austin, Texas 78763-5080		13. Type of Report and Period Covered Technical Report: September 2003-August 2004
		14. Sponsoring Agency Code

15. Supplementary Notes

Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration.

Project Title: Develop Statewide Recommendations for Application of PCC Joint Reflective Cracking Rehabilitation Strategies Considering Lufkin District Experience

URL:http://tti.tamu.edu/documents/0-4517.2/pdf

16. Abstract

Selecting the optimal rehabilitation strategy for old Jointed Concrete Pavements (JCP) continues to be a daunting challenge for TxDOT engineers. The variability of joint load transfer efficiency and slab support along projects are major issues contributing to both strategy selection and performance. The presence of water trapped beneath slabs can also be a major limiting factor in strategy selection.

In this project two new technologies were investigated which show tremendous potential for providing 100 percent coverage of the JCP sections. The Rolling Dynamic Deflectometer (RDD) was evaluated on a number of rehabilitation studies and in some control tests. The second technology is Ground Penetrating Radar which shows great promise to identify areas of trapped moisture beneath slabs.

The strengths and weaknesses of these devices are described in this report. The overall conclusion is that these technologies are ideal for testing jointed concrete pavements. More development work is recommended, and future versions of the RDD should incorporate both GPR and video logging capabilities.

17. Key Words	18. Distribution Statement
Jointed Concrete Pavement, JCP, Crack Retarding Layers, Reflection Cracking, Rubblization, Flexible Base Overlays	No restrictions. This document is available to the public through NTIS: National Technical Information Service Springfield, Virginia 22161 http://www.ntis.gov

19. Security Classif.(of this report)	20. Security Classif.(of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	40	

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USING ROLLING DEFLECTOMETER AND GROUND PENETRATING RADAR TECHNOLOGIES FOR FULL COVERAGE TESTING OF JOINTED CONCRETE PAVEMENTS

by

Tom Scullion, P.E.
Research Engineer
Texas Transportation Institute

Report 0-4517-2
Project Number 0-4517
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July 2005
Published: April 2006

TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

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There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new useful improvement thereof, or any variety of plant, which is or may be patentable under the patent laws of the United States of America or any foreign country.

ACKNOWLEDGMENTS

This project was made possible by funding from the Texas Department of Transportation in cooperation with the Federal Highway Administration. The author thanks the many personnel who contributed to the coordination and accomplishment of the work presented herein. Special thanks are extended to Dr. Dar-Hao Chen, P.E., and Mr. Charles Gaskin, P.E., for serving as the project director and project coordinator, respectively. This project is funded by Research Management Committee 1 (RMC1) in TxDOT's research program. The assistance and direction of the project steering committee is acknowledged. Mr. Jeffrey Lee from the Center for Transportation Research in Austin collected the Rolling Dynamic Deflectometer (RDD) data presented in this report. Many TxDOT district personnel made valuable contributions to this effort; the support from Ralph Self, P.E., of the Wichita Falls Districts and Sara Moreno, P.E., of the Houston District is acknowledged. Mr. Chris Von Holdt processed the RDD from the replicate runs at Texas Transportation Institute (TTI's) Riverside Campus.

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CHAPTER 1
INTRODUCTION

TTI Research Project 0-4517 was established to summarize the results from the Lufkin experiment on US 59 and to develop statewide guidelines on how to select rehabilitation options for jointed concrete pavements (JCP). The objects of Project 0-4517 were well summarized in the project statement, and an extract is presented below:

"Reflective cracking continues to be a major problem in the rehabilitation of jointed concrete pavements. A study is proposed to summarize the performance of the results obtained on the Lufkin experiment and to determine how applicable these results are statewide. The proposed investigation will focus on why a particular approach worked well and others did not and to identify the lessons that can be learned for use on future projects. This will involve post-mortem studies on the Lufkin project, together with evaluations of similar type treatments in different areas of the state. The objective is to develop statewide methods for rehabilitating jointed concrete pavements (JCP) to avoid joint reflective cracking."

The findings from the Lufkin projects and a survey of other strategies TxDOT have recently used to minimize reflection cracking were summarized in Report 0-4517-1, "Performance Report on Jointed Concrete Pavement Repair Strategies in Texas" (Scullion and Von Holdt, 2004). Based on the findings of the year 1 report, researchers proposed that in year 2 recommendations will be prepared on how to perform structural evaluations on Jointed Concrete Pavements in order to select the optimal pavement rehabilitation procedures. TxDOT has a range of unique nondestructive testing (NDT) equipment including the Rolling Dynamic Deflectometer (RDD) and Ground Penetrating Radar (GPR) which provide 100 percent coverage of candidate projects. In addition to these devices, TxDOT has a fleet of Falling Weight Deflectometers and Dynamic Cone Penetrometers which can provide substantial information about both joint and sub-slab conditions.

In selecting the optimal repair strategy for JCP's, the District Pavement Engineer typically has to select between six general types of strategies, namely:

- full depth repair and milling for smoothness or skid,
- overlays,
- slab fracturing techniques,
- granular base overlays,
- bonded concrete overlays,
- full reconstruction.

Clearly, the decision on which strategy to use involves many factors in addition to structural condition; for example, traffic level, surface condition, and long-range district plans. However, structural evaluations remain a critical element in the decision-making process. In the following sections of this report, a discussion will be given on applications of using both the RDD and GPR technologies to evaluate Jointed Concrete Pavements. The results presented are from actual TxDOT rehabilitation projects. The strengths and weakness of each device will be discussed as well as future directions. Recommendations are given on how TxDOT personnel should evaluate future candidate projects with both of these new technologies and also with existing nondestructive testing equipment.

CHAPTER 2

THE ROLLING DYNAMIC DEFLECTOMETER

2.1 BACKGROUND

The Rolling Dynamic Deflectometer, shown in Figure 1, was developed in the 1990s at the Center for Transportation Research in Austin under the direction of Dr.'s Ken Stokoe and Jim Bay (Bay and Stokoe, 1998). The RDD places a cyclic load on the pavement as it rolls along at 1.5 mph; for pavement testing, the load is usually fixed at 10,000 lb with a frequency of 30 Hz. One innovative feature of the RDD is the four rolling geophones, as shown in Figure 2, which continuously measure the movement of the pavement surface at different offsets from the load wheels.

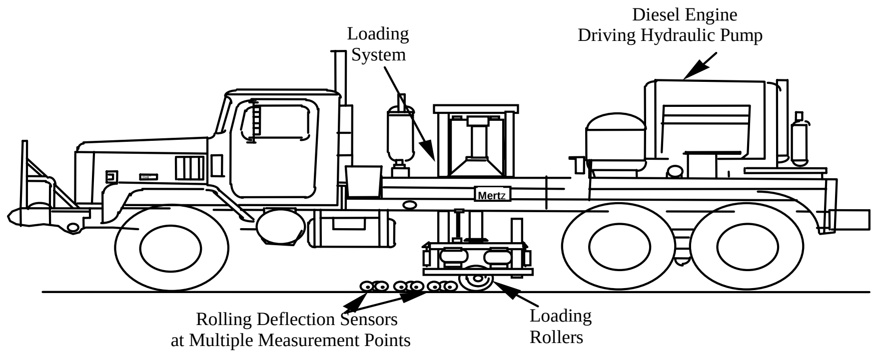


Figure 1. TxDOT's Rolling Dynamic Deflectometer (Lee et al., 2004).

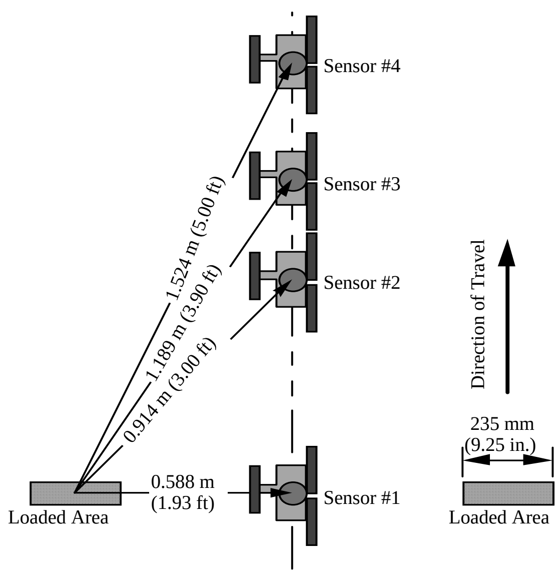


Figure 2. RDD Loading and Sensor Locations (Lee et al., 2004).

The RDD is the only known operational rolling deflection system which provides sufficient data to make project level decisions on Jointed Concrete Pavements. The current data acquisition system collects continuous pavement deflections at a frequency of 30 Hz. The operator typically summarizes the data into a 2-second window and calculates an average pavement deflection for that time interval. Under normal operating speed, this corresponds to an average deflection measurement for every 2 to 3 feet of pavement. The data are supplied for analysis in a spreadsheet form, and a typical data set (for three channels) with the distance offsets is shown in Table 1.

Table 1. Raw RDD Data.

Distance (ft)			Deflection (mils / 10-Kips)		
Sensor #1	Sensor #3	Sensor #4	Sensor #1	Sensor #3	Sensor #4
0	2.9037	4.4067	1.76498	1.39031	1.10129
2.2775	5.4545	6.9575	1.92297	1.35933	1.13472
5.3749	8.5519	10.0549	2.16399	2.01865	0.594924
9.5655	12.7425	14.2455	1.94617	1.80177	1.12401
12.0252	15.2022	16.7052	1.74404	1.87227	1.52156
14.6671	17.8441	19.3471	1.6259	1.78634	1.36034
17.4912	20.6682	22.1712	1.9699	1.8906	1.18904
21.4085	24.5855	26.0885	1.81311	1.27399	0.844339
24.2326	27.4096	28.9126	2.07403	1.57865	1.03076
27.4211	30.5981	32.1011	1.86796	1.85789	1.21181

The operator also provides a log of distances and markers along the roadway as shown in Table 2. This will permit the engineer to locate areas of interest in the field.

Table 2. Event Log Produced by RDD Operators.

File	Start	End	Length (ft)
TC4	Sign: **200 ft South of Milepost 9**	Sign: **End Road Work**	199.6
TC6	Sign: **End Road Work**	Sign: **EXIT 7 Off-Ramp**	4114
TC7	Sign: **EXIT 7 Off-Ramp**	Sign: **EXIT 6 Off-Ramp**	6450.8
TC8	Sign: **EXIT 6 Off-Ramp**	Sign: **EXIT 5 Frontage Road 1/4 mile**	4469.2

The current RDD software also generates a strip map showing the location along the highway where the rolling deflection survey was conducted. More details on the RDD operation can be found in a recent paper by Lee et al., (2004).

Figure 3. Map Showing Where RDD Data Were Collected.

2.2 INTERPRETING RDD DATA FROM JOINTED CONCRETE PAVEMENTS

The RDD is ideal for testing jointed concrete pavements where it is important to assess both sub-slab support and load transfer efficiency. The RDD data are best displayed in graphical form. Although the RDD has the capabilities of monitoring four rolling sensors, in all studies described in this report only three sensors–1, 3 and 4– were used. The data from a short section of jointed concrete pavement from IH 45 in Houston is shown in Figure 4. The blue line is the deflections measured between the load wheels, and the red and yellow lines are deflections measured at offsets of 38 and 56 inches from the center of the load wheels (sensors 1, 3, and 4 from Figure 2). The large periodic increases in the blue line are the deflections measured as the load wheels pass over a joint. Researchers propose that the difference between the maximum deflection over the joint and the deflection measured at sensor 3 is an indication of the load transfer efficiency of the joint. For example, the difference in deflection at a distance of 9840 feet is approximately 8 mils (14 mils – 6 mils). The magnitude of this value is important if an asphalt overlay is being proposed for the concrete pavement. Clearly, if the load transfer efficiencies are consistently poor (higher values), then an overlay may not be the best repair option for the highway. Later in this report, criteria will be proposed as to what constitutes good and poor load transfer efficiency.

In Figure 4, the magnitude of the sensor 1 deflection between the joints is the center slab deflections, and this is an indication of the quality of the subgrade support. This would be important data if slab fracturing techniques are being considered for this highway. For the section shown in Figure 4, the center slab deflections are consistently in the range of 5 to 6 mils.

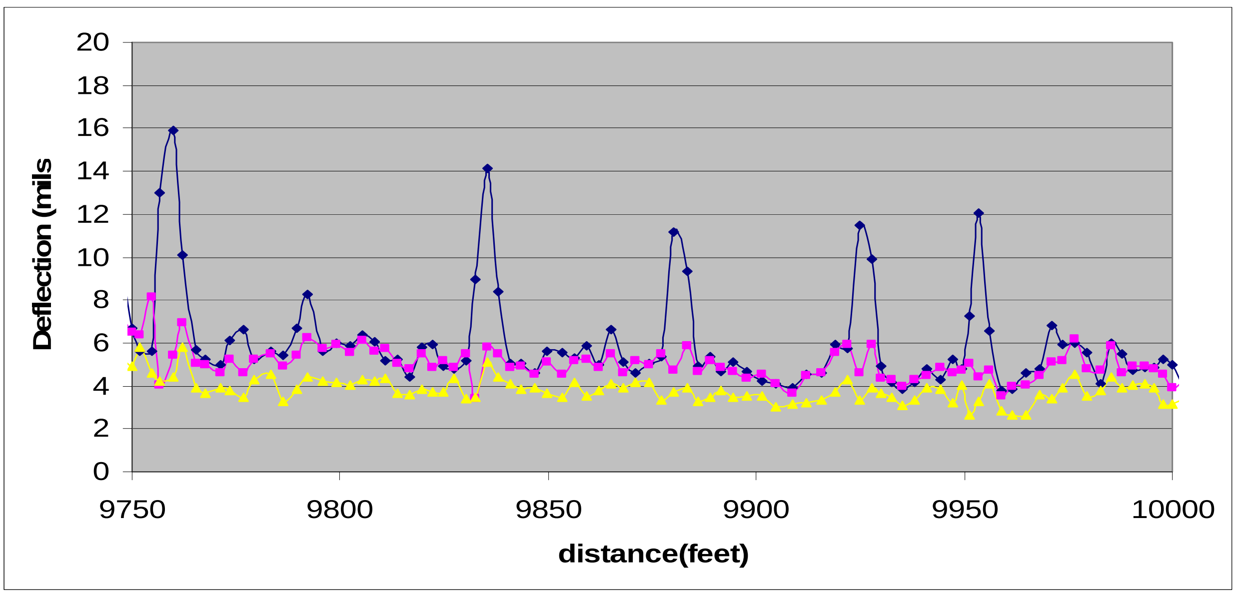
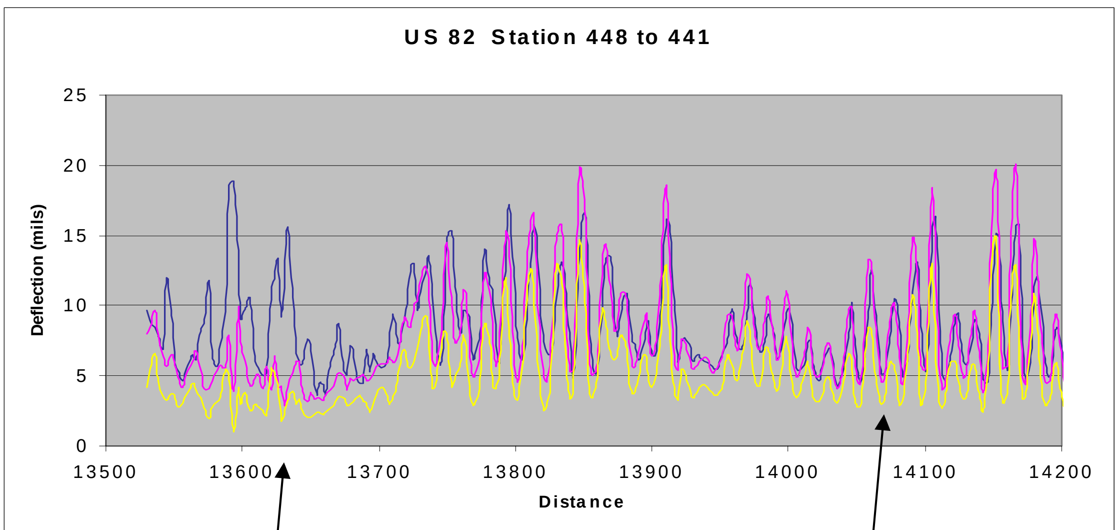


Figure 4. Typical RDD Three-Channel Deflection Plot for a Section of Jointed Concrete Pavement (Blue, red, and yellow lines are sensors 1, 3, and 4 as described in Figure 2).

In Project 0-4517 Rolling Dynamic Deflection data were collected on an 11-mile section of US 82 in the Wichita Falls District. Considerable variation in RDD deflection profiles was observed along the highway. The upper plot in Figure 5 shows the RDD deflection profile for a 700 feet section of US 82 where high pavement deflections were recorded. The blue, red, and yellow plots are for the sensor locations 1, 3, and 4 as described in Figure 2. At the left of Figure 5 the pavement condition was poor with several broken slabs and some slab faulting. At the right of the figure, the slab condition was good with no apparent surface distress. The RDD deflection data are different from the distressed to the non-distressed section. In the distressed section, there is considerable variation in sensor 1 but relatively small changes in sensors 3 and 4 are the RDD rolls over joints. However, in the non-distressed areas, all three sensors show substantial changes in deflection. The interpretation is that in the distressed areas the joint load transfer is poor and this results in large differences in deflections as the sensors straddle the joint. Whereas in the non distressed area, the load transfer is good but the sub-slab support is poor.



a) Raw RDD data for a 700 feet section of highway

b) Pavement condition in the distressed and non distressed area

Figure 5. Rolling Deflectometer Data from US 82 in the Wichita Falls District.

The RDD patterns in the high deflection areas should be contrasted with those obtained in other areas of US 82 such as shown in Figure 6. The vertical deflection axis in Figure 6 uses the same scale (0 – 25 mils) as that used in Figure 5 to emphasize the difference in magnitudes that are observed as the RDD tests a project. Figure 6 shows deflections from five slabs. Clearly, in this location, the load transfer is excellent and the center slab deflections are very low.

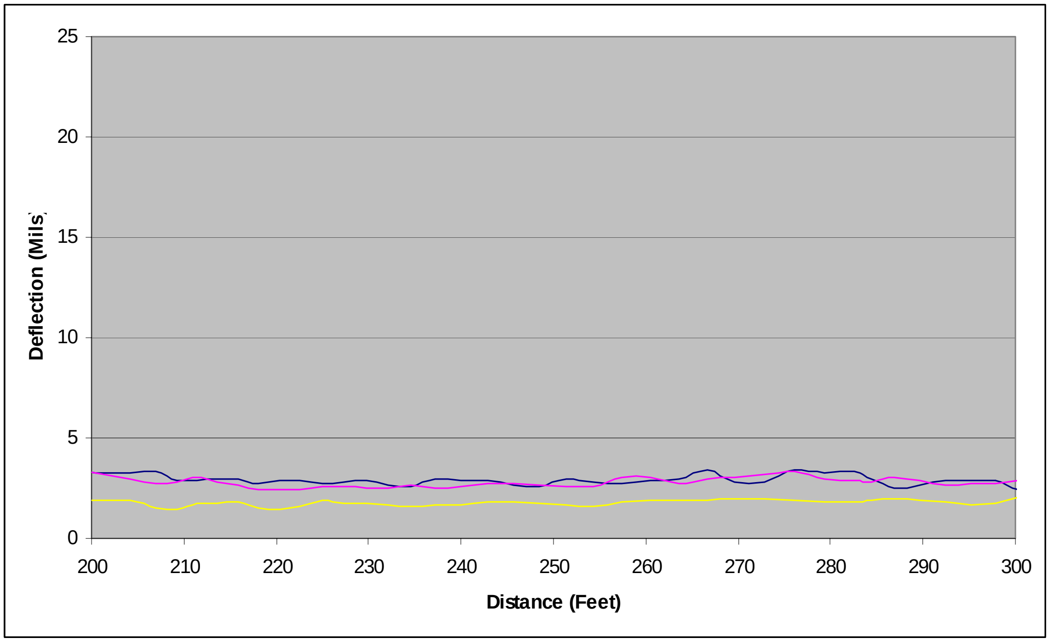


Figure 6. RDD Deflections from a Low Deflection Area on US 82 (contrast with Figure 5).

Supplemental field testing was conducted with both a Falling Weight Deflectometer (FWD) and Dynamic Cone Penetrometer (DCP) to validate the interpretation of these data. In the location shown in Figure 6, the load transfer efficiencies were all measured to be 95 percent and above. The load transfer efficiencies in the distressed area marked in Figure 5 were highly variable ranging from 20 percent to 85 percent. From the DCP, the US 82 slabs were measured to be sitting on a thin layer of select material over a highly variable subgrade. For the low deflection section shown in Figure 6, the rate of penetration of the DCP through the subgrade ranged from 0.5 to 0.8 inches/blow. For the high deflection area shown in Figure 5, the top of the subgrade penetrations ranged from 2.2 to 3.2 inches/blow.

The RDD data to this point have not been widely used by TxDOT engineers to make rehabilitation decisions on jointed concrete pavements. The device shows great potential, but more work needs to be done as will be described later. However, until further work is completed, there is an urgent need to develop and implement criteria to support decisions that TxDOT engineers are making today. The area of main interest is in determining if the existing pavement is a good candidate for an asphalt overlay. This involves an estimation of the number of poor joints which will need to be replaced or improved before placing an overlay. The results from the US 82 study lead to the conclusion that researchers need criteria based on the difference in deflection between sensors as the load wheels pass over a joint. A second set of criteria is needed to estimate the quality of sub-slab support that will be based on the magnitude of sensor 1 deflections. In developing tentative criteria, three sets of RDD data obtained from rolling over single joints in US 82 shown in Figures 7, 8, and 9 are used to illustrate the variations observed in the field.

Figure 7 shows a joint with very low deflections on sensors 3 and 4 for the entire length with only a small increase in deflection (2 mils) as the RDD passes directly over the joint. In this area, the highway was performing excellently; it must be recalled that US 82 was over 40 years old at the time of testing. This is judged to be the ideal case.

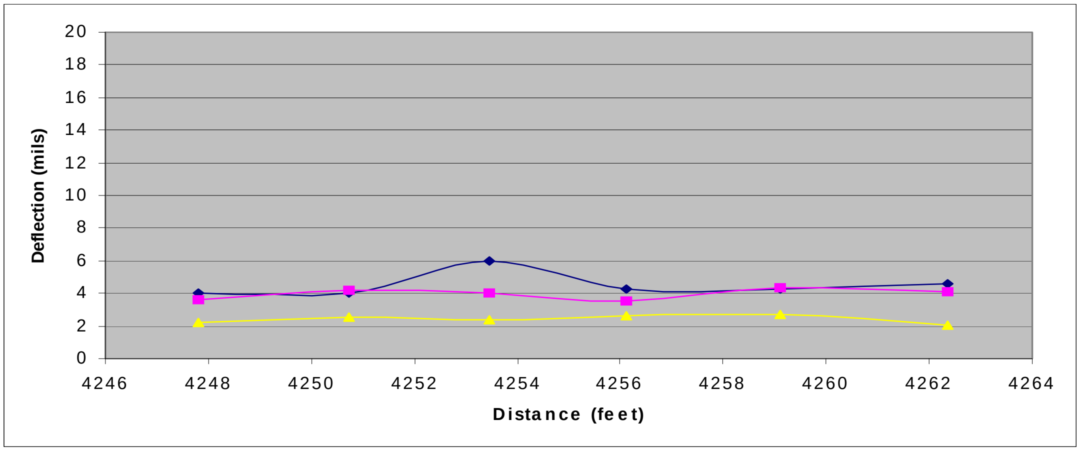


Figure 7. RDD Data from an Ideal Joint Deflection Pattern. (Interpretation: excellent load transfer, good slab support)

The ideal case in Figure 7 should be contrasted with the higher deflection joint shown in Figure 8. In this case, there are large increases in all three RDD sensors as the load wheels pass over the joint. Sensors 3 and 4 both peak before sensor 1. Where sensor 1 peaks, it is assumed that this is equivalent to the upstream FWD tests location with the FWD plate on one side of the joint and the other sensors on the other. (This may not be the case if the slab has substantial voiding.) With the current level of understanding, researchers propose that the instantaneous difference in deflection (between sensors 1 and 3) when sensor 1 peaks is a good measure of the load transfer efficiency of the joint. Furthermore, the magnitude of the sensor 1 deflection over the joint and at center slab is an indication of subgrade quality. The interpretation for Figure 8 would be reasonable load transfer, poor subgrade support. The roadway was in good condition at this location.

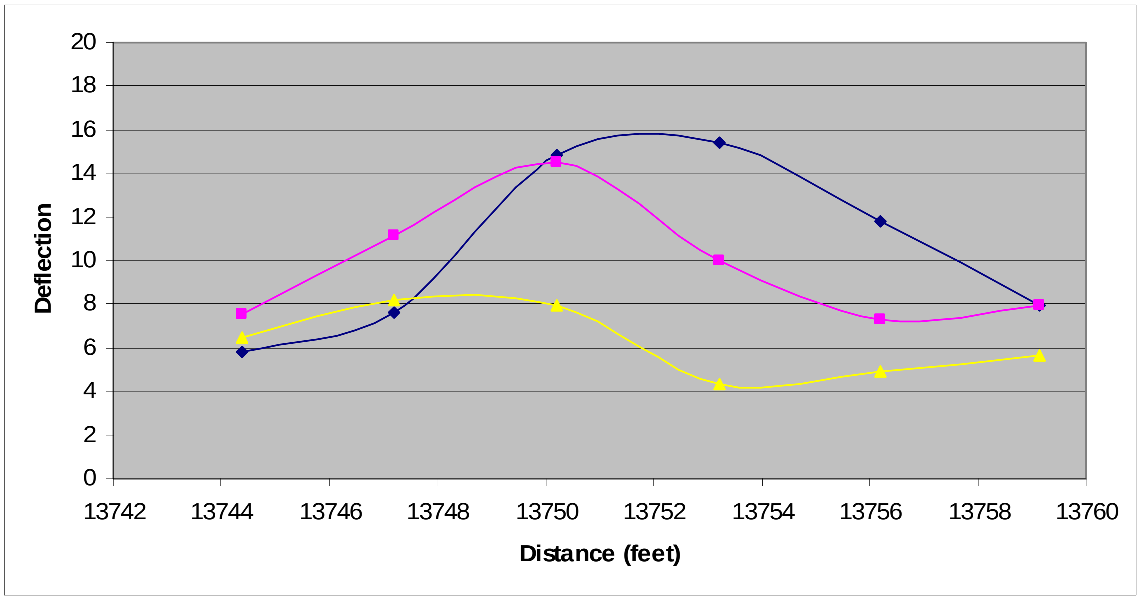


Figure 8. RDD Data from a High Joint Deflection Location on US 82. (Interpretation: reasonable load transfer, poor slab support)

A third deflection patterns was also observed on US 82 in contrast to the cases shown in Figures 7 and 8. This is shown below in Figure 9. This was from a badly distressed area on US 82. In this location, several of the slabs had corner breaks or longitudinal cracks. In this case, there is some variation in sensors 3 and 4 but clearly not as much as observed in Figure 8, but there is a large increase in sensor 1. When sensor 1 peaks, there is a difference of over 13 mils between sensors 1 and 3, indicating substantial vertical movement of the joint. This is judged to be an active joint with a poor load transfer efficiency. It is proposed that an asphalt overlay would do poorly over this joint because of the high shear stresses that would be induced for each passage of a heavy truck.

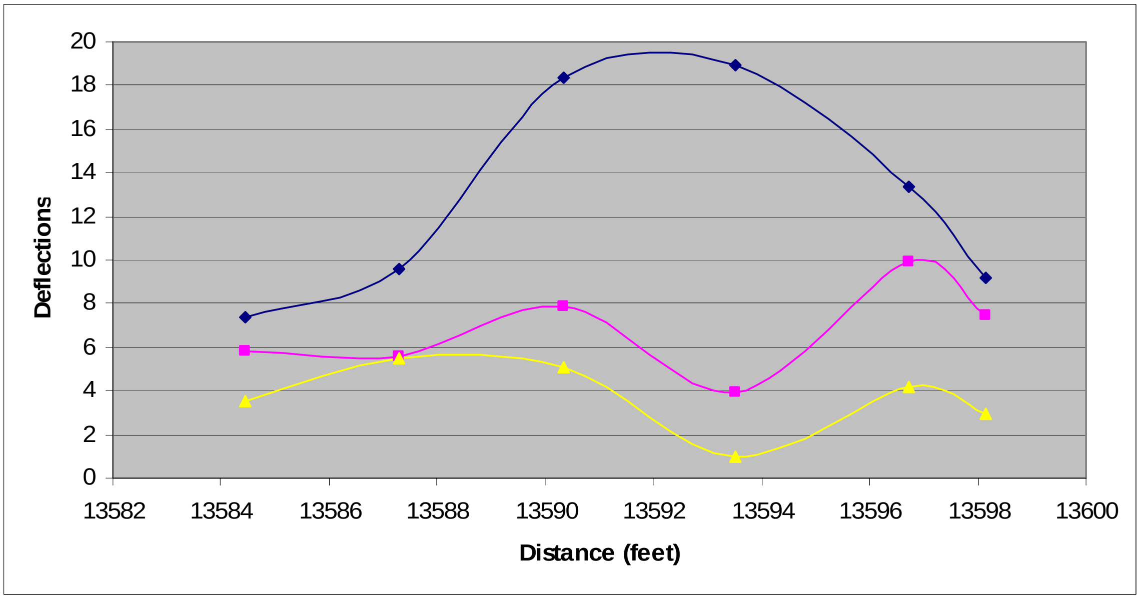


Figure 9. RDD Data from a Poorly Performing Section on US 82. (Interpretation: very poor load transfer, poor slab support)

The interpretations given above and the criteria presented below are tentative at this time. Figures 7, 8, and 9 show three of the most commonly observed deflection patterns as the RDD rolled over joints on US 82. Other patterns were found; clearly, more work is required in this area. US 82 consisted of jointed plain concrete with dowel bar load transfer at the joints and a select material base beneath the concrete. In Texas, the jointed concrete pavement tested with the RDD are typically 30 to 70 years old with a range of reinforcement and joint load transfer devices. The work presented here is for one case, and clearly, more work is required before general guidelines can be developed. This work must include different slab types, testing control slabs (voided and non-voided), and modeling to fully understand the deflection patterns with rolling deflection equipment. The RDD supplies a tremendous amount of information about jointed concrete pavements; more work is required to fully understand and interpret this information.

Based on the data collected on US 82, the following criteria are proposed:

For load transfer efficiency – (Instantaneous difference in deflection between sensors 1 and 3 when sensor 1 peaks, with RDD operating at the 10 kip load level)

Good	< 6 mils
Marginal	6 – 8 mils
Poor	> 8 mils

For center slab support – (Mid-slab deflections on sensor 1)

Good	< 5 mils
Marginal	5 – 7 mils
Poor	> 7 mils

The US 82 project was an actual TxDOT rehabilitation study where the Wichita Falls District was planning to place the same overlay along the entire length of the project. This included 1.5 inches of dense-graded asphalt level up, 3 inches of Stone Matrix Asphalt, and 1.5 inches of Porous Friction Course (PFC). Based on the RDD data and the interpretation scheme presented above, the US 82 project was broken into three distinct joint condition classes as follows:

- Class 1 – Ideal case, good sub-slab support, and good load transfer (this was the predominate case with almost 70 percent of the highway falling into this class);
- Class 2 – Good load transfer, poor subbase support (about 20 percent of the highway fell into this class); and
- Class 3 – Poor load transfer and poor subgrade support (around 10 percent fell here; these were normally the areas that were exhibiting some form of surface distress).

Based on this classification, it was proposed that the district modify its rehabilitation plan. In the solid area (Class 1), the Load Transfer Efficiency (LTE) were measured to be greater than 90 percent. In these areas, the district could reduce the overlay thickness design. For Class 2, researchers recommend that the district consider thickening the overlay required. For Class 3, some full depth repair would be required, but this would be problematic as the sub-slab support is very poor; therefore, substantial undercutting would be required. The main finding of this evaluation is that the RDD appears to be an excellent tool for sub-sectioning jointed concrete pavements in the rehabilitation planning stage. The FWD is not practical for this work. The pavement on US 82 was over 11 miles long with over 3000 joints. At the current operational speed, the RDD completed the testing in about 11 hours.

2.3 CONTROLLED TESTING WITH THE RDD

The observed potential of the RDD leads to a very limited set of control tests conducted at TTI's Riverside Campus. In this study, replicate runs were made of the same set of 20 slabs. Also, the center slab deflection and load transfer efficiency were recorded with the FWD.

Typical RDD repeatability results are shown in Figures 10 and 11. Figure 10 shows the replicate maximum deflections from RDD sensor 1 for seven of the 20 slabs tested. In general, the deflection patterns were similar, but there were some variations in both magnitude and deflection shape. For example, the maximum deflections at the joint at 300 feet were 13.1, 13.2, and 11.8 mils. It was also noted that the shape of the deflection patterns change from run to run. For example, see the deflection peaks at 390 feet in Figure 10, the shape of the deflection pattern as the RDD passes over the joint is different for each run.

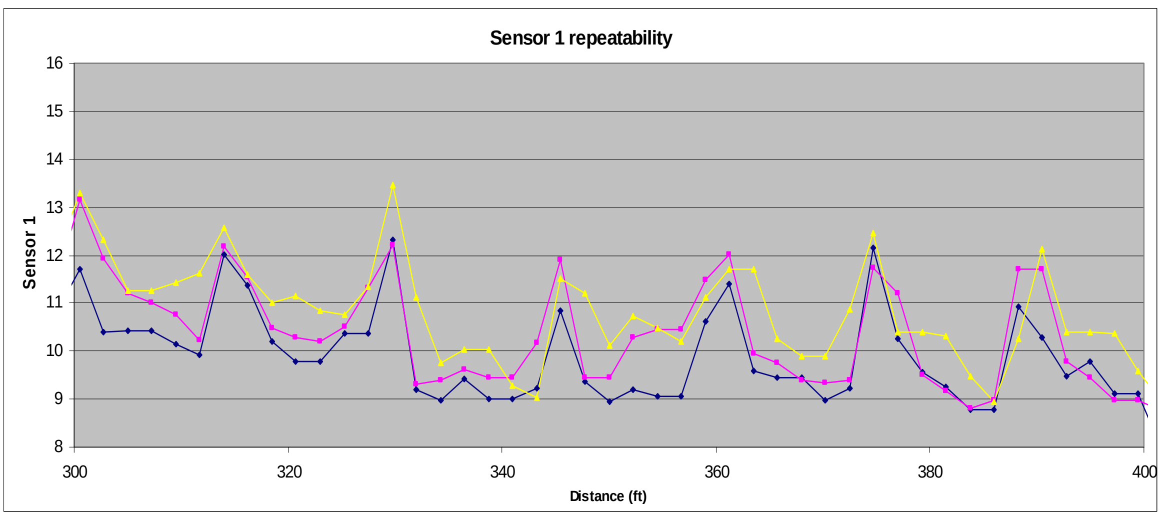


Figure 10. Maximum Deflection from Replicate Runs of the RDD over Seven Joints at TTI's Riverside Campus.

As proposed earlier, it appears that the quality of the LTE of any joint is best measured by the difference in deflection. Figure 11 shows the difference in deflection between sensors 1 and 3 for three runs. There is some concern about repeatability; for example, for the first joint, the differences for the three runs were 9.4, 9.7, and 5.4 mils. Based on the criteria from US 82, this joint could have been ranked as poor or good based on this variation.

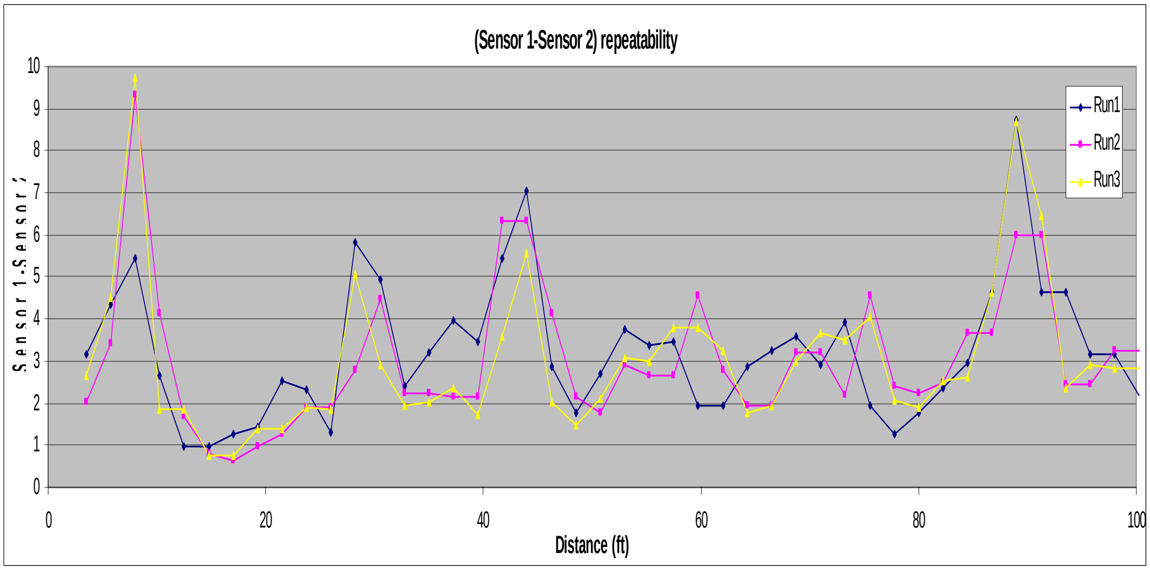


Figure 11. Difference in Deflection (mils) versus Distance from Three Repeat Runs of the RDD.

The comparison of the RDD maximum deflection with the FWD center slab maximum deflection normalized to 10,000 lb is shown in Figure 12. The yellow line is the center slab deflection measured with the FWD. For this data set, the FWD deflections are constantly between 10 and 20 percent less than the RDD deflection. One problem with these data is that the data set is very limited; there is not a wide variation of either center slab deflections or LTE.

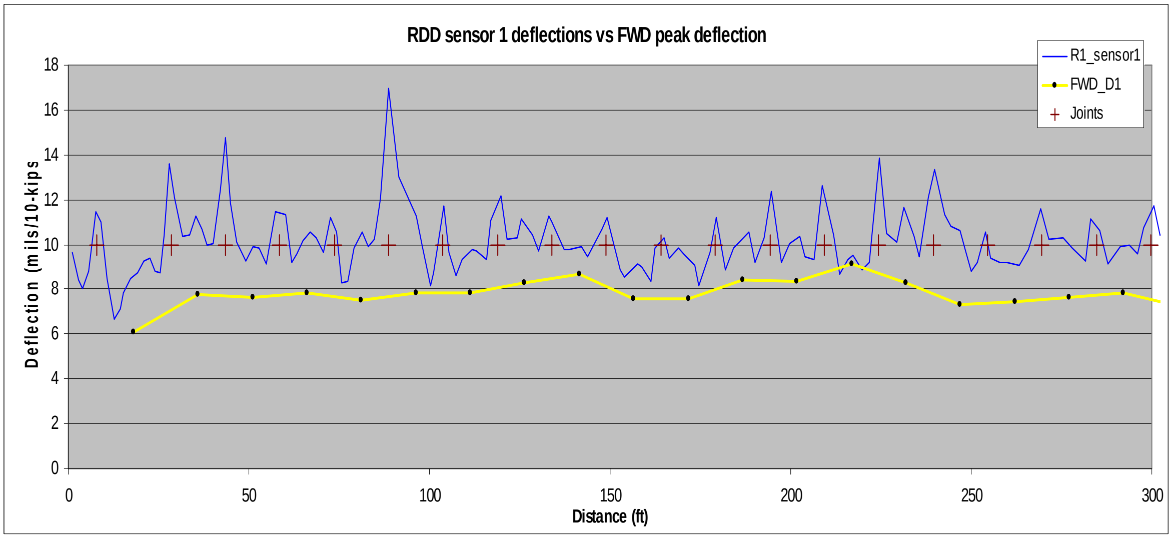


Figure 12. Comparison of RDD Deflection Profile with FWD Center Slab Deflection Normalized to 10,000 lb.

The plot of LTEs from the FWD against difference in deflection between sensors 1 and 3 is shown in Figure 13. The FWD load transfer efficiency scale is on the right of this figure, and the RDD deflection scale is on the left. Again, the limitation of this data set is that other than the first joint all of the other joints have very similar LTE between 84 and 95 percent. For this particular data set with the criteria presented earlier from the RDD data, 18 of the joints would have been classified as good, one as marginal, and one as poor. From the FWD using the 80 percent LTE criteria, only one of the joints would have been classified as marginal while all the others would have been classified as good. The concern is that the poor joint detected by the RDD was not detected by the FWD.

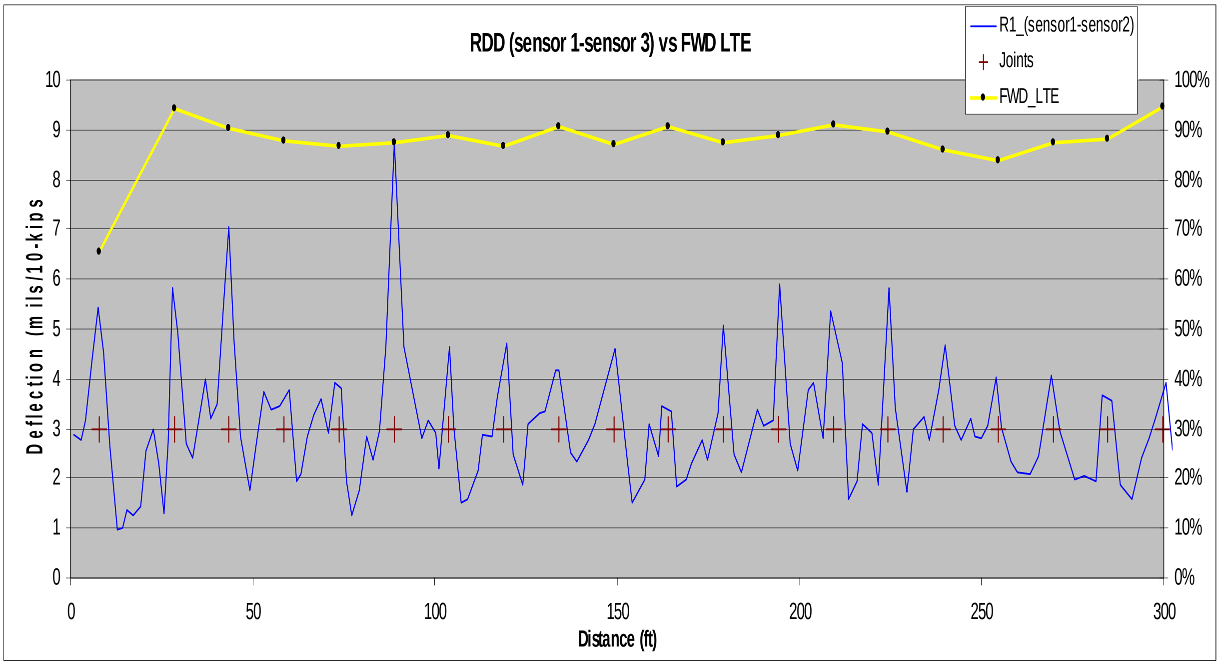


Figure 13. Comparison of RDD Difference in Deflection between Sensors 1 and 3 against Load Transfer Efficiency Measure by the FWD.

2.4 SUMMARY OF RECOMMENDATIONS FOR RDD

The Rolling Dynamic Deflectometer shows great potential for helping TxDOT evaluate jointed concrete pavements. The results obtained on US 82 were of great interest to the Wichita Falls District. An RDD deflection interpretation scheme was also proposed for US 82 based on observed field performance and a limited validation testing with the DCP and FWD. Whether this scheme is appropriate for other JCPs is subject to question.

The benefits of the RDD are clearly demonstrated in the data plot shown below in Figure 14 which is for a short section of US 82. Under the existing test conditions, the RDD can clearly define where the slab deflections are ideal for an asphalt overlay and where they are not. The left section of Figure 14 shows very good load transfer with very little vertical movement. In the overlay design, the designer will, therefore, have to concentrate on designing a mix which will accommodate horizontal movements initiated by changes in temperature rather than shear stresses initiated by truck loadings. TxDOT can design for thermally induced movements using the updated overlay tester recently developed in Project 0-4467 (Zhou and Scullion, 2004). The right section of Figure 14 shows an area where a simple overlay will have problems. In this location the joint will have to be improved or replaced before a simple overlay should be considered.

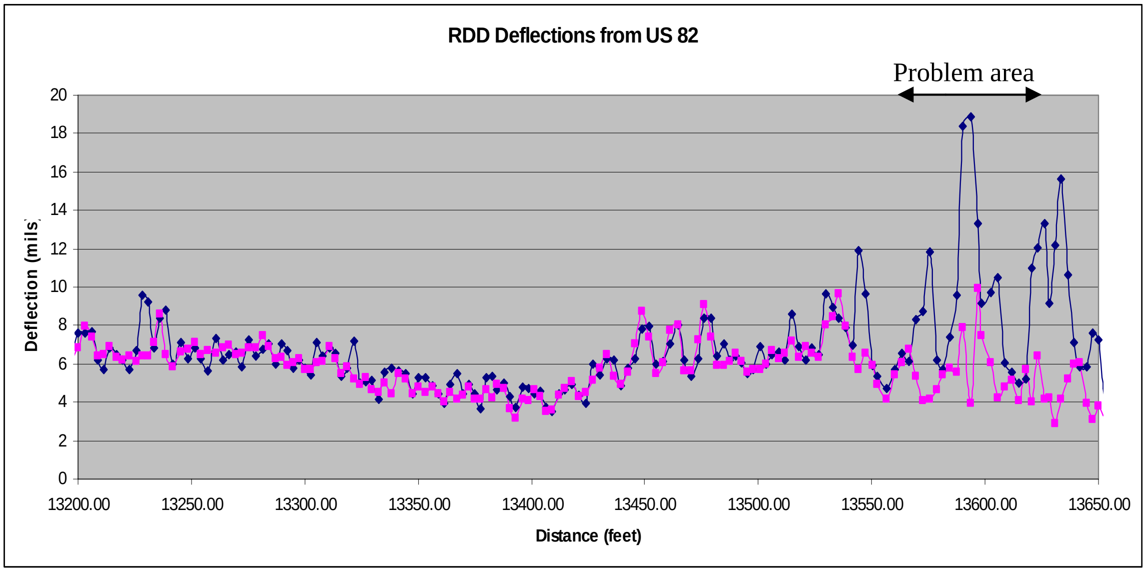


Figure 14. Typical RDD Profile from US 82 Showing both Low Deflection and High Deflection Areas.

The limited controlled testing at the Riverside Campus raised some concerns about the repeatability of the RDD. Based on the above discussion, the following conclusions and recommendations are offered.

1. The main limitations of the RDD are the speed of travel, its reliability, and limitations in the data acquisition system. The data collection speed of 1 to 1.5 mph can be dangerous in urban situations, especially at on-ramps and off-ramps. The current system is built on a 30-year-old vibrosies frame; this unit frequently breaks down. The data acquisition system is very basic; this limitation will be discussed later.
2. The positive results obtained on the US 82 and several other projects around Texas indicate that TxDOT should consider funding the development of the next generation RDD system. The remaining JCP pavements in Texas are very old and often very problematic. TxDOT districts urgently need new tools to evaluate them and to assist in planning the optimal rehabilitation strategy.
3. The RDD's data acquisition system urgently needs to be upgraded. Currently, it collects data in a time mode and reports data as average deflections in 2-second intervals. The time reporting is a problem, as the RDD's speed can be somewhat variable. There is also no guarantee that 2 seconds is the best reporting interval. More work is required here, especially in the area of reporting deflections over joints. The reporting interval should be user defined in the distance rather than time mode. It is proposed that this 2-second reporting interval may be the source of some of the variability and repeatability concerns discussed earlier.
4. In the geophone-based Dynaflect system, the geophones were calibrated in the field before every data collection run. A similar calibration system should be incorporated into future versions of the RDD.
5. One limitation of the RDD on long runs is that it is very difficult to locate individual problematic joints in the field after data collection. This could best be solved by incorporating a video system as part of the data acquisition unit.
6. More repeatability and comparisons with the FWD should be made but future tests must be made in areas which show substantial changes in deflection. The comparison with the FWD should take into consideration the differences in the loading mechanisms between the two devices. The RDD is a vibratory system which can induce both constructive and destructive amplitude changes from waves reflected from lower layers, whereas the FWD is an impact loading device with a very short data collection period. The comparison should also be conducted on control slabs with built-in voids and known soil support conditions.

7. Studies need to be made on the variations in RDD deflection patterns over joints. For projects such as US 82, the different types of RDD patterns should be identified and investigated with other tools such as GPR, DCP, and FWD. Finite Element modeling should also be conducted to model the passage of the RDD over joints.
8. The impact of temperature on RDD joint deflections should be studied. Temperature is a major limiting factor in interpreting LTEs from the FWD.

CHAPTER 3
GROUND PENETRATING RADAR

3.1 BACKGROUND

Ground Penetrating Radar technology was implemented with TxDOT in the mid 1990s. TxDOT has a fleet of three units which are used routinely for forensic investigations and pavement rehabilitation studies. Figure 15 shows one of TxDOT's most recent air-coupled GPR units. The background to GPR and the discussion of TxDOT's analysis program COLORMAP are given elsewhere (Scullion et al., 1995). However, most GPR applications have focused on its usage on flexible pavements where it is possible to estimate layer thicknesses and identify subsurface defects.

Figure 15. TxDOTs Air-Coupled GPR Unit.

In Project 0-4517 GPR data were also collected on several jointed concrete pavements. On JCP pavements, the main purpose is to identify possible sub-slab defects, primarily areas of trapped moisture. On composite pavements where the jointed concrete is covered by an asphalt overlay, the GPR can be used to identify any problems with the Hot Mix Asphalt (HMA) layers in addition to detecting water-filled voids.

On an investigation of the concrete pavements on IH 45 in the Houston District, a wide variety of GPR signatures were obtained. Figures 16, 17, and 18 show the typical COLORMAP displays several GPR signatures.

Figure 16 shows the ideal case. This is a GPR display of approximately 600 feet of IH 45. The depth scale is on the right, and the distance scale in miles and feet is at the bottom of the figure. The faint line at a depth of 4 inches below the surface is reflections from reinforcing steel. The additional faint line at a depth of 12 inches is the reflection from the bottom of the concrete/top of the base. The intensity of reflection at this location indicates the presence of moisture or the presence of air-filled voids. Water will produce a strong reflection, and this would be shown as a strong red reflection at a depth of 12 inches. There are no strong reflections in Figure 16. An air-filled void would give a completely different reflection; with the color coding scheme used in COLORMAP, an air-filled void would be represented as a blue line. Again, there are no indications of air-filled voids in Figure 16.

The graph at the bottom of the figure is a plot of surface dielectric from the JCP. The periodic increases in the plot coincide with the joints in the pavement. The increase in surface dielectric is associated with increases in near surface moisture content. These patterns occur in some JCPs but not all; they are either associated with build up in moisture in the joint itself or in the concrete immediately surrounding the joint.

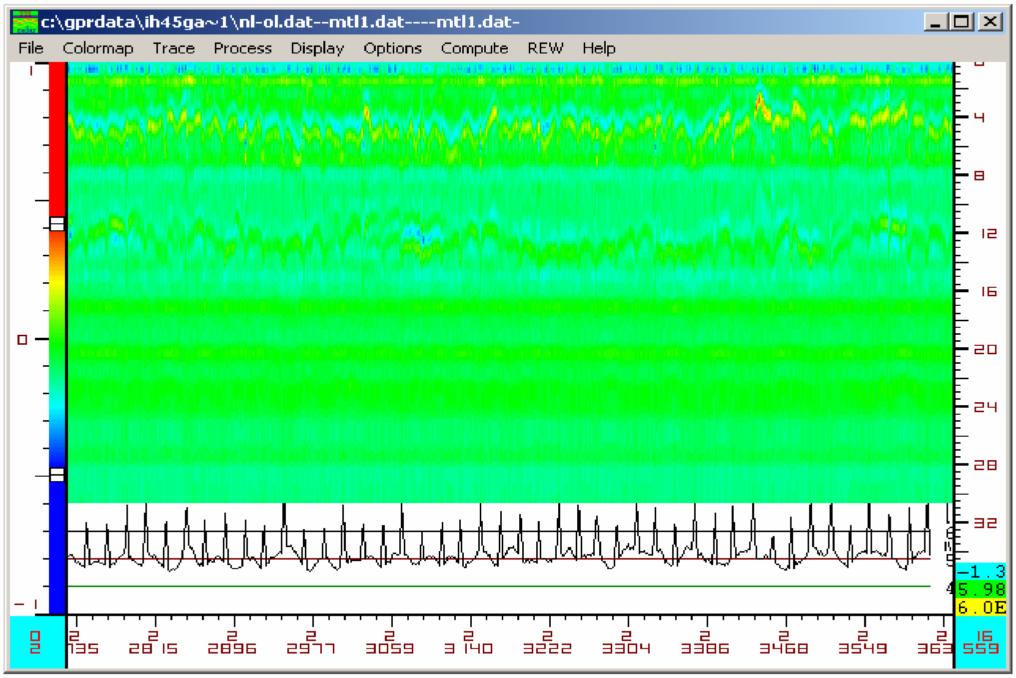


Figure 16. GPR Data from a JCP with no Obvious Sub-Slab Problems.

The COLORMAP display shown above should be contrasted with that shown below in Figure 17. In this case, there are stronger periodic reflections both at the bottom of the slab and from within the slab itself at the depth of the reinforcing steel. The red reflections beneath the slab indicate the presence of additional moisture. The presence of free water beneath the slab could have a major impact on the selection of rehabilitation options. For overlays, if the overlay sealed the concrete surface, the concern would be that the trapped moisture may migrate up through the joints and cause layer debonding or stripping with the HMA layer. For projects where slab fracturing techniques are proposed, it would be critical to drain the moisture before proceeding.

As with all other investigations with GPR, it is critical to verify the interpretations. In this case, pilot holes were drilled through the concrete slab. The red areas at the bottom of the slab were found to be areas of wet clay rather than water-filled voids. The original base was select sand material, which in these few locations now has become contaminated by clay. From coring, it was determined that the stronger reflections at mid depth were found to be associated with areas of corrosion of the reinforcing mesh used on this slab.

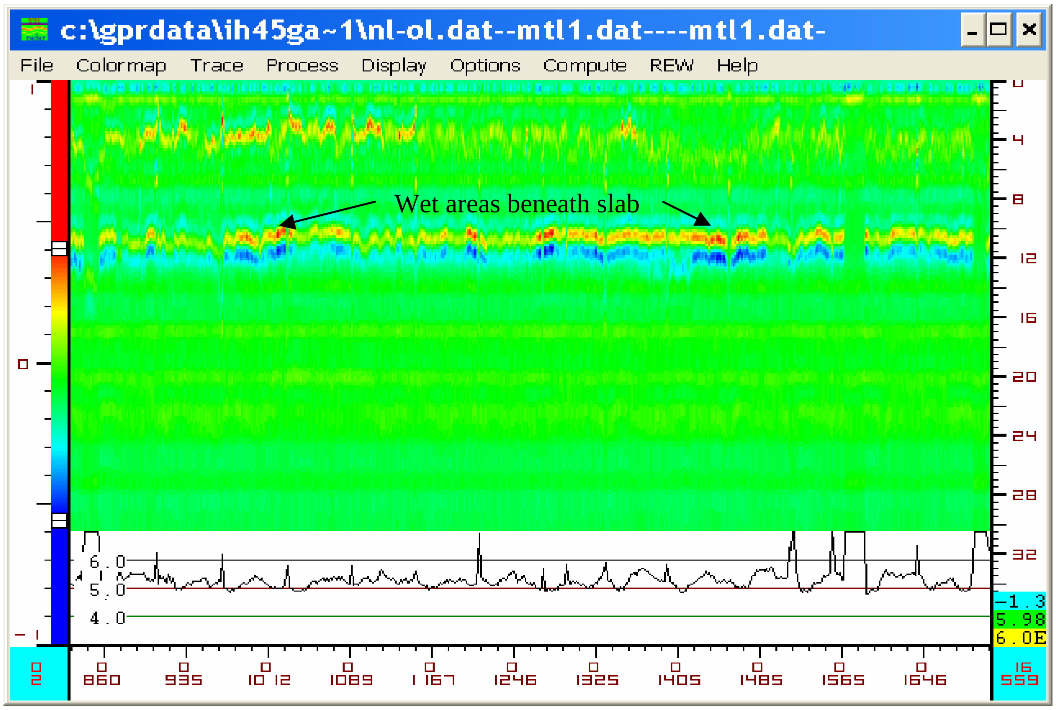


Figure 17. COLORMAP Display from a Section of JCP with Possible Problem Areas.

The third case shown below in Figure 18 displays a different portion of the same highway. In this case, there is almost a continuous strong red reflection followed by a continuous blue reflection. The one gap in the middle of the plot is where a full depth patch has been placed. This location had already been undersealed; however, there was substantial staining along the shoulder of the pavement. This section was cored, and it was found that free water was present beneath the slab. In places, there was a localized 2 to 3 inch thick void beneath the slabs. Clearly, the rehabilitation options for this highway are limited because of the presence of the water.

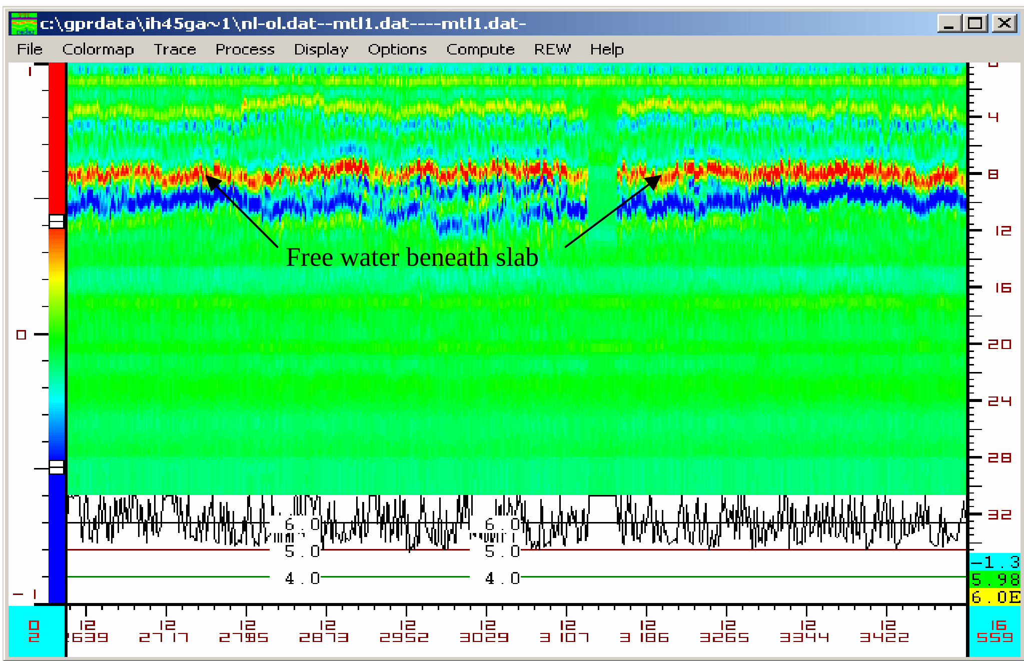


Figure 18. COLORMAP Display from an Area Where Free Water Exists Beneath the Slab.

3.2 SUMMARY FOR GPR APPLICATIONS

Based on experience with GPR testing on concrete, the following conclusions and recommendations are offered.

1. GPR can be used to locate major defects in either the asphalt covering of JCPs or major defects such as water-filled voids beneath the slab. By combining GPR data with the continuous deflection data provided by the RDD, the pavement engineer will have a comprehensive evaluation of pavement conditions.

2. GPR will detect major defects, but it is doubtful if it will detect minor defects such as thin air-filled voids. The current 1 GHz GPR units also have restrictions on depth of penetration; little useful information will be obtained from deeper than 20 inches. This is not usually a restriction on old JCP; it could be a problem if the old concrete has a thick asphalt overlay, or if the slab is sitting on a thick base and the problem is in the subgrade layer.
3. COLORMAP cannot provide quantitative values (layer dielectric) for the base layer beneath the slab. This is because concrete is a highly attenuative medium for GPR waves (whereas asphalt has little or no attenuation). Work by Peterson (2004) found that the signal strength through concrete can be attenuated by up to 8 percent for every inch of transmission through the slab. The attenuation through a concrete slab is not adequately addressed in the current version of COLORMAP. Based on signal attenuation, if a strong positive reflection is observed beneath a concrete slab, then the base must have a very high dielectric indicating possible trapped moisture.
4. All GPR interpretations require validation. As with the case on IH 45, strong reflections beneath a slab do not automatically mean a water-filled void. As found in this project, it could be areas of saturated base or wet clay, with no void.

CHAPTER 4

CONCLUSIONS AND RECOMMENDATIONS

Chapters 2 and 3 have described the new technologies which have been used in this project to evaluate concrete pavements. The RDD is a unique deflection device, and TxDOT should consider developing the next generation of the unit. It will also be necessary to conduct additional RDD work to address the issues raised at the end of Chapter 2.

Efforts should also be initiated to continue the integration of digital video, GPR and continuous deflection technologies. A pilot integration effort was initiated under Project 0-4517, and the modified RDD unit shown in Figure 19 was field tested. A 1 GHz horn antenna and a video camera were fixed to the front of the RDD. TTI developed a new data acquisition program to collect and integrate the information from all three devices. This approach holds much promise, and it will address some of the deficiencies noted in Chapter 2.

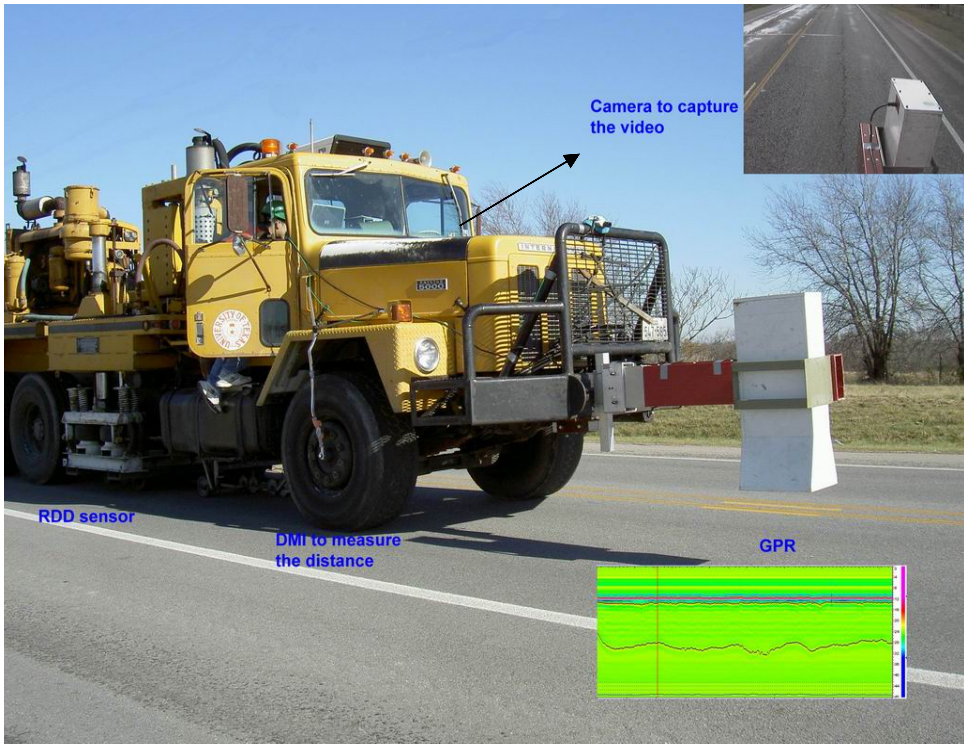


Figure 19. Integrated RDD Unit.

4.1 RECOMMENDATIONS ON FUTURE NDT TESTING OF JOINTED CONCRETE PAVEMENTS

The recommended approach to evaluating JCP as the first step in the pavement design process includes the following:

1. **Assemble all existing project information.** This will include typical sections and recent maintenance history. It is important to know if the slab is reinforced, the type of joint, type of shoulders, and the type of base beneath the slab.
2. **Conduct a GPR survey and visual inspection.** The GPR data should be collected in the outside lane/outside wheel path of the project with a data collection interval of 1 foot per trace. The data will identify areas of high sub-slab reflection indicating possible trapped moisture. If the slab has an existing hot-mix surface, the GPR can measure the thickness of the overlay and determine if there is any deterioration in the overlay. The information generated from the GPR will be used to assist in interpreting the RDD data.

   If the shoulders are different from the main lanes, a GPR survey should be made to identify the layer thicknesses and their subsurface condition. This is important if the section is a candidate for slab fracturing.

   During the GPR survey, collect a video of the pavement surface, and use this to make a log of pavement conditions. Areas of shattered slabs, wide longitudinal joints, and faulted joints should be identified. If the project is being considered for slab fracturing, note the drainage condition, and evaluate the feasibility of retrofitting edge drains if the GPR indicates trapped subsurface moisture.
3. **Conduct an RDD survey.** Collect RDD data, and use the interpretation criteria presented in Chapter 2 to locate problem joints or identify weak support areas. Segment the project based on the RDD data; in each segment calculate the number of poor joints and locate the poor support areas for validation testing. If an overlay is to be considered for a segment, then all joints classified as having poor load transfer should be scheduled for replacement or for improvement (dowel bar retrofit).

4. **Conduct validation testing.** All projects require additional testing to validate both the GPR and RDD interpretations. It is normal to select at least one location in each project segment to validate that the correct interpretations have been made.

   Poor joints identified by the RDD can be tested with the FWD; if the GPR also indicated possible voiding, access holes should be drilled through the concrete to validate what is beneath the slab.

   If the project is a candidate for slab fracturing techniques, areas with either trapped moisture or high center slab deflections should be tested with the DCP. Techniques for interpreting both center slab FWD deflections and DCP data to determine if the slab is a good candidate for rubblization are under development in Project 0-4687.

The approach described above is ideal because it provides 100 percent coverage of the proposed pavement section. Both the GPR and RDD collect full coverage data. However, because of reliability problems and breakdowns with the existing RDD, it is important to make a backup plan in case the RDD is not available for project testing. In this case, the RDD survey described above should be replaced with the following FWD survey.

5. **Conduct and FWD survey.** Conduct an FWD survey of the entire project. At each location, perform an FWD test initially at the center of the slab, and then move forward to the next joint location. At the joint, perform an upstream test where the load plate is placed on one side of the joint and the remaining six sensors are placed on the other side. Test at a minimum of 30 locations along the project, but do not collect data at intervals of greater than 0.1 mile. For very long projects, for example greater than 10 miles, at the engineer's discretion the data collection interval can be extended to every 0.2 miles.

   All FWD data should be collected in the outside lane and outside wheel path. Mid-depth slab temperatures should be measured at the start and end of the test.

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