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16. Abstract

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 strategies for jointed concrete pavements (JCP). This year 1 report reviews the performance of the six experimental sections on US 59 and makes recommendations for statewide implementation. The best performing section in Lufkin was the flexible base overlay, which involved placing high-quality crushed limestone directly over the JCP followed by an underseal and thin asphalt overlay. This was also one of the least expensive treatments used in the experiment. The large stone mix also gave good performance, but the crack and seat and full-depth repair techniques did not perform well. A forensic investigation was conducted to attempt to explain the variation in treatment performance.

To complement the Lufkin results, a review is also presented of the performance of other JCP rehabilitation techniques recently evaluated by TxDOT districts. An evaluation of crack retarding asphalt layers (Strata®), grid layers (GlasGrid®) and slab fracturing techniques is also included in the report.

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PERFORMANCE REPORT ON JOINTED CONCRETE PAVEMENT REPAIR STRATEGIES IN TEXAS

by

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Report 0-4517-1
Project 0-4517
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DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Texas Department of Transportation (TxDOT) or the Federal Highway Administration (FHWA). This report does not constitute a standard, specification, or regulation. The engineer in charge was Tom Scullion, P.E. (Texas, # 62683).

Strata® is a registered tradename for a proprietary crack retarding layer. Strata® is registered to Koch Materials. Glasgrid® is a proprietary grid manufactured by Industrial Fabrics Inc.

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

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CHAPTER 1
INTRODUCTION

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 objectives of Project 0-4517 were well summarized in the project statement. 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."

This report summarizes the year 1 activities. These activities consisted of surveying the TxDOT districts, which are actively evaluating methods of minimizing reflection cracking. The first goal was to present conclusions and recommendations from the Lufkin experiment on US 59. The next step was to survey the district and identify new and innovative treatments, which have been applied in recent years. These improvements involve application of crack retarding asphalt layers, the use of grid type fabrics and the use of slab fracturing techniques. These different techniques have been applied in numerous districts around the state of Texas. In this project, the experimental sections are identified, and the performance of the sections is presented.

The performance evaluation consisted of the three distinct phases described below:

1) In Phase 1 the visual performance of the treatment was documented with regard to its ability to retard reflection cracking. The local area engineers (AE) were interviewed to obtain project construction details and their evaluations of the effectiveness of the treatment.

2) In Phase 2 a nondestructive test evaluation was made of the project. This evaluation typically involves both ground penetrating radar (GPR) and falling weight deflectometer (FWD) surveys.

3) Phase 3 involves coring of the test pavement and in some instance removing samples for laboratory testing at the Texas Transportation Institute (TTI).

This report concludes the first year of Project 0-4517. In year 2 the monitoring will be continued on selected projects, and the information gained will be summarized into guidelines to direct future JCP rehabilitation efforts.

CHAPTER 2

LESSON LEARNED FROM THE LUFKIN EXPERIMENT

BACKGROUND

In the 1980s the district engineer in the Lufkin District, Mr. J. L. Beard, proposed the construction of experimental sections on US 59. The predominant pavement structure on this major highway was jointed concrete, which had received numerous widenings and overlays. Under heavy traffic the pavement had deteriorated rapidly. This led to the planning, design and construction of a major experiment, which was intended to provide inputs to the long-term rehabilitation plan for this important highway.

The Lufkin project was designed to evaluate the performance of the seven strategies shown in Figure 1 and to minimize reflection-cracking problems (*1*). These strategies included a) full depth repair, b) crack and seat, c) crushed stone base interlayer, d) open graded asphalt concrete (AC) interlayer, e) styrene-butadiene modified seal coat, f) dense graded overlay and g) thin dense graded overlay. A technique involving sawing and sealing of joints was also investigated. The seven sections were constructed under carefully controlled conditions, with continuous documentation of materials properties, layer thickness and environmental conditions. The traffic in the early 1990s was very high with an average annual daily traffic (AADT) of 16,500 vehicles per day and 2.3 million 18-kip equivalents (ESALs) per year.

The sections were opened to traffic in April 1992, and in 1995 the preliminary performance results were reported (*2, 3*). After two years in service Moody reported that (*3*):

- The most expensive treatment (R1) (full depth repair) was not successful at reducing reflection cracking, with 100 percent of the joints reflecting.
- The crack and seat section was not successful. It did not appear to provide adequate structural support.
- The flexible base overlay (R3) was showing rutting, but this appeared to be stabilizing. More monitoring was recommended.
- Section R4 (open graded asphalt) was performing well.
- Section R5 failed and was replaced, but the cause of the failure was thought to be due to problems with the surface layer.

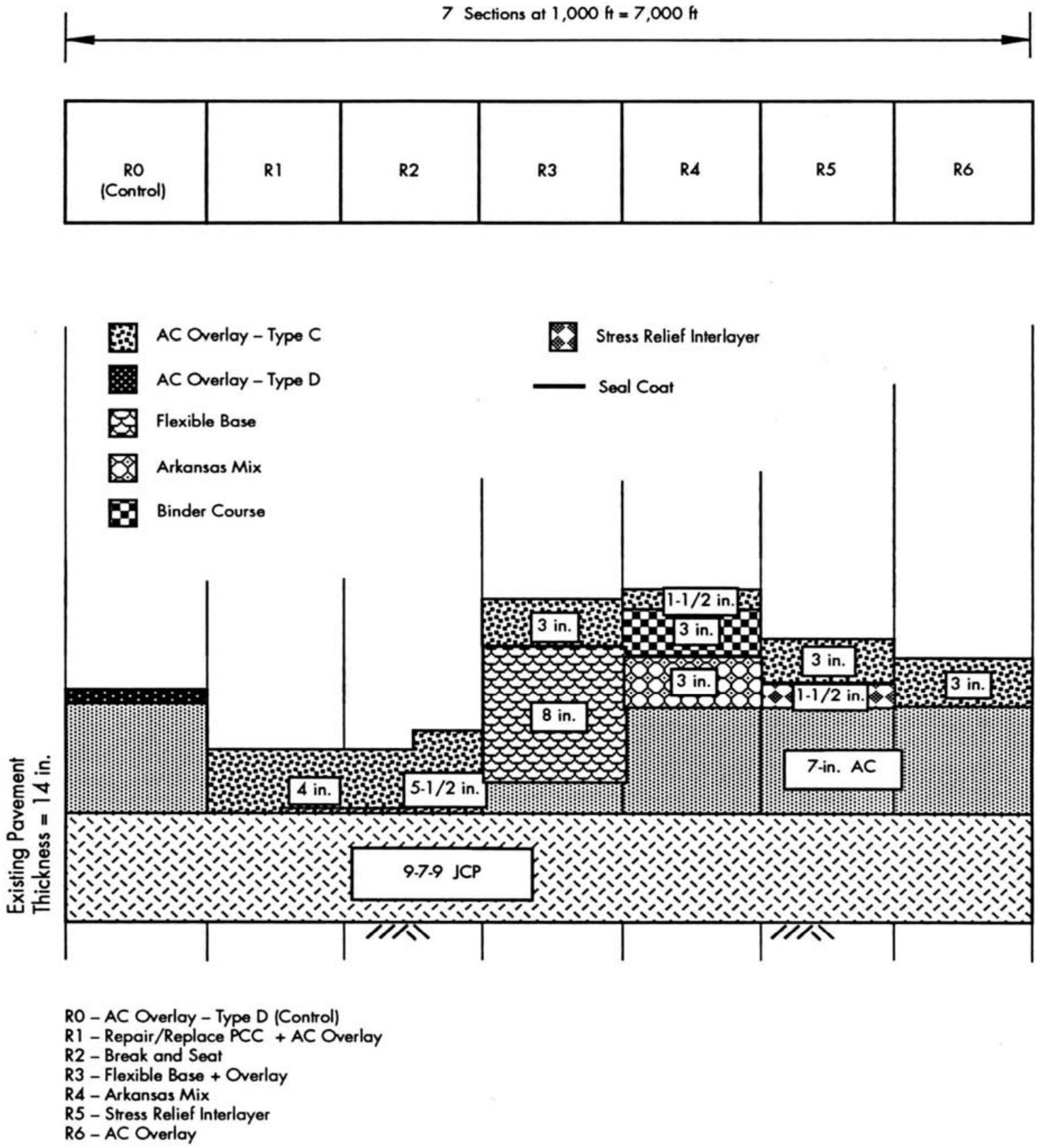


Figure 1. Layout of the JCP Sections on the Lufkin Experiment (2).

- Section 6, which was the cheapest overlay, was performing relatively well with 35 percent reflection cracking.

Moody did provide detailed construction and maintenance costs for each of the sections through 1995. Table 1 summarizes these costs.

Table 1. Initial Results from US 59 (*3*).

Section Number	Treatment	Total Cost $/sqyd	Performance after 2 years
R0	Control - 1.5 inch overlay	8.95	Fair - 40% reflection cracking
R1	Full depth repair. Joint replacement, restoring load transfer, crack sealing and 4 inch overlay	35.63	Poor - 100% reflection cracking
R2A	Crack and seat plus 4 inch overlay	26.73	Very Poor - 20% reflection cracking, and severe block cracking and pumping
R2B	Crack and seat plus 5 inch overlay	30.13	Very Poor - some reflection cracking, block cracking and pumping
R3	Crushed stone interlayer plus 3 inch overlay	17.96	Good - no reflection cracking. Initial rutting (0.25 inch)
R4	Open-graded interlayer plus 4.5 inch overlay	28.15	Very good - no rutting or reflection cracking
R5	Styrene-butadiene Styrene (SBS) modified interlayer plus a 3 inch overlay	15.10	Very poor - section judged to have failed after 1 year. Surface deterioration and roughness problems
R6	3 inch overlay	11.62	Fair - 35% reflection cracking

The cost includes both construction and maintenance cost over the first two years of service.

A follow-up survey was reported by Cho et al. who found continued deterioration in all sections (*2*). In particular, the survey noted additional alligator cracking in section R3. In the mid 1990s, the area engineer (Mr. Harry Thompson) performed some maintenance and repair work on several of these sections. The area engineer concluded that the surfacing mixes were badly segregated, and that this segregation had caused some of the initial performance problems.

In 1997 the hot mix asphalt (HMA) in the travel lane on several of the sections was milled and replaced with good quality material. In late 1999, interest in these sections increased. It was proposed to place TxDOT accelerated pavement tester, the Mobile Load Simulator (MLS), on at least one or two of these experimental sections. A preliminary condition survey was conducted under the direction of Dr. Dar-Hao Chen from TxDOT's construction division. The photographs shown in Figure 2 were taken in 2000, three years after resurfacing. It was found that:

- The best performing section was section R3 with no distress and a good ride.
- Section 4 was performing well. There was minor block cracking in some areas, but the overall ride was very good.
- The other sections were not performing well. The worst performing section was the crack and seat (R2), which was continuing to give structural problems.

These conclusions were very interesting. Section 3 was the granular overlay, which initially rutted and cracked badly. Cho et al. reported that in 1995 (three years after original construction) the section had 450 sq. feet of alligator cracking (*2*). In 1997, the 3 inch surfacing in the outside lane was removed and replaced with 3 inches of new HMA. As reported in 2000, after three additional years in service the repaired lane in Section 3 was performing excellently with no apparent surface distress and excellent ride. The outside lane has the original HMA surface and it is severely cracked. It was, therefore, concluded that the initial poor performance of this section was attributed to a poor surfacing layer.

Section R4 has also reasonably good performance. This section consists of the large stone Arkansas mix, which is largely still the 1992 materials. There are some reflection cracks in the surface, but the overall ride is still good.

a) Section R4 Arkansas Mix (2000) **b) Section R2 Crack and Seat (2000)**

c) Flexible Base Overlay (section R3) (2000)

Figure 2. Several Sections from the Lufkin Experiment, Three Years after a New Surfacing Layer Was Placed.

CONDITION REPORT ON SECTION IN 2003

In early 2003, a visual condition survey was conducted on the test sections. This survey was six years after the outside lane was resurfaced. Several of the sections had received additional maintenance since the photographs shown in Figure 2 were taken in 2000. The condition of each of the experimental sections as of January 2003 is discussed below.

Section R1 continued to exhibit reflection cracking. Maintenance forces have placed additional patches on the section. Figure 3 shows the condition.

Figure 3. Section R1 in 2003, Additional Patching with Reflection Cracking.

Sections R2 A and B continued to perform poorly. The cracked slabs appear to be unstable, and there are substantial failures with pumping, as shown below in Figure 4. Maintenance forces have placed additional patches on the section.

Figure 4. Continued Failures in Section 2B the Crack and Seat Section after Multiple Repairs.

Section R3 was performing very well. As shown below in Figure 5 there was no reflection cracking and only a small area of alligator cracking. However, the alligator cracking was confined to an area where the pavement shoulder had been replaced in 2000 to accommodate the MLS testing. The new shoulder was cement treated base (CTB), and a wide longitudinal crack was evident at the interface between the CTB shoulder and the flexible base main lanes. The alligator cracking was attributed to water entering this construction joint and weakening the flexible base. The normal and cracked areas of R3 are shown in Figure 5.

Figure 5. Section R3 in 2003 Six Years after Resurfacing.

Section R4 was performing well but did exhibit extensive block cracking in the original surface layer as shown in Figure 6. Some reflection cracking was also evident in the outside lane. The ride on the section was good.

Figure 6. Condition of Section R4 in 2003.

After the removal of the original surface, R5 and R6 were essentially identical. They were judged to be performing well with approximately 40 percent reflection cracking. However, the ride on both sections was judged to be good. As discussed below, very little can be concluded about the performance of stress relief layers from this experiment.

In 2003 it was concluded that sections R3 and R4 were still performing well. The full depth repair (R1) and the crack and seat sections (R2) continued to perform poorly. The crack and seat continues to be the worst performing section. The early failure of section R5 in the original experiment was probably attributed to the poor quality of the original surface, therefore it is impossible to gain any information on the ability of SBS modified interlayer to retard reflection cracking.

In 2003 researchers decided to conduct a forensic investigation to determine the cause of the poor performance of the crack and seat sections and to gain additional information of the other sections.

Nondestructive Testing

In January 2003 both GPR and FWD data were collected on each experimental section, and dynamic cone penetrometer (DCP) data were collected on section R2.

Ground Penetrating Radar Data

The COLORMAP display from Section R1 is shown in Figure 7 together with a typical trace shown in Figure 8. In Figure 7 the depth scale in inches is at the right of the figure, and the distance scale in miles and feet is at the bottom. All of the pavement layers are visible. A surface removal technique has been used, and the pavement surface is at the top of the figure. The yellow line at approximately 2.5 to 3 inches below the surface is the bottom of the new HMA layer placed in 1997. The next yellow line at approximately 5 inches in depth is the top of the JCP. The line at approximately 13 to 14 inches deep is the bottom of the JCP. This reflection is strong indicating the presence of moisture beneath the slab.

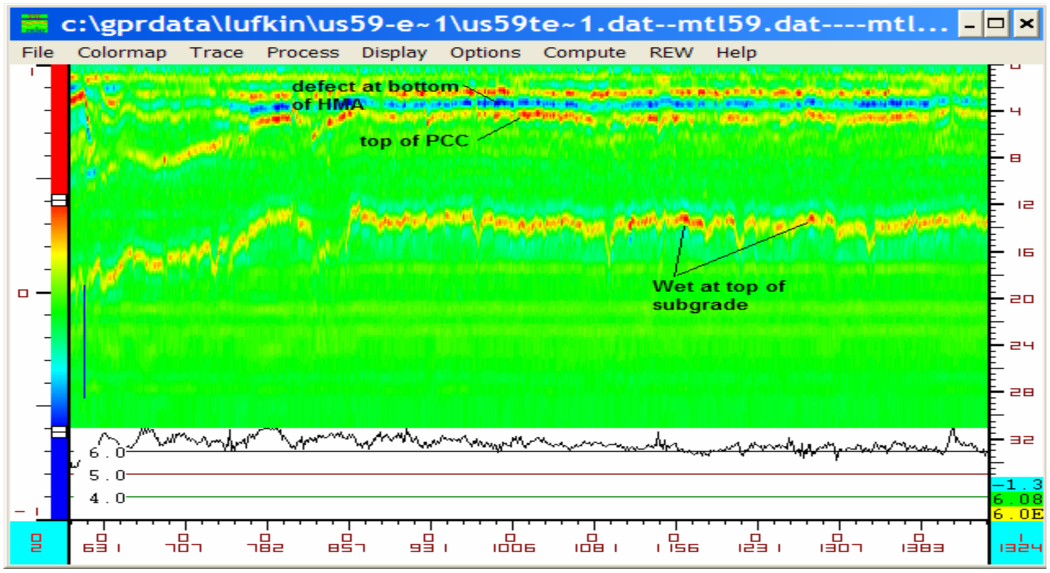


Figure 7. COLORMAP Display from Section R1.

Figure 8 shows an individual GPR reflection from section R1. The most significant feature of this trace is the large reflection from beneath the concrete. This reflection is an indication that the layer beneath the PCC is wet. The amplitude is higher than normally found from beneath concrete slabs.

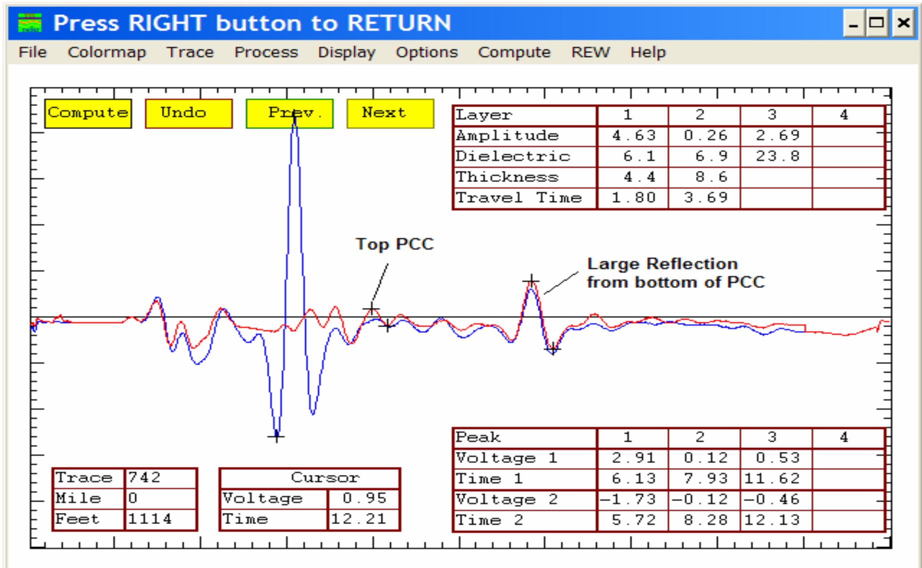


Figure 8. Individual Trace from Section R1.

Figure 9 below shows the COLORMAP plot for section R2. In the middle of section R2A (crack and seat plus 4 inches of HMA) the original pavement section has been replaced with full depth repair. This section consists of a thick cement treated base and 3 inches of HMA.

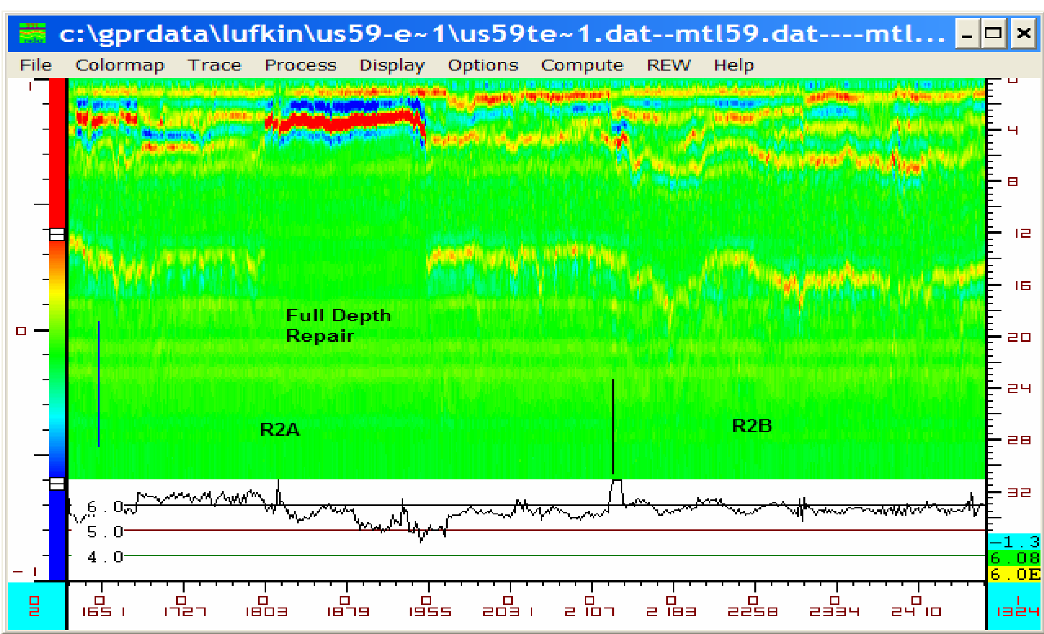


Figure 9. COLORMAP Display from Section R2.

In the original structure strong reflections are still found from beneath the concrete slab. The GPR data from section R3 (flexible base overlay) is shown in Figure 10. All layers in the pavement structure are clear in this figure. The surface is at the top of the figure. The strong red line approximately 4 inches down is from the top of the flexible base overlay. The blue-red-blue interface 12 to 16 inches down is from the bottom of the flexible base. This interface is complex as there was about 1 inch of original HMA left on top of the JCP before the flexible base was placed. The bottom of the JCP is the faint line towards the bottom of the figure. One observation here is that the average flexible base thickness was 8 inches, but that varied from around 10 inches at the beginning of the section to around 6 inches in the middle.

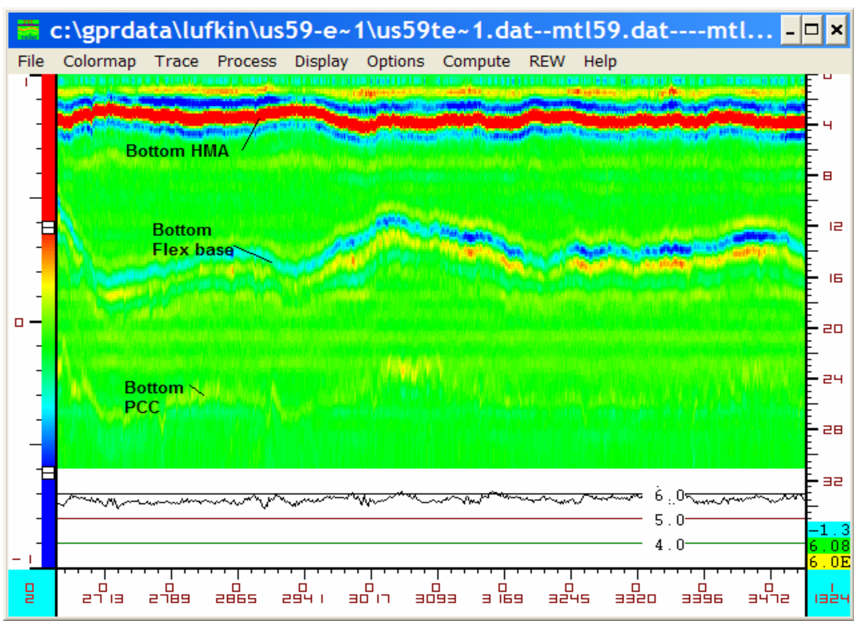


Figure 10. COLORMAP Display from Section R3.

Figure 11 shows the GPR data from section 4. This display is complex because there are many different layers within the HMA layer. The total HMA thickness is between 14 and 16 inches. The reflections from the bottom of the PCC are still visible but fainter than in the earlier figures. The complex nature of the reflections from section R4 is shown in Figure 12, which shows a single reflection. There are multiple reflections from within the structure, and it would have been difficult to estimate layer thickness without prior knowledge of the existing structure.

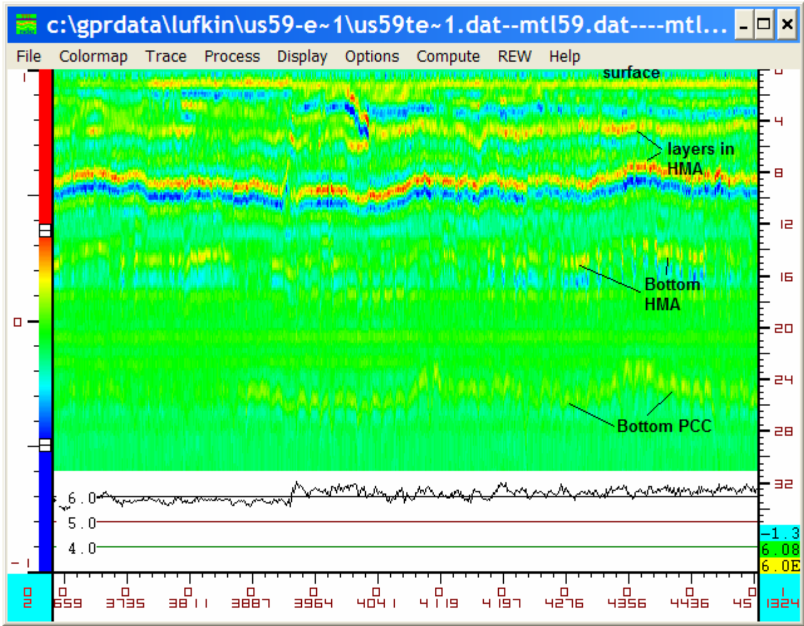


Figure 11. COLORMAP Display from Section R4.

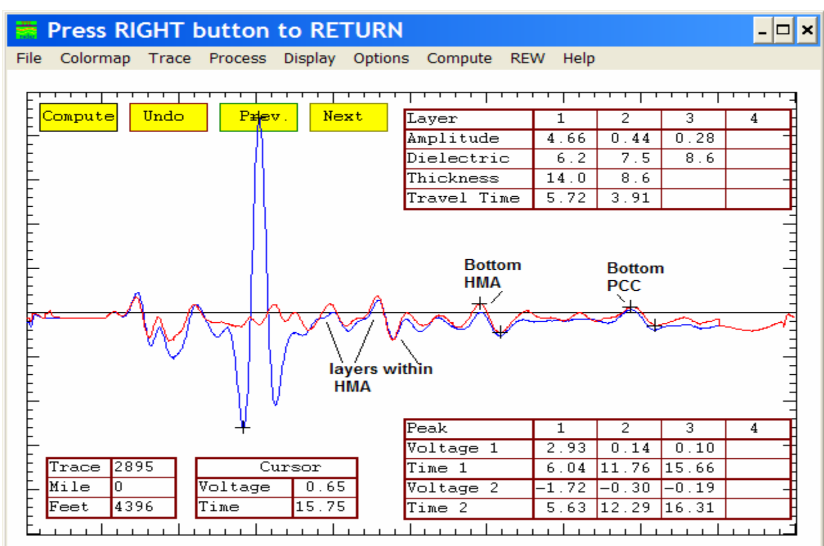


Figure 12. Typical Trace from Section R4.

Falling Weight Deflectometer Data

The MODULUS 6 outputs from the six experimental sections are given in Appendix A of this report. Table 2 shows the subgrade moduli values for each of the intact concrete sections.

Table 2. Subgrade Moduli from Lufkin Experiment.

Section	Average Subgrade Modulus (ksi)	% of section with a subgrade modulus less than 14 ksi
R1	16.2	43%
R3	17.1	18%
R4	27.8	0%
R5	21.8	0%
R6	22.7	0%

It must be recalled that sections R1 and R2 on US 59 are near the bottom of a hill, that section 3 is on the uphill portion and that section R4 is at the top of the hill. It appears that the subgrade support in <u>Section R1</u> is significantly less than the other sections.

One interesting feature from the FWD is the results from section R3, the flexible base overlay. The section is very uniform with an average deflection of 8.6 mils with a coefficient of variation of only 7 percent. The average backcalculated value for the flexible base material is 186 ksi, which is very high for an untreated material. But several issues must be remembered. Firstly, the base was measured to be 1 or 2 percent below optimum moisture content. Secondly, the base is resting on a solid concrete slab, which gives it excellent support and very high confining stresses. The lab results for the granular base will be reported later in this section.

Dynamic Cone Penetrometer Data

Several DCP tests were conducted in sections R1, R2 and R3 to evaluate the subgrade in these sections. The most important information is shown in Figure 13 and Table 3. From the data collected there is a 2 to 3 inch thick layer of weak material just beneath the slab in sections R1 and R2. The base material beneath the slab is unstabilized select material. The estimated DCP modulus for this material is in the mid 20 ksi range, however directly below the slab this drops to around 8 ksi. However, no significant weak layer was found beneath the slab on section R3.

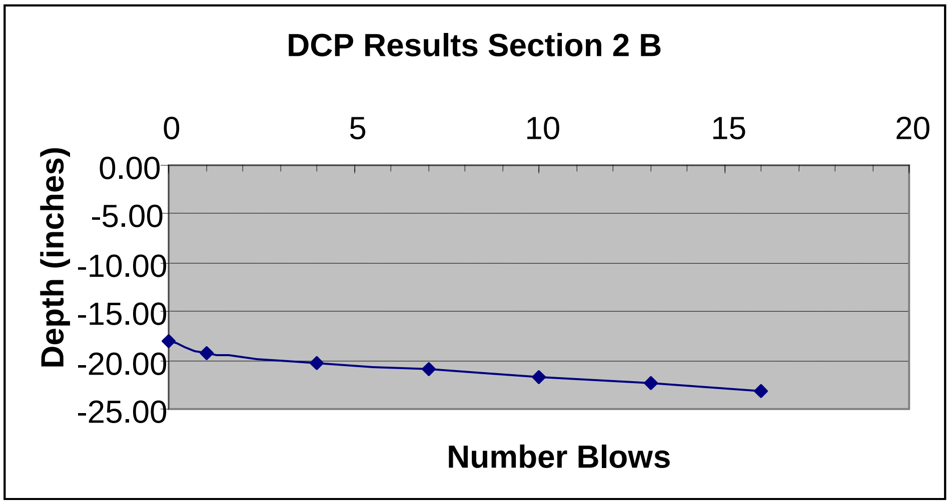


Figure 13. Dynamic Cone Penetrometer Data from Section R2.

Table 3. CBR and Moduli Values Estimated from DCP Data for Section R2.

	mm/blow	CBR	E mod
Just beneath Slab	32	6.020255	8.044401
Base	6	39.25125	26.70585

LABORATORY TESTING

In the summer of 2003, cores were removed from all sections of the experiment to validate the GPR data but also to conduct laboratory testing. Of specific interest were the materials used in section R3. Based on the discussion given above, the flexible base overlay was judged to have performed the best after the original surface was replaced in 1997. One major conclusion from this project is that the flexible base overlay option should be considered when selecting rehabilitation options for badly distressed JCP pavements. However, both the quality of the base and the use of an effective surface seal are critical issues. The original flexible base used on US 59 consisted of crushed limestone from Georgetown, Texas; it was specified as Texas Triaxial class 1 base material.

In order to further evaluate this material, researchers decided to sample material from the roadway and return this material to TTI for laboratory testing. The sampling operation is shown in Figure 14. The major engineering property of interest for the base material is its moisture susceptibility. This property was measured by a new TxDOT test (Draft method 144E), the Tube Suction Test (TST). The samples of the Lufkin material under test are shown in Figure 15.

Figure 14. Sampling Materials from Section R3 on US 59.

Figure 15. Samples of Base in TST Test.

Details of the tube suction procedure can be found elsewhere (*4*). In summary the test measures the capillarity of base samples compacted at optimal moisture content. The sample is dried back, and then moisture is introduced at the bottom of the sample. The top of the sample is monitored with a Percometer, which measures the surface dielectric. The dielectric is an electrical property, which is an indication of the amount of unbound moisture reaching the surface of the material. The test is run for 10 days, and the final moisture content and (asymptotic) dielectric is used to evaluate the moisture susceptibility of the base material. This test has been under study for about 10 years in both Texas and Finland. Based on field observations and validation tests conducted in the laboratory, interpretation criteria, shown in Table 4, have been proposed for this test.

The results for the Lufkin material are shown in Table 5 and Figure 16. The final dielectric for this material was found to be 12.5, which from the classification shown in Table 4 was judged to be "good." The final moisture content after 10 days capillary rise was found to be 6.8 percent, which is below the optimum moisture content of 7.8 percent. In summary, if flexible base overlays are to be used on future JCP projects in Texas the base material uses should have equal or better properties to the material, which performed well on US 59. This included:

- Class 1 Texas Triaxial class (after 10 days capillarity), and
- good classification in TST (TxDOT method 144 E).

Table 4. Classification of Base Materials Based on Tube Suction Test.

Final Dielectric	Classification
< 10	Excellent – no moisture-related problems
10-13	Good – typical of most Texas Class 1 aggregate. Should perform well except in very cold/wet climates.
13-16	Moderate – some concern about moisture problems. Consider chemical modification (low levels of cement or lime) if this is to be used on a high volume roadway.
16 +	Fair-Poor – moisture susceptible, consider for modification for all applications.

Table 5. TST Results from R3 Materials.

Sample	Asymptotic Dielectric Value, ϵ	Gravimetric Water content, W	%Water Loss in Drying	Actual Density (dry)	Actual Compaction Moisture %	Target Dry Density	Target Compaction Moisture %
US 59	12.5	6.8	43.9	133.5	7.8	137.6	7.8

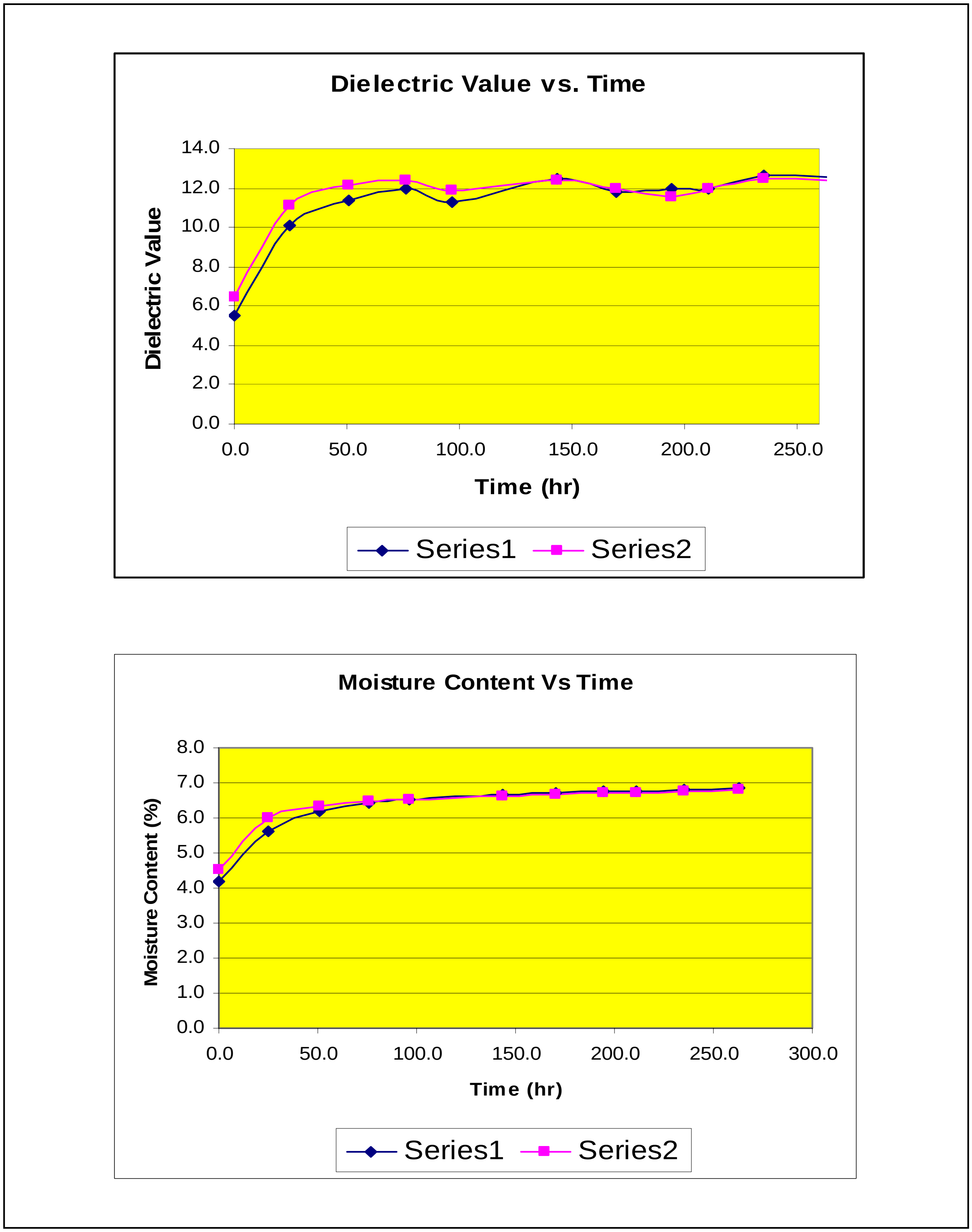


Figure 16. Detailed Results from the Tube Suction Test on the Section R3 Base Material.

CONCLUSIONS FROM THE LUFKIN EXPERIMENT

The performance of the Lufkin experiment is influenced by two external factors. Firstly, sections R1 and R2 were constructed towards the bottom of a hill, and the subgrade support in this area was worse than that found in sections R3 and R4. Secondly, the initial variable performance of the sections was largely attributed to the poor quality of the surfacing layer. With these factors in mind the following conclusions are offered for the Lufkin experiment:

- The best performing section was the flexible base overlay. This strategy should be considered as a rehabilitation option for other badly distressed JCP pavements in Texas. This is also one of the lowest cost strategies.
- The flexible base overlay will work only if a) material of equivalent or better properties (Triaxial class and moisture susceptibility) is used and b) the base layer can be effectively sealed from moisture intrusion from the surface.
- The Arkansas mix (old TxDOT Type G) can give good performance.
- Crack and seat should not be used if a weak layer exists beneath the slab. On US 59 the original slab was an old 9-6-9 design with an untreated select material support layer. From the data presented, “weak” would be in terms of the DCP as penetrations of more than 1 inch per blow and in terms of the FWD with a backcalculated subgrade modulus of less than 15 ksi.
- Crack and seat should not be attempted if moisture is observed beneath the slab unless the moisture can be drained. Subsurface moisture can be detected with ground penetrating radar testing.
- The poor performance of section R1 (full depth repair, restoring load transfer, etc.), which was the most expensive rehabilitation option, could not be explained. The only partial explanation was that this was in the area of worst subgrade support.

CHAPTER 3
DOWEL BAR RETROFIT

INTRODUCTION

Dowel bar retrofit (DBR) is currently being considered as a rehabilitation technique for jointed concrete pavements within Texas. However, there is currently very limited experience and no performance data from this practice within the state of Texas. DBR has been used extensively around the US, and this information has been summarized to provide a general status report on this rehabilitation technique. This report provides some background to DBR; provides a brief description of the method application and current level of adoption; provides insights into some of the issues and problems related to DBR; provides feedback on performance results; gives an indication of the rehabilitation cost; and provides some insight into the potential for the application of the rehabilitation technique in Texas.

BACKGROUND TO DBR

New jointed concrete pavements contain dowel bars across the joints for load transfer. Prior to the 1980s the importance and cost-effectiveness of dowel bars for load transfer were not fully understood, and dowel bars were not installed. As a result, older pavements cannot transfer load effectively between slabs, which results in faulting, and which in turn results in a poor quality of ride.

Dowel bar retrofit is a technique used to rehabilitate concrete pavements where faulting is a problem, but the pavement is in otherwise good condition. Slots are cut into the pavement over existing joints and transverse cracks, dowels are placed in the slots at mid-pavement depth, and the slots are backfilled with patch material. Later the pavement is diamond ground so the tops of the slabs are flush with each other. The objective of DBR is to restore the ride quality and to extend the pavement life (*8*).

DESCRIPTION

A brief description of the general construction process is provided below (9):

1) Saw cuts: Using a gang saw, make saw cuts across joint or transverse crack parallel to centerline.
2) Clear concrete: Using a 30 lb jackhammer, clear concrete between saw cuts and scrape debris away with pickaxes. Sandblast slots and then blow out sand and dust with compressed air.
3) Prepare dowel bars: Apply bond breaker and fit foam board, chairs and end caps.
4) Place dowel bars: Caulk slots and set dowel bar in place with foam board in line with joint or crack and resting on bead of caulk. Apply bead of caulk to each side of foam board to ensure no patching material seeps into joint or crack.
5) Calibrate mobile mixer.
6) Pour patch material: Clean slots with leaf blower, wet down with water using a pressure sprayer and backfill the patch material into the slots with shovels taking care not to damage the foam board. Consolidate the patch material with a vibrator, smooth off with trowels and apply concrete curing compound.
7) Finish joint: Saw cut to restore transverse joints or cracks, diamond grind to restore riding quality and seal joints.

A schematic illustration of the dowel bar placement is shown in Figure 17 (9).

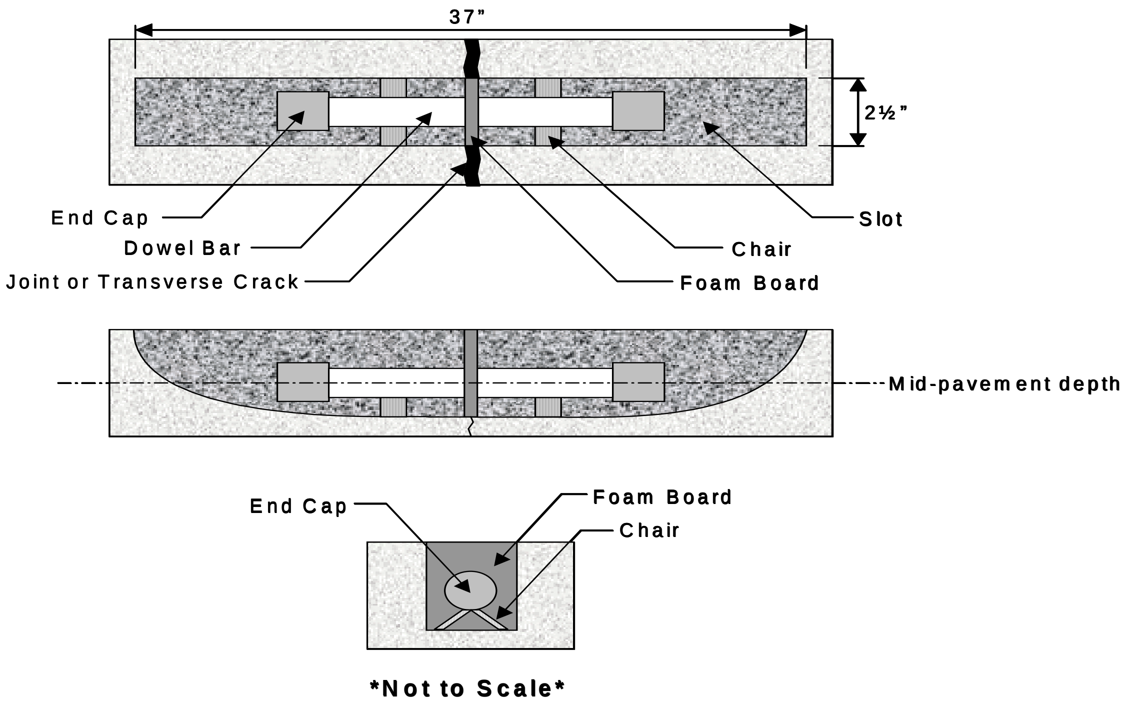


Figure 17. Schematic Illustration of Dowel Bar Placement.

An indication of the current level of adoption of the DBR technique was obtained from the International Grooving and Grinding Association's DBR project database (9). The number of placed bars in various states is illustrated schematically in Figure 18.

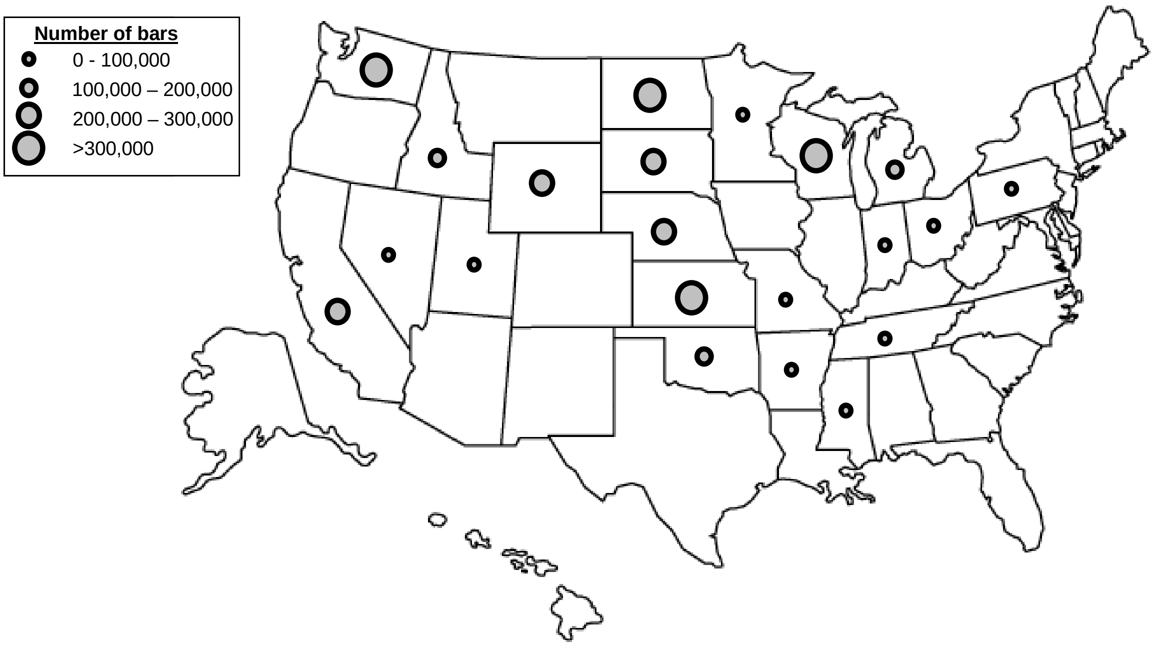


Figure 18. Illustration of the Level of DBR Adoption in the United States.

DBR PERFORMANCE

To get an indication of the issues related to DBR and the performance of the technique, reports from studies undertaken by the Wisconsin Department of Transportation (WisDOT), Washington State Department of Transportation (WSDOT) and the University of California, Davis were reviewed. Relevant factors from the studies were identified and are discussed in a brief format below.

WisDOT STUDY

WisDOT conducted a study to evaluate if DBR is a viable and effective concrete rehabilitation technique and to refine material and construction specifications.

Construction Complications

A brief summary of the main construction complications and a performance evaluation one year later is given below (*8*).

- The wearing away of the concrete slab interfaces at joints creates gaps that need to be caulked to prevent mortar intended to fill the slot from entering these gaps.

- Severe deterioration of the concrete pavement at the base of the slots was noticed. This deterioration appeared to be bottom up deterioration of the slab and was worse in areas of low elevation and near the outside of the roadway. An effective way of filling these voids or repairing this deterioration was not found. Guidance is needed on how to deal with voids and how to maintain the joints in the slots with voids.
- The cutting of slots in areas where pavement sections differ in thickness, such as intersections, creates problems. It was recommended that ground penetrating radar or cores be used to determine pavement thickness where differing pavement thickness is suspected.
- Problems with mixing a "single component" mortar mix, with sand and cement premixed, were found with different methods of mixing. A mobile mixer did not provide satisfactory results, and a paddle mixer had low productivity.
- Poor consolidation of the patch mixture around the dowels and poor bonding to the sidewalls were encountered. The slump requirements and the cement type were considered to be influencing factors. It is suspected that the debonding was a result of drying, plastic and chemical shrinkage.
- Problems with bars of inconsistent diameter, where end caps were either too tight or too loose, were encountered.
- Poor workmanship in foam board alignment and saw cutting for joint restoration were found and created unnecessary problems.

Field Performance (One Year Later)

The following observations and recommendations were made one year after placement.

- Some debonding of the patch material from the sidewall and microcracks at the interface were evident. Alternative methods to alleviate shrinkage, such as different patching materials, should be investigated.
- Cracks from the outside dowel slots to the outside edge of the pavement were found. The cracks occurred primarily on the upstream side of joints and seemed to propagate from areas of debonding. Current thinking is that the debonding resulted in the "isolation" of the slab edge and resulted in greater susceptibility to fatigue damage. It was

recommended that a minimum distance to the outer edge of the pavement be included in specifications. Cracks also extended from the dowel bar slots to the ends of full depth repair saw cuts.

WSDOT STUDY

WSDOT conducted a study to determine the appropriateness of DBR and diamond grinding to restore the functionality of the pavement and provide a smooth riding surface. A brief summary of the main findings is given below (*8*):

- DBR sections showed better performance than control sections and tied shoulder only sections.
- Based on current evaluations, it is estimated that DBR will extend the pavement life by 10-15 years.
- Performance equations relating truck volumes to faulting are in the process of being developed to aid in prediction.
- Decision trees, such as the one shown in Figure 19, are being considered.

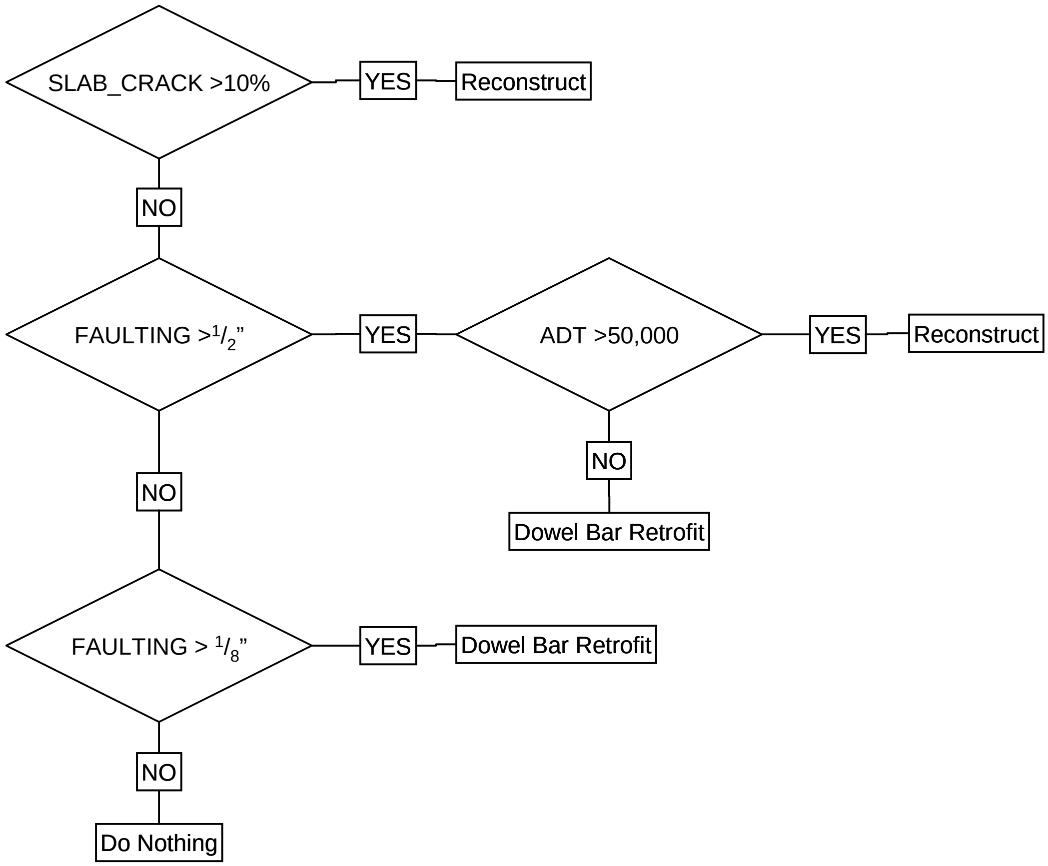


Figure 19. Example of a Decision Tree for DBR.

DBR PERFORMANCE UNDER HVS LOADING STUDY

The University of California, Davis undertook a study of DBR performance under the action of a heavy vehicle simulator (HVS). The primary objective was to determine whether DBR provides adequate performance relative to its cost and other rehabilitation alternatives (*11*). A brief summary of the main findings is given below:

- DBR provides very good load transfer efficiency (LTE), reduced deflections, and reduced deflection differences between slabs.
- No cracking or other failure of the DBR or new slab cracking was evident after the application of 11,000,000 ESALS with the HVS.

- LTE was heavily reduced on the control section, but not on the retrofitted sections, although some damage in terms of maximum deflections and deflection differences was evident on the retrofitted sections.
- It was not clear why damage in terms of deflection was evident on the retrofitted sections, but not in terms of LTE.
- Daily air temperature fluctuations had a significant effect on the LTE and deflection measurements.

Cost

WSDOT refers to a cost estimate of $320,000 per lane mile, including all costs, which may be used as a first order estimate (*10*). Further comment is that the DBR is cheaper than an overlay, and not all the lanes have to be treated.

Opportunities for DBR Adoption in Texas

Based on the review of the studies above, the following comments are made:

- Although the performance of DBR has not yet been determined with certainty and a life cycle cost analysis has not yet been undertaken, it is clear the DBR does improve load transfer efficiency between slabs and does have the potential to increase the life of the pavement.
- Potential problems related to the construction process and materials do exist and should be appropriately addressed by means of specifications.
- Decision trees and performance curves have not yet been developed, but it does appear as if steps are being taken in that direction.

Based on the TxDOT pavement management information system (PMIS), there are approximately 4000 lane miles of jointed concrete pavements in Texas, representing approximately 2 percent of the state's road network (*12*). Although the review does not give definite answers, it does indicate the significant potential for testing and developing DBR as a means of rehabilitation for jointed concrete pavements in Texas.

CHAPTER 4
PERFORMANCE OF CRACK RETARDING LAYERS

Over the last 40 years a whole range of products have been evaluated to attempt to reduce the occurrence of reflection cracking on Texas highways. Numerous research studies and field trials have been conducted but no single product has gained widespread acceptance. In recent years two proprietary products have been tested in several experimental sections around Texas. These are Strata® from Koch Industries and GlasGrid® from Bay Mills Industries. In this section, it is intended to review the field performance of these two proprietary products.

The two products are described below; this information was extracted largely from the manufacturers' brochures.

Strata® is a highly elastic, fine graded, polymer modified mixture. The aggregates are crushed and natural sands with 80 to 100 passing the No. 4 sieve. Strata® is an impermeable hot mix reflective crack relief interlayer that is designed to retard the reflective cracking in the AC pavements. In applications involving jointed concrete pavements in Texas, a thin level-up HMA layer is placed to provide a uniform level support layer. After this layer, the Strata® interlayer is applied. The surface is then compacted suitably by using different types of rollers. The appearance of the reflective crack relief interlayer after final rolling is black in color. The surface texture is tight. Proper inspection is done to see that flushing has not occurred. On major highways it is recommended that the Strata® layer be covered with between 2 or 3 inches of traditional HMA surfacing.

GlasGrid® is a self-adhesive reinforcing mesh used as a stress relief interlayer to retard the reflective cracking in the new asphalt overlays on the older paved surfaces. It consists of high tensile strength fiberglass strands arranged in a grid structure and covered with a polymer coating and a pressure sensitive adhesive. On jointed concrete pavements a thin 1-inch thick level-up layer is placed to provide a uniform support and to bridge over existing cracks and joints. Then the GlasGrid® is placed on the surface by mechanical means. The manufacturer does not recommend a tack coat because the grid is self adhesive. The final surface is recommended to be at least 2 inches of traditional HMA.

With both of these products the manufacturer's representatives are present during placement.

CRACK RETARDING OVERLAY (STRATA®)

The Strata® product has been evaluated on a variety of highways in Texas. Strata® is a proprietary product manufactured by Koch Materials. Most of the sections are relatively new with one of the oldest being the SH 3 section in the Galveston Area Office of the Houston District. The contact person for this project is Bill Babington, the assistant area engineer. The original structure was an old jointed concrete pavement with 4 to 5 inches of asphalt overlay. The section had extensive reflection cracking. In late 2001, most of the original hot mix was removed. After milling, 1 inch of existing hot mix asphalt was left on top of the PCC slab. Three experimental sections were then placed on this highway. The first consisted of a 1 inch layer of Strata® followed by 2 inches of Type C mix with a PG grade 76-22 binder. A second section consisting of a Petromat fabric and the same 2 inch surfacing was placed in the lane adjacent to the Strata® section. The third section was the control section, which simply received the 2 inch surfacing course.

Pavement inspection and sampling were conducted after one year in service. Figures 20 and 21 show the condition of the experimental and control sites. Neither the Strata® or Petromat section had any reflection cracks after one year, but most of the reflective cracks were present in the control section. The left lane has the Strata® whereas the right lane has the Petromat. After 1 year in service no cracks were observed in either lane.

Figure 20. SH 3 Houston District.

Figure 21. Reflection Cracks in the Control Section after One Year.

In the second year of Project 0-4517, researchers proposed to continue monitoring this section. Samples of the Strata® and Type C material were removed from the pavement and returned to TTI for testing in the overlay tester. This device, which simulates thermal stress in hot mix overlays and a core with the different layers, is shown in Figure 22. The upgraded overlay tester has been developed at TTI in Project 0-4467. In the test the hot mix sample is glued to split test plates. One plate is fixed, and the other is cycled back and forward at a slow rate. The SH 3 materials were tested at the standard test condition of room temperature (77 ° F), crack opening 0.025 inches and loading cycle 10 seconds. The crack opening is intended to simulate the anticipated movements for a concrete slab with 15 ft joint spacings experiencing a 30 degree temperature variation.

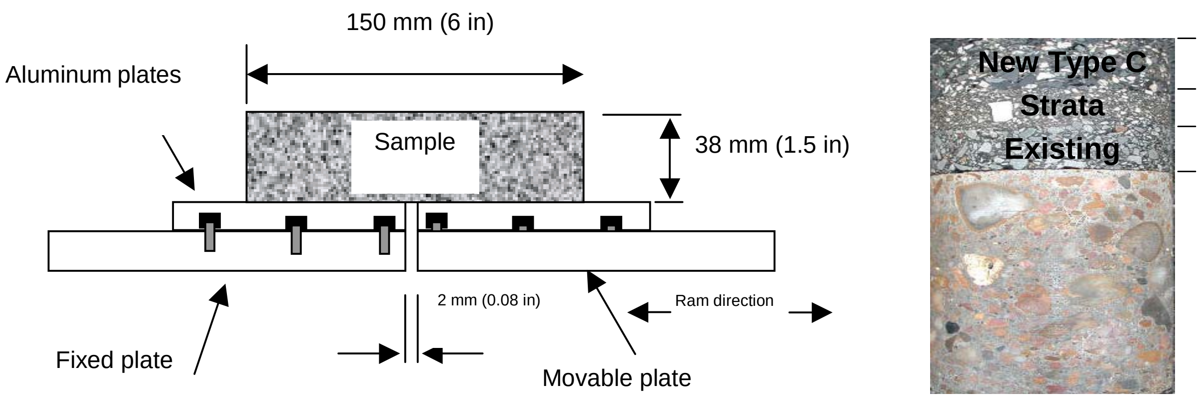


Figure 22. TTI's Upgraded Overlay Tester and Core Showing Different Layers.

The results obtained for the two mixes used on SH 3 are shown in Figure 23. The Type C material failed after 30 cycles, whereas the Strata® material lasted more than 700 cycles. The test was stopped after 700 cycles, and the Strata® material had not cracked. These results are to be compared with the results obtained on a good performing latex modified mix from the Dallas District. The mix on US 175 was placed over 10 years ago on a cracked cement treated base, and after 10 years of service very few cracks were found in the surface. From the overlay tester results the Strata® material has superior crack retarding properties. Similar tests were conducted at a reduced temperature of 50 ° F, and the Strata® material still performed very well.

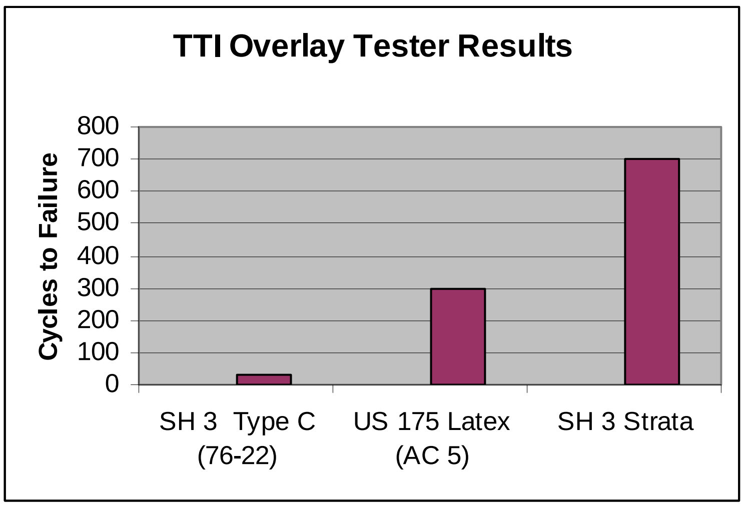


Figure 23. Overlay Tester Results from SH 3 Materials (Average of Two Tests).

In order to further evaluate the Strata® product, samples were taken from a new project in the Dallas District, and these were returned to TTI for both reflection cracking and rut potential measurements. To evaluate the rutting potential of Strata®, samples were compacted to densities similar to those specified in the field and subjected to the asphalt pavement analyzer (APA) rutting test. The results are shown in Figures 24 and 25. This test is run at 64 ° C, and the acceptance criteria recommended is that the rut depth should be less than 8 mm after 8000 load repetitions.

As shown in Figure 24 the Strata® material did poorly in the rutting test. The deformations shown in the specimen after less than 3000 load repetitions are shown in Figure 25. In Figure 24 the Strata® performance is compared to the following three mixes:

- Two lab molded samples are coarse matrix high binder (CMHB) mixes containing asphalt rubber. The mixes have good reflective cracking resistance and good rutting resistance.
- One field mix was from field cores obtained from IH 20 on a mix that rutted badly after only two days in service. It was found that this mix was placed with more asphalt than designed.

These data show the Strata® has a very similar rutting curve to the failed Texas mix. The results raise a concern that it may be a problem to drive on the Strata® product for extended periods during construction; this would be a concern if the construction was occurring in the middle of a Texas summer.

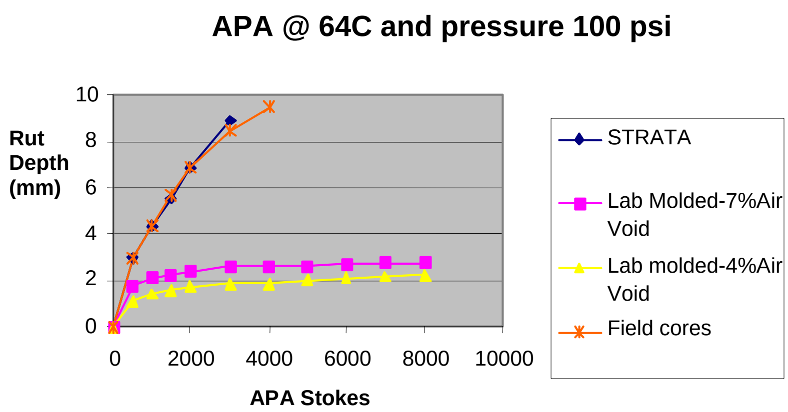


Figure 24. APA Results Evaluating the Rutting Potential of the Strata® Material.

Figure 25. Strata® Samples after 300 Load Repetitions.

The test results presented above show both the strengths and the weaknesses of the Strata® system. Firstly, it demonstrates superior resistance to reflection cracks. This material should perform excellently in the field. The weakness is that it is a soft mix with a flushed appearance. The Houston District expressed concerns about its skid resistance in wet conditions. In general traffic should not drive on the mix for extended periods during construction. The manufacturer recommends no more than five days, but that could be a problem in mid summer in Texas. Houston placed the overlay on top of Strata® within 72 hrs. The potential for the mix to rut also governs the amount of cover material required in high traffic load situations. For example the Strata® manufacturer material usually recommends a minimum of 3 inches of HMA over the Strata®. That means that the minimum application thickness will be on the order of 4 inches.

Further research should be conducted to develop a crack retarding material with a better blend of both rutting and crack resistance properties. This is because many of the JCPs and other cracked concrete pavements are in urban areas where the ideal thickness of the asphalt overlay should be less than 2 inches to avoid other geometric problems. To meet the height restrictions, it would be desirable to sacrifice some resistance to reflection cracking to gain more rutting resistance.

GRID PRODUCTS (GLASGRID®)

GlasGrid® is a proprietary product manufactured by Industrial Fabrics Inc. A limited literature search was conducted to evaluate the performance of GlasGrid® reinforcement outside of Texas. The results obtained were not too encouraging. In one documented study by New York DOT both the "general use" (full width treatment) and "single strip" (joint repair) were placed on US 20 in October 1993 (*5*). The report concluded that after four years in service there was no statistical difference in performance between the control and treated sections.

A series of case studies conducted in Texas with the GlasGrid® product are discussed in the following sections.

Case Study 1 Lufkin District US 59 (Contact: Cheryl Flood, Area Engineer)

In November 1999 a 1000 ft section of GlasGrid® was placed on the northbound lanes of US 59 on control/section/job 0176-03-110. A 1 inch leveling course was placed, followed by the

GlasGrid®, followed by a 2 inch CMHB level up. The only problem noted by Ron Evers, the district pavement engineer at the time, was that because of the low (45 °F) air temperature during construction, the GlasGrid® did not adhere well to the leveling course at the beginning of the operation. A control section was placed adjacent to the grid section with 4 inches of HMA without the grid. Visual inspections were first conducted in August 2002. At that time all of the cracks had reflected through the control section, and the first reflection cracks were observed in the grid section. This is shown in Figure 26 where the reflection cracks are clear in the control section. The cracks were observed in about 20 percent of the joints in the GlasGrid® section but they were less severe than the control section. Ron Evers commented that the grid had retarded the reflection cracking by about 1 year.

Cores were taken from the GlasGrid® site in July 2003. At that time approximately 60 percent of the cracks had reflected through the grid. The cores are shown in Figure 27. In all cores taken it was clear that the grid did not bond to the lower HMA (level-up) layer. This entire project was resurfaced in late 2003.

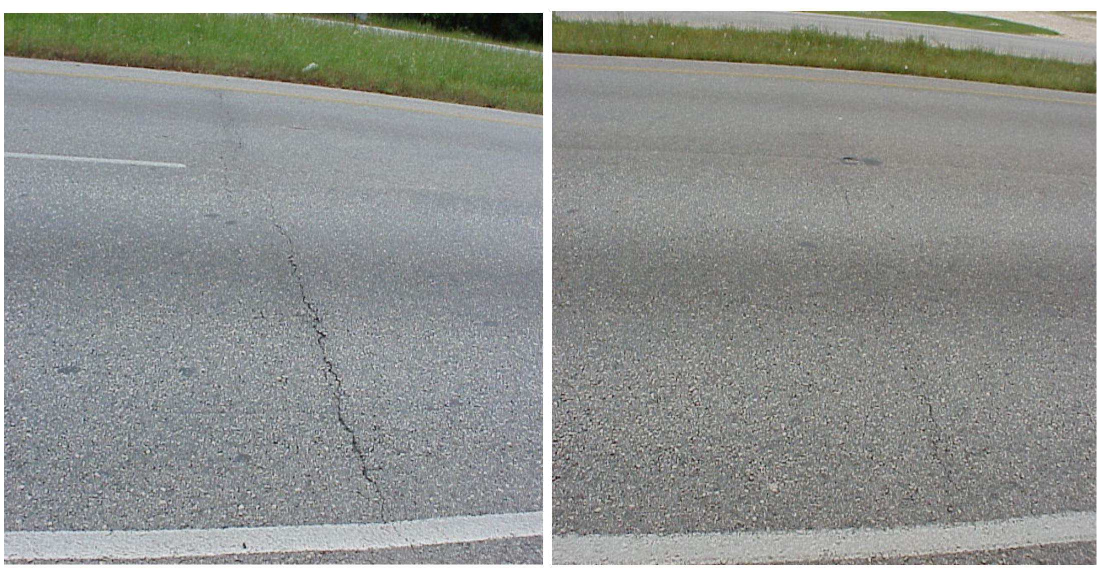


Figure 26. US 59 GlasGrid® Evaluation Site.

Figure 27. Cores Removed from the Lufkin Section on US 59.

Case Study 2 Houston District (Contact: Charles Gaskin, District Construction Engineer)

The Houston District placed GlasGrid® on two heavily trafficked sections. The first section was on FM 2920 from IH 45 to the Tomball city limit. The second was a section of IH 45 just north of Conroe.

FM 2920 Comparison of GlasGrid® and Petromat

In October 1998, an 11 mile section on FM 2920 was resurfaced. The existing structure had a cement treated base and 3 inches of asphalt. Block cracks were observed in the existing pavement structure. In resurfacing the first 2.5 miles, a GlasGrid® interlayer and a 1.5 inch (40 mm) overlay were placed. In the rest of the section the Petromat fabric was placed followed by the same 1.5 inch overlay.

A condition survey was completed in May 2003. At that time both sections had substantial reflection cracking, but the condition of the Petromat section was much better than the GlasGrid® section. The GlasGrid® section is shown in Figures 28 and 29; longitudinal and transverse cracks were present but there was also severe alligator cracking in both wheel paths. In some areas pop-outs were occurring, and the top layer of asphalt was breaking loose. The section was also rough to drive. The typical condition of the Petromat section is shown in Figure 30. The Petromat did not significantly delay the reflection cracks, but the overall condition of the section was reasonable and the ride was still good.

Figure 28. GlasGrid® Section on FM 2920 after Four Years of Service.

Figure 29. GlasGrid® Section on FM 2920 – Note Deterioration in Wheel Paths.

Figure 30. Petromat Section on FM 2920 after Four Years of Service.

To further evaluate the performance, ground penetrating radar data were collected on FM 2920. The GPR display from the transition from the GlasGrid® to the Petromat section is shown in Figure 31. The key feature of this figure is the strong reflection from the location of the grid. This red/blue reflection indicates that there is a high moisture interface within the HMA in the GlasGrid® section where none was found in the Petromat section. The moisture buildup could be at the bottom of the surface layer or in a small void left by the grid. More work is needed to identify the cause of this reflection. The interface in the Petromat section is judged as ideal, indicating no discontinuities and good bonding between layers.

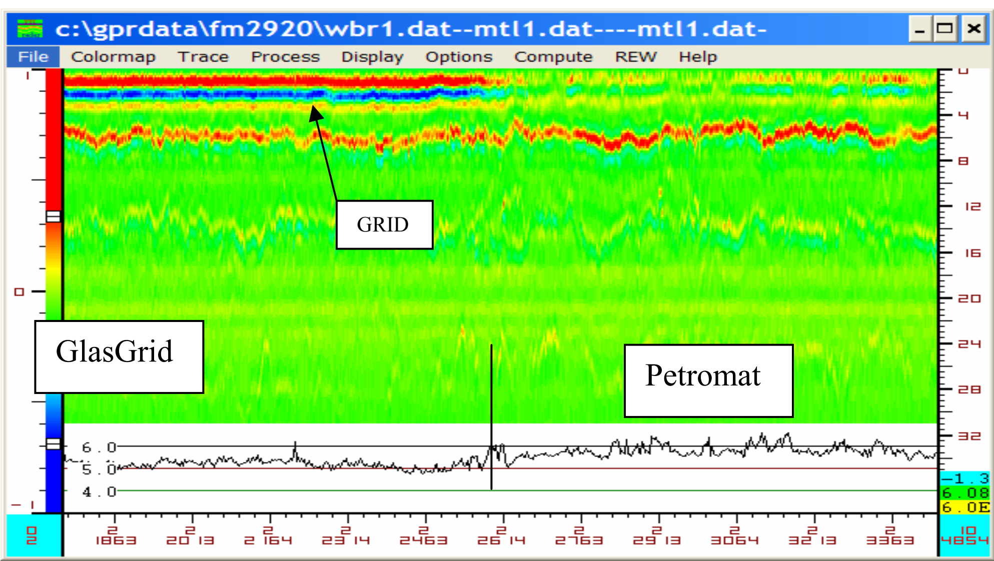


Figure 31. GPR Data from FM 2920 Houston from the Transition between GlasGrid and Petromat Sections.

IH 45 North of Conroe

In the summer of 1999, a second GlasGrid® section was placed by the Houston District on IH 45 just north of Conroe. The existing pavement was jointed concrete. The existing HMA was milled and a level-up layer; full width GlasGrid® and thin HMA surface layer was placed. In all of this work and in other installations in Texas a representative from the grid manufacturers was present during grid placement.

The IH 45 section performed very poorly. The first indication of problems was the occurrence of white powder in the wheel paths. This was followed by localized pop-outs of the surface layer. A photograph of the surface condition is shown in Figure 32. No detailed evaluation was possible as the section had to be replaced within six months of being constructed. The Conroe area office laboratory cored the failed section and noted in all areas that the grid caused debonding with the surface layer.

Figure 32. Failure of the GlasGrid® Section on IH 45, Houston District (1999).

Just prior to replacement a ground penetrating radar survey was conducted on the failed section. Figure 33 shows the GPR results.

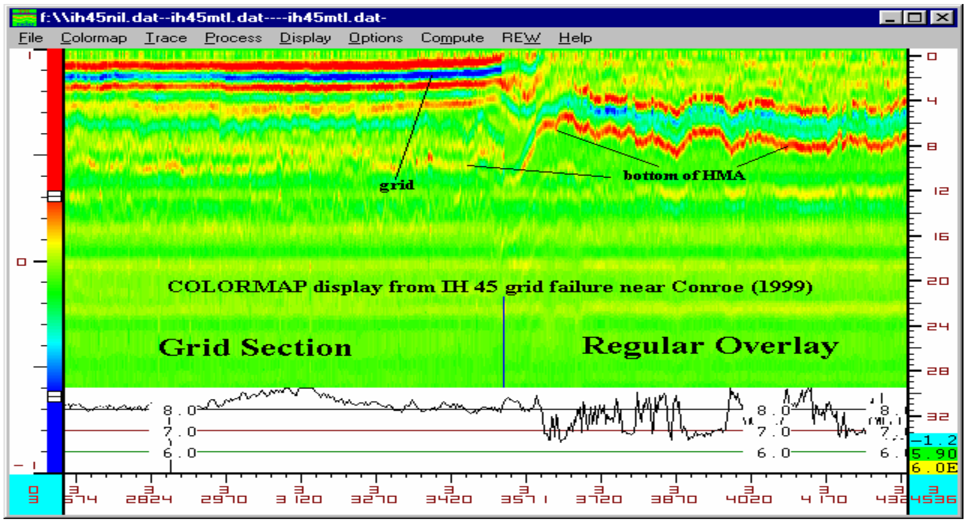


Figure 33. GPR Results from the GlasGrid® Failure on IH 45 (1999).

A limited forensic investigation was conducted by district staff. In some localized areas it was noted that the surface thickness was slightly less than the recommended 1.5 inches. Some field cores were found to be 1.3 to 1.4 inches thick. No problems were reported with the asphalt mixture or with field construction. The section on IH 45 failed very dramatically and had to be replaced with an emergency contract, therefore no additional testing was possible. It is difficult to believe that the magnitude of the failure on IH 45 was attributed to small differences in surface mat thickness.

Case Study 3 IH 45, Bryan District (Contact: Darlene Goehl, District Lab Engineer)

In late 2001, a 6 mile section of GlasGrid® was placed on IH 45 between mileposts 112 and 118. The construction included removing the old HMA down to the concrete, doing localized full depth repairs to the concrete. A 2 inch type C level-up layer was placed over the repaired PCC. In this application the GlasGrid® was placed in 5 foot wide strips over only the transverse joints. This application used the "GlasGrid® 8502" product, which has improved tensile strength in the transverse direction over the traditional 8501 product used in Houston and Lufkin. In this application an emulsion tack coat was applied prior to placing the GlasGrid® and a 2 inch layer of CMHB-C was placed as the final surface.

This project did not perform well. It started to demonstrate many of the same failure mechanisms as the failure on IH 45 in the Houston District. The first indication of problems was fine white powder in the wheel paths as shown in Figure 34. This was later found to be fiberglass indicating that the grid was disintegrating under the action of traffic.

Figure 34. Initial Indications of Problems on GlasGrid® Installation on IH 45 in the Bryan District.

TxDOT's forensic team conducted field coring and a full forensic investigation. This testing included nondestructive testing, field coring, lab testing and a complete review of observations during construction. The coring and field trenching observations are shown in Figure 35. It was clear the layers of HMA were debonding at the GlasGrid® interface.

Figure 35. IH 45 Field Cores and Trench from the Failed GlasGrid® Section.

The ground penetrating radar data collected on the GlasGrid® section of IH 45 in the Bryan District is shown in Figure 36. It must be recalled that the grid was placed in 5 foot wide strips, and that the joints were every 15 ft. Figure 36 shows about 700 ft of the section. The distance scale in miles and feet is at the bottom of the figure. From the GPR there is a clear signature from each strip of grid. This is the red/blue interface at a depth of approximately 2 inches (depth scale on right of figure). This reflection shows that there is a discontinuity in the mat at each grid location. If the two asphalt layers were fully bonded together, only minor reflections would be observed at the interface between the CMHB and Type C mix. The GPR signature is similar to that observed in the Houston sections. The pattern indicates the presence of moisture at the level of the grid. This pattern is most probably associated with debonding of the HMA layers at grid, allowing moisture to concentrate at this level.

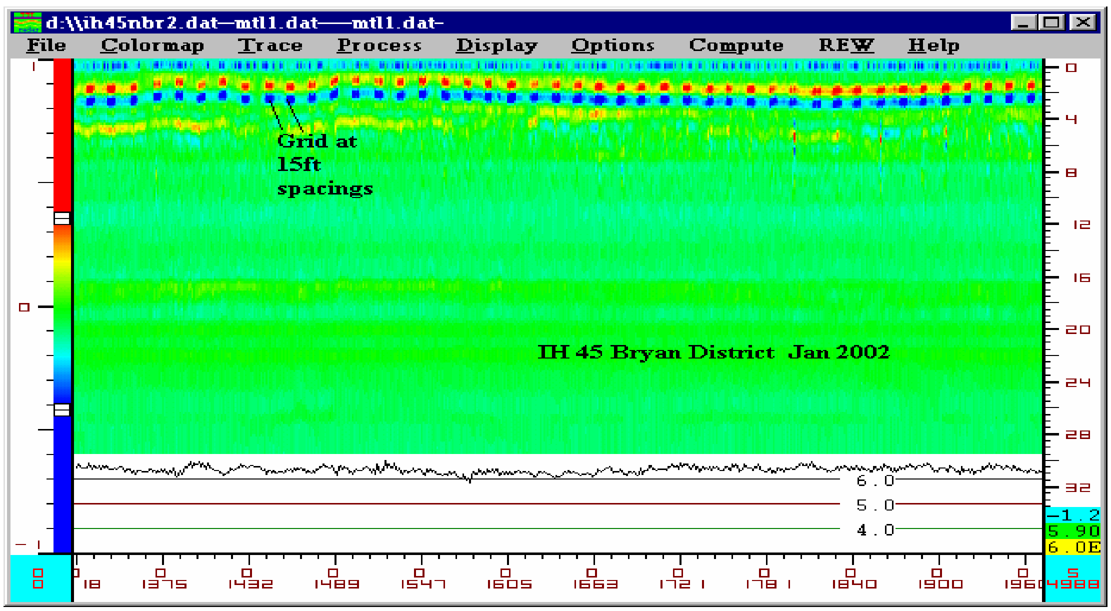


Figure 36. GPR Data Collected on the GlasGrid® Section on IH 45.

In conclusion the failure on IH 45 was attributed to one or more of the following factors:

- the placement of the GlasGrid® on an emulsion based tack coat (SS1) before the emulsion had fully broken;
- rainfall during construction, before the grid was covered;
- debonding during construction (the TxDOT field inspector also observed a wave in front to the roller and a small "bump" as the roller went over each grid strip); and
- the fact that each strip of grid may not be directly over each joint.

COMPARING GRID AND MODIFIED ASPHALTS

US 96 near Lumberton in the Beaumont District is the only site in Texas that provides a direct comparison of different approaches to minimize reflection cracking. The existing section has a cement treated base. In the early 1990s, the following three overlay test sections were constructed: a) crumb rubber modified asphalt overlay, b) GlasGrid® with asphalt overlay and c) a control section with standard Type C overlay. The condition of the section was monitored by Dr. Chen from TxDOT. Photographs of the pavement condition are shown in Figure 37. Based on discussion with Jack Moser, the area engineer, the conclusions after eight years in service were the following:

- The crumb rubber section was performing best, and after eight years showed very few reflection cracks.
- With regard to reflection cracking, the GlasGrid® section was performing about the same as the traditional asphalt overlay.

RECENT DEVELOPMENTS

The use of Strata® and GlasGrid® has continued around Texas, and as of January 2004 several additional sections are now in place.

Application of GlasGrid® in the Dallas District (Contact: Maurice Pittman, Dallas)

Since 2001 the Dallas District has placed four sections of GlasGrid® on major highways. These sections include:

- IH 35E from IH 635 to Sandy Lake Road (2 miles),
- SH 183 from Loop 12 to the Tarrant County Line,
- Loop 12 from US 75 to Lawther,
- US 80 from Terrell east for 3 miles, and
- US 175 from IH 635 east for 4 miles.

a) Crumb Rubber after 8 Years, Excellent condition

b) Transition from Crumb Rubber to GlasGrid® Section

c) GlasGrid® Significant Longitudinal Cracking and Some Transverse

d) Control Section Regular Transverse Cracking

Figure 37. Pavement Condition of US 96 after Eight Years in Service.

SH 183 also contains sections where both the large opening (8511) and small opening grid were placed. All of these sections had a 2 inch surfacing over the grid with approximately a 1 inch level up. In the first three projects the grid was placed without a tack coat as recommended by the manufacturer, but on the US 80 project a tack coat was used to hopefully minimize the potential for debonding between layers. After one or two years in service, all of these sections appear to be performing well. Monitoring of the section was initiated in 2003, and this will continue in the second year of Project 0-4517.

US 377 Tarrant County Fort Worth (Contact: Richard Williammee or Andrew Wimsatt, Fort Worth)

In mid 2003 a short 0.5 mile demonstration section of GlasGrid® was placed on US 377. In the remainder of the project a Petromat grid was used, which is standard practice in the district. Very soon after placement, major delaminations occurred in the GlasGrid® section. This entire section was replaced less than three months after placement. Lab engineer Richard Williammee can provide addition information on this failure.

US 175 Dallas District (Contact: Gary Moonshower, Dallas)

In 2003 a major experiment was placed on roughly a 4 mile section of US 175 just south of the intersection with IH 635. This was an old jointed concrete pavement with an array of joint spacings from 15 to 40 ft. The section was over 40 years old and had performed reasonably well. However, due to the increasing maintenance cost, TxDOT personnel decided to use this section to evaluate the benefits of both Strata® and GlasGrid®. A visual inspection and some FWD and rolling deflectometer data were collected prior to construction.

In both directions a 1 inch level up layer was placed. In the northbound direction, the 1 inch thick Strata® layer was placed followed by 3 inches of HMA surfacing. In the southbound lane, 2 inches of HMA were placed over the GlasGrid®.

These sections were completed in late 2003. The performance will be monitored in the final year of Project 0-4517.

CHAPTER 5
RUBBLIZATION

INTRODUCTION

Rubblization has been widely used in many states, but it has only been used on a limited basis in Texas. The first nationwide performance comparison of the various methods of fracturing PCC slabs was conducted by Witczak and Rada in the early 1990s (*14*). A comparison shown in Figure 38 was developed to show the variation in backcalculated modulus following each of the treatments. Based on the variability of final modulus, rubblization was recommended as the preferred treatment for PCC pavements.

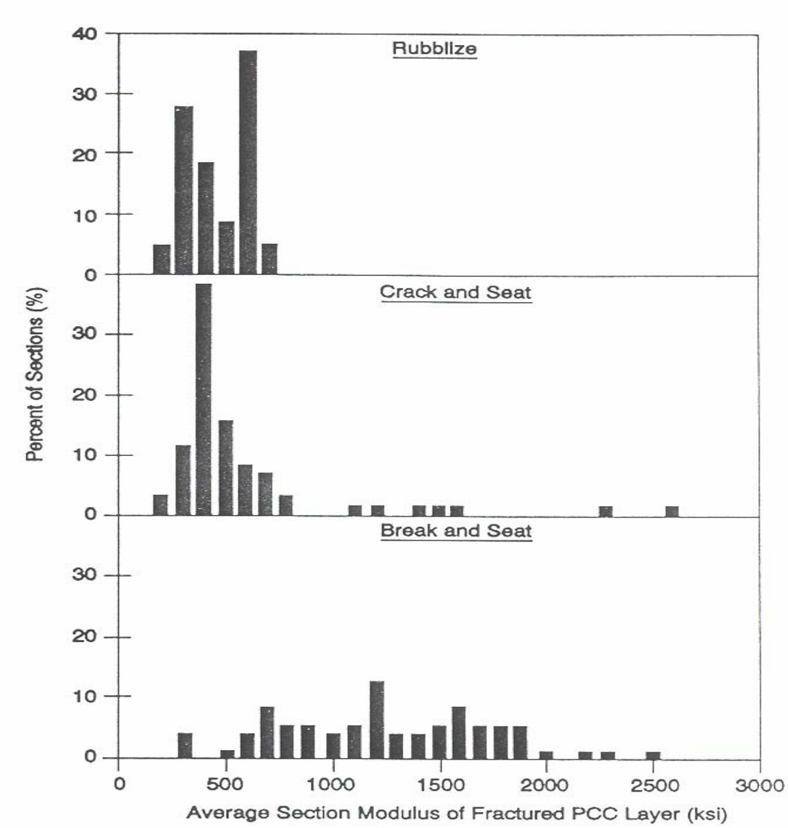


Figure 38. Frequency Distribution of In-site PCC Moduli Values after Treatment (*14*).

Ksaibati et al. presented a summary of the results of a nationwide survey performed by Florida DOT to collect information about the practices performed by other state DOTs with regard to rubblization and also determine the performance of rubblized sections in various states

(*15*). A wide variation was seen in the practice of construction techniques, overlay thicknesses and field performance among the various states. The survey results indicated that most of the states were satisfied with rubblization as a rehabilitation technique to eliminate the reflective cracking in all the types of concrete pavements.

A review was also made of the rubblization projects included in the strategic highway research program (SHRP) long-term pavement performance studies. Appendix B contains performance and deflection results from rubblization sections constructed by the Oklahoma DOT in the early 1990s. The description of the sections tested were obtained from a project report prepared by Brent Rauhut Engineering (*6*). The condition data and deflection analysis were performed on data collected from the long-term pavement performance (LTPP) data base. The performance of these sections was not good. The growth in alligator cracking is shown in Figure 39. The average backcalculated moduli values for the rubblized PCC slab in the two sections (400607 and 400608) were 90 ksi and 225 ksi, substantially below the average value of 412 ksi reported by Witczak and Rada (*14*).

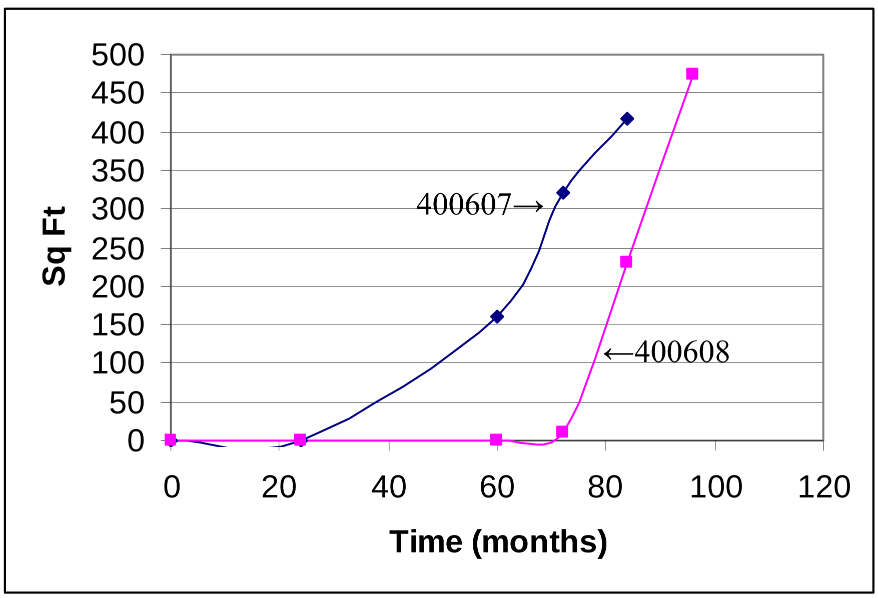


Figure 39. Growth in Alligator Cracking from Rubblized Concrete Sections in Oklahoma.

In section 400607 substantial alligator cracking was found in the section after five years in service. This was the section with the low PCC modulus. Both sections received substantial patching and maintenance in year 2001, nine years after construction. The poor performance is attributed to the low base modulus resulting in a structurally inadequate section to carry the traffic loads. The cause of the low PCC modulus was not identified in this evaluation.

From a review of existing information, there are no criteria for when and where to use rubblization. Most DOTs report favorable performance, but as shown in Oklahoma, this may not always be the case. It also appears that there are a whole range of design and construction practices in use by DOTs. In the remainder of this section a review will be made of the TxDOT experience with rubblization as well as a discussion of field data and a review of construction recommendations provided by Louisiana DOT.

RESULTS FROM TEXAS

TxDOT has little experience with rubblization, and there are only two projects identified from discussion with the TxDOT districts. The two existing projects are on US 79 in the Bryan District (contact person - Darlene Goehl) and US 67 Atlanta District (contact person - Miles Garrison).

The Bryan District reported problems with rubblizing on the US 79 project. On about 20 percent of the project the slabs would not break and they had to be replaced with full depth hot mix, leading to an expensive field change. They placed 7 inches of new flexible base and 3.5 inches of hot mix on top of the rubblized concrete. The rubblized section is now part of a major intersection. Most of the rubblized pavement is in the central turn lanes. Performance to date has been good; the district reported one longitudinal crack, which is in the area where they widened the existing slab with full depth hot mix. The section was constructed around 1998.

The Atlanta section was constructed by an area office near Mt. Pleasant. The process was not considered a success. The district lab engineer reported that "The process was not effective over joint, big unbroken pieces remained which had to be replaced with full depth ACP. The rubblizer rutted the processed section and displaced the concrete. Upon coring it was found that water was seen in the rubblized concrete beneath the ACP." The FWD data collected after rubblization are shown in Figure 40. These are troubling. The average rubblized concrete moduli value is less than 50 ksi, and this is less than would be anticipated for a class 1 flexible base. This may be attributed to the fact that water may be trapped in the base; as with the Bryan District no edge drains were installed in this project.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 6.0)

District:19 (Atlanta)
County :225 (TITUS)
Highway/Road: US0067

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	3.50	380,000	1,630,000	H1: v = 0.35
Base:	7.00	10,000	150,000	H2: v = 0.35
Subbase:	0.00			H3: v = 0.00
Subgrade:	143.77(by DB)	12,000		H4: v = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
301.000	10,304	24.50	15.34	8.27	4.90	3.10	2.14	1.64	470.8	50.0	0.0	11.3	2.62	160.7
403.000	10,244	29.63	16.21	7.50	3.95	2.34	1.76	1.36	435.7	19.5	0.0	13.0	2.73	102.6
500.000	10,276	20.33	14.53	8.59	5.06	3.07	2.01	1.44	1630.0	36.4	0.0	11.4	0.84	135.0 *
600.000	10,244	18.29	13.04	7.69	4.39	2.63	1.87	1.49	1630.0	39.4	0.0	12.9	2.78	129.4 *
701.000	10,260	17.59	12.02	6.54	3.65	2.26	1.61	1.22	1519.6	37.6	0.0	15.0	2.78	133.2
800.000	10,117	20.10	12.08	5.87	3.25	2.04	1.49	1.17	679.2	37.7	0.0	15.9	3.39	127.4
902.000	10,022	21.00	12.83	6.46	3.68	2.38	1.77	1.27	627.6	42.3	0.0	14.1	4.42	149.1
1001.000	10,236	19.92	12.60	6.48	3.69	2.30	1.63	1.18	821.3	42.5	0.0	14.6	2.67	149.0
1102.000	10,216	16.84	9.67	4.98	3.06	2.11	1.49	1.07	541.8	70.9	0.0	18.1	6.46	294.8 *
1201.000	9,966	20.21	11.97	6.45	3.87	2.50	1.76	1.25	420.7	63.2	0.0	14.0	3.67	181.1 *
1302.000	10,026	18.44	11.66	6.48	3.89	2.50	1.71	1.24	518.7	77.4	0.0	14.0	2.73	176.1
1401.000	10,705	19.83	11.98	6.26	3.79	2.52	1.82	1.31	456.7	69.6	0.0	15.2	5.09	220.2 *
1500.000	9,998	23.50	14.74	8.01	4.76	3.05	2.09	1.57	459.1	53.0	0.0	11.3	2.82	171.8
1601.000	10,280	21.80	14.90	9.23	5.94	3.89	2.70	2.02	484.0	104.3	0.0	9.7	2.34	197.3
1706.000	10,165	19.85	14.56	9.60	6.76	4.81	3.37	2.47	824.4	150.0	0.0	8.4	3.74	226.6 *
1803.000	9,752	29.19	17.98	10.49	6.52	4.22	2.86	2.05	380.0	49.4	0.0	8.3	3.69	181.1 *
1901.000	9,771	28.75	18.90	9.42	5.21	3.27	2.35	1.85	737.3	20.0	0.0	9.7	3.72	125.2
2002.000	9,958	23.20	14.78	8.45	5.47	3.79	2.80	2.03	380.0	82.7	0.0	10.0	6.70	300.0 *
2101.000	9,752	29.58	15.63	7.63	4.69	3.39	2.68	2.17	380.0	29.1	0.0	10.9	10.79	287.4 *
2304.000	9,807	31.89	18.13	8.86	5.15	3.37	2.35	1.87	380.0	24.9	0.0	9.9	4.58	168.2 *
2401.000	9,676	36.34	18.70	7.11	3.37	2.29	1.80	1.51	380.0	10.0	0.0	12.6	6.33	72.4 *
2502.000	9,918	27.41	15.44	6.70	3.48	2.22	1.74	1.38	497.7	17.8	0.0	13.8	4.57	97.8
2603.000	9,791	37.06	22.06	10.13	4.97	2.89	2.15	1.71	497.5	10.0	0.0	9.7	3.24	83.0 *
Mean:		24.14	14.77	7.70	4.50	2.91	2.08	1.58	658.8	49.5	0.0	12.3	4.03	154.3
Std. Dev:		5.93	2.93	1.47	1.03	0.74	0.50	0.38	394.0	32.8	0.0	2.6	2.05	61.5
Var Coeff(%):		24.57	19.82	19.04	22.98	25.31	24.11	24.06	59.8	66.3	0.0	20.9	50.77	38.3

Figure 40. FWD Results from Candidate Rubblization Section on US 67 in Atlanta.

The performance of the rubblized sections in Texas and Oklahoma has not been good. However the state of Louisiana has a very aggressive rubblization program underway that will eventually be used on long sections of both IH 10 and IH 20. As part of Project 0-4517 two visits were made to Louisiana to discuss construction and performance issues as well as collect nondestructive testing (NDT) data for existing projects on IH 10. The results of this survey are discussed in the next section of this report.

RESULTS FROM LOUISIANA

Louisiana Department of Transportation (LaDOT) has over 10 years experience with rubblization and is currently in the process of rubblizing and overlaying large sections of both IH 10 and IH 20. A preliminary meeting with LaDOT construction personnel was conducted in Lafayette in December 2002. The notes from that meeting are shown in Appendix C. Many critical factors are included in these notes such as the need for installing edge drains prior to starting the rubblization. Based on the reported success of the treatment, researchers decided to use NDT techniques to test two recently completed sections on IH 10. The goal of the testing was to identify if the sections on IH 10 were trapping moisture as reported in the Atlanta District and to use the FWD to measure the in-place moduli for the rubblized section.

The GPR testing was conducted on January 31, 2003. GPR data were collected on two rubblization jobs on IH 10. Job 1 was just over a 10 mile section approximately between mileposts (MP) 82 and 93. Job 2 was also a 10 mile section, from MP 93 to MP 103. Data were collected at 60 mph with one trace taken every 3 feet of travel. In both runs 1 and 2 all the data were collected in the outside lane, outside wheel path.

Figure 41 shows typical data from Job 2 (east of MP 93). These data are judged as ideal. The blue line in Figure 41 is the raw data collected at this location. The distance from the start of the run (7 miles and 943 feet) is shown in the box at the lower left-hand corner. The red line superimposed on the blue line in Figure 42 is the resulting trace once the large surface reflection has been removed by the COLORMAP software package (*13*). This "surface removal" technique is useful in exposing small near surface reflections, which are typically from the bottom of the last overlay placed. In this case, a small peak is detected just to the left of the surface reflection; this is the bottom of the wearing course.

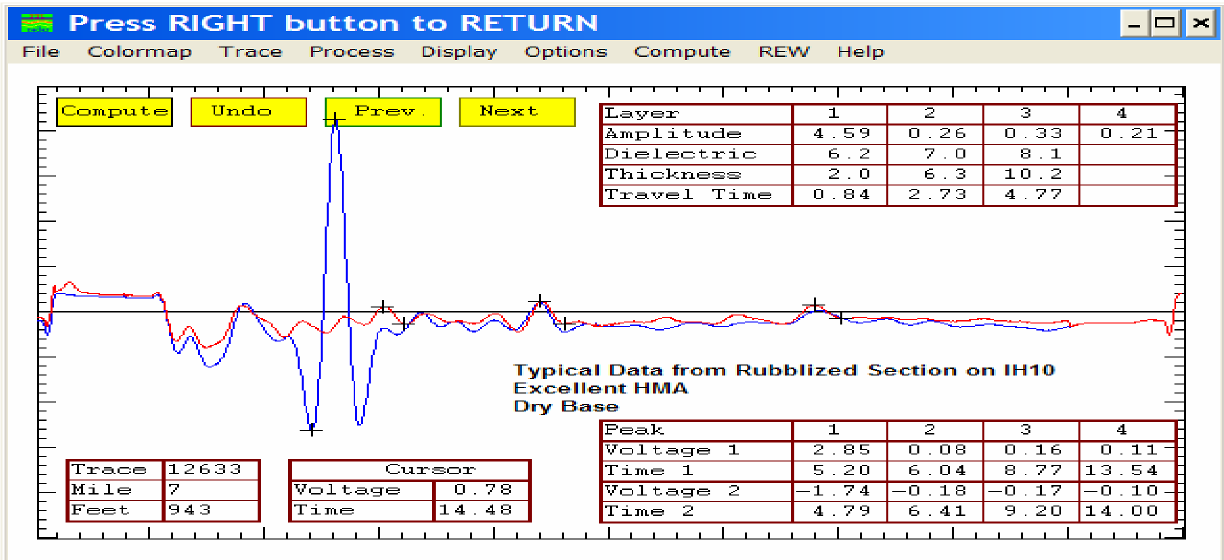


Figure 41. Typical Trace Obtained on IH 10 Rubblization Project (Ideal Case - No Defects).

The numbers in the box in the upper right part of Figure 41 are the results of the calculations made on this trace. The amplitudes and travel times between peaks are listed together with the computed layer dielectrics and thicknesses. In this particular case the trace was analyzed to show 2 inches of wearing course, 6.3 inches of HMA base and 10.2 inches of rubblized concrete. The individual layer dielectrics are 6.2, 7.0 and 8.1. The dielectric for the rubblized concrete is low at 8.1; this is very dry. This number for a granular base is judged to be ideal if it is less than 10, and it would be a cause for concern if it was greater than 16.

Using the COLORMAP color coding scheme, 1500 ft of IH 10 is shown in Figure 42. The scale at the bottom is the distance scale from 5 miles and 810 feet to 5 miles and 2310 feet. In this case, the surface has been moved to the top of the screen. The only significant reflection is a faint yellow line at a depth of approximately 10 inches. This is the top of the rubblized concrete layer. No defects are apparent in either the HMA or base layers. The line at the bottom of the figure is the surface dielectric plot. This is an indicator of the uniformity of the surfacing, and sudden localized dips would indicate areas of segregation. No significant segregation was found on either job. There is one small defect area to the far right of Figure 42. There is a localized strong red/blue reflection at approximately mid depth in the HMA layer.

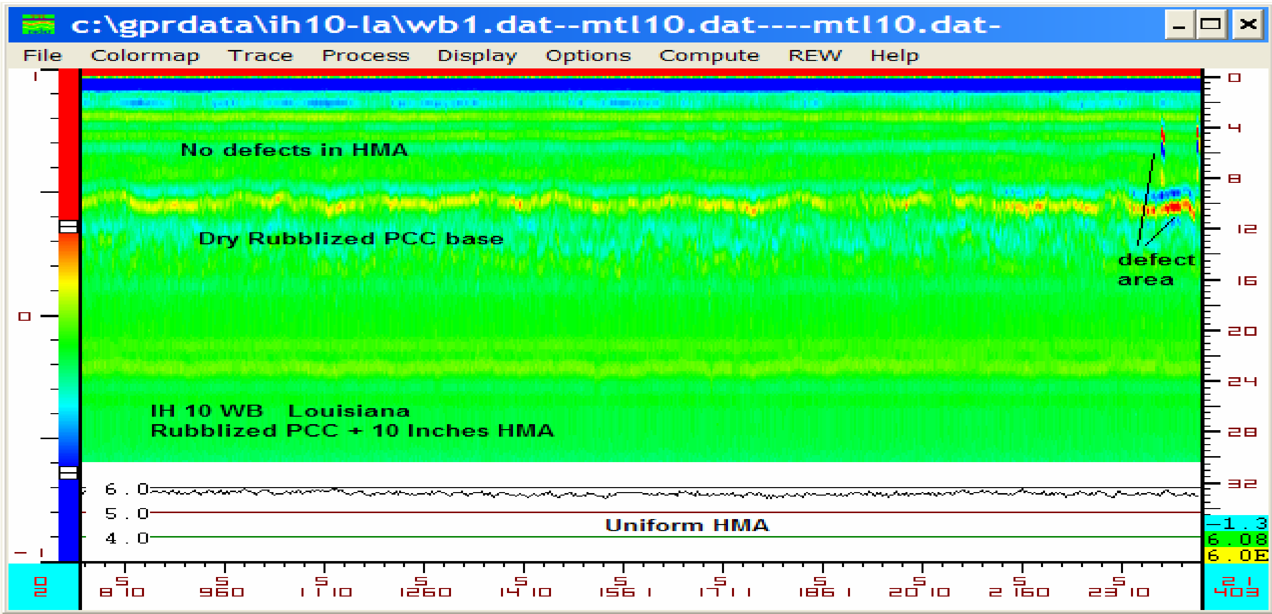
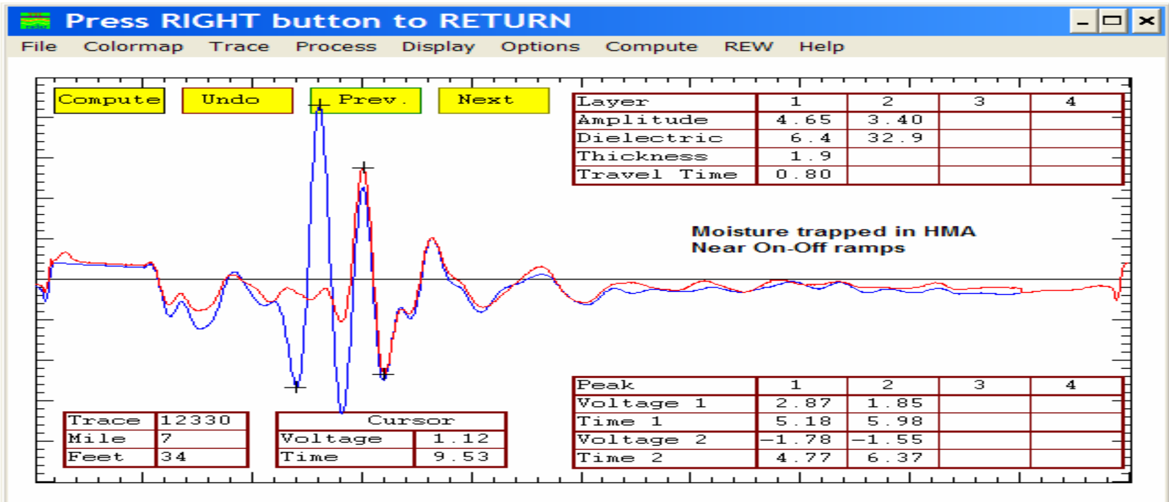


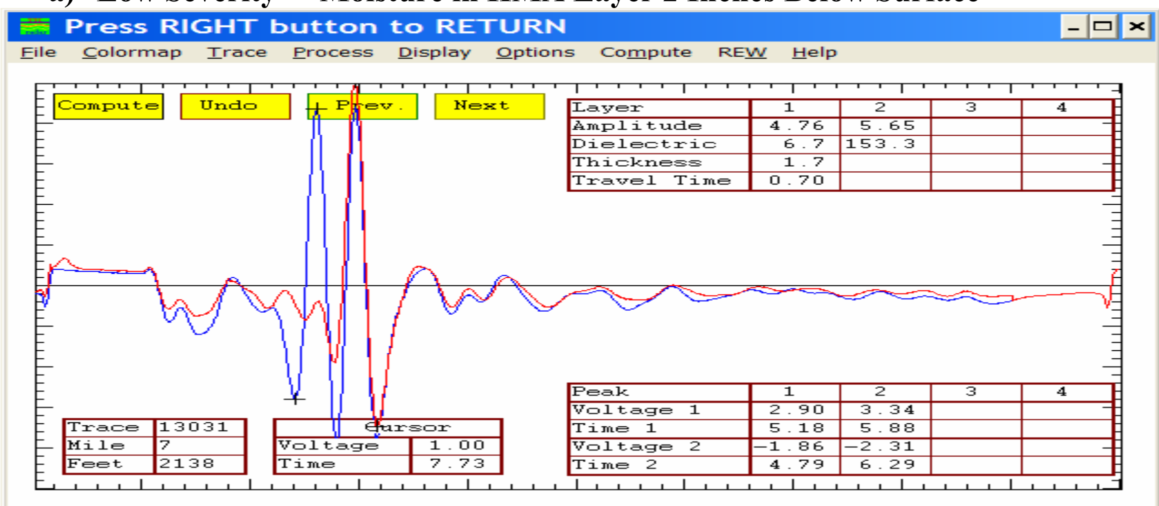
Figure 42. Typical Colormap Display for Job 2 (Ideal Case), Small Defect Area at Right.

The results from Job 2 are judged as ideal and provide clear evidence that this is a well compacted, top quality, defect free HMA layer. For the entire project the surface dielectric was extremely uniform, and the base was dry. However, a few localized problems were detected. These are illustrated in Figure 42. All of the defects in Job 2 were detected at either on or off ramps.

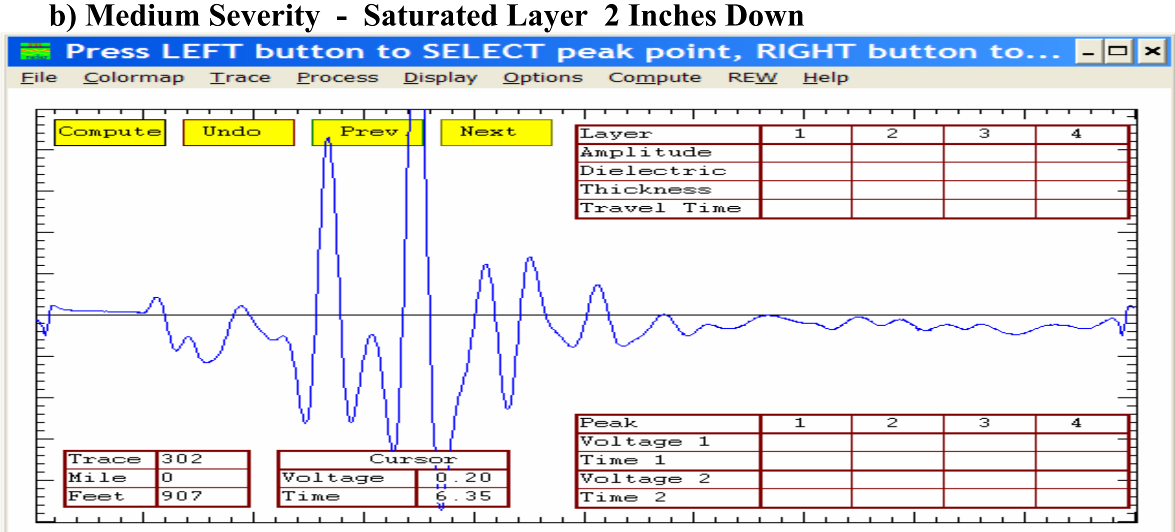
It appears that the drainage systems may not be functioning where the old concrete ramp meets the rubblized main lanes. These problem areas are localized to a few GPR traces. However, the reflections in these areas are very high. It was decided to group the GPR reflections from these problem areas into the three severity levels shown in Figure 43. At the low severity level it appears that there is a moist layer 2 to 3 inches below the surface. This is not a real cause for concern. The moderate level indicates a very high concentration of moisture. The severe level is a mystery. This is too high for moisture, and it is assumed to be related to a metal object beneath the upper HMA layers. Clearly there is a problem at the intersection between the rubblized main lanes and the old jointed concrete on/off ramps, which were not rubblized. More work is needed in these transition areas.



a) **Low Severity - Moisture in HMA Layer 2 Inches Below Surface**



b) **Medium Severity - Saturated Layer 2 Inches Down**



c) **High Severity - Unknown Metallic Object, Foil or Metal Paint 4 Inches Below Surface**

Figure 43. Different Severity of Problems Detected at On/Off Ramps on Job 2 EB IH 10.

FALLING WEIGHT DEFLECTOMETER RESULTS FROM IH 10 IN LOUISIANA

To provide an assessment of the structural strength of the rubblized pavement, FWD data were collected on a section of Job 2. The TxDOT falling weight deflectometer is shown in Figure 44. The FWD data are processed using the MODULUS 6.0 backcalculation program. On IH 10 the FWD data were collected in the outside lane EB direction starting at milepost 94, with data collected at 0.2 mile intervals for approximately 6 miles. The temperature of the HMA layer at the time of testing was measured by drilling a hole to a depth of 2 inches. The temperature of the HMA at the time of testing averaged 67 °F. This section was approximately three years old at the time of testing.

Figure 44. TxDOT's Falling Weight Deflectometer.

Figure 45 shows the FWD data and the results from the MODULUS run on the 6 miles of data. The first observation from these data is that the maximum deflections are very low at this load level. The average maximum deflection at the 6000 lb test load is 2 mils, which is very low. The average backcalculated moduli for both the surface and base layers are very high with average values of over 1400 ksi and 847 ksi, respectively. Using standard TxDOT temperature correction factors the temperature-corrected modulus for the HMA layer would be 977 ksi at the design temperature of 77 °F. This is well above the standard HMA design value used in Texas of 500 ksi.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 6.0)

District: Louisiana			MODULI RANGE(psi)		
County :		Thickness(in)	Minimum	Maximum	Poisson Ratio Values
Highway/Road: IH 10 EB	Pavement:	10.00	540,000	1,780,000	H1: v = 0.38
	Base:	11.00	50,000	2,000,000	H2: v = 0.25
	Subbase:	0.00			H3: v = 0.00
	Subgrade:	234.39(by DB)	15,000		H4: v = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	6,907	2.51	2.07	1.81	1.53	1.35	1.18	0.96	1277.6	1008.5	0.0	18.1	1.16	142.1
0.200	6,506	2.09	1.82	1.58	1.13	1.07	0.88	0.75	1780.0	415.2	0.0	25.8	5.24	300.0 *
0.400	6,407	1.89	1.61	1.41	1.20	1.02	0.86	0.72	1780.0	957.7	0.0	22.6	1.00	300.0 *
0.600	6,323	1.73	1.42	1.28	1.09	0.97	0.83	0.72	1707.3	1532.0	0.0	22.5	0.77	300.0
0.800	6,303	1.74	1.48	1.32	1.14	1.00	0.83	0.68	1780.0	986.0	0.0	24.0	2.42	132.8 *
1.000	6,256	1.91	1.63	1.44	1.16	1.04	0.87	0.77	1780.0	838.2	0.0	22.3	1.92	300.0 *
1.200	6,212	1.96	1.63	1.46	1.15	1.07	0.90	0.78	1733.3	879.0	0.0	21.5	2.29	300.0
1.400	6,164	1.74	1.37	1.21	0.98	0.92	0.77	0.67	1240.9	1656.2	0.0	25.0	2.16	300.0
1.600	6,164	1.95	1.60	1.42	1.18	1.04	0.85	0.74	1630.3	858.7	0.0	22.2	1.14	300.0
1.801	6,216	1.66	1.35	1.24	1.00	0.90	0.72	0.55	1780.0	1125.2	0.0	26.0	2.30	300.0 *
2.000	6,180	1.89	1.44	1.17	1.01	0.88	0.72	0.68	988.8	1154.3	0.0	28.0	1.80	300.0
2.401	6,176	1.98	1.54	1.39	1.14	1.04	0.83	0.74	1128.0	1277.3	0.0	22.7	1.94	300.0
2.600	6,208	2.04	1.68	1.44	1.25	1.05	0.89	0.77	1538.2	809.9	0.0	21.9	0.44	300.0
2.800	6,164	1.96	1.67	1.39	1.08	1.01	0.82	0.67	1780.0	424.8	0.0	26.5	4.01	300.0 *
3.000	6,156	1.80	1.41	1.18	0.99	0.88	0.73	0.61	1200.1	1164.1	0.0	27.3	1.70	300.0
3.400	6,240	2.09	1.71	1.44	1.17	1.03	0.86	0.77	1441.4	659.0	0.0	23.8	1.46	300.0
3.600	6,156	2.42	1.80	1.39	1.05	0.88	0.70	0.61	881.7	313.4	0.0	30.8	2.04	300.0
3.800	6,113	2.95	2.36	1.83	1.36	1.13	0.87	0.77	1109.8	142.1	0.0	24.4	1.89	300.0
4.000	6,105	2.37	1.78	1.43	1.13	0.95	0.78	0.70	863.4	458.5	0.0	27.2	1.53	300.0
4.201	6,117	1.96	1.67	1.40	1.20	1.05	0.89	0.72	1780.0	767.1	0.0	21.6	1.43	127.8 *
4.400	6,180	1.70	1.46	1.20	1.02	0.87	0.72	0.63	1780.0	833.1	0.0	27.3	1.30	300.0 *
4.800	6,133	1.87	1.43	1.18	0.93	0.82	0.68	0.57	1109.3	773.7	0.0	30.8	2.02	300.0
5.000	6,148	1.92	1.69	1.38	1.30	1.21	1.05	1.00	1780.0	1618.8	0.0	16.2	3.12	300.0 *
5.200	6,141	2.07	1.57	1.27	0.98	0.85	0.67	0.51	1064.7	481.9	0.0	31.3	1.77	300.0
5.400	6,148	2.12	1.66	1.20	0.87	0.68	0.52	0.37	1445.5	124.8	0.0	41.6	2.14	89.9
5.600	6,133	1.90	1.56	1.34	1.11	0.98	0.85	0.70	1484.7	985.4	0.0	23.1	1.44	150.1
5.800	6,109	2.18	1.72	1.48	1.14	0.96	0.77	0.67	1418.5	382.1	0.0	26.6	1.39	300.0
6.000	6,129	1.89	1.48	1.28	1.06	0.94	0.78	0.71	1199.1	1100.2	0.0	25.1	1.12	300.0
Mean:		2.01	1.63	1.38	1.12	0.99	0.82	0.70	1445.8	847.4	0.0	25.2	1.89	255.4
Std. Dev:		0.28	0.21	0.16	0.14	0.13	0.12	0.12	318.1	409.9	0.0	4.8	0.97	117.7
Var Coeff(%):		13.81	13.18	11.98	12.16	12.97	15.02	17.43	22.0	48.4	0.0	18.8	51.16	46.1

Figure 45. MODULUS 6.0 Results from Job 2 IH 10.

A second set of deflection data were also collected mid-slab on a section of jointed concrete on IH 10, which was about to be rubblized. This was an intact 10 inch thick PCC slab made with gravel aggregates. Figure 46 compares the normalized maximum deflection data from the rubblized section on Job 2 to the unbroken concrete section. In both cases the data were normalized to 9000 lbs, and both sets of data were collected with a pavement temperature close to 67 ° F. It is interesting to note that the deflections from the rubblized pavement are very similar to the deflections from the concrete slab. In fact, the rubblized section has slightly lower deflections than the uncracked slab.

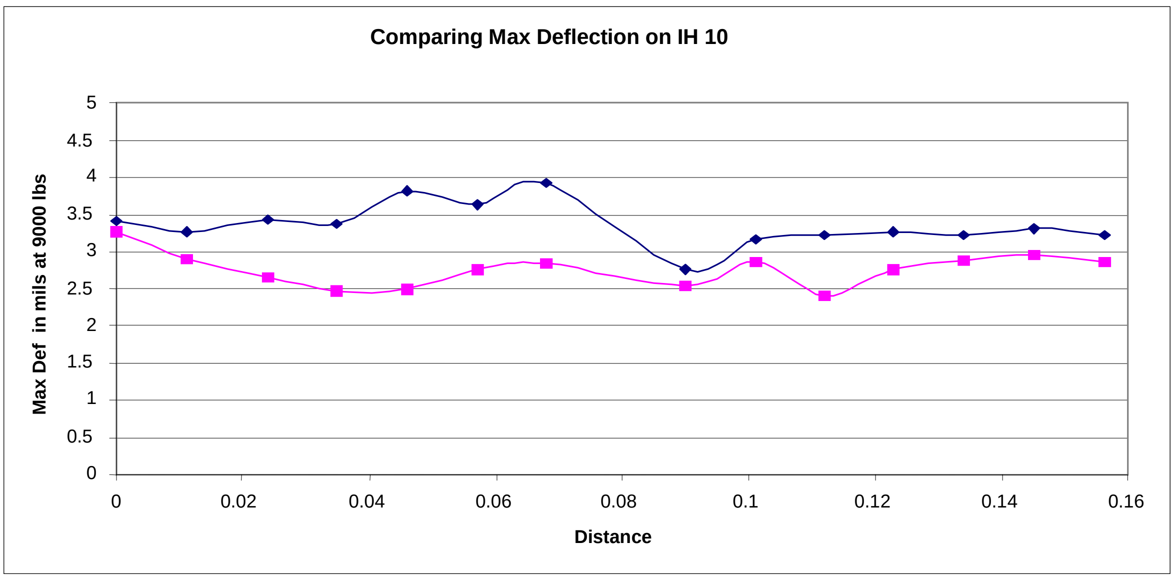


Figure 46. Comparing Deflection from the Rubblized Section (Job 2) with an Uncracked Concrete Slab.

In summary, the structural strength of the rubblized section looks very good. The rubblized concrete base modulus of almost 850 ksi is well above that traditionally found with Class 1 flexible bases of 70 to 80 ksi. This indicates that the rubblized base is not being reduced to a flexible base. It is still retaining many features of a fractured slab. It was commented that the rubblization process rubblizes the upper layers but only introduces diagonal cracks in the lower part of the slab. These very high moduli values would support that observation.

The extremely high values for the Superpave mix are also very interesting. This is a modified binder with a large stone aggregate. Field performance to date has been excellent. It would be worth doing follow-up FWD testing during summer months to evaluate the impact of high temperatures on the backcalculated moduli value. It would also be worthwhile to conduct laboratory tests on this material to compare the laboratory moduli value with those obtained on traditional dense graded mixes.

CONCLUSIONS

Based on the FWD and GPR results, the rubblization projects in Louisiana are judged to be excellent. The design and construction practices used by LaDOT have produced a sound, defect-free pavement, which is judged to be structurally adequate to carry interstate traffic for many years to come. The rubblized concrete has a moduli value over 10 times higher than that of a traditional flexible base, and the base is draining well with no evidence of trapped water. The Superpave mixes with the Roadtech shuttle buggy and the warranty bond are providing an outstanding HMA surface over the solid rubblized concrete base.

Discussions with LaDOT pointed out that the following items are critical in ensuring a successful project:

- The existing mid-slab support should be good, otherwise problems will be encountered when rubblizing the slab.
- Rubblization is preferred over crack and seat when failed/voided joints are present. Crack and seat may simply bridge the joints, whereas rubblization will fill the voids.
- Installing edge drain before rubblization is essential.
- Design and quality control (QC) of the asphalt layers is critical. LaDOT pointed out that the only problems they have encountered in earlier projects was with rutted asphalt layers. The new design and construction techniques have eliminated these problems.

Is this concept transferable to areas in Texas? The answer must be “yes.” It should be recalled that subgrade conditions in this area of Louisiana are extremely challenging. The water table is very near the surface and the area receives over 50 inches of rain each year. Furthermore the traffic loadings on IH 10 are very high. No rutting or cracking was observed in any of the projects tested. Based on these findings, it is recommended that this section of JCP in Texas be identified where rubblization would be a good rehabilitation option. The ideal candidate would have a stabilized base and sufficient elevation to install an edge drain system.

CHAPTER 6
RECENT DEVELOPMENTS

There are many ongoing activities in the state of Texas in the area of selecting rehabilitation alternatives for jointed concrete pavements. Four unique case studies will be discussed in this chapter of the report. The projects described will be tested further in the second year of Project 0-4517.

CASE STUDY 1 JOINTED CONCRETE PAVEMENT: STATE HIGHWAY 82, WICHITA FALLS DISTRICT

In this study an evaluation was made of State Highway US 82 in the Wichita Falls District. This highway is a plain jointed concrete pavement, which was built in 1957. The majority of this JCP has been performing well for the past 46 years. Over these years, full depth repairs were made at selected locations to improve the roadway condition. However, the area engineer was planning to place a hot mix asphalt overlay on top of the existing JCP in year 2004, which was going to be the first major rehabilitation since the JCP was built. The proposed rehabilitation strategy was to place 1.5 inches of porous friction course (PFC), over 3 inches of stone matrix asphalt (SMA) and with 1.5 inch of Type D mix. The design thicknesses were obtained using American Association of State Highway and Transportation Officials' (AASHTO's) Darwin design program. The district requested that an evaluation be made of the existing highway to determine if the proposed strategy was adequate for the existing structural condition. An 8.4 mile long test section was established along the westbound outside lane. The test section is located approximately 12 miles east of the city of Gainesville.

The rolling dynamic deflectometer (RDD), shown in Figure 47, was used to collect a continuous deflection profile along the westbound outside lane of the test section on June 4 – 6, 2003. Details of the RDD and results generated can be found reports by Lee (*7*). The test section is located between station markers 582+00 and 140+00. During the RDD testing, other nondestructive testing devices such as the FWD and the GPR were used to measure load transfer at selected joints and to detect sub slab anomalies. Furthermore, the DCP was used at selected locations to determine the quality of the subgrade material underneath the JCP.

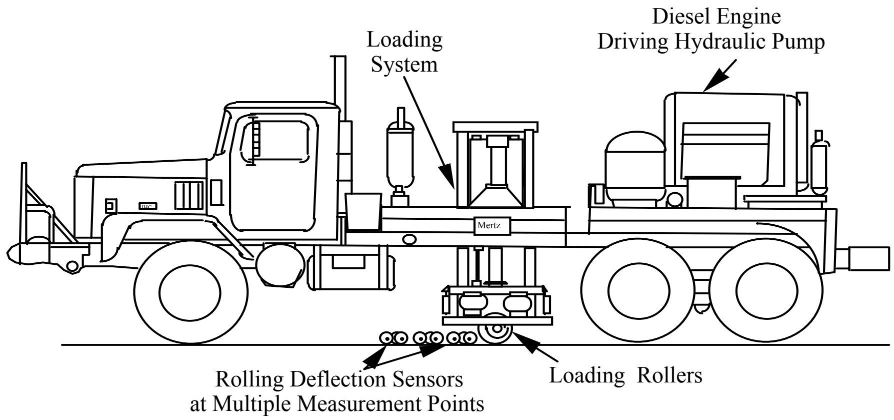


Figure 47. TxDOT's Rolling Dynamic Deflectometer (7).

The loading and sensor arrangement under the RDD is shown schematically in Figure 48. In the arrangement used on US 82 only three sensors were used.

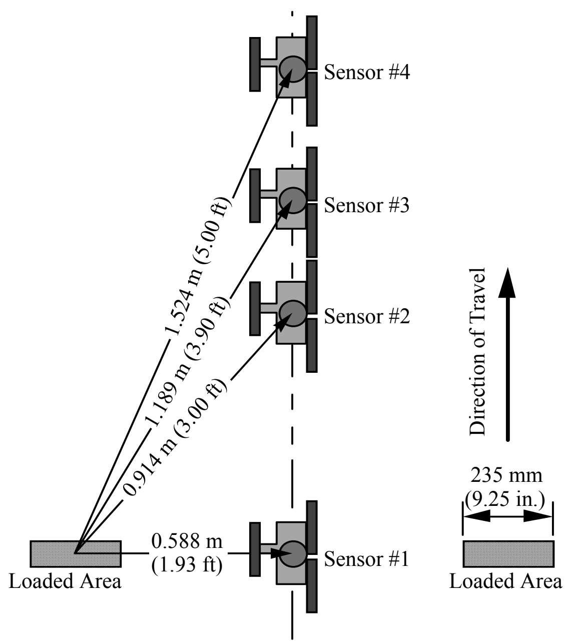


Figure 48. RDD Loading and Sensor Locations (7).

With the RDD, deflections are continually monitored at all three rolling geophones. These data are processed to provide an average surface deflection for every 2 feet of pavement. A typical deflection profile for a single joint in the pavement is shown in Figure 49. The blue line is the sensor directly between the loading wheels; the red and yellow lines are sensors 2 and 3.

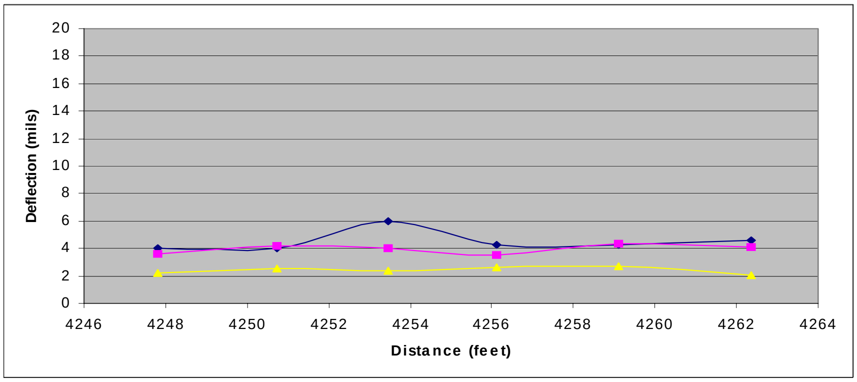
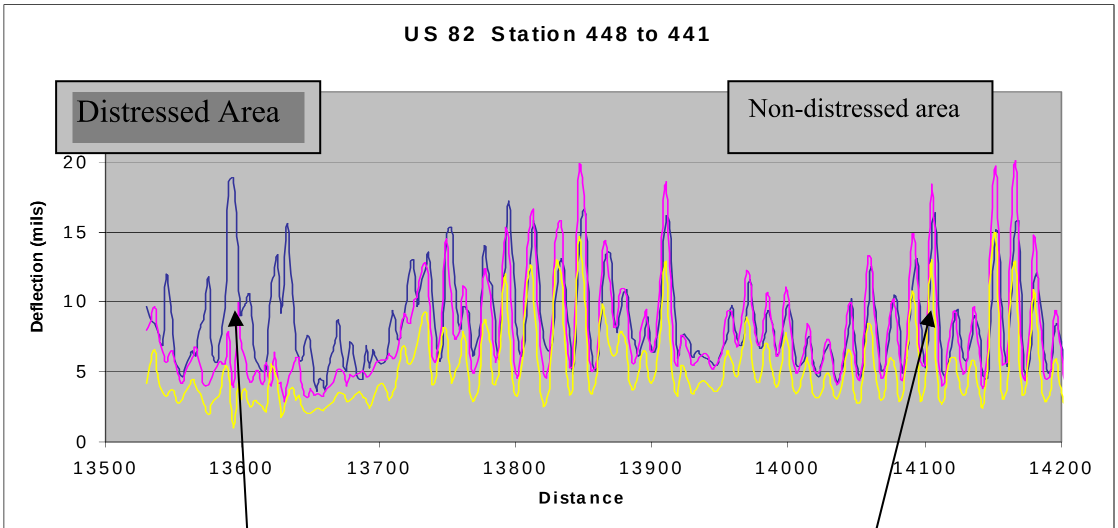


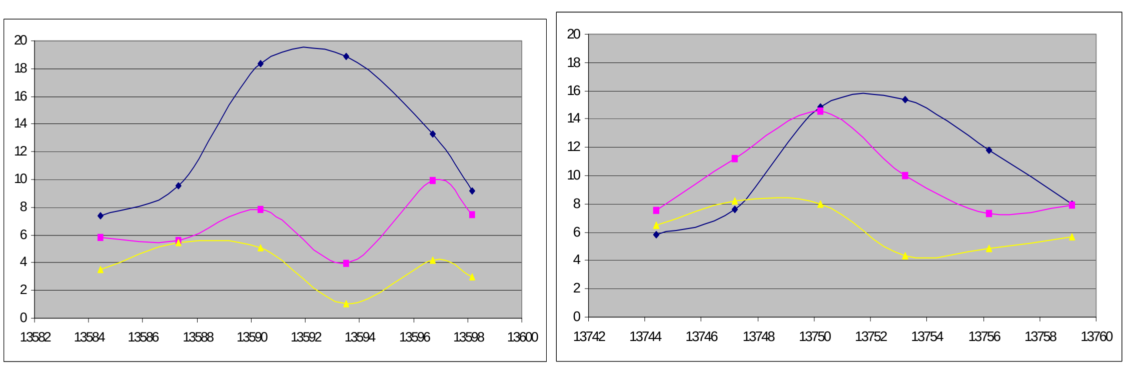
Figure 49. RDD Deflection for One Joint (Ideal Case).

The deflection pattern from the joint shown in Figure 49 is judged to be the ideal case. At this location the pavement condition was excellent; the joints had zero faulting and the load transfer test with the FWD indicated that the LTE was between 95 and 100 percent. For US 82 the majority of the section gave joint deflection profiles similar to that shown in Figure 49. However, as shown in Figure 50, significantly different deflection patterns were also observed in other sections on US 82.



a) Raw RDD Data for a 700 ft Section of Highway

b) Pavement Condition in the Distressed and Non-distressed Area



c) RDD Deflection Patterns over Typical Joints in the Distressed and Non-distressed Area

Figure 50. Rolling Deflectometer Data from US 82 in the Wichita Falls District.

The upper plot in Figure 49 shows the RDD deflection profile for a 700 ft section of US 82. The left of the figure shows that the pavement condition was poor with several broken slabs and some slab faulting. The right of the figure shows that the slab condition was good with no apparent surface distress. However, in both cases the two deflection patterns shown in Figure 50c are clearly different from the ideal case shown in Figure 49.

As part of this evaluation a tentative interpretation was made of the deflection patterns shown in Figure 50c. The deflection pattern on the left of Figure 50c is for the distressed area. It shows a large increase in sensor 1 deflections and some variations in sensors 2 and 3. However, the main characteristic is that there is a large difference in deflections between sensors 1 and 2 as the loaded wheel passes over the joint. The interpretation is that this area has poor load transfer and low subgrade support. The deflection pattern for the non-distressed section at the right of Figure 50c is very different from either the ideal case or the distressed case. In this area, large variations in all three sensors are occurring. In particular sensor 2 deflections are similar in amplitude as sensor 1. The interpretation here is that the subgrade support is poor, but the load transfer efficiency on the joint is good.

In an attempt to validate the subgrade support and load transfer efficiency classifications a limited number of DCP and FWD tests were conducted. The poor subgrade area was confirmed with the DCP. In the problem areas it was found that approximately 2 feet of very weak soil exists beneath the slab.

Once the interpretation scheme was defined, it was possible to use the RDD data to break the US 82 project into three distinct zones as follows:

- Zone 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).
- Zone 2 Good load transfer, poor subbase support (about 20 percent of the highway fell into this class).
- Zone 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 (zone 1) the LTE were measured to be greater than 90 percent. In these areas the district could reduce the overlay thickness design. In zone 2 it is recommended that the district consider thickening the overlay required. In zone 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. More work was suggested to define rehabilitation options for the zone 3 areas. These full-depth repair areas could be a candidate for a flexible base overlay similar to that used in the Lufkin experimental sections described in Chapter 2 of this report.

The main finding of this evaluation is that the rolling deflectometer device 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 had over 3000 joints. At the current operational speed the RDD completed the testing in about eight hours. More work with the RDD is recommended in year 2 of this project. The data acquisition system in the RDD can be improved, the RDD deflections should be integrated with video and GPR data, and more work should be done on building interpretation schemes for the RDD data.

CASE STUDY 2 US 175 IN DALLAS

In November of 2003 the Dallas District constructed a major experimental section to evaluate the ability of both GlasGrid® and Strata® to retard reflection cracking. The original pavement condition is shown in Figure 51. The original structure consisted of a 45 year old JCP pavement, which had performed well. The pavement had variable joint spacings; in some areas the joints were at 50 ft spacing whereas in others a 15 ft joint spacing was found. The project length was almost 3 miles. In both directions a 1 inch level-up layer was placed on top of the JCP; in the southbound direction the GlasGrid® product was placed followed by a 2 inch surface layer. In the northbound direction the Strata® product was used with a 3 inch overlay. Both sections were completed in December 2003. It is recommended that this section be included in the monitoring in the second year of Project 0-4517.

Figure 51. View of the Section of US 175 that Received Both the GlasGrid® and Strata® Treatment in December 2003.

CASE STUDY 3 RUBBLIZATION OF US 83 IN CHILDRESS

As described in Chapter 5, there are only two known rubblization projects in Texas. However, in the fall of 2003 work was initiated on a third project on US 83 in the Childress District. This is a JCP with three inches of HMA overlay. This highway was constructed in the 1930s as a 9-6-9 concrete pavement with a select sand base. Over the years many sections of the existing JCP have been replaced. In the current project, the plan is to remove the existing HMA and rubblize the remaining sections of old JCP pavement. The remaining sections are relatively short in length from 500 to 4000 ft long. No edge drains will be installed on this section.

Nondestructive testing data were collected on the site prior to construction. The view of the project is shown in Figure 52. This is a relatively low volume roadway. The subgrades in the test area are good and the climate is dry, with some freeze/thaw cycling in winter.

Figure 52. Rubblization Site on US 83 South of Childress.

The NDT testing consisted of rolling deflectometer, FWD and GPR. In a few cases dynamic cone penetrometer data were also collected. In this test program, three different segments, which were to be rubblized, were tested. A sample of the RDD data collected is shown in Figure 53. In all of the sections tested, the deflections were substantially higher than previously encountered on JCP pavement in other parts of Texas. The RDD maximum deflections were over 20 mils, and in several locations the sensor 2 deflections were higher than sensor 1. Based on previous experience on US 82, this would be classified as an area of good load transfer but very poor sub-slab support. The three adjacent joints starting at 495 feet have very high deflections.

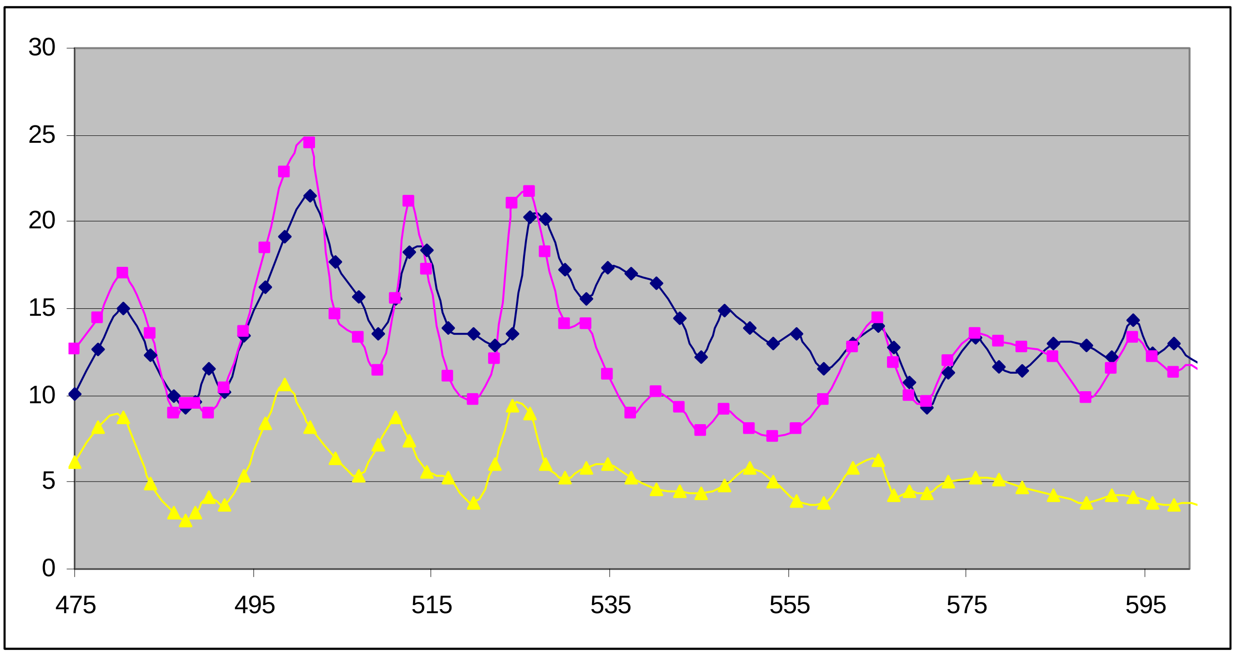


Figure 53. Rolling Deflectometer Data from Candidate Rubblization Site on US 83.

The FWD data collected on any of the three rubblization candidates were equally interesting. A typical FWD data set and the interpreted moduli values from MODULUS 6 are shown in Figure 54. The deflections, particularly the deflections from the outer sensors, are very high. The average value is 3.6 mils with a high value of over 8 mils. The outer deflections are measured 72 inches away from the load plate, and they are good indicators of the subgrade stiffness for the section. The backcalculated moduli value for this site is very low with an average value less than 10 ksi and several values at 5 ksi or less. These values are exceptionally low for a concrete pavement.

The high deflections on the US 83 project are also shown in Figure 55. These are maximum deflection values normalized to 9000 lb load. The US 83 results are compared with those from two other rubblization projects. The IH 10 results are from Louisiana where rubblization has been documented to be working very well. The US 67 project is from the Atlanta District of Texas where rubblization was a problem. These data were collected center slab before attempting rubblization. One concern about the US 83 project is that the initial deflections were substantially higher than on the other two projects, one of which was judged to be a failure.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 6.0)

District:			MODULI RANGE(psi)		
County :		Thickness(in)	Minimum	Maximum	Poisson Ratio Values
Highway/Road:	Pavement:	3.00	50,000	300,000	H1: v = 0.20
	Base:	7.00	2,000,000	7,000,000	H2: v = 0.20
	Subbase:	0.00			H3: v = 0.00
	Subgrade:	290.00(by DB)	10,000		H4: v = 0.40

	Load	Measured Deflection (mils):							Calculated Moduli values (ksi):				Absolute	Dpth to	
Station	(lbs)	R1	R2	R3	R4	R5	R6	R7	SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	ERR/Sens	Bedrock	
5.000	10,006	10.15	8.73	7.37	5.91	4.50	3.45	2.69	300.0	3260.9	0.0	10.9	1.87	300.0	*
195.000	9,728	12.07	8.06	6.13	5.02	3.98	3.24	2.59	70.2	4148.8	0.0	12.4	2.94	300.0	
205.000	9,831	16.78	13.92	13.48	12.65	11.22	9.96	8.54	116.4	5360.3	0.0	3.9	6.92	300.0	*
390.000	9,871	10.15	7.73	6.43	4.95	3.67	2.82	2.14	181.6	2808.3	0.0	13.4	1.13	300.0	
400.000	9,577	13.89	12.00	10.53	9.56	8.12	7.05	5.68	156.4	5274.0	0.0	5.2	2.91	300.0	*
590.000	9,859	11.50	10.13	9.00	7.61	6.17	5.03	4.11	212.9	5435.6	0.0	7.1	2.00	300.0	*
615.000	9,803	9.33	7.91	6.85	5.88	5.05	4.31	3.57	290.0	5353.6	0.0	9.7	3.66	300.0	*
785.000	9,636	10.64	7.74	8.93	8.06	6.86	6.21	5.56	196.9	7000.0	0.0	6.6	11.10	300.0	*
810.000	9,696	9.64	8.09	7.09	6.00	4.76	3.80	2.85	300.0	4383.8	0.0	10.0	1.75	300.0	*
975.000	9,756	7.91	6.42	5.59	4.67	3.76	3.10	2.51	248.7	6624.5	0.0	11.9	0.45	300.0	
1000.000	9,541	19.69	18.39	14.52	12.14	9.32	6.72	4.34	146.2	2000.0	0.0	4.9	4.66	164.0	*
1190.000	9,601	9.08	7.36	6.21	5.17	3.97	3.14	2.41	244.0	4188.9	0.0	11.6	0.80	300.0	
1220.000	9,581	11.10	9.91	8.89	7.59	6.07	4.66	3.27	216.0	5178.8	0.0	7.2	3.50	213.8	*
1395.000	9,676	7.89	6.68	5.65	4.65	3.52	2.76	2.05	300.0	4575.0	0.0	13.2	1.79	288.8	*
1400.000	9,819	7.19	6.64	5.96	5.00	3.85	2.97	2.17	300.0	6212.2	0.0	12.1	4.60	260.9	*
Mean:		11.13	9.31	8.18	6.99	5.65	4.61	3.63	218.6	4787.0	0.0	9.3	3.34	300.0	
Std. Dev:		3.40	3.26	2.78	2.63	2.33	2.06	1.79	74.0	1382.3	0.0	3.2	2.74	58.4	
Var Coeff(%):		30.56	34.96	34.03	37.58	41.17	44.67	49.40	33.8	28.9	0.0	34.7	82.23	20.3	

Figure 54. FWD Results from Candidate Rubblization Site on US 83 in Childress.

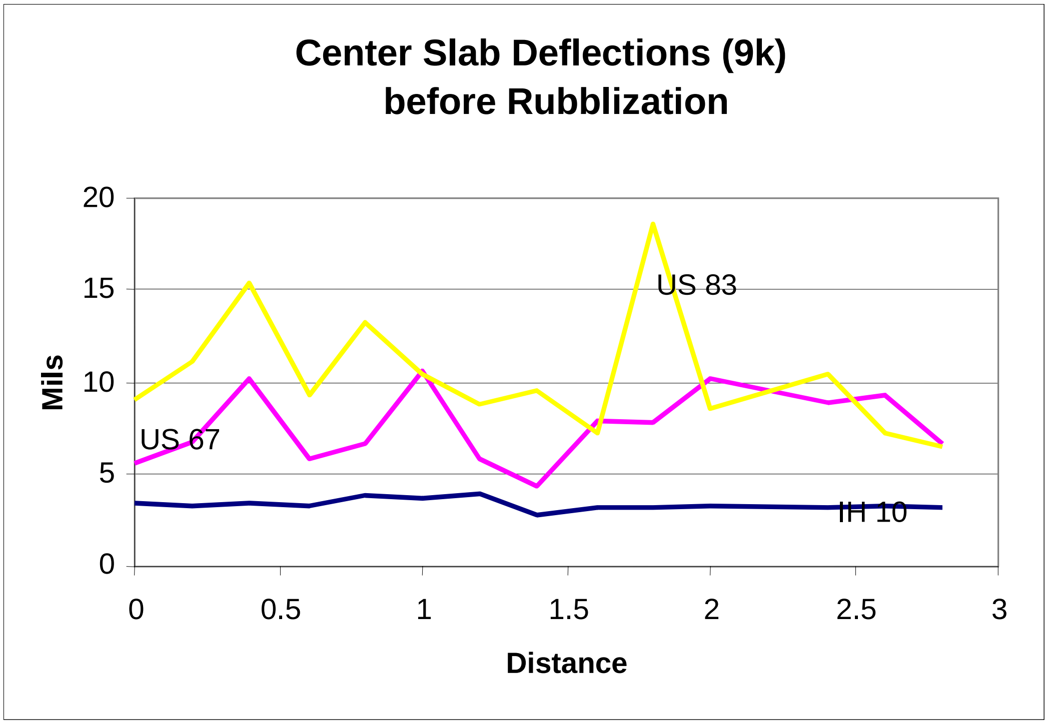


Figure 55. Comparison of Center Slab Deflections on Three Candidate Rubblization Projects.

To complete the NDT testing on US 83, ground penetrating radar data were collected on all the candidate rubblization projects. The data collected on site 3 are shown in Figure 56; this is the project with the extremely low moduli values shown in Figure 54. The key feature of this figure is the strong negative reflection (blue line) at the bottom of the slab. This is an indication that the select fill may be of low density or have eroded in several places. The select fill material is also very dry. The fill appears to be 4 to 5 inches thick, and then the GPR wave encounters a higher dielectric soil layer, marked as the bottom of the base in Figure 56.

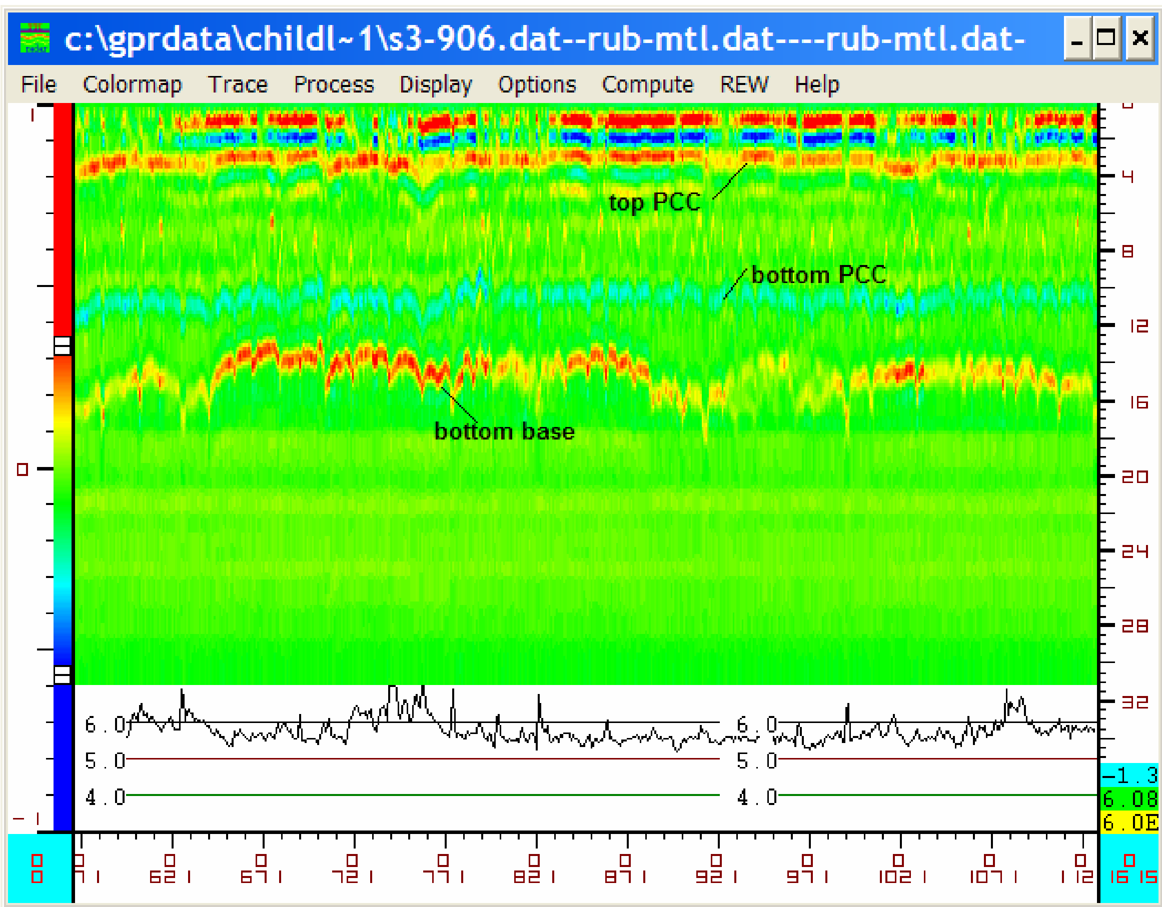


Figure 56. GPR Data on Candidate Rubblization Site on US 83.

The conclusion from this evaluation is that the slab on US 83 is resting on a weak dry sand layer. If deflection is a good criteria for determining if US 83 is a good candidate for rubblization, then based on the results in Figure 55 it could be assumed that US 83 is not a good candidate. If the selection criteria should be based on the presence of sub-slab moisture, then US 83 would be judged to be a good candidate. It will be critical to follow the construction of US 83 in the spring of 2004, to determine if the slab can be effectively rubblized and if the resulting pieces can be adequately compacted. It will also be important to measure the deflection of the resulting pavement.

CASE STUDY 4 EXPERIMENTAL SECTIONS ON US 83

The rubblization project on US 83 is just one of the methods used by the Childress District to rehabilitate the old JCP on this highway. US 83 runs for about 120 miles, north to south, through the entire length of the Childress District. Most of this highway was originally constructed with the old 9-6-9 JCP design. The Childress District Engineer, Craig Clark, informed the project team that the district has rehabilitated most of this highway over the past 10 years using a wide variety of rehabilitation options, including:

- crack and seat (four different projects),
- fly ash base overlay plus 3 inch overlay,
- geocomposite underseal plus 2 inches of HMA,
- lime rock asphalt base with one course surface treatment,
- thick HMA overlay, and
- mill and inlay.

It was commented that the sections on US 83 would be ideal for inclusion in the Project 0-4517 monitoring effort. No monitoring is being performed at the moment. The district can supply complete construction histories, and they are very interested in including these sections into a long-term monitoring effort.

As a minimum FWD, GPR and visual condition data should be collected on at least one section with each rehabilitation option. It is recommended that a modification be sought to Project 0-4517 to establish a monitoring program for this highway. A detailed proposal will be prepared for the consideration of the project monitoring panel.

CHAPTER 7
CONCLUSIONS AND RECOMMENDATIONS

The objectives of this year 1 report are to:

- Summarize the performance of the strategies that have been recently used within TxDOT to retard reflection cracking.
- Identify promising projects that should be considered for long-term monitoring.
- Identify strategies that have been working well in one district and could potentially be used statewide.
- Make recommendations for future research in terms of laboratory testing or equipment development.

CONCLUSIONS

For the Lufkin Experimental Sections on US 59

The following conclusions are proposed:

- The original performance of the rehabilitation options was impacted by the low quality of the original HMA surface. This surface was replaced in the travel lanes in 1997.
- Based on the observed performance from 1997 to 2003 the best performing section was the flexible base overlay. This was also one of the least expensive options. This strategy should be tried elsewhere in Texas. The flexible base overlay material should be of an equal or better quality than that used in Lufkin (Triaxial Class 1, with a TST final dielectric value of 12.5 or less). The base must also be sealed to prevent the ingress of surface moisture.
- The crack and seat was the worst performing strategy. The blocks were not effectively seated and continued to cause problems with block cracking and pumping. Slab fracturing should not be considered in East Texas if a weak layer exists beneath the slab. On US 59 the original slab was an old 9-6-9 design with an untreated select material support layer. From the data presented "weak" should be defined in terms of the DCP

more than 1 inch per blow and in terms of the FWD with a backcalculated modulus of less than 15 ksi.

- Crack and seat should not be attempted if free moisture is observed beneath the slab unless the moisture can be drained. Free moisture can be detected with ground penetrating radar testing.
- The second best performing layer in Lufkin was the open graded large stone asphalt layer (Arkansas mix).
- The poor performance of sections R1(extensive maintenance plus overlay) and R2 (crack and seat) was also influenced by the poorer support conditions in the other test sections.

With Regard to the Use of Strata® (Crack Retarding Interlayer)

Strata® is a proprietary product from Koch Materails. The following conclusions are based on the limited data collected in this study.

- No construction problems have been reported to date.
- The performance to date has been good, however the oldest section is less than two years old so long-term monitoring is recommended.
- The Strata® product performed very well in the laboratory in TTI's overlay tester with regard to resistance to reflection cracking. Tests were run at both 77 °F and 50 °F with excellent results.
- However, the product did not have good rut resistance, which raised concerns about driving on this layer during construction in hot weather. The layer is very rich. One concern that was raised was that it may have poor skid properties if it is driven on during rainfall.

With Regard to the GlasGrid® Product

GlasGrid® is a proprietary product from Koch Materials. The following conclusions are based on the limited data collected in this study.

- Several construction problems and failures shortly after construction were found around the state. In all cases, the grid was placed on a thin HMA level-up layer, and the observed problems were related to debonding of the two layers of HMA. The cause of debonding was attributed to several factors including: a) the small openings in the grid (about 0.25 inch) preventing intrusion of aggregates (particularly a problem with coarse TxDOT surfacing mixes such as CMHB or SMA), b) rainfall during construction, c) moisture problems with the self-adhesive glue used on the grid and d) inadequate surface thickness.

- The grid is placed in tension with specialized equipment. However, if the grid does not bond to the lower layer, then the "spring" action of the grid will cause rapid debonding.
- The long-term performance of the grids, which were successfully placed, could not be determined. However, in the oldest installation in Texas (US 96 Beaverton) the GlasGrid® performed significantly worse than the rubber modified HMA layer.
- Monitoring of the most recent grid sections in Dallas should continue, especially the new sections with the larger grid opening (up to 1 inch).

With Regard to Rubblization

Based on the limited data collected in this study the following conclusions are presented.

- This strategy, to date, has not worked well in Texas. However, it is working very well on Interstate pavements in Louisiana, and the lessons learned in terms of pavement design and construction sequence (Appendix C) should be applied in Texas.
- The ideal candidate would be a distressed JCP pavement with a stiff treated base. If sub slab moisture exists the edge drains must be installed prior to rubblization. GPR appears to be the best tool for determining if a JCP project is a good candidate for rubblization.
- For the conditions found in Louisiana the in-place stiffness from the rubblized concrete is 850 ksi. This is over 10 times higher than that value found with typical flexible bases. Clearly in Louisiana, the slabs are not completely rubblizing full depth.
- From the literature the final modulus of the rubblized concrete appears to be a function of the sub-slab support. With untreated bases the resulting modulus of the rubblized slab is somewhat less than that found in Louisiana. Little before and after deflection data are available nationwide in this area, and it is critical that TxDOT continue monitoring on-going projects such as the US 83 project in the Childress District.

Based on the treatments evaluated in this project a ranked list of rehabilitation options for jointed concrete pavements is presented in Table 6.

Table 6. Ranked List of JCP Rehabilitation Approaches.

Approach	Performance in Texas	Recommendation	Rank*	Comment
Dowel Retrofit	Not used	Use Washington DOT criteria and specs.	Unknown	Working well in other states. To be evaluated on US287 (Childress) in 2004.
Flexible Base Overlay	Best performing section in Lufkin experiment.	Use on low volume roads only.	A	Need non moisture susceptible flexible base (TST < 12.5). Under-seal and dense surfacing critical.
Crack and Seat with ACP Overlay	Worst performing section in Lufkin experiment. Works well in West Texas.	Do not use on wet/clay subgrades.	B	With water filled voids – must install edge drains prior to cracking
Rubblization with Asphalt Concrete Pavement (ACP) Overlay	Poor performance to date in Texas. Excellent performance in Louisiana.	Do not use on wet/clay subgrades.	B	With water filled voids – must install edge drains prior to rubblization.
Strata® (Crack Relief Layer)	Initial performance very good in Houston.	Use if no major structural distress.	B	Concerns: Cost Skid Layer is rut susceptible and therefore needs a thick overlay.
GlasGrid® (Reinforcing High Strength Grid)	Numerous failures in Texas. (Recent sections in Dallas look promising)	Do not use unless the debonding problem is addressed.	C	Concerns Cost Debonding of HMA layers Long term performance

*A Good Candidate
B Good Candidate but address concerns
C Not recommended

RECOMMENDATIONS

The rehabilitation of JCP pavements continues to be a problematic issue for many TxDOT districts. Every district in the state has old JCP sections, which are frequently buried under HMA overlays. These sections have typically exceeded their design lives. The findings developed in the first year of this project have attempted to pull together performance data on an array of different treatments constructed around the state. In the course of this project, it was determined that there are a variety of treatments being tried by districts on an experimental basis, but often the benefits of these test sections are not obtained because of a lack of monitoring or because the results obtained were not communicated to the district, which are often facing the same problem. The work initiated in year 1 should be continued into the final year of this project. In particular, the following sections should be visited in the second year.

Continuing Monitoring Sections

NDT data, visual condition results, and, where required, field coring/laboratory testing should be conducted on the following sections:

- SH 3 Houston (Strata® vs. Petromat),
- US 175 Dallas (Strata® vs. GlasGrid®),
- US 83 Childress (Rubblization), and
- four recent GlasGrid® sections in Dallas.

New Flexible Base Overlay Projects

The flexible base overlay did exceptionally well on US 59. This is a very heavily trafficked highway in a wet part of the state. Efforts should be made to locate other deteriorated JCP sections around the state. The base used in these new projects should be of an equal or better quality than that used in Lufkin, and the new base should be sealed to prevent the intrusion of surface moisture.

New Rubblization Projects in Texas

Efforts should be made to locate candidate rubblization projects in Texas based on the guidelines obtained from LaDOT.

New Equipment Development

As demonstrated on the US 82 project in Wichita Falls, the rolling deflectometer is an excellent tool for testing deteriorated jointed concrete pavements. Funds should be allocated to improve this system including:

- upgrading the data acquisition system,
- integrating a video and GPR system into the RDD vehicle to obtain a comprehensive JCP investigation in one pass,
- attempting to increase the speed of data collection (the current 1 mph is too dangerous for many urban highways), and
- using controlled testing in areas of known sub-slab condition and load transfer efficiencies to develop interpretation schemes for the three sensors.

In year 2 of Project 0-4517, it is proposed to use the RDD on several upcoming JCP rehabilitation projects to demonstrate its use to district personnel.

New HMA Mix Development (Improved Stress Relieving Layer)

During the course of this project it was determined that the proprietary stress relieving layers currently being placed by several districts have excellent reflection cracking properties. However, they are very soft and have poor rutting resistance. This may not be a problem if the layer is placed beneath 3 to 4 inches of traditional HMA. However from discussions with district staff, there is an urgent need to develop new mixes to be used in urban areas where the total overlay thickness must be no more than 2 inches. The new design could consist of 1 inch of level-up material and 1 inch of PFC. The tools described in this report, in particular TTI's new overlay tester, should be used to design the level-up materials to have both superior cracking resistance and adequate rutting resistance.

Monitoring the Experimental Sections on US 83 in Childress

Project 0-4517 was established to draw conclusions from the experimental pavements in the Lufkin District and to distribute the results statewide. However, in the course of this project it was determined that over the last five years the Childress District has constructed many different strategies on US 83 to rehabilitate the existing old JCP pavement. Funding should be provided to document these sections in Childress and to establish a long-term monitoring program.

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APPENDIX A

FWD RESULTS FROM LUFKIN EXPERIMENTAL SECTIONS ON US 59

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 6.0)

District: Lufkin
County : Polk
Highway/Road: US 59
Section R 1

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	4.00	120,000	660,000	H1: v = 0.35
Base:	8.00	2,000,000	7,000,000	H2: v = 0.20
Subbase:	0.00			H3: v = 0.00
Subgrade:	288.00(by DB)	8,000		H4: v = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	10,085	7.74	4.97	4.44	3.43	2.55	1.88	1.43	137.3	2990.5	0.0	20.3	2.68	300.0
0.000	10,010	7.15	4.48	4.41	3.77	3.04	2.48	1.93	138.0	7000.0	0.0	15.2	2.84	300.0 *
0.000	9,918	7.87	5.19	4.87	4.06	3.14	2.68	2.17	120.5	6181.7	0.0	14.0	2.13	300.0
0.000	9,958	10.38	7.38	6.37	4.96	3.80	2.97	2.19	127.1	2149.3	0.0	13.0	0.94	275.3
0.000	9,946	8.04	5.16	5.15	4.24	3.26	2.31	1.70	128.0	4684.4	0.0	15.0	5.84	252.5
0.000	9,879	10.17	7.91	7.61	6.10	4.50	3.55	2.68	239.2	2270.2	0.0	10.3	4.09	300.0
0.000	9,946	7.60	5.13	4.84	4.02	3.24	2.65	2.00	131.7	6623.2	0.0	13.7	1.80	300.0
0.000	9,831	6.94	5.30	5.35	4.56	3.63	2.89	2.19	264.3	6883.4	0.0	11.7	3.86	300.0
0.000	9,950	7.04	5.07	4.72	3.69	2.59	1.93	1.35	255.5	2511.6	0.0	18.8	4.63	300.0
0.000	9,907	9.06	4.34	3.65	2.73	2.03	1.48	1.11	120.0	2440.9	0.0	24.4	6.48	300.0 *
0.000	9,918	5.90	4.05	3.94	3.19	2.27	1.77	1.37	220.3	4734.3	0.0	20.7	4.99	300.0
0.000	9,950	6.54	4.61	4.44	3.64	2.84	2.18	1.64	201.1	5577.1	0.0	16.6	3.31	300.0
0.000	9,914	7.39	5.11	4.50	3.46	2.51	1.91	1.52	179.4	2561.7	0.0	19.6	2.56	300.0
0.000	9,994	5.02	3.35	3.11	2.45	1.87	1.39	1.02	227.0	5488.8	0.0	26.8	3.22	268.7
0.000	9,811	8.12	6.39	3.73	2.88	2.10	1.44	0.99	143.4	2056.3	0.0	20.6	14.88	300.0 *
0.000	9,895	6.43	4.59	4.31	3.52	2.74	2.06	1.52	215.1	4881.6	0.0	17.5	3.23	278.1
0.000	9,863	6.59	4.65	4.86	4.04	3.25	2.43	1.86	218.8	7000.0	0.0	13.9	5.36	300.0 *
0.000	9,875	6.59	4.81	5.07	4.34	3.42	2.72	2.17	284.5	7000.0	0.0	12.7	4.70	300.0 *
0.000	9,922	7.68	5.20	5.62	4.84	3.91	3.38	2.75	208.2	7000.0	0.0	10.9	4.74	300.0 *
0.000	9,855	12.22	9.02	7.82	5.54	3.90	2.61	1.81	120.0	2000.0	0.0	11.4	11.13	182.8 *
0.000	9,863	11.28	8.39	7.10	5.17	3.74	2.56	1.76	120.0	2000.0	0.0	12.4	7.80	197.7 *
Mean:		7.89	5.48	5.04	4.03	3.06	2.35	1.77	180.9	4477.8	0.0	16.2	4.82	300.0
Std. Dev:		1.81	1.48	1.26	0.93	0.71	0.60	0.49	55.7	2044.2	0.0	4.6	3.22	46.5
Var Coeff(%):		22.87	27.03	24.89	23.08	23.13	25.57	27.55	30.8	45.7	0.0	28.3	66.91	15.9

Figure A1. MODULUS 6.0 Results from Section R1 on US 59.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 6.0)

District:	Lukin		Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
County :	Polk					
Highway/Road:	US 59	Pavement:	6.30	60,000	540,000	H1: v = 0.35
Section	R 2	Base:	8.00	50,000	5,000,000	H2: v = 0.20
		Subbase:	0.00			H3: v = 0.00
		Subgrade:	288.00(User Input)	8,000		H4: v = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	9,767	12.68	9.14	7.71	6.10	4.48	3.32	2.39	196.0	598.4	0.0	10.8	2.22	106.5
0.000	9,783	10.30	7.40	6.75	5.39	4.03	3.06	2.40	222.0	1254.1	0.0	11.8	3.16	151.2
0.000	9,803	9.01	5.99	5.09	3.80	2.73	2.07	1.34	206.2	919.8	0.0	17.6	2.28	84.4
0.000	9,839	9.81	6.69	5.59	4.04	2.94	2.30	1.72	219.9	634.8	0.0	16.3	1.87	201.9
0.000	9,720	11.20	7.66	6.64	5.04	3.80	3.06	2.22	165.6	1083.9	0.0	12.4	1.76	300.0
0.000	9,696	15.56	9.21	7.13	4.95	3.16	2.04	1.36	115.2	180.2	0.0	14.5	5.99	91.9
0.000	9,716	12.79	9.19	7.87	6.14	4.02	2.29	1.37	386.4	151.4	0.0	12.1	9.73	74.1
0.000	9,664	10.84	8.22	6.75	4.53	3.14	2.09	1.39	540.0	127.8	0.0	14.6	5.52	98.3 *
0.000	9,732	10.22	6.74	5.78	4.20	2.99	2.38	1.78	177.3	796.5	0.0	15.6	2.39	162.3
0.000	9,752	9.66	6.26	5.62	4.44	3.23	2.27	1.62	171.6	1333.0	0.0	15.1	3.89	98.9
Mean:		11.21	7.65	6.49	4.86	3.45	2.49	1.76	240.0	708.0	0.0	14.1	3.88	115.0
Std. Dev:		1.96	1.24	0.94	0.82	0.58	0.47	0.43	126.9	450.3	0.0	2.2	2.54	40.9
Var Coeff(%):		17.53	16.22	14.51	16.81	16.90	19.01	24.45	52.9	63.6	0.0	15.7	65.37	35.6

Figure A2. MODULUS 6.0 Results from Section R2b on US 59.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 6.0)

District: Lufkin
County : Polk
Highway/Road: US 59
Section R 3

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	3.70	130,000	680,000	H1: v = 0.35
Base:	8.00	50,000	250,000	H2: v = 0.35
Subbase:	8.00	2,000,000	7,000,000	H3: v = 0.20
Subgrade:	280.30(by DB)	8,000		H4: v = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	10,113	8.49	4.19	4.06	3.44	2.67	2.24	1.83	130.0	196.2	3787.4	16.5	3.60	300.0 *
0.000	10,220	8.65	4.69	4.52	3.96	3.14	2.58	2.01	130.0	237.9	3519.9	13.5	3.21	300.0 *
0.000	10,185	8.26	4.45	4.33	3.70	2.83	2.33	1.83	137.5	250.0	2585.3	15.4	3.93	300.0 *
0.000	10,006	8.19	4.49	4.30	3.67	2.90	2.35	1.84	138.1	250.0	2581.0	14.9	3.13	300.0 *
0.000	9,998	7.84	4.49	4.02	3.44	2.72	2.21	1.77	159.5	250.0	2000.0	16.1	1.86	300.0 *
0.000	9,946	9.67	5.30	4.36	3.69	2.81	2.10	1.58	292.4	92.8	2000.0	15.8	3.91	300.0 *
0.000	9,942	8.83	4.45	4.04	3.47	2.70	2.18	1.73	130.0	180.2	2566.1	17.0	2.59	300.0 *
0.000	9,867	8.36	4.57	4.57	3.98	3.23	2.67	2.13	130.0	250.0	4480.2	12.0	3.46	300.0 *
0.000	9,839	8.24	4.50	4.58	4.06	3.19	2.65	2.12	133.2	250.0	4640.9	12.0	4.39	300.0 *
0.000	9,903	8.48	4.81	4.61	3.90	3.00	2.43	1.80	137.8	250.0	2000.0	14.0	3.80	285.5 *
0.000	9,907	8.47	4.12	4.63	3.92	3.10	2.45	1.83	130.0	206.9	5483.9	12.8	6.70	300.0 *
0.000	9,978	8.39	4.02	4.00	3.40	2.83	2.18	1.70	130.0	187.6	4906.3	15.8	4.28	300.0 *
0.000	9,958	9.02	4.61	4.07	3.52	2.86	2.18	1.61	130.0	177.0	2347.0	16.7	2.39	279.1 *
0.000	10,089	9.94	4.69	3.82	3.29	2.70	2.11	1.57	171.0	98.7	3072.3	18.4	0.95	292.9
0.000	10,050	7.80	3.52	3.26	2.83	2.44	1.97	1.55	130.0	184.8	7000.0	18.9	1.95	300.0 *
0.000	10,248	8.21	4.03	3.96	3.35	2.66	2.14	1.65	130.0	219.1	3866.6	17.2	3.74	300.0 *
0.000	10,228	9.33	4.71	4.19	3.64	2.64	2.07	1.50	144.6	154.2	2000.0	18.0	4.19	300.0 *
0.000	10,232	9.30	4.22	3.70	3.17	2.59	2.04	1.60	130.0	137.0	3740.1	19.4	1.72	300.0 *
0.000	10,208	7.50	3.42	3.28	2.83	2.23	1.81	1.43	130.0	222.3	5069.8	21.0	3.47	300.0 *
0.000	10,232	9.08	4.58	3.82	3.15	2.26	1.67	1.17	333.1	90.6	2005.5	20.1	5.52	212.8 *
0.000	10,101	8.77	3.61	2.97	2.44	1.91	1.47	1.03	150.4	116.1	2695.3	27.0	1.44	213.0 *
0.000	10,061	8.97	4.12	3.19	2.74	2.18	1.69	1.26	239.5	90.6	2872.4	22.5	0.95	296.9
Mean:		8.63	4.34	4.01	3.44	2.71	2.16	1.66	157.6	186.0	3419.1	17.1	3.24	300.0
Std. Dev:		0.61	0.45	0.49	0.44	0.34	0.31	0.28	56.1	59.0	1369.1	3.6	1.43	33.3
Var Coeff(%):		7.03	10.28	12.17	12.77	12.66	14.41	16.73	35.6	31.7	40.0	20.9	44.23	11.2

Figure A3. MODULUS 6.0 Results from Section R3 on US 59.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 6.0)

				MODULI RANGE(psi)		
District:	Lufkin		Thickness(in)	Minimum	Maximum	Poisson Ratio Values
County :	Polk	Pavement:	14.50	160,000	720,000	H1: v = 0.35
Highway/Road:	US 59	Base:	8.00	2,000,000	7,000,000	H2: v = 0.20
Section	R 4	Subbase:	0.00			H3: v = 0.00
		Subgrade:	265.48(by DB)	8,000		H4: v = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	10,228	4.14	2.36	2.36	2.06	1.68	1.41	1.10	439.2	6991.1	0.0	22.4	4.01	105.4 *
0.000	10,328	3.43	1.89	1.95	1.69	1.39	1.17	0.98	551.5	7000.0	0.0	28.1	4.24	300.0 *
0.000	10,216	3.41	1.92	1.96	1.63	1.33	1.21	0.98	542.8	7000.0	0.0	28.1	4.09	300.0 *
0.000	10,189	3.55	2.02	2.13	1.90	1.56	1.38	1.19	581.6	7000.0	0.0	22.9	4.54	300.0 *
0.000	10,244	3.56	2.01	2.13	1.94	1.57	1.35	1.19	574.1	7000.0	0.0	23.3	4.73	300.0 *
0.000	10,189	4.57	2.48	2.43	2.13	1.57	1.32	1.11	373.8	3398.5	0.0	25.5	5.48	134.2
0.000	10,244	3.03	1.72	1.58	1.40	1.17	1.00	0.87	588.5	7000.0	0.0	34.5	1.67	300.0 *
0.000	10,244	3.04	1.64	1.60	1.41	1.09	1.02	0.89	581.9	7000.0	0.0	35.3	4.75	300.0 *
0.000	10,268	3.39	1.74	1.78	1.56	1.31	1.16	0.95	527.7	7000.0	0.0	31.2	4.94	122.4 *
0.000	10,224	3.41	1.66	1.78	1.55	1.28	1.13	0.96	513.8	7000.0	0.0	32.0	6.11	300.0 *
0.000	10,185	3.66	1.54	1.81	1.44	1.13	0.96	0.98	398.6	7000.0	0.0	37.0	7.44	300.0 *
0.000	10,280	3.67	1.61	1.72	1.52	1.15	0.96	0.88	404.6	7000.0	0.0	36.6	6.29	300.0 *
0.000	10,181	3.83	1.65	1.81	1.52	1.29	1.12	0.93	396.6	7000.0	0.0	33.6	7.50	146.8 *
0.000	10,181	3.64	1.62	1.81	1.61	1.27	1.16	1.13	452.7	7000.0	0.0	32.4	7.96	300.0 *
0.000	10,232	3.96	2.05	2.13	1.79	1.40	1.23	1.07	418.4	7000.0	0.0	27.6	4.99	300.0 *
0.000	10,185	4.65	1.99	2.09	1.80	1.43	1.20	1.10	299.2	7000.0	0.0	29.1	5.37	300.0 *
0.000	10,193	4.27	2.19	2.24	1.92	1.61	1.37	1.21	391.3	7000.0	0.0	24.8	3.76	300.0 *
0.000	10,185	4.37	2.13	2.22	1.91	1.63	1.37	1.15	374.2	7000.0	0.0	25.2	4.82	300.0 *
0.000	10,165	4.33	1.95	2.24	1.88	1.57	1.36	1.19	376.0	7000.0	0.0	26.2	7.34	300.0 *
0.000	10,085	4.12	2.02	2.09	1.87	1.47	1.31	1.17	393.7	7000.0	0.0	26.6	5.38	300.0 *
0.000	10,034	4.22	2.17	2.11	1.91	1.52	1.38	1.33	386.9	7000.0	0.0	25.1	4.16	300.0 *
0.000	10,109	3.84	1.85	2.11	1.89	1.59	1.55	1.19	514.7	7000.0	0.0	23.6	8.57	300.0 *
0.000	10,097	3.69	2.01	2.07	1.97	1.52	1.28	1.07	506.1	7000.0	0.0	24.6	5.36	300.0 *
0.000	10,081	3.70	2.09	2.22	1.99	1.61	1.32	1.17	523.5	7000.0	0.0	22.8	5.50	300.0 *
0.000	10,089	3.94	1.89	2.09	1.95	1.52	1.38	1.25	453.9	7000.0	0.0	25.4	7.36	300.0 *
0.000	10,006	3.91	2.01	2.06	1.93	1.45	1.40	1.25	452.5	7000.0	0.0	24.9	6.60	300.0 *
0.000	10,030	4.34	2.31	2.33	2.09	1.63	1.48	1.23	393.2	7000.0	0.0	22.5	4.67	300.0 *
0.000	9,974	4.51	2.47	2.26	1.84	1.46	1.20	1.05	349.0	2799.7	0.0	28.0	3.08	300.0 *
Mean:		3.86	1.96	2.04	1.79	1.44	1.26	1.09	455.7	6721.0	0.0	27.8	5.38	288.0
Std. Dev:		0.44	0.26	0.23	0.21	0.17	0.16	0.13	82.2	1026.2	0.0	4.5	1.57	120.2
Var Coeff(%):		11.43	13.19	11.13	12.01	11.85	12.35	11.62	18.0	15.3	0.0	16.2	29.23	41.7

Figure A4. MODULUS 6.0 Results from Section R4 on US 59.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 6.0)

				MODULI RANGE(psi)		
			Thickness(in)	Minimum	Maximum	Poisson Ratio Values
District:	Lufkin	Pavement:	10.80	120,000	660,000	H1: v = 0.35
County :	Polk	Base:	8.00	2,000,000	7,000,000	H2: v = 0.20
Highway/Road	US 59	Subbase:	0.00			H3: v = 0.00
Section	R 5	Subgrade:	277.82(User Input)	8,000		H4: v = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	10,002	4.63	2.40	2.13	1.86	1.41	1.19	1.02	288.8	6482.6	0.0	31.0	2.03	300.0
0.000	10,097	4.76	2.32	2.15	1.93	1.38	1.19	0.97	274.9	7000.0	0.0	31.7	3.69	102.9 *
0.000	10,169	4.31	2.25	2.13	1.83	1.36	1.12	1.02	330.3	6281.0	0.0	32.4	3.83	141.6
0.000	10,125	5.39	2.55	2.41	2.02	1.48	1.25	1.13	236.2	5925.7	0.0	30.5	3.78	140.3
0.000	10,069	4.85	2.85	2.85	2.43	2.01	1.65	1.36	364.5	7000.0	0.0	21.1	3.00	159.3 *
0.000	10,069	4.48	2.65	2.75	2.33	1.81	1.52	1.28	403.4	7000.0	0.0	22.7	4.74	300.0 *
0.000	10,014	6.33	3.46	3.30	2.78	2.22	1.86	1.54	233.0	5476.9	0.0	19.2	2.55	179.2
0.000	10,018	7.44	3.72	3.58	3.06	2.42	1.98	1.62	176.1	5644.4	0.0	17.6	3.13	163.2
0.000	10,133	6.59	3.39	3.50	3.01	2.44	2.00	1.66	223.8	7000.0	0.0	17.6	3.88	179.5 *
0.000	10,121	5.50	2.28	2.63	2.28	1.93	1.66	1.43	253.1	7000.0	0.0	24.6	9.41	300.0 *
0.000	10,061	6.04	2.75	3.00	2.67	2.26	2.00	1.67	242.1	7000.0	0.0	20.3	7.86	300.0 *
0.000	10,061	7.24	3.27	3.39	3.04	2.44	2.12	1.77	181.2	7000.0	0.0	17.6	4.90	300.0 *
0.000	10,014	6.16	2.88	3.21	2.75	2.26	1.97	1.69	237.9	7000.0	0.0	19.3	6.52	300.0 *
0.000	10,069	5.33	2.67	3.37	3.03	2.46	2.24	1.82	418.0	7000.0	0.0	16.1	10.07	300.0 *
0.000	9,831	8.29	4.61	4.42	3.74	2.96	2.33	1.94	183.0	3119.4	0.0	14.8	3.37	186.5
0.000	10,026	7.00	3.06	3.00	2.53	1.93	1.62	1.35	168.5	7000.0	0.0	23.1	3.81	300.0 *
0.000	10,034	8.02	3.59	3.46	3.07	2.56	2.12	1.73	153.1	7000.0	0.0	17.3	3.00	144.1 *
0.000	9,982	6.28	2.87	2.76	2.35	1.98	1.52	1.20	196.8	7000.0	0.0	23.7	3.43	121.5 *
0.000	9,978	7.43	2.57	2.42	1.98	1.63	1.31	1.07	134.4	7000.0	0.0	30.7	3.01	151.5 *
Mean:		6.11	2.95	2.97	2.56	2.05	1.72	1.44	247.3	6522.6	0.0	22.7	4.53	196.6
Std. Dev:		1.23	0.60	0.60	0.52	0.46	0.39	0.31	82.2	966.8	0.0	5.9	2.29	71.6
Var Coeff(%):		20.16	20.33	20.10	20.35	22.25	22.45	21.40	33.2	14.8	0.0	25.8	50.53	36.4

Figure A5. MODULUS 6.0 Results from Section R5 on US 59.

TTI MODULUS ANALYSIS SYSTEM (SUMMARY REPORT) (Version 6.0)

				MODULI RANGE(psi)		
			Thickness(in)	Minimum	Maximum	Poisson Ratio Values
District:	Lufkin	Pavement:	11.00	60,000	540,000	H1: v = 0.35
County :	Polk	Base:	8.00	2,000,000	7,000,000	H2: v = 0.20
Highway/Road:	US 59	Subbase:	0.00			H3: v = 0.00
Section	R 6	Subgrade:	287.93(User Input)	8,000		H4: v = 0.40

Station	Load (lbs)	Measured Deflection (mils): R1	R2	R3	R4	R5	R6	R7	Calculated Moduli values (ksi): SURF(E1)	BASE(E2)	SUBB(E3)	SUBG(E4)	Absolute ERR/Sens	Dpth to Bedrock
0.000	9,942	8.14	3.33	3.31	2.73	2.16	1.76	1.31	137.1	6980.8	0.0	21.2	3.81	105.6
0.000	10,026	7.86	3.04	2.91	2.51	2.02	1.69	1.26	140.5	7000.0	0.0	23.9	3.27	96.6 *
0.000	10,018	6.15	2.80	2.75	2.31	1.82	1.40	1.05	200.1	6808.1	0.0	25.5	4.18	100.8
0.000	9,907	7.45	3.48	3.37	2.83	2.34	1.87	1.43	163.5	6788.3	0.0	18.8	2.83	114.2
0.000	9,883	5.89	2.67	3.06	2.61	2.11	1.72	1.39	236.1	7000.0	0.0	21.0	6.70	145.6 *
0.000	9,879	5.99	2.48	2.46	2.01	1.54	1.24	1.02	189.6	7000.0	0.0	30.3	4.29	162.2 *
0.000	9,843	5.93	2.67	2.65	2.23	1.79	1.49	1.18	209.2	7000.0	0.0	24.9	3.22	125.2 *
0.000	9,791	5.98	2.66	2.87	2.40	1.89	1.50	1.19	210.1	7000.0	0.0	23.2	5.53	132.0 *
0.000	9,863	5.62	2.54	2.58	2.13	1.54	1.24	0.96	219.7	5616.2	0.0	29.2	5.93	136.3
0.000	9,799	8.06	3.54	3.26	2.73	2.13	1.70	1.35	142.0	4865.0	0.0	21.8	2.74	135.7
0.000	9,811	8.25	3.74	3.93	3.40	2.81	2.35	1.93	150.6	7000.0	0.0	14.7	3.94	173.1 *
0.000	9,938	10.06	4.66	3.70	3.36	2.83	2.28	1.88	113.0	5193.0	0.0	16.7	1.79	143.4
0.000	9,835	7.86	3.54	3.70	3.22	2.69	2.24	1.81	159.0	7000.0	0.0	15.7	4.22	147.8 *
0.000	9,811	8.08	3.31	3.86	3.20	2.53	2.02	1.59	148.6	7000.0	0.0	17.0	7.03	142.8 *
0.000	9,827	9.58	3.41	2.83	2.43	1.80	1.22	0.89	101.6	2824.0	0.0	28.2	4.16	78.7 *
0.000	9,771	9.42	3.89	3.81	3.17	2.47	1.98	1.56	116.8	5499.2	0.0	18.4	3.98	139.2
0.000	9,883	6.50	3.23	3.28	2.82	2.26	1.85	1.45	207.5	7000.0	0.0	18.7	3.95	122.3 *
0.000	9,910	4.94	2.38	2.44	2.00	1.50	1.14	0.89	268.9	5367.2	0.0	30.3	5.94	118.4
0.000	9,863	4.89	2.39	2.46	2.01	1.49	1.12	0.87	276.6	4719.8	0.0	30.4	6.34	116.7
0.000	9,879	8.04	3.13	2.90	2.36	1.87	1.50	1.10	130.7	5973.6	0.0	25.6	2.50	98.8
0.000	9,696	7.52	3.87	4.00	3.51	2.91	2.37	1.89	187.5	7000.0	0.0	14.0	4.13	137.8 *
0.000	9,847	7.54	4.02	4.08	3.49	2.88	2.43	1.89	191.9	7000.0	0.0	14.0	3.26	127.1 *
0.000	9,748	11.61	3.65	3.66	3.22	2.75	2.31	1.85	82.7	7000.0	0.0	17.6	5.52	135.5 *
Mean:		7.45	3.24	3.21	2.73	2.18	1.76	1.38	173.2	6288.5	0.0	21.8	4.32	126.9
Std. Dev:		1.69	0.60	0.54	0.51	0.48	0.43	0.36	51.4	1103.4	0.0	5.5	1.41	24.2
Var Coeff(%):		22.70	18.52	16.95	18.68	22.01	24.51	26.27	29.7	17.5	0.0	25.2	32.71	19.1

Figure A6. MODULUS 6.0 Results from Section R6 on US 59.

APPENDIX B

LTPP RUBBLIZATION CASE STUDIES

As part of Project 0-4517 a review was made of experimental sections within the LTPP database where rubblization was performed on JCP pavements and deflection and long-term performance data are available. Two sections were found and a summary of the construction records and performance data are presented below. The construction records were extracted from the Brent Rauhut Engineering (BRE) long-term performance report.

SECTION 400607, US 59, OKLAHOMA

Original Section

The original pavement consisted of 9.1 inch jointed reinforced concrete pavement (JRCP) on a 4 inch sand base laid on an 8 inch subbase layer made of a soil aggregate mixture, predominantly clay, resting on silty clay (SC) subgrade.

Final Section

The construction of final section was done in the following three stages.

1. Preconstruction Monitoring of the Section

Preconstruction monitoring included the following measurements before the start of rubblization. This was done to assess the pavement conditions prior to the application of rehabilitation treatment.

Pavement Surface Distress:
From the distress surveys conducted on October 11, 1991, and July 28, 1992, moderate faulting, low severity spalling and corner breaks were the main distresses identified on the pavement section.

Surface Profile:
Rod and level measurements of the pavement section were taken prior to rubblization. Also, longitudinal profile of the section was obtained from SHRP's high-speed profilometer on January 14, 1992.

Structural Capacity:
Structural Capacity of the pavement was evaluated from deflection measurements using a SHRP falling weight deflectometer from January 28 – February 6, 1992.

Materials Sampling and Testing:
Oklahoma DOT, conducted preconstruction sampling on June 3, 1992. The sampling operation mainly involved extraction of 4 inch and 6 inch diameter cores, 6 inch auger probes and three test pits of 6 foot x 4 foot size to a depth of 12 foot below the top of the untreated subgrade.

2. Construction

The rubblization on section 400607 began on the afternoon of July 27, 1992. The concrete pavement was rubblized with a RMI (resonant frequency) breaker. With 8 inch wide shoe, it operated on the pavement at a frequency of 44 beats per second making 20 passes per lane. The concrete pieces on the surface were about 2 to 3 inches in size and those below, the steel were closer to 6 inches in size. The outside lanes were rubblized on July 27, 1992. The inside lanes were rubblized on July 28, 1992. Two sets of deflection measurements were taken before the start of the seating operation.

To seat the newly rubblized pavement, a 39 ton pneumatic roller was used. The roller made seven passes per lane. After the seating operation, the entire pavement was water blasted and then air blasted. This was done to remove the dust and fines that could inhibit the bonding of the asphalt concrete AC to the surface. Deflection measurements were taken on the newly seated pavement section before the application of the overlay.

The paving operation was done on the section using a SS-1H tack coat with a 50 percent dilution rate (1 part diluents to 1 part asphalt). It was started on July 29, 1992, and was completed by August 7, 1992.

A Caterpillar 2000 Drum Mixer plant was used to lay the hot mix asphalt concrete overlay on the section. A first lift of Type B mix AC overlay was placed on July 29, 1992. The second lift of Type B mix was placed on August 7, 1992.

Three different rollers were used to compact the overlay. A 10 ton steel-wheeled vibratory roller was used as a breakdown roller. This roller made two passes over the section. A 12 ton pneumatic roller was used as intermediate roller. This roller made five passes over the section. A 13.5 ton steel-wheeled static roller was used as a final roller. This roller made two passes over the section.

The installation of edge drains for subdrainage started on July 30, 1992. The main purpose of the subdrainage installation was to remove the free water from the drainage layers. The Advanedge® pipe system was used for the subdrainage. It had 2 inch x 18 inch corrugated plastic rectangular channels encased within the filter fabric. High modulus geotextile wrap was used as a primary filter. It was closely placed to the slab. The top of the channel was placed 1 inch below the PCC slab surface, and the horizontal distance of the pipe from the outer edge of the pavement was 3 inches. Laterals were then cut through the shoulders to dispose the drainage of the system through the shoulders. The lateral drains were placed by August 3, 1992.

All the traffic had been detoured during the rubblizing operation and installation of edge drains for the subdrainage system. The road was opened to the traffic after the completion of the above operations.

3. Post Construction Monitoring of the Section

The post construction monitoring, similar to the preconstruction monitoring of the section, was initiated after the completion of the above operations. It was mainly done to assess the effect of various operations on the performance of the road section.

<u>Pavement Surface Distress</u>

The manual distress data obtained on November 5, 1992, did not show any signs of distress on the road sections. Table B1 describes the various types of cracking that occurred in Section 400607.

Table B1. Types of Cracking in Section 400607.

Time in Years	Alligator Cracking (square feet)	Transverse Cracking (linear feet)	Longitudinal Cracking (linear feet)	Alligator + Patching (linear feet)
11/5/1992	0	0	0	0
3/30/1994	0	8	0	0
11/2/1994	0	14	0	0
5/22/1997	161	213	1025	161
11/17/1998	322	292	1060	322
9/14/2000	418	12	0	5395
8/3/2001	0	0	0	5581
9/17/2001	0	0	0	0

Surface Profile

Longitudinal profile of the section was obtained from SHRP's high-speed profilometer on March 16, 1993. Rod and level measurements were also taken on the section.

Structural Capacity

The structural capacity of the pavement was evaluated from the deflection measurements using a SHRP falling weight deflectometer during April 1993.

Sampling and Testing of Materials

Oklahoma DOT conducted sampling and testing of materials on August 31, 1992. Precise sampling was done to extract 20 4 inch diameter asphalt cores without any splitting of the samples from the section overlaid with hot mix. The obtained samples, and the samples obtained from the preconstruction sampling, were sent to the laboratory for testing.

Backcalculation of Layer Moduli using MODULUS 6.0

MODULUS 6.0 was used to calculate the elastic modulus of the different pavement layers. Table B2 shows the elastic modulus of different pavement layers for Section 400607.

Table B2. Layer Moduli for Section 400607.

Year	Temperature (°F)	E_{HMA}	$E_{CONCRETE}$	E_{BASE}	$E_{SUBGRADE}$
Preconstruction	75	--------	5220.5	18.6	19.5
1993	93.2	786	47.6	67.3	16.3
1996	17.6	1814.5	143.1	32.4	18.5
1998	62.6	978.4	83.4	98.1	18.5

SECTION 400608, US 59, OKLAHOMA

Original Section
The original test section consisted of 9.1 inch JRCP on a 4 inch sand base layer laid on an 8 inch subbase layer of soil aggregate mixture, predominantly clay, resting on silty clay subgrade.

Final Section
The construction of the final section was done in the following three stages.

1. Preconstruction Monitoring of the Section
Preconstruction monitoring included the following measurements before the start of rubblization. This was done to assess the pavement conditions prior to the application of rehabilitation treatment.

Pavement Surface Distress:
From the distress surveys conducted on October 11, 1991, and July 28, 1992, moderate faulting, low severity spalling and corner break were the main distresses identified on the pavement section.

Surface Profile:
Rod and level measurements of the pavement section were taken prior to rubblization. Also, longitudinal profile of the section was obtained from SHRP's high-speed profilometer on January 14, 1992.

Structural Capacity:
Structural capacity of the pavement was evaluated from deflection measurements using a SHRP falling weight deflectometer from January 28 – February 6, 1992.

Materials Sampling and Testing:
The Oklahoma DOT, conducted preconstruction sampling on June 3, 1992. Sampling operation mainly involved extraction of 4 inch and 6 inch diameter cores, 6 inch auger probes and three test pits of 6 foot x 4 foot size to a depth of 12 inches below the top of the untreated subgrade.

2. Construction
The rubblization on section 400608 began on the afternoon of July 27, 1992. The concrete pavement was rubblized with a RMI (resonant frequency) breaker. With an 8 inch wide shoe, it operated on the pavement at a frequency of 44 beats per second making 20 passes per lane. The concrete pieces on the surface were about 2 to 3 inches in size, and those below the steel were closer to 6 inches in size. The outside lanes were rubblized on July 27, 1992. The inside lanes were rubblized on July 28, 1992. Two sets of deflection measurements were taken before the start of the seating operation.

A 39 ton pneumatic roller was used to seat the newly rubblized pavement. The roller made seven passes per lane. After the seating operation, the entire pavement was water blasted and then air blasted. This was done to remove the dust and fines that could inhibit the bonding of AC to the surface. Deflection measurements were taken of the newly seated pavement section before the application of the overlay.

A Caterpillar 2000 Drum Mixer plant was used for laying the hot mix AC overlay on the section. A first lift of Type A mix AC overlay was placed on July 29, 1992. The second lift of Type A mix AC overlay was placed on August 3, 1992. The paving operation was done on the section using a SS-1H tack coat with 50 percent dilution rate (1 part diluents to 1 part asphalt). The paving operation was started on July 29, 1992 and was completed on August 3, 1992. Finally, a surface friction course of Type B mix was placed on August 7, 1992.

Three different rollers were used to compact the overlay. A 10 ton Hyster steel-wheeled vibratory roller was used as breakdown roller. This roller made two passes over the section. A 12 ton Bomag pneumatic roller was used as intermediate roller. This roller made five passes over the section. A 13.5 ton Hyster steel-wheeled static roller was used as a final roller. This roller made two passes over the section.

The installation of edge drains for subdrainage system started on July 30, 1992. The main purpose of the subdrainage installation was to remove the free water from the drainage layers. The Advanedge pipe system was used for the sub drainage. It had 2 inch x 18 inch corrugated plastic rectangular channels encased within filter fabric. High modulus geotextile wrap was used as a primary filter. It was closely placed to the slab. The top of the channel was placed 1 inch below the PCC slab surface, and the horizontal distance of the pipe from the outer edge of the pavement was 3 inches. Laterals were then cut through the shoulders to dispose the drainage of the system through the shoulders. The lateral drains were placed by August 3, 1992.

All the traffic had been detoured during the rubblization operation and installation of edge drains for the subdrainage system. The road was opened to the traffic after the completion of the above operations.

3. Postconstruction Monitoring of the Section

The postconstruction monitoring, similar to the preconstruction monitoring of the section, was initiated after the completion of the above operations. It was mainly done to assess the effect of various operations on the performance of the road section.

Pavement Surface Distress
The manual distress data obtained on November 5, 1992, did not show any signs of distress on the road sections.

Surface Profile
Longitudinal profile of the section was obtained from SHRP's high-speed profilometer on March 16, 1993. Rod and level measurements were also taken on the section. From the profile obtained, it can be seen that the ride quality of the pavement has increased after the application of these processes. Table B3 describes the various types of cracking that occurred in Section 400608.

Table B3. Types of Cracking in Section 400608.

Time in Years	Alligator Cracking (square feet)	Transverse Cracking (linear feet)	Longitudinal Cracking (linear feet)	Alligator + Patching (linear feet)
11/5/1992	0	0	0	0
3/30/1994	0	0	0	0
11/2/1994	0	0	0	0
5/22/1997	0	49	1004	0
11/17/1998	11	92	1001	11
9/14/2000	231	277	1021	231
8/3/2001	474	437	1027	474

Structural Capacity

The structural capacity of the pavement was evaluated from the deflection measurements using a SHRP falling weight deflectometer during April 1993.

Sampling and Testing of Materials

Oklahoma DOT conducted sampling and testing of materials on August 31, 1992. In spite of precise sampling, only one complete core could be obtained for the 8 inch AC overlay placed on Section 400608. The splitting of the samples during coring indicated that the aggregates were not bonded well in the bottom part of the overlay.

Backcalculation of Layer Moduli Using MODULUS 6.0

MODULUS 6.0 was used to calculate the elastic modulus of the different pavement layers. Table B4 shows the elastic modulus of different pavement layers for Section 400608.

Table B4. Layer Moduli for Section 400608.

Year	Temperature (°F)	E_{HMA}	$E_{CONCRETE}$	E_{BASE}	$E_{SUBGRADE}$
Preconstruction	75	---	5114	1837	19.7
1993	95	626.8	168.1	25.8	19.6
1996	19.4	2418.6	196.8	30.1	23.0
1998	66.2	830.3	298.5	40.7	25.6

APPENDIX C

RUBBLIZATION OF JOINTED CONCRETE PAVEMENTS

Notes from a meeting with Louisiana DOT Personnel

Dec 4th 2002, Lafayette, LA

Objective **To discuss LaDOT experience with Rubblization of the Jointed Concrete Pavement on IH 10 and to judge how appropriate this treatment would be to major projects under consideration in Texas**

Present:	TxDOT	Dar-Hao Chen, Moon Won, John Bilyeu
	LaDOT	Mike Eldridge, District Construction Engineer
		Lester LeBlanc District Construction Engineer Luanna Cambas Bituminous Engineer various Field and Lab personnel
	TTI	Tom Scullion, Lee Gustavus

LaDOT is in the process of rubblizing long sections of IH 10 from the Texas State line to Baton Rouge. Several sections are complete and several more are under construction. The oldest section is 3 years. All of the pavements were old faulted JCP with wire mesh reinforcement on a sand/shell soil-cement base. LaDOT had tried a range of rehabilitation options including under-sealing, overlaying and grinding. All of these had not performed well with the distresses reappearing in a short period.

The concept of rubblization was initiated in the mid-1990s. The state is now sold on it and plans to rubblize all of IH 10 and large sections of IH 20. Performance to date (3 years max) has been very good. The sections ride well and show no defects. However no NDT data has been collected on the completed sections.

Below is a list of the topics discussed in the Dec 4th meeting;

CRITICAL FACTORS IN RUBBLIZATION PROJECTS

LADOT stated that there are 3 critical factors

Factor 1 Quality of existing base

Soft spots beneath the JCP slab are a big problem. The slabs will not shatter if they are sitting on "jello". However the DOT does not have any criteria for what constitutes a "good base". The IH 10 sections are rubblizing fine, however they do anticipate hitting soft spots on a % of each section (10%), in these areas they do full depth repair, with the slab being replaced with full depth flexible base.

"*Comment---- It is possible to find poor support conditions with either the FWD or RDD; from the limited data we have it looks as if the center slab subgrade modulus should be greater than 20 ksi*"

The DOT stated that poor load transfer and small voids at the joints are not a problem on IH 10, the joints shatter and settle. They viewed this as one of the advantages over crack and seat where cracked slabs may bridge voids, causing stability problems later.

"Comment----Joints were a big problem on the Rubblization Job on US 79 in the Atlanta District, see Miles Garrision's comments in the attachments. However on US 79 the base was poor and the drainage system was not used"

Factor 2 Drainage System

The LaDOT feels that it is critical to install the shoulder drains as the first step in the construction sequence. On most of IH 10 there are adequate ditches and run-off areas to drain the pavement. However in a few locations they deepen the ditches prior to starting construction. They currently use a pipe drainage system, after 3 years the drainage is reported to be working well with no evidence of clogging. They do not feel that the rubblization process severely impacts the drains. However they are concerned. They plan to initiate video inspections and are actively seeking, through research, alternatives to the existing systems. However the DOT feels drainage is critical. They would not do a rubblization job if they could not install a subsurface drainage system.

Factor 3 Traffic Handling

Where possible the LaDOT would like to keep 2 lanes open at all times. This means that they must add width to the inside lane. Therefore they have to let traffic run on the outside shoulder during construction. This has caused problems. They use the FWD to structurally evaluate their shoulders in the planning stage.

In many cases they simply close the IH down to one lane.

Other important Topics

1) Specifications LaDOT provided the project team with a complete set of specifications. They find that the test pit is critical early in the project to ensure that the process is given the correct size pieces.

2) Exceptions They do not rubblize on bridge approaches and on underpasses. They use a simple overlay with sawed and sealed joints.
"*Comment…This is a problem you can clearly see reflection cracks even on new sections…this is something we need to look at*"

3) Construction Sequence The sequence used in Louisiana is shown in Figures C-1 thru C-7.
"Comment….. this is very interesting and we need to study this carefully"

4) Drainage system Details of the current drainage system were also supplied to the project team. They use a rodent guard on the outlet pipes. It appears to be doing OK now but it is a cause for concern.

"*Comment… one idea could be to move the drain from the edge of the concrete by using a permeable asphalt base on the shoulder, then installing a pipe system at the edge of the shoulder to daylight the layer. This would be expensive but it will move the drain away from the pavement edge where it could be more easily maintained*"

5 Steel in Slab All of the JCP on IH 10 has wire mesh in the slab. This causes a few minor problems. In some jobs a few areas are found where the wire moves to the surface after rubblization, in these cases the DOT simply cuts and removes the wire. Most times the wire stays buried in the slab.
"Comment….we have no idea if this process will work as well on plain JCP, LaDOT thought the wire helps to hold the concrete together during rubblization"

6 Priming the Surface LaDOT found problems with placing a prime on the rubblized concrete. The found that the fines on the surface causes the prime not to stick, it was easily picked up by construction traffic. Currently they place hot mix directly on top of the broken concrete
"*Comment…This is a concern, somehow I think we need to seal the rubblized concrete, perhaps a chip seal placed on top of the concrete or a seal on top of the first hot mix layer"*

7 Type of Aggregate All of their concrete is made from river gravel; this is very hard on the rubblizing equipment. LaDOT commented that they better have plenty of back up equipment.

8 Warranty on Hot Mix In all cases LaDOT placed a 7 – 8 inch layer of hot mix on top of the rubblized concrete. They use a warranty on the HMA overlay. This is shown in attachment 6-1 thru 6-5. The commented that the only real performance problems on early projects was with the quality of the Hot Mix. This they feel they have corrected with the warranty.

9 Additional Comments Figure C9 has additional written answers to comments TxDOT submitted in advance to the meeting. These were written by Mr. Lablanc the District construction engineer.

Where to Go from Here

1) As soon as possible we should complete the NDT testing on the sections in Louisiana. We need to collect FWD and GPR data on perhaps their oldest section. We need a minimum of 20 FWD drops spaced equally along the project. GPR data should be collected to assess if moisture is trapped in the rubblized base. LaDOT also has a large project under construction next to the Texas line. We perhaps need to get some center slab and joint deflections on this project.
2) We need to agree on acceptance criteria for rubblization candidate projects, based on a) FWD testing, b) drainage criteria and c) traffic levels. I will continue to search published literature to see what I can come up with.
3) Discuss the criteria and design recommendations with districts that have substantial amounts of JCP pavements who will consider rubblization as an option (Beaumont, Houston, Dallas).

Conclusion

The visual performance of the sections in Louisiana is clearly impressive. However we need to collect more NDT data to verify this. The experiences of East Texas Districts have not been too impressive, however they may not be a fair representation of what to expect from rubblization. Clearly although it looks promising we still have unanswered questions about the performance of the longitudinal drainage systems and the long-term performance of the IH 10 projects. I would not recommend this treatment on any major IH projects in Texas without more data from Louisiana and without constructing a few demonstration projects on less critical routes.

F.A.P.	STATE PROJECT	PARISH	SHEET NO.
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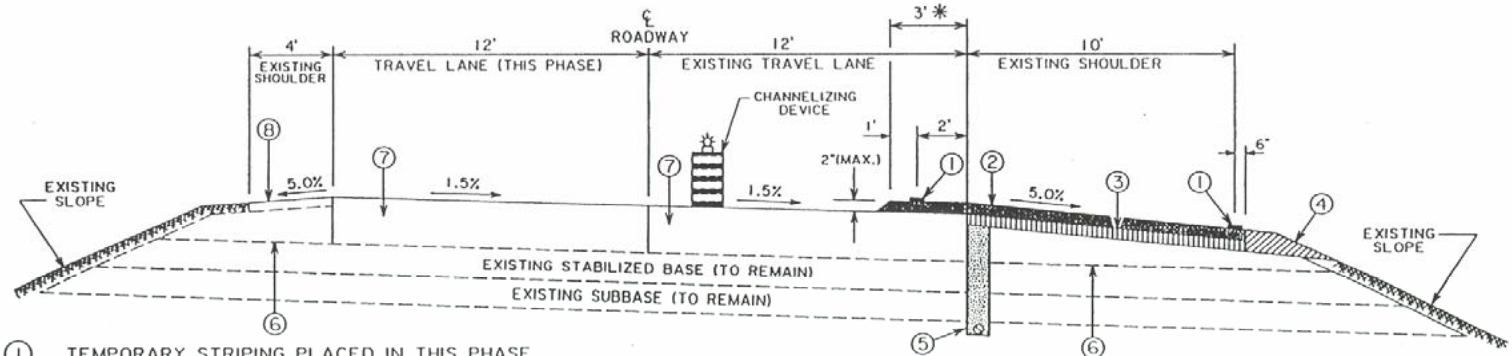


1. TEMPORARY STRIPING PLACED IN THIS PHASE
2. 2.0" SUPERPAVE ASPHALTIC CONCRETE BINDER COURSE ③
3. 2.0" SUPERPAVE ASPHALTIC CONCRETE BINDER COURSE
4. EMBANKMENT MATERIAL TO MATCH GRADE OF ADJACENT ASPHALTIC CONCRETE
5. REQUIRED UNDERDRAIN SYSTEM
6. EXISTING SHOULDER BASE COURSE TO REMAIN (PATCH AS REQUIRED)
7. EXISTING 10" P.C.C. PAVEMENT TO REMAIN AS IS DURING THIS PHASE
8. EXISTING ASPHALTIC CONCRETE SURFACING TO REMAIN DURING THIS PHASE

NOTES:

UNDERDRAIN SYSTEM SHALL BE BACKFILLED, COMPACTED, AND GRADED TO THE TOP OF EXCAVATED TRENCHES AT THE END OF EACH WORK DAY.

FOR MORE INFORMATION SEE "TYPICAL FINISHED SECTION."

PHASE I

(1) CHANNEL TRAFFIC ONTO EXISTING 12' TRAVEL LANE LEFT OF ℄
(2) COLD PLANE FULL DEPTH ASPHALTIC CONCRETE FROM RIGHT SHOULDER (2" AVG.)
(3) REMOVE EXISTING UNDERDRAIN SYSTEM (RIGHT SHOULDER)
(4) INSTALL NEW SHOULDER UNDERDRAIN SYSTEM AND OUTLETS USING MOVING OPERATION
(5) BACKFILL UNDERDRAIN SYSTEM AS REQUIRED (SEE "SHOULDER UNDERDRAIN DETAILS")
(6) PATCH EXISTING BASE COURSE (RIGHT SHOULDER) AS DIRECTED BY THE PROJECT ENGINEER
(7) APPLY 2.0" SUPERPAVE ASPHALTIC CONCRETE ON RIGHT SHOULDER
(8) APPLY 2.0" SUPERPAVE CONCRETE ON RIGHT SHOULDER AND ROADWAY AS SHOWN
(9) USING EMBANKMENT MATERIAL, BRING FORESLOPES UP TO MATCH BINDER COURSE ON RIGHT SHOULDER
(10) STRIPE AS SHOWN FOR NEXT PHASE OF OPERATION
(11) APPLY EROSION CONTROL WHEN REQUIRED BY PROJECT ENGINEER

✻ THE 2" x 3' ASPHALTIC CONCRETE PLACED OVER THE EXISTING P.C.C. PAVEMENT SHALL BE REMOVED BEFORE PERFORMING THE RUBBLIZING OPERATION IN PHASE III.

ALL SEQUENCE OF CONSTRUCTION SHEETS ARE SUGGESTED AND MAY BE CHANGED AT THE DISCRETION OF THE PROJECT ENGINEER. ③

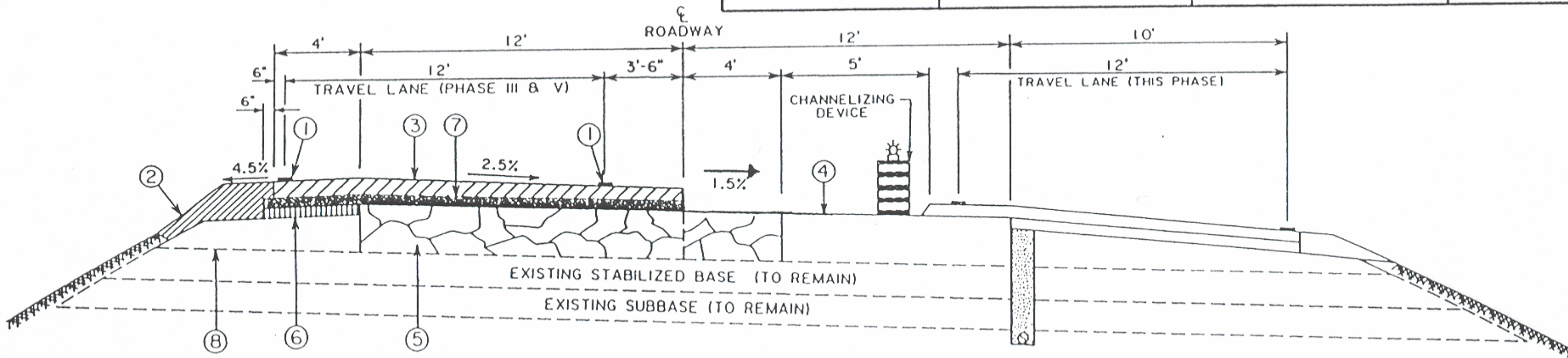
SHEET 1 OF 7

REVISIONS					
Date	Description	By	SEQUENCE OF CONSTRUCTION (DETAIL)	DESIGNED: J.E.F.	DETAILED: L.L.,M.P.
③ 6-12-1	ADDED/REMOVED NOTE	NKI		DATE:	DATE: 03-05-2001
				I:/PavPresProg/450030037/	CHECKED:
				rubblize/(seq-of-const)-mod.dgn	DATE:

K-73 (Revised 06/14/01)

Figure C1. Construction Sequence for Rubblization Projects in Louisiana (Step 1 of 7).

F.A.P.	STATE PROJECT	PARISH	SHEET NO.
IM-10-1(161)051	450-03-0037	JEFF DAVIS	65



(1) TEMPORARY STRIPING PLACED IN THIS PHASE
(2) EMBANKMENT MATERIAL TO MATCH GRADE OF ADJACENT ASPHALTIC CONCRETE
(3) SUPERPAVE ASPHALTIC CONCRETE BINDER COURSE (THICKNESS VARIES: 2.95" TO 3.19" ON SHOULDER AND 3.19" TO 2.47" ON ROADWAY)
(4) EXISTING 10" P.C.C. PAVEMENT TO REMAIN AS IS DURING THIS PHASE
(5) EXISTING 10" P.C.C. PAVEMENT (LEFT LANE) TO BE RUBBLIZED DURING THIS PHASE
(6) 2.0" SUPERPAVE ASPHALTIC CONCRETE BINDER COURSE
(7) 2.0" SUPERPAVE ASPHALTIC CONCRETE BINDER COURSE
(8) EXISTING SHOULDER BASE COURSE TO REMAIN (PATCH AS REQUIRED)

PHASE II

(1) CHANNEL TRAFFIC ONTO TEMPORARY 12' TRAVEL LANE RIGHT OF ℄
(2) COLD PLANE FULL DEPTH ASPHALTIC CONCRETE FROM LEFT SHOULDER (2" AVG.)
(3) PATCH EXISTING BASE COURSE (LEFT SHOULDER) AS DIRECTED BY THE PROJECT ENGINEER
(4) APPLY 2.0" ASPHALTIC CONCRETE SUPERPAVE BINDER COURSE ON LEFT SHOULDER
(5) RUBBLIZE EXISTING 10" P.C.C. PAVEMENT (LEFT LANE)
(6) BLADE BACK EXISTING TOPSOIL AND VEGETATION AND WINDROW FOR FUTURE USE
(7) APPLY SUPERPAVE ASPHALTIC CONCRETE BINDER COURSE AS SHOWN ON LEFT LANE AND SHOULDER
(8) USING EMBANKMENT MATERIAL, BRING FORESLOPES UP TO MATCH BINDER COURSE ON LEFT SHOULDER
(9) STRIPE AS SHOWN FOR NEXT PHASE OF OPERATION
(10) APPLY EROSION CONTROL WHEN REQUIRED BY PROJECT ENGINEER

NOTE:

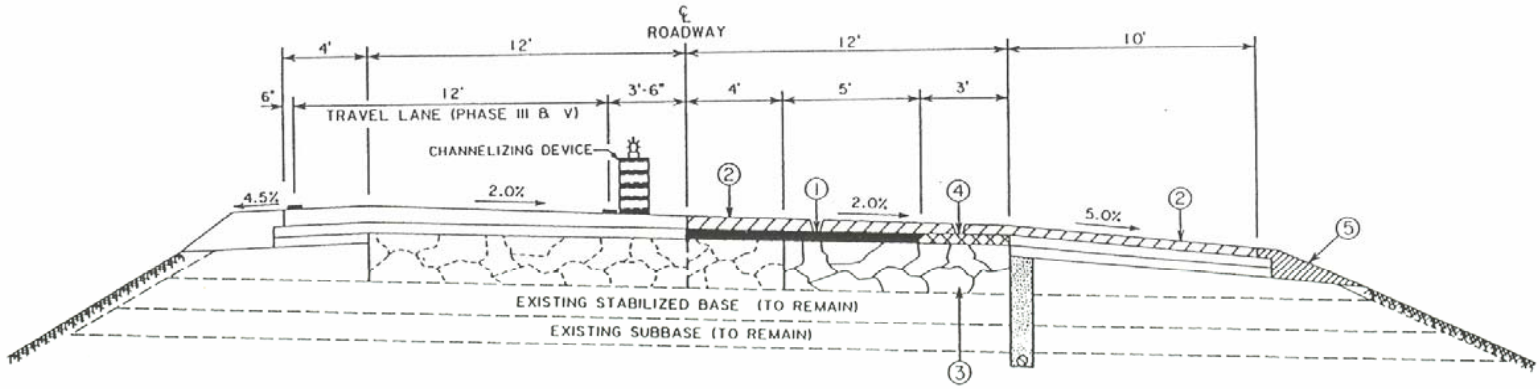
FOR MORE INFORMATION SEE "TYPICAL FINISHED SECTION."

SHEET 2 OF 7

REVISIONS			SEQUENCE OF CONSTRUCTION (DETAIL)	DESIGNED: J.E.F.	DETAILED: L.L..M.P.
Date	Description	By		DATE:	DATE: 03-05-2001
				I:/PovPresProg/450030037/	CHECKED:
				rubblize/(seq-of-const)-mod.dgn	DATE:

Figure C2. Construction Sequence for Rubblization Projects in Louisiana (Step 2 of 7).

F.A.P.	STATE PROJECT	PARISH	SHEET NO.
IM-10-1(161)051	450-03-0037	JEFF DAVIS	66



(1) 2" SUPERPAVE ASPHALTIC CONCRETE BINDER COURSE
(2) SUPERPAVE ASPHALTIC CONCRETE BINDER COURSE (THICKNESS VARIES: 2.47" TO 1.75")
(3) EXISTING 10" P.C.C. PAVEMENT (RIGHT LANE) TO BE RUBBLIZED DURING THIS PHASE
(4) ASPHALTIC CONCRETE (PLACED IN PHASE I) TO BE REMOVED AND REPLACED WITH (1)
(5) EMBANKMENT MATERIAL TO MATCH GRADE OF ADJACENT ASPHALTIC CONCRETE

NOTE:

FOR MORE INFORMATION SEE "TYPICAL FINISHED SECTION."

PHASE III

(1) CHANNEL TRAFFIC ONTO TEMPORARY 12' TRAVEL LANE LEFT OF ℄
(2) COLD PLANE THE 2"x3' ASPHALTIC CONCRETE (PLACED IN PHASE I) TO PERFORM RUBBLIZING OPERATION
(3) RUBBLIZE EXISTING 10" P.C.C. PAVEMENT (RIGHT LANE)
(4) APPLY SUPERPAVE ASPHALTIC CONCRETE BINDER COURSE AS SHOWN ON RIGHT LANE AND SHOULDER
(5) USING EMBANKMENT MATERIAL, BRING FORESLOPES UP TO MATCH BINDER COURSE ON RIGHT SHOULDER
(6) APPLY EROSION CONTROL WHEN REQUIRED BY PROJECT ENGINEER

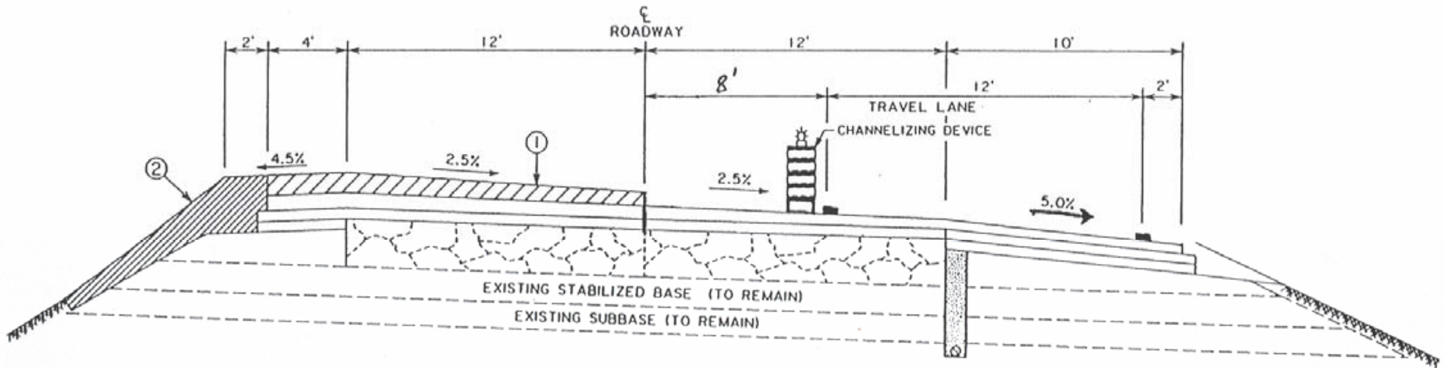
SHEET 3 OF 7

REVISIONS			
Date	Description	By	
			SEQUENCE OF CONSTRUCTION (DETAIL)

DESIGNED: J.E.F.	DETAILED: L.L..M.P.
DATE:	DATE: 03-05-2001
I:/PavPresProg/450030037/	CHECKED:
rubblize/(seq-of-const)-mod.dgn	DATE:

Figure C3. Construction Sequence for Rubblization Projects in Louisiana (Step 3 of 7).

F.A.P.	STATE PROJECT	PARISH	SHEET NO.
IM-10-1(161)051	450-03-0037	JEFF DAVIS	67



① SUPERPAVE ASPHALTIC CONCRETE BINDER COURSE (THICKNESS VARIES: 3.19" TO 2.35")
② EMBANKMENT MATERIAL TO MATCH GRADE OF ADJACENT ASPHALTIC CONCRETE

NOTE:

FOR MORE INFORMATION SEE "TYPICAL FINISHED SECTION."

PHASE IV

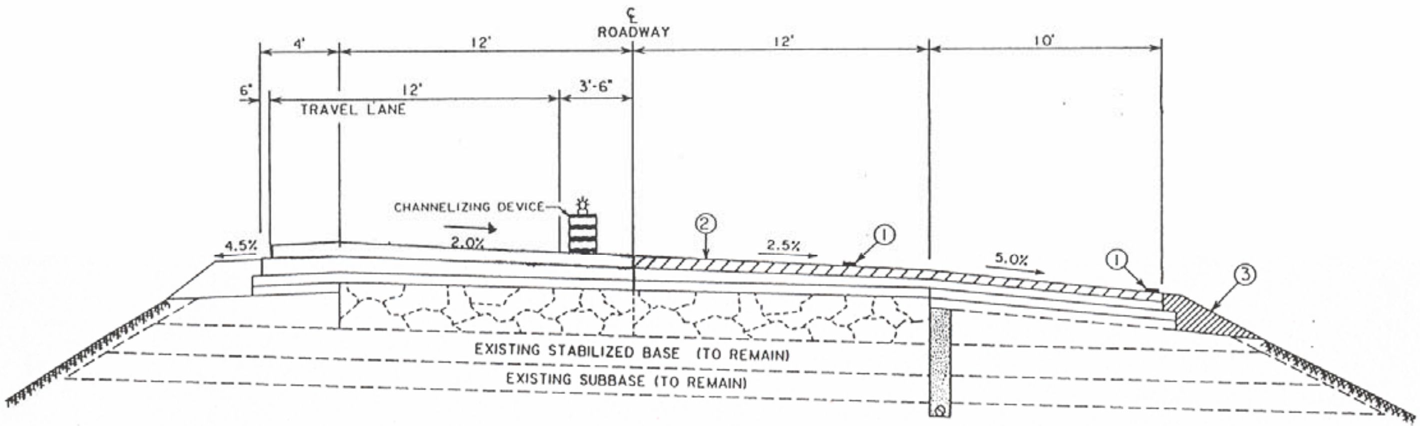
(1) CHANNEL TRAFFIC ONTO TEMPORARY 12' TRAVEL LANE RIGHT OF ℄
(2) APPLY ASPHALTIC CONCRETE BINDER COURSE AS SHOWN ON LEFT LANE AND SHOULDER
(3) USING EMBANKMENT MATERIAL, BRING FORESLOPES UP TO MATCH WEARING COURSE ON LEFT SHOULDER
(4) APPLY EROSION CONTROL WHEN REQUIRED BY PROJECT ENGINEER

SHEET 5 OF 7

REVISIONS			SEQUENCE OF CONSTRUCTION (DETAIL)	DESIGNED: J.E.F.	DETAILED: L.L.,M.P.
Date	Description	By		DATE:	DATE: 03-05-2001
				I:/PavPresProg/450030037/	CHECKED:
				rubblize/(seq-of-const)-mod.dgn	DATE:

Figure C4. Construction Sequence for Rubblization Projects in Louisiana (Step 4 of 7).

F.A.P.	STATE PROJECT	PARISH	SHEET NO.
IM-10-1(161)051	450-03-0037	JEFF DAVIS	68



(1) TEMPORARY STRIPING PLACED IN THIS PHASE
(2) SUPERPAVE ASPHALTIC CONCRETE BINDER COURSE (THICKNESS VARIES: 2.35" TO 1.75")
(3) EMBANKMENT MATERIAL TO MATCH GRADE OF ADJACENT ASPHALTIC CONCRETE

NOTE:

FOR MORE INFORMATION SEE "TYPICAL FINISHED SECTION."

PHASE V

(1) CHANNEL TRAFFIC ONTO TEMPORARY 12' TRAVEL LANE LEFT OF ℄
(2) APPLY ASPHALTIC CONCRETE BINDER COURSE AS SHOWN ON RIGHT LANE AND SHOULDER
(3) USING EMBANKMENT MATERIAL, BRING FORESLOPES UP TO MATCH BINDER COURSE ON RIGHT SHOULDER
(4) STRIPE AS SHOWN FOR NEXT PHASE OF OPERATION
(5) APPLY EROSION CONTROL WHEN REQUIRED BY PROJECT ENGINEER

SHEET 4 OF 7

REVISIONS					
Date	Description	By	SEQUENCE OF CONSTRUCTION (DETAIL)	DESIGNED: J.E.F.	DETAILED: L.L.,M.P.
				DATE:	DATE: 03-05-2001
				I:/PavPresProg/450030037/	CHECKED:
				rubblize/(seq-of-const)-mod.dgn	DATE:

Figure C5. Construction Sequence for Rubblization Projects in Louisiana (Step 5 of 7).

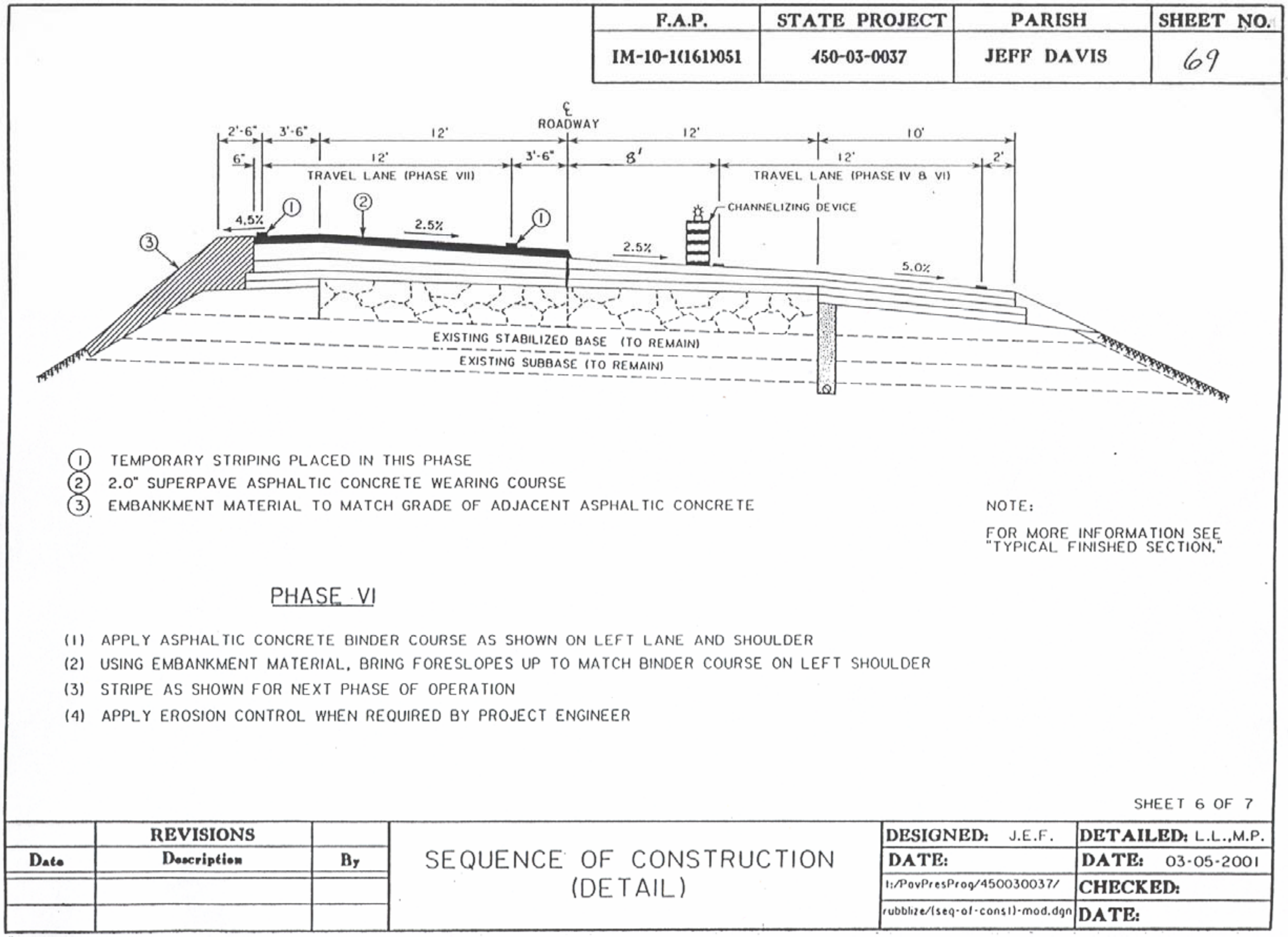
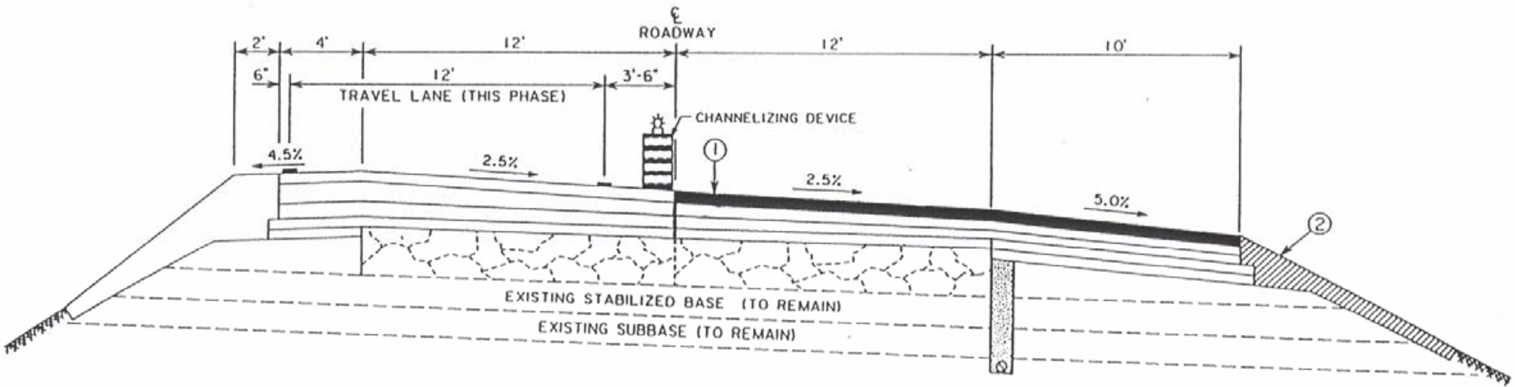


Figure C6. Construction Sequence for Rubblization Projects in Louisiana (Step 6 of 7).

F.A.P.	STATE PROJECT	PARISH	SHEET NO.
IM-10-1(161)051	450-03-0037	JEFF DAVIS	70



① 2.0" SUPERPAVE ASPHALTIC CONCRETE WEARING COURSE
② EMBANKMENT MATERIAL TO MATCH GRADE OF ADJACENT ASPHALTIC CONCRETE

NOTE:

FOR MORE INFORMATION SEE "TYPICAL FINISHED SECTION."

PHASE VII

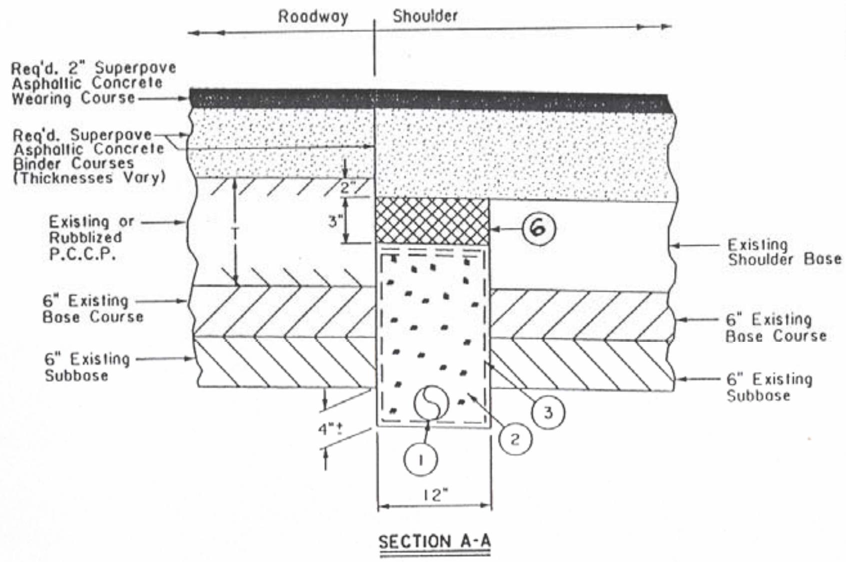
(1) CHANNEL TRAFFIC ONTO TEMPORARY 12" TRAVEL LANE LEFT OF ℄
(2) APPLY ASPHALTIC CONCRETE WEARING COURSE AS SHOWN ON RIGHT LANE AND SHOULDER
(3) USING EMBANKMENT MATERIAL, BRING FORESLOPES UP TO MATCH WEARING COURSE ON RIGHT SHOULDER
(4) APPLY REQUIRED STRIPING, PAVEMENT MARKERS, AND EROSION CONTROL
(5) COMPLETE ALL REMAINING ITEMS OF WORK

SHEET 7 OF 7

REVISIONS			SEQUENCE OF CONSTRUCTION (DETAIL)	DESIGNED: J.E.F.	DETAILED: L.L.,M.P.
Date	Description	By		DATE:	DATE: 03-05-2001
				I:/PavPresProg/450030037/	CHECKED:
				rubblize/(seq-of-const)-mod.dgn	DATE:

Figure C7. Construction Sequence for Rubblization Projects in Louisiana (Step 7 of 7).

F.A.P.	STATE PROJECT	PARISH	SHEET NO.
IM-10-1(161)051	450-03-0037	JEFF DAVIS	15



LEGEND

① 4" PLASTIC PIPE (PERFORATED)

② AGGREGATE

③ PLASTIC FILTER CLOTH

④ 4" PLASTIC PIPE (NON PERFORATED)

⑤ CONCRETE SLOTTED HEADWALL

⑥ ASPHALTIC CONCRETE CAP
(COST TO BE INCLUDED IN THE SHOULDER
UNDERDRAIN PAY ITEM)

Sheet 2 of 2
Not To Scale

REVISIONS			SHOULDER UNDERDRAIN DETAILS (PERFORATED PIPE)	DESIGNED: J.E.F.	DETAILED: M.P., L.H.L.
Date	Description	By		DATE:	DATE: 02-26-2001
				CHECKED:	CHECKED:
				T:/PavPresPro(1)/450-03-0037/ Rubblize/udraindtls.dgn	DATE:

Figure C8. Details of Underdrain System Used by LaDOT on Rubblization Projects.

P. O. BOX 1430
LAKE CHARLES, LA 70602
337-437-9103
December 3, 2002

DAR:

An attempt to answer your questions is attached herewith:

(1) What are good candidates for rubblization?

Answer: Old existing busted up PCC Pavement that is beyond repair conventionally, and generally has good bases and adequate drainage.

(2) When would we not rubblize?

Answer: Rubblization could not be done in areas where there are overhead clearance problems, depth coverage over existing cross-drains (box culverts, pipes) is insufficient, and where there may exist significant sub grade moisture. minimum of clearance approach ramp.

(3) When did they install drainage? (If before have they checked that it is still working?)

Answer: All new drainage shoulder under drain systems and shoulder outlet under drains were installed prior to the rubblizing process.

(4) What pavement design did they use?

Answer: The Darwin Design program used.

(5) What thickness and type of HMA used?

Answer: Please review typical sections of each project being rubblized. However, normally the mix placed varies from 6" to 9" of Level 3 Superpave binder with a min. of 2" of Level 3 Superpave or SMA wearing courses.

(6) Any weather problems? (What happens if it gets wet?)

Answer: We have weather limitations (temperature requirements) on the type of HMAC being placed on the roadway. Rains temporarily delay lay down operations, but in general the contractor makes up time by laying HMAC 24 hours/day. If it gets wet, with the shoulder drainage system in place, a higher percentage of the water

• If there is bad base, reconstruction is only way.
the [illegible] is not set. Sand shale.

Figure C9. Comments on Rubblization Provided by LaDOT District Construction Engineer (Mr. Leblanc).

escapes away for the roadway in a short time and the rubblized pavements dry up fairly rapidly.

(7) What about tie-ins with existing ramps, bridge decks, etc.?

Answer: At these locations, the contractor is made to stop the rubblizing, clean and re-seal joint in the concrete, replace concrete roadway panels that are required, taper the overlay, and then saw and seal the roadway joints coming from the concrete. Transition details are found in the plans.

(8) Can you do this in urban areas?

Answer: It can be accomplished in urban areas. However, there are latent problems that could affect the work. For example, urban areas are usually limited by drainage issued and normally contain some curb and gutter sections. Overlay thicknesses required for rubblization would affect every one of these areas.

(9) What about costs?

Answer: The costs for rubblization are a lot less than for reconstruction, but higher that overlay costs. For example, costs usually run from $300K-$400K per lane mile depending on traffic control costs, and other factors.

(10) What monitoring data do they have?

Answer: None that we are aware of unless LTRC has some monitoring system in place.

(11) What problems were encountered during construction?

Answer: Some of the problems encountered on any of these projects are usually traffic flow during peak hours of travel. Keeping the rubber-neckers moving is always a problem. Whenever there were accidents, traffic backups were a major problem. Additionally, the hard siliceous gravel encountered by the rubblizing equipment caused many breakdowns in the contractor's equipment. This has temporarily slowed down operations for the contractor.

(12) How did they prepare the surface for the HMAC?

Answer: After completion of the rubblization, one pass was made with a 10 ton steel wheel vibratory roller; one pass with a pneumatic tire roller, and two additional passes with a 10 ton steel wheel vibratory roller. Then any reinforcing steel and protruding wires were removed from the rubblized surface.

(13) How did they now how to stop breaking?

Figure C9. Comments on Rubblization Provided by LaDOT District Construction Engineer (Mr. Leblanc) (Continued).

Answer: Rubblizing is a one-pass operation with a 10-inch shoe beginning at one edge of the pavement and proceeding lengthwise and across the roadway to the opposite edge. The quality of the rubblizing process is very dependent on the existing stability of the base.

(14) Can we have copies of their latest specs and typical sections?

Answer: Yes.

Hopefully this data can be of assistance to you.

LESTER J. LEBLANC, P. E.
DISTRICT CONSTRUCTION ENGINEER

Figure C9. Comments on Rubblization Provided by LaDOT District Construction Engineer (Mr. Leblanc) (Continued).