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16. Abstract

Selecting rehabilitation options for Jointed Concrete Pavements (JCP's) continues to be one of the most challenging tasks for pavement engineers. In project 0-4517, the performance of numerous treatments was investigated. Reports 0-4517-1 and 0-4517-3 identified treatments that are performing well and those that are not. Report 0-4517-2 proposed a field investigation plan for testing future candidate projects that combines visual inspections with nondestructive testing (NDT). A sequenced approach is proposed for NDT evaluation that includes Ground Penetrating Radar (GPR) and deflection testing and in some instances Dynamic Cone Penetrometer (DCP) testing. Deflections can be taken with either the Falling Weight Defelctometer (FWD) or Rolling Dynamic Deflectometer (RDD).

Three treatments were found to be performing well. For JCPs without major failures, the use of an asphalt overlay should be considered. Good agreement was found between field reflection cracking performance and the results from laboratory testing with TTI's overlay tester. Future overlay of JCPs should follow the overlay tester criteria proposed in this report. For sections with moderate levels of deterioration, with adequate support and no trapped moisture the use of rubblization is a good alternative. The rubblized concrete was found to provide a more uniform support than Crack and Seat. For those sections that are not candidates for rubblization because of poor slab support, then a flexible base overlay appears to be a good alternative. The flexible base overlays on US 59 in Lufkin and US 83 in Childress are performing very well.

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METHODS OF REDUCING JOINT REFLECTION CRACKING: FIELD PERFORMANCE STUDIES

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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).

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.

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CHAPTER 1
INTRODUCTION

Project 0-4517 was established to develop statewide guidelines on how to select rehabilitation options for jointed concrete pavement (JCP). The objectives are 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 to avoid joint reflective cracking."

This report covers the activities in Year 3 of this project. Two earlier reports have been submitted to TxDOT. Report 0-4517-1 entitled "Performance Report on Jointed Concrete Pavement Repair Strategies in Texas" (Scullion and Von Holdt, 2004) summarized the findings from the Lufkin experiment and also surveyed the performance of several additional treatments around the state. In Lufkin, it was shown that the best performing strategies were either those with large stone (Arkansas) mixes or flexible base overlays. Crack and Seat techniques did not perform well in the Lufkin experiment due to the weak base layer and the inability to effectively seat the broken concrete. Variable performances were also found from three rubblization projects; slab fracturing techniques can work well, but they are not appropriate for all cases. In other studies, it was found that grids placed between asphalt layers did not perform well primarily because of layer debonding problems.

Report 0-4517-2 entitled "Using Rolling Deflectometer and Ground Penetrating Radar Technologies for Full Coverage Testing of Jointed Concrete Pavements" (Scullion, 2005) presented a field testing plan for jointed concrete pavements. Key components of the plan are the extensive use of advanced nondestructive testing equipment available in Texas. The use of Ground Penetrating Radar (GPR) is recommended to measure the thickness of cover over JCP and to identify pockets of sub-slab trapped moisture. Deflection techniques are also required to measure both load transfer efficiency and center slab strength. The use of either the Rolling Dynamic Deflectometer (RDD)or the Falling Weight Deflectometer (FWD) was recommended.

In the first two years of this project, numerous additional test sections where districts had used innovative techniques to rehabilitate jointed concrete pavements were also identified. To complete the evaluation of treatments used in Texas, a 1-year extension was given to Project 0-4517 to include the evaluation of the following experimental sections:

1) Experimental Section on US 83 in Childress

US 83 runs the entire length of the Childress District. This roadway is largely an old jointed concrete pavement, which over the past 10 years has been largely rehabilitated. The Childress District has employed a whole range of rehabilitation approaches, including rubblization, crack and seat, various types of asphalt overlays, flexible base overlays, use of fabrics, etc. In this evaluation, the performance of each will be monitored. As a minimum, this will include visual condition data, FWD data, and GPR data. If performance problems are encountered, a forensic study will be initiated to identify the cause of the problem. District design, lab, and construction staff will be interviewed to document their experiences with each treatment.

2) Experimental Section on US 175

In 2003, the Dallas District rehabilitated a 3-mile section of jointed concrete on US 175 just south of the IH 635 loop. Two propriety treatments were used; Strata® (Koch Materials) and the GlasGrid® product (Bay Mills) were placed beneath a thin hot mix surface layer. The performance of these sections will be determined and the district's construction experiences documented. Testing will include GPR, field coring, and laboratory testing.

This Year 3 report contains an evaluation of both of these experimental sections.

CHAPTER 2
EVALUATION OF GRID PRODUCTS

In Report 0-4517-1, construction and early pavement performance problems were documented with grid products. These are summarized in the following paragraph taken from the Report 0-4517-1:

> *"Several construction problems and failures shortly after construction were found around the state. In all cases, the grid was placed on a thin Hot Mix Asphalt (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 Coarse Matrix High Binder (CMHB) or Stone Matrix Asphalt (SMA), b) rainfall during construction, c) moisture problems with the self-adhesive glue used on the grid, or d) inadequate surface thickness."*

However, in these studies, it was commented that no evaluations were made of the numerous grid projects that have been placed in the Dallas District from 2001 to 2004. Several of these projects were thought to be performing well. Whereas many of the reported problems were found with a grid with small openings, the Dallas District has used grids with the larger grid opening (up to 1-inch).

The performance of the sections from the Dallas District are described in the remainder of this chapter.

LOOP 12, DALLAS DISTRICT (CONTACT: MAURICE PITTMAN)

A 2-mile section of Loop 12 between Shady Brook Lane and Lawther Drive was rehabilitated using the ½-inch GlasGrid product (8501). The section is three lanes per direction, and both directions were rehabilitated. The existing pavement was a 10-inch thick jointed concrete pavement.

A 2-ft wide membrane material was placed over the joints as part of the preparation for an overlay. The membrane was placed as a barrier for horizontal movement between the slab below and the layers above. A ¾-inch Type D hot mix asphalt level-up course was then applied. This was followed by a continuous layer of the grid, which was not tacked. The grid was overlaid with 2-inches of Type C HMA with polymer and was opened to traffic in July 2002. A typical pavement section is shown in Figure 1 below.

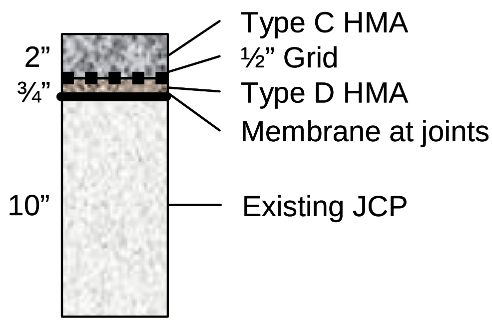


Figure 1. Typical Section on Loop 12.

The performance of this section was evaluated in June 2005, at an age of 3 years. A windshield visual survey revealed numerous transverse cracks and failures. A detailed visual inspection of 2500-ft of the eastbound section revealed 23 transverse cracks and 3 failures. Photographs of the transverse crack and failure distresses found on the section are shown in Figure 2 below.

Figure 2. Photograph of Distresses on Loop 12.

A GPR survey of the outside wheel path of the slow and middle lanes was conducted. The data show high reflections within the HMA layers at the level of the grid. For sections with grid products, this radar pattern has in the past indicated areas of debonding. The pattern was found over most of the section length of all surveyed lanes in both directions. Typical GPR data are shown in Figure 3 below.

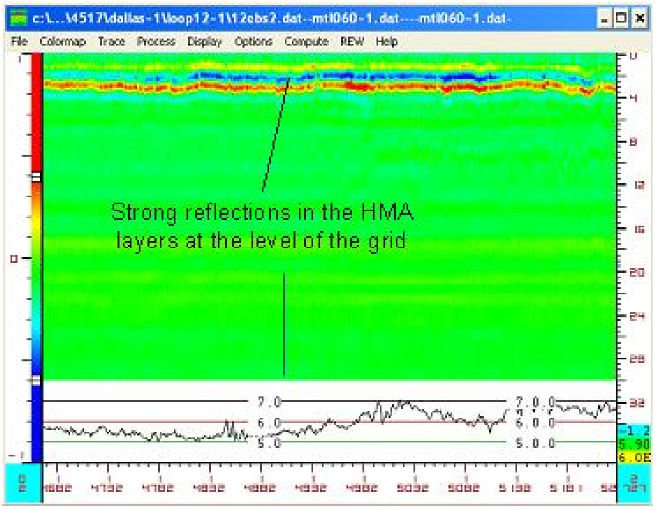


Figure 3. GPR Data Collected on Loop 12.

To gain a better understanding of the factors contributing to the development of the distresses, three trenches were evaluated on this section. The trenches were cut at a transverse crack, a failure, and on a section with no visible distress.

The trench with the reflection crack revealed that the surface course was poorly bonded to the grid and the underlying level-up course. The grid easily separated from the surface course when researchers lifted it out. The grid was torn at the crack, poorly bonded to the level-up course, and very weak. It could easily be torn apart by hand. Photographs of the trenching are shown in Figure 4.

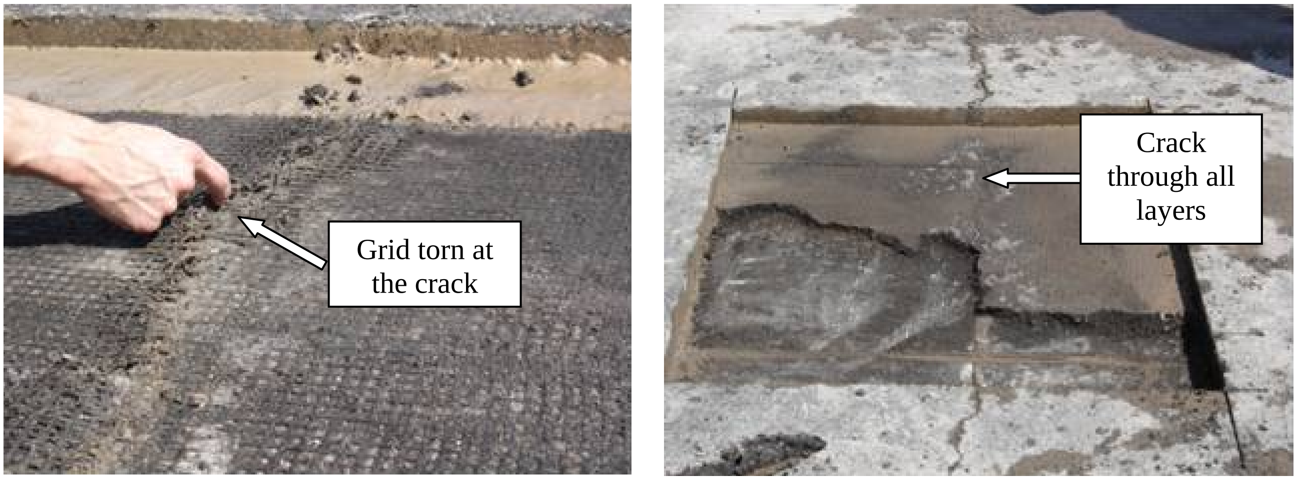


Figure 4. Photographs of a Trench at a Transverse Crack on Loop 12.

The trench with the failure revealed poor bonding between the grid and the HMA layers. The grid was severely deteriorated and could easily be torn apart by hand. The level-up layer was severely deteriorated and not well bonded to the underlying membrane over the joint. The membrane was intact and saturated with moisture. The failure did not occur at the joint, but at the edge of the membrane about 1½ ft from the joint where some cold patching seems to have been placed prior to the overlay. The mechanism of how this failure developed is still unclear, but the merits of the membrane at the joint have been questioned by the area office involved on this project. Photographs of the trenching are shown in Figure 5 below.

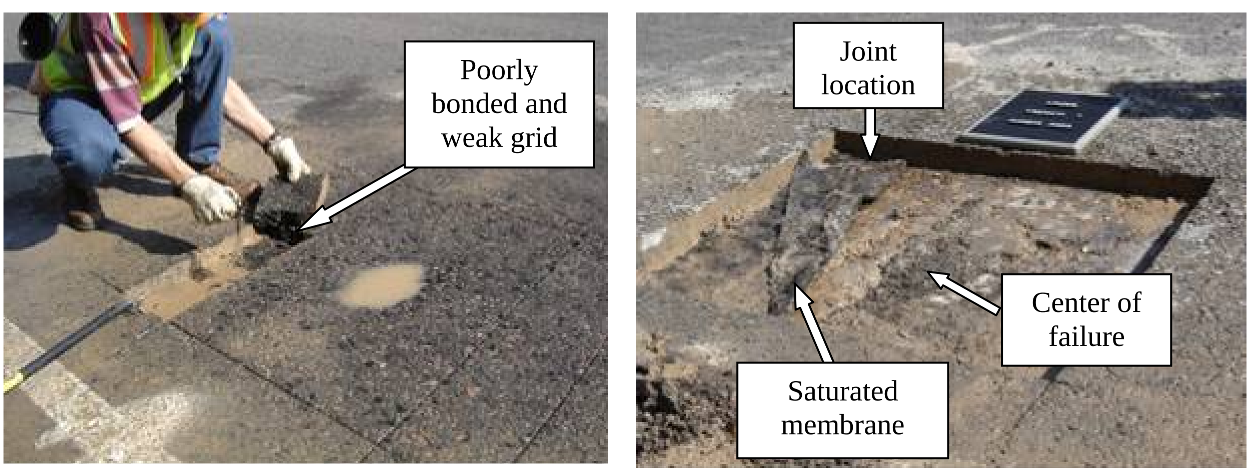


Figure 5. Photographs of a Trench at a Failure on Loop 12.

The trench on the section with no visible distress showed the same debonding and weak grid as seen at the trenches with the distresses.

Based on the radar data collected on this section and the findings from the trenching, it is apparent that the debonding of the asphalt layers at the grid is widespread on Loop 12 and the strength of the grid has deteriorated over the past 3 years. The grid was not highly effective in retarding the reflection of cracks through the HMA layers from the joints in the concrete pavement below.

SH 183, DALLAS DISTRICT (CONTACT: MAURICE PITTMAN)

A 4-mile section of the heavily trafficked SH 183 between Loop 12 and SH 161 was rehabilitated using the ½-inch GlasGrid product (8501). The 1-inch GlasGrid product (8511) was used on a 1000-ft section of the westbound outer lane at the start of the section on the Loop 12 end. The section is three lanes per direction, and both directions were rehabilitated. The existing pavement was a 9-inch thick jointed concrete pavement.

A 2-ft wide membrane material was placed over the joints as part of the preparation for the overlay to limit the transfer of horizontal movement from the slab below to the layers above. A ¾-inch Type D HMA level-up course was applied, followed by a continuous layer of the ½-inch opening grid, which was not tacked. The grid was overlaid with 2 inches of Type C HMA with latex. The section was opened to traffic in September 2002. A typical pavement section is shown in Figure 6.

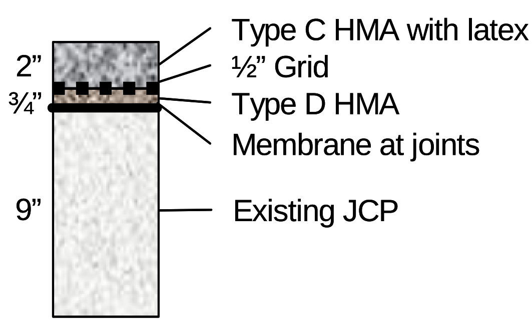


Figure 6. Typical Section on SH 183.

The performance of this section was evaluated in June 2005, at an age of 3 years. A windshield visual survey did not reveal any major distresses along the section. A detailed visual inspection of 1200-ft of the eastbound section revealed no distresses in the travel lanes. Some transverse cracks were noted in the shoulder, but these did not extend through to the travel lanes.

A GPR survey of the outside wheel path showed patterns of high reflections in the HMA layers, which may be associated with debonding. The GPR data from this section is shown in Figure 7.

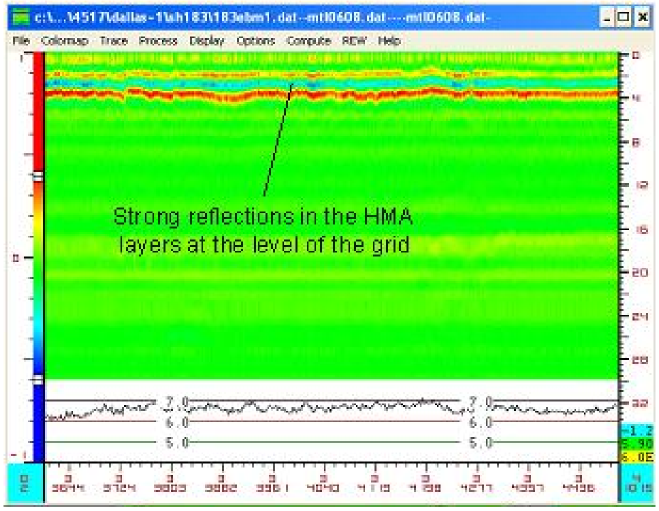


Figure 7. GPR Data Collected on SH 183.

A trench was cut on the section with the ½-inch GlasGrid product (8501) with no distresses to evaluate the bonding between the layers. The trench was cut in the middle of the lane and not in the wheel path. The trench revealed poor bonding between the grid and both the surface and level-up layers. This condition can be seen in Figure 8. The grid could be torn apart by hand, but not very easily.

Figure 8. Photograph of a Trench on a Good Section on SH 183.

Two trenches were cut in the section with the 1-inch GlasGrid product (8511); one at a location without a transverse crack and another at a location with a transverse crack. The trench at the location without the transverse crack revealed good bonding between the asphalt layers. The upper asphalt layer could not be easily lifted out from the trench due to the bond with the lower layers. The grid was firmly stuck to the level-up layer below. With a steady pull, the grid could be removed with tearing, and it dislodged some aggregate from the level-up layer. The bonding of the grid to the HMA layers was good and the condition of the grid was fairly good. A photograph of the grid is shown in Figure 9.

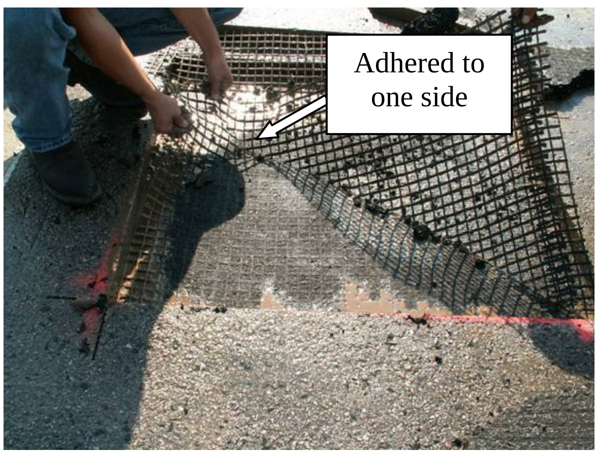


Figure 9. Photograph of a Well-Bonded Grid on SH 183.

The second trench, at the transverse crack, revealed a grid that was poorly bonded to the upper HMA, but adhered to the level-up layer. The condition of the grid appeared to be fairly good out of the wheel path, but poor within the wheel path area. This may indicate that the grid is deteriorating as a result of traffic loading. In this trench, the grid was not severed at the location of the crack. This may indicate that the crack can propagate through the grid without the grid actually severing. A photograph of the grid is shown in Figure 10.

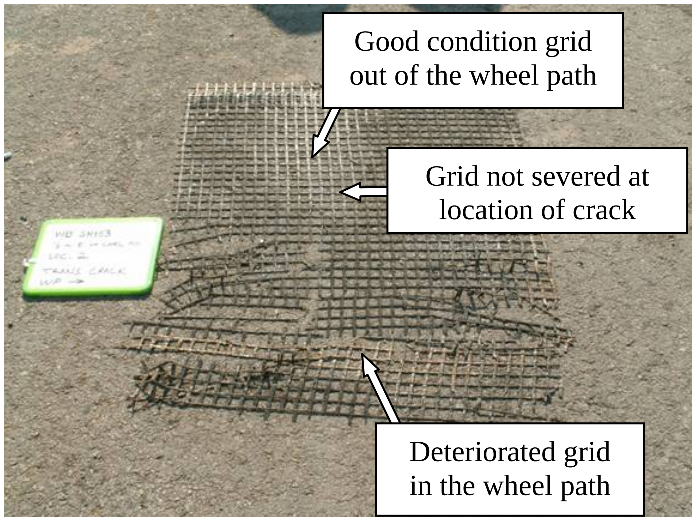


Figure 10. Photograph of an Extracted Grid on SH 183.

Based on the radar data collected on this section and the findings from the trenching, it appears that the debonding of the layers at the grid is widespread on SH 183. It also appears as if the grid may deteriorate quicker under the action of traffic loading. This section was performing well after 3 years, with few distresses.

US 175, DALLAS DISTRICT (CONTACT: GARY MOONSHOWER)

A 2½-mile section of US 175 was overlaid with HMA and the 1-inch GlasGrid product (8511). The section includes both lanes of the westbound direction between Seagoville Road and Woody Road. This was an old 8-inch thick jointed concrete pavement of over 40 years old that had performed reasonably well in the past.

The existing HMA surface was planed down and a 1-inch Type D HMA layer was placed as a level-up layer. This was followed by 10-ft wide continuous strips of the grid that was tacked to the level-up layer. The grid was overlaid with a 2-inch Type C HMA layer. This section was completed in October 2003. A photograph of construction and a typical section are shown in Figure 11.

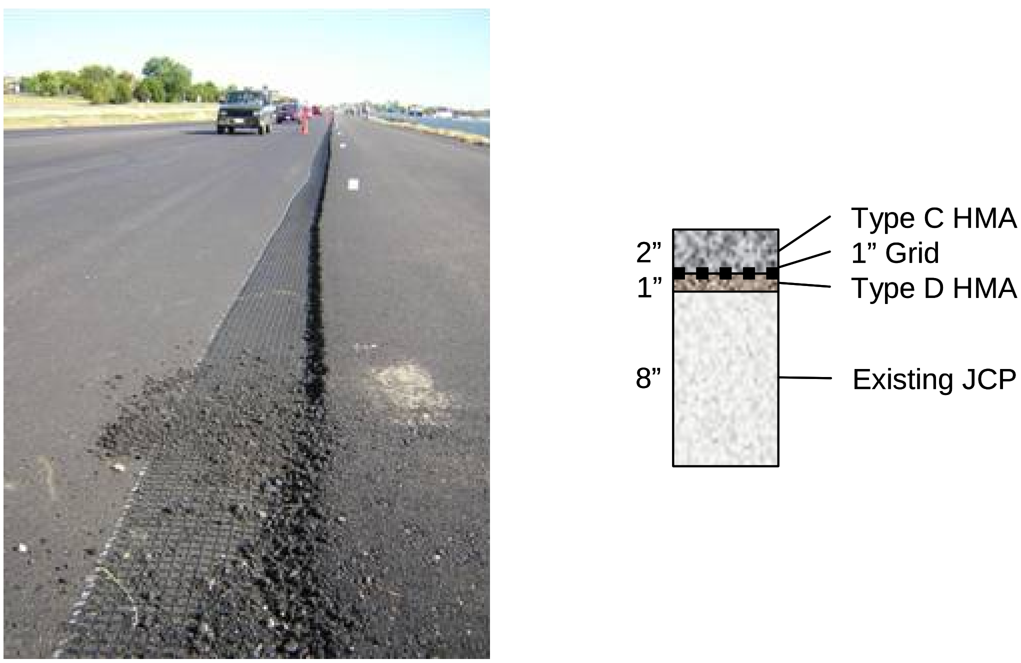


Figure 11. Photograph of Construction on US 175 and Typical Section.

The performance of this section was evaluated in June 2005, at an age of 2 years. A windshield visual survey revealed several transverse cracks across the full width of the pavement on one stretch of the road. A photograph of a typical distress is shown in Figure 12.

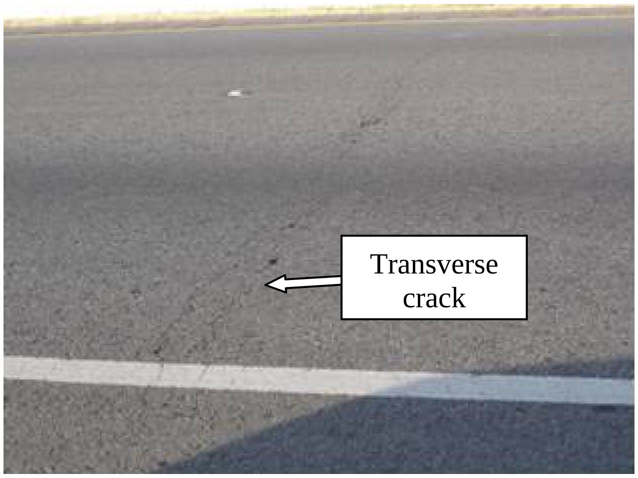


Figure 12. Photograph of a Transverse Crack on US 175.

A GPR survey of the outside wheel path of both lanes was conducted. The data show high reflections within the HMA layers at the level of the grid, which may indicate debonding. The pattern was found over almost the entire section length of both the outside and inside lanes. Typical GPR data are shown in Figure 13.

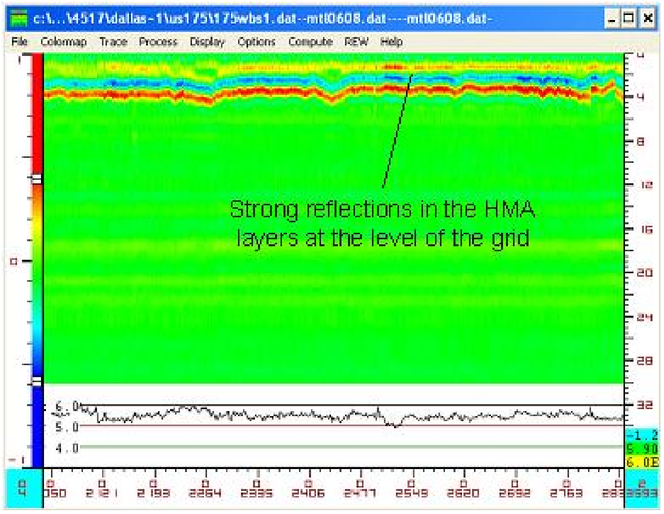


Figure 13. GPR Data Collected on Westbound US 175.

Two trenches were cut on the section. One trench was cut at a joint with a transverse crack and another at a joint with no transverse crack. The trench with the crack revealed that the grid was poorly bonded to the upper asphalt, but well bonded to the level-up layer, which was probably a result of the tack that was used. The grid was torn at the location of the crack. The grid was fairly strong, but could be torn apart by hand with some effort. A photograph of the trench is shown in Figure 14.

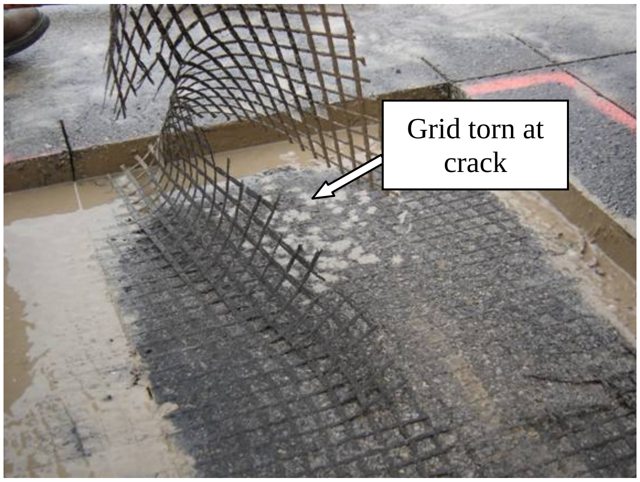


Figure 14. Photograph of a Trench at a Transverse Crack on US 175.

The trench at the joint with no transverse crack revealed a grid that was adhered to the level-up layer, but poorly bonded to the upper HMA layer. The grid was strong and could not be torn apart by hand. No evidence of cracking was found in the level-up layer beneath the grid.

Based on the radar data collected on this section and the findings from the trenching, it appears that the debonding of the layers at the grid is widespread on US 175. The bonding to the level-up layer appeared to be good where the tack coat was placed, but the bonding to the upper HMA layer was poor. The grid was torn through at the location of the transverse crack, indicating that the grid was not strong enough to withstand the stresses at the location of the crack. The grid was also easily torn apart, which indicates that it had deteriorated over time.

IH 35E, DALLAS DISTRICT (CONTACT: MAURICE PITTMAN)

A 4-mile section of IH 35E was overlaid with HMA and the 1-inch GlasGrid product (8511). The heavily trafficked section includes all lanes of both directions of IH 35E between Loop 12 and IH 635. The underlying pavement is an 8-inch continuously reinforced concrete pavement (CRCP).

The pavement consists of a ¾-inch Type D HMA level-up layer followed by the 1-inch GlasGrid product (8511) that was tacked to the level-up layer. The grid was overlaid with a 2-inch Type C HMA with polymer modified penetration grade binder. This section was completed in early 2005. A typical pavement section is shown in Figure 15.

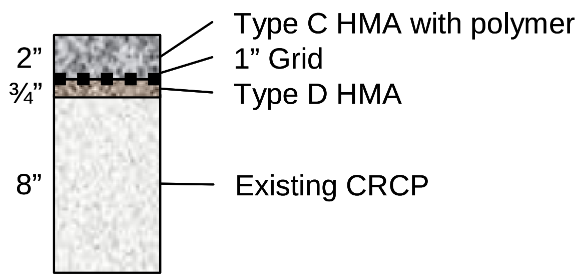


Figure 15. Typical Section on IH 35E.

The performance of this section was evaluated in June 2005, so the pavement was less than 1 year old. A windshield visual survey revealed no distresses.

A GPR survey of the outside wheel path of both lanes was conducted. The data show high reflections within the HMA layers at the level of the grid, which may indicate debonding. The pattern was found over almost the entire section length of both the outside and inside lanes. Typical GPR data are shown in Figure 16.

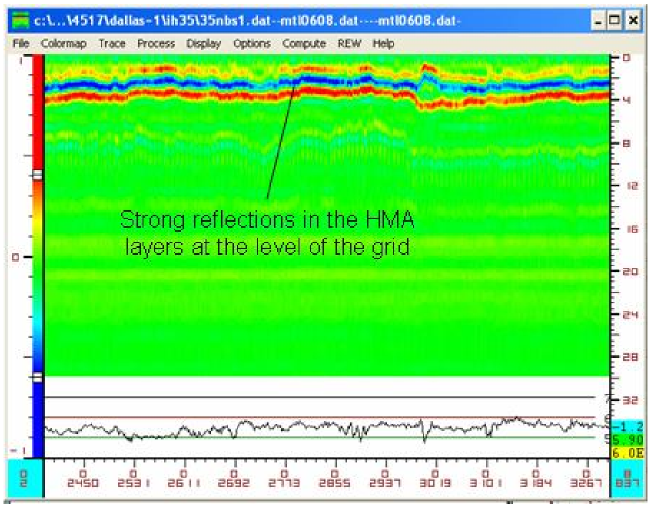


Figure 16. GPR Data Collected on IH 35E.

Two trenches were cut on this section. Both were at locations with no distresses. Both trenches revealed poor bonding to the upper HMA layer, but good bonding to the level-up layer. The good bonding may be a result of the tack coat that was used. The grid was in good condition and strong. A photograph of the well-bonded grid is shown in Figure 17.

Figure 17. Photograph of a Trench on IH 35E.

Based on the radar data collected on this section and the findings from the trenching, it appears that the debonding of the layers at the grid is widespread on IH 35E. The bonding to the level-up appeared to be good where the tack coat was placed, but the bonding to the upper HMA layer was poor. The grid was in good condition.

SUMMARY OF FINDINGS OF THE GLASGRID® PRODUCTS

The evaluation of the five road sections with GlasGrid products produced several findings, which are discussed below.

Several sections rehabilitated with the GlasGrid products had transverse cracks in the early life of the pavement. These transverse cracks reflected through from the underlying joints of the jointed concrete pavements. Some sections cracked as early as 2 years and 3 years after placement of the grid. The GlasGrid products did not prove to be highly effective in retarding the development of reflection cracks on old jointed concrete pavements.

The sections all appear to have widespread debonding at the level of the grid in the HMA layers. When a tack coat was used on the underlying HMA layer, the bond between the 1-inch grid and the underlying layer improved significantly. No tack coat was used on sections with the ½-inch grid, and no tack coat was used between the grid and the upper HMA layer. Debonding between the layers is a concern for future pavement performance. If the pavement starts to crack, moisture may penetrate the pavement and collect within the debonded area. This moisture penetration may lead to the accelerated deterioration of the pavement.

The GlasGrid products seem to deteriorate in strength under the action of traffic. The greatest deterioration appeared in the wheel path. Several of the grids had deteriorated to such a degree that they could easily be torn apart by hand. The lowered strength of the deteriorated grid may reduce its ability to retard the propagation of reflection cracks.

CHAPTER 3

CRACK RETARDING OVERLAYS

With regard to crack retarding layers, Report 0-4517-1 had the following conclusions about the use of crack retarding products such as Strata.

- The performance to date has been good; however, the oldest section is less than 2 years old so long-term monitoring is recommended.
- The Strata product performed very well in the laboratory in the Texas Transportation Institute (TTI)'s overlay tester with regard to resistance to reflection cracking. Tests were run at both 77 and 50 °F with excellent results.
- The product does not have good rut resistance, which raised concerns about driving on this layer during construction in hot weather.

However, these conclusions were based largely on the performance of a single section of the product on a single highway in Houston. Since that time, the product has been used in several other areas in Texas. Of particular interest is its use on a section of US 175. In one direction, the Glasgrid product described in Chapter 2 was used, and in the other, the Strata product was used. The performance of this section is described below.

US 175, DALLAS DISTRICT (CONTACT: GARY MOONSHOWER)

A 2½-mile section of US 175 was overlaid with a 1-inch layer of the Strata product with 3-inches of HMA above it. The section includes both lanes of the eastbound direction between Seagoville Road and Woody Road. This was an old 8-inch thick jointed concrete pavement of over 40 years old that had performed reasonably well in the past.

The existing HMA surface was planed down and a 1-inch Type D HMA layer was placed as a level-up layer. This was followed by the 1-inch layer of Strata and then 3-inches of Type C HMA as a surface course. This section was completed in October 2003. A typical pavement section is shown in Figure 18.

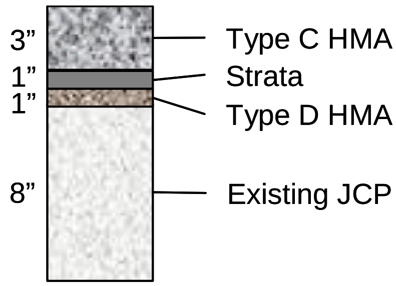


Figure 18. Typical Section on US 175.

The performance of this section was evaluated in June 2005, at an age of 2 years. A windshield visual survey revealed no sign of transverse cracks or other distresses on the pavement.

A GPR survey of the outside wheel path of both lanes was conducted. The data show strong reflections at the Strata layer, which is expected due to the difference in material types. Inconsistent surface thickness, which indicates some construction problems, is the only problem noted from the GPR data. Typical GPR data are shown in Figure 19.

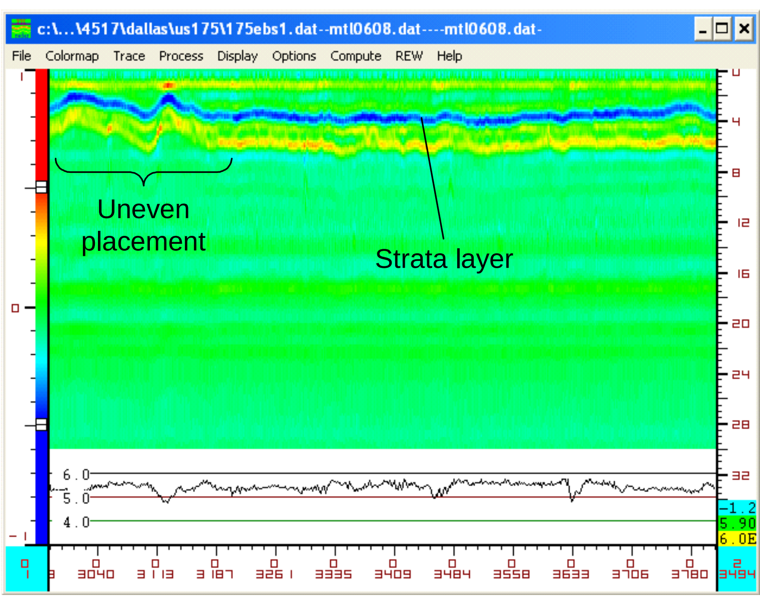


Figure 19. GPR Data Collected on Eastbound US 175.

Cores were taken on this section for laboratory testing. TTI's overlay tester (Zhou and Scullion, 2004) was used to evaluate the resistance of the mix to reflection cracking, and the Hamburg Wheel Test was used to evaluate the resistance of the mix to rutting. TTI's overlay tester is shown in Figure 20.

Figure 20. TTI's Overlay Tester.

The Strata layer and the Type C HMA surface course were tested separately using the overlay tester. The following criteria were used for the test:

- specimen failure criterion: $\dfrac{tensile\,load}{initial\,tensile\,load\,(2^{nd}\ cycle)} < 0.2$;
- design criteria for crack resistant materials: cycles > 750; and
- design criteria for Type C and D HMA mixes: cycles > 300.

The Strata performed well on the test and passed the design criteria for crack resistant materials by exceeding 750 cycles of the test. The results of the test are shown in Figure 21. The yellow line in the figure shows the failure load level based on the criteria given above.

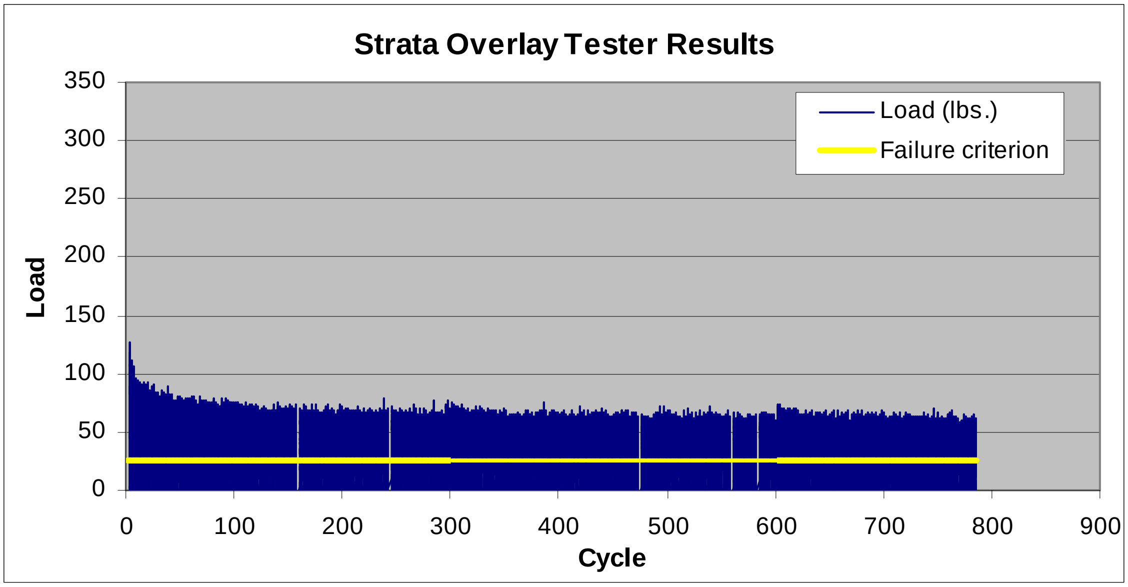


Figure 21. Strata Overlay Tester Results.

The Type C HMA mix did not pass the design criteria for Types C and D mixes. Failure occurred at 169 cycles, which is below the design criterion of 300 cycles for this type of mix. The results of the test are shown in Figure 22.

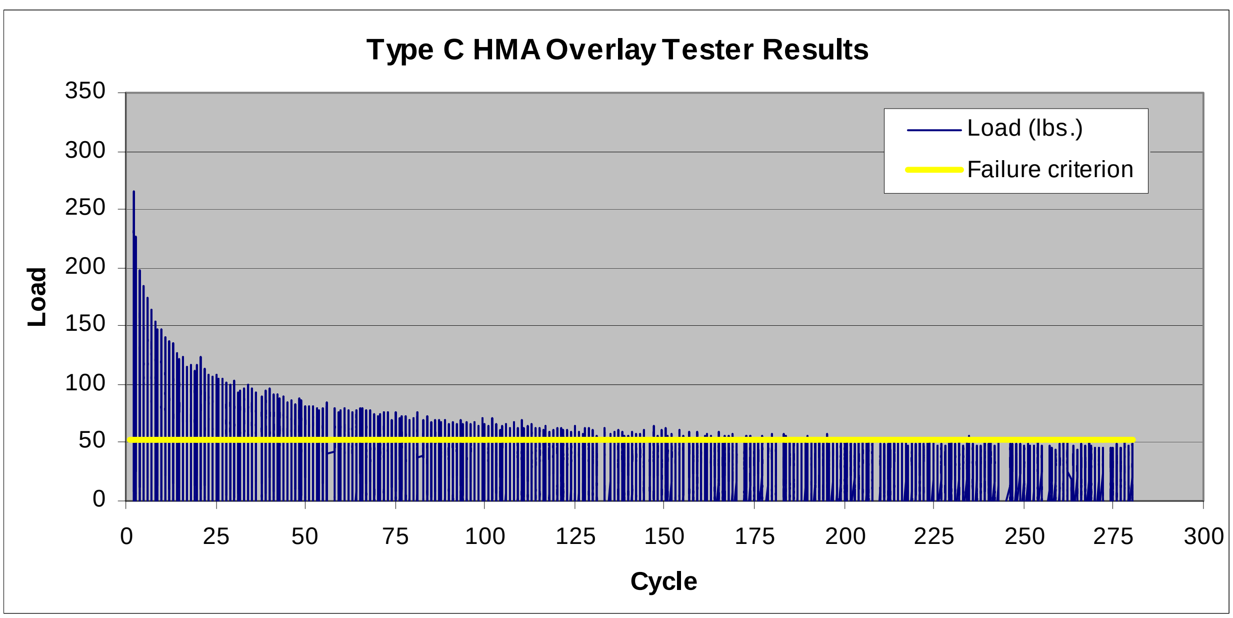


Figure 22. Type C HMA Overlay Tester Results.

The Strata and Type C HMA combination were tested in the Hamburg Wheel Test to evaluate the resistance of the combination of materials to rutting. The Hamburg Wheel Test is shown in Figure 23.

Figure 23. Hamburg Wheel Test.

A 2½-inch specimen was prepared with 1-inch of Strata at the bottom and 1.5 inches of Type C HMA at the top. The actual pavement has 3-inches of Type C HMA above the Strata layer. The wheel was applied directly to the Type C HMA. The following criteria were used for the test:

- specimen failure criterion: rut depth > 12½-mm (½-inch); and
- design criteria for overlays: passes > 20,000.

The Strata and Type C HMA combination performed well on the test and passed the design criteria by not failing within 20,000 passes. The final rut depth was 7-mm after 20,000 passes. The results of the test are shown in Figure 24.

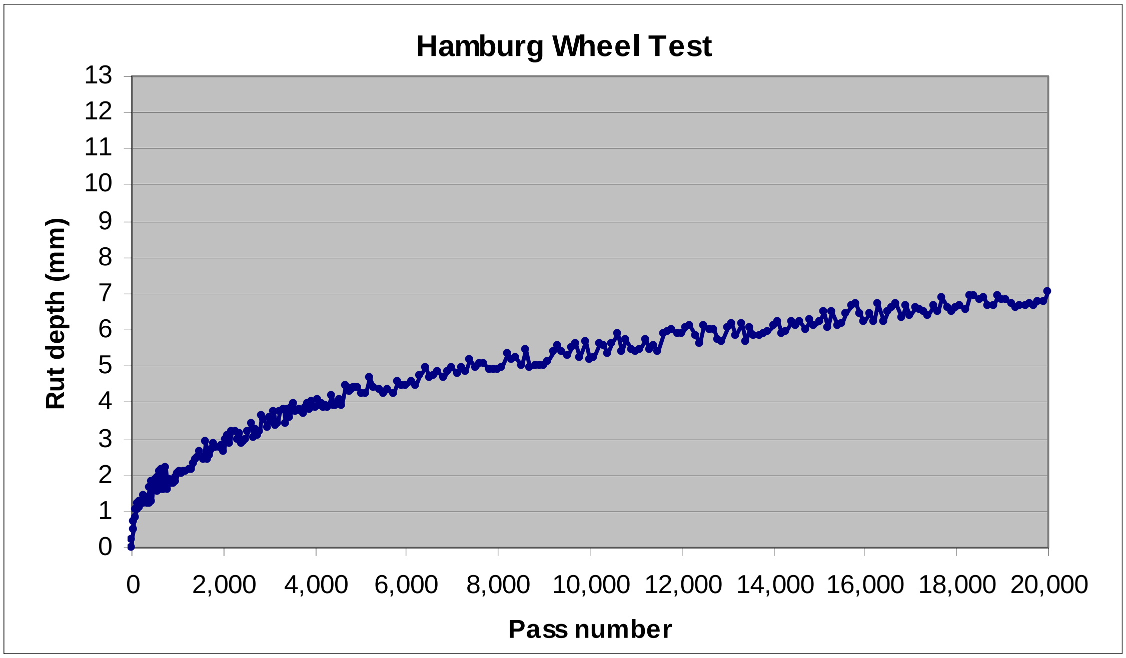


Figure 24. Hamburg Wheel Test Results for the US 175 Material.

Summary of Findings for the Strata Product

Based on the field investigation and testing conducted on the Strata section of US 175, this section is currently performing well and is expected to continue performing well. The Strata layer shows good resistance to reflective cracking propagating from the bottom upward, and the combination of Type C HMA and Strata shows good resistance to rutting.

The Strata product appears to be capable of restricting the development of reflective cracking in jointed concrete pavement overlays. However, the cost implications of using these materials for jointed concrete overlays should also be considered.

CHAPTER 4

THIN CRUMB RUBBER HMA OVERLAY

The Strata product described in Chapter 3 was not the only crack retarding asphalt layer evaluated in Report 0-4517-1. A crumb rubber modified mix was placed on an experimental section on US 96 near Lumberton in the Beaumont District. Although not a JCP (the existing section has a cement-treated base), this experiment gave the researcher the opportunity to compare the performance of different treatments to retard reflection cracking. The following three overlay test sections were constructed in the early 1990s: 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. Dar-Hao Chen from TxDOT. Based on discussion with Jack Moser, the area engineer, the conclusions after 8 years in service were the following:

- The crumb rubber section was performing best, and after 8 years the section showed very few reflection cracks.
- With regard to reflection cracking, the GlasGrid section was performing about the same as the traditional asphalt overlay.

During the monitoring of the test sections in Dallas, Maurice Pittman, P.E., asked the research team to include a section of SH 114 in the evaluation. This section had a thin layer of crumb rubber hot mix placed directly over jointed concrete pavement. The evaluation results are described below.

SH 114 FRONTAGE ROADS, DALLAS DISTRICT (CONTACT: MAURICE PITTMAN)

A 3-mile section of frontage roads on SH 114 between Loop 12 and Spur 348 was overlaid with a 1-inch layer of crumb rubber HMA. An AC-10 binder was used with a binder content of 7.8 percent by weight of the mix and a crumb rubber content of 16 percent by weight of the binder content. The sections have two lanes per direction with curbs and gutters, and both directions were rehabilitated. The existing pavement has several long sections of jointed concrete pavement. In other areas, the jointed concrete pavement was removed and full-depth asphalt pavement was used. The concrete surface was planed down by 1-inch prior to laying the 1-inch crumb rubber HMA layer. The section was opened to traffic in November 2003. A typical pavement section is shown in Figure 25.

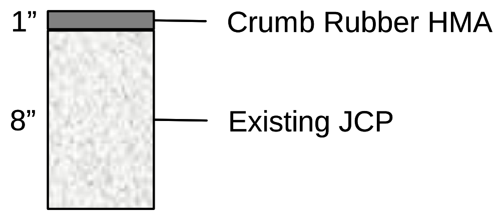


Figure 25. Typical Section on SH 114.

The performance of this section was evaluated in June 2005, at an age of 2 years. A windshield visual survey revealed isolated cases of shoving in braking areas and fairly widespread transverse cracks on the sections with concrete underneath. Photographs of the distresses found on the section are shown in Figure 26.

Figure 26. Photographs of Shoving and Transverse Cracks on SH 114.

The transverse cracks were very narrow and were closed in the wheel paths where the binder had resealed the cracks. The cracks did not have any effect on the riding quality of the pavement. It should be borne in mind that the investigation was conducted during the summer, and the cracks may widen in the winter.

A GPR survey of the outside wheel path of both lanes was conducted. The data clearly show the consistent thickness of the crumb rubber layer on the concrete. No major moisture problems were noted in the pavement structure. Typical GPR data are shown in Figure 27.

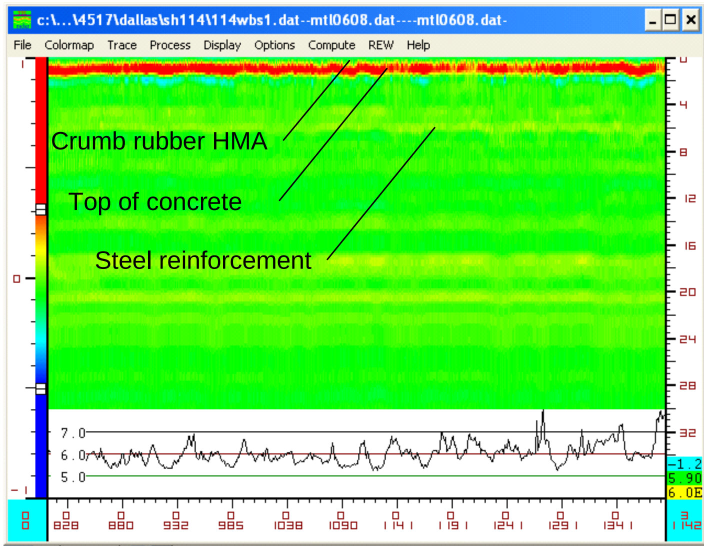


Figure 27. GPR Data Collected on SH 114.

Cores for laboratory testing were taken at an isolated thick surface location identified from the radar data. TTI's overlay tester was used to evaluate the resistance of the mix to reflection cracking, and the Hamburg Wheel Test was used to evaluate the resistance of the mix to rutting. Photographs of the core and test are shown in Figure 28.

Figure 28. TTI's Overlay Test.

The following criteria were used for the test:

- specimen failure criterion: $\dfrac{tensile\,load}{initial\,tensile\,load\,(2^{nd}\ cycle)} < 0.2$;
- design criteria for crack resistant materials: cycles > 750; and
- design criteria for Types C and D HMA mixes: cycles > 200.

The crumb rubber performed well on the overlay test and passed the design criteria for crack resistant materials by exceeding 750 cycles of the test. The results of the test are shown in Figure 29.

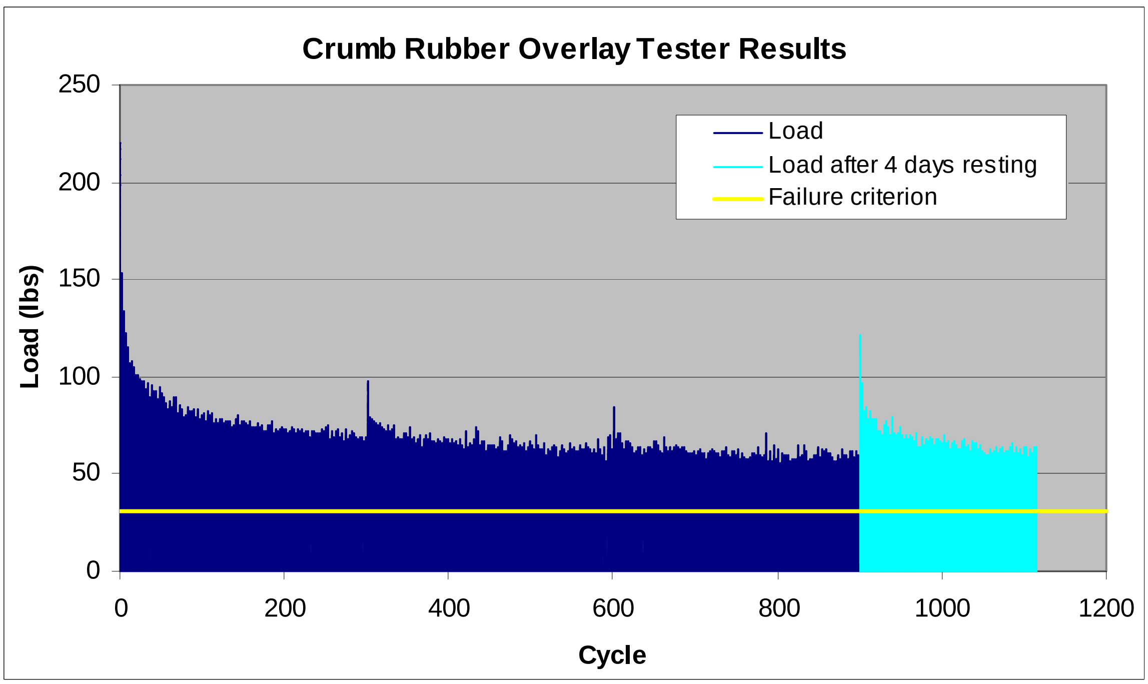


Figure 29. Crumb Rubber Overlay Test Results.

Since it was suspected that some healing of the crumb rubber layer may have been taking place in the field, it was decided to retest the sample after completion of the usual test. The sample was retested 4 days after the initial test. The secondary test showed a noticeable increase in the resistance to the strain in the secondary test when compared to the state of the sample at the end of the first test. This may be evidence of healing of the sample, but further investigation will be needed to provide conclusive proof of healing.

The crumb rubber was tested in the Hamburg Wheel Test to evaluate resistance to rutting. The Hamburg Wheel Test is shown in Figure 30.

Figure 30. Hamburg Wheel Test of Crumb Rubber.

The following criteria were used for the test:

- specimen failure criterion: rut depth > 12½-mm (½-inch); and
- design criteria for overlays: passes > 10,000.

The crumb rubber performed very poorly on the Hamburg test. The sample was approaching failure within 2000 passes. The binder used in this sample was AC-10, which is judged to be equivalent to the current PG 64-22, which has a Hamburg requirement of 10,000 passes. The results of the test are shown in Figure 31.

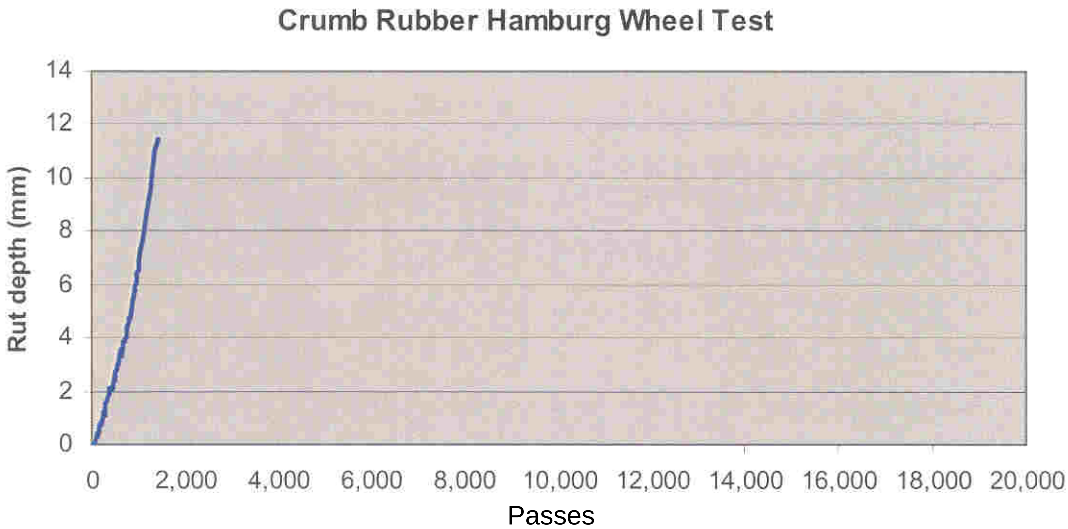


Figure 31. Hamburg Wheel Test Results on Crumb Rubber.

The results show that the crumb rubber layer is very susceptible to rutting and permanent deformation. No severe rutting was noted in the field, but the traffic on the frontage road is very light and of low volume. The frontage roads provide access to office complexes off SH 114. Some evidence of permanent deformation was noted in the braking areas on the frontage roads, but this was not widespread.

The overlay tester results show a high resistance to transverse cracks formed from the expansion and contraction of concrete slabs beneath the surface layer. However, widespread transverse cracks were noted in the field. The occurrence of these cracks may be the result of several factors. Firstly, some of the underlying concrete sections consisted of 60-ft jointed reinforced concrete slabs. The expansion and contraction of these long slabs result in large joint movements. The strain levels at the joints of these large slabs may exceed the strain levels used in the overlay tester. This may explain why the test did not fail, but the pavement layer failed in the field. Secondly, the crumb rubber layer was very thin at a thickness of 1-inch. The thin layer is susceptible to cracking by vertical shear stress at joints where load transfer is less than perfect. The crumb rubber layer may, therefore, be cracking by another mode than that tested using the overlay tester. No load transfer measurements were made on any of the joints on SH 114.

Based on the field investigation and laboratory testing of the thin crumb rubber layer, it is expected that rutting and permanent deformation will continue to develop on SH 114. The rate of development will depend primarily on the traffic loading. If the traffic mix and volume remain at current levels, the thin crumb rubber layer may provide reasonable service for several years. The transverse cracks that have developed on SH 114 are fairly tight and do not negatively affect the riding quality of the road. These cracks are expected to have little effect on the performance of the road over the next few years.

The ability to lay a thin asphalt overlay on concrete pavements with curb and gutter restrictions is very attractive, but due to the expected susceptibility of the mix to rutting, it is not recommended for use on heavily trafficked roads. Thin surface overlays of this type are attractive and alternative mix designs should be sought that can provide resistance to cracking and rutting on heavily trafficked roads.

Conclusions and Recommendations from the SH 114 Evaluation

It is difficult to draw conclusions from this project. The traffic loadings are very light, and joint condition was not measured prior to the application of the thin surfacing. In general, the district is happy with the performance of this thin overlay. As far as the Hamburg and overlay tester criteria, the following conclusions are made:

- The mix performance in the Hamburg test was very poor, but the material has not shown major rutting in the field. Some localized wash-boarding was observed in braking areas. However, the traffic loadings are very low so that it is difficult to generalize and make any recommendations on how the Hamburg criteria should be modified. TxDOT has on-going studies on the feasibility of using Hamburg criteria for crumb rubber modified mixes.
- The mix did very well in the overlay tester but exhibited reflection cracking in the field. Again, it is difficult to generalize as no load transfer measurements were made. The cracks themselves are minor and not currently a cause for concern. However, the one issue that should be considered is the slab length of 60-ft. The current overlay tester opening criteria of 25-mils is based on a 15-ft slab experiencing a 30 °F drop in temperature. With the longer slabs, either the opening criteria should be increased or the test temperature lowered to make the test more severe.

CHAPTER 5

EVALUATION OF THE REHABILITATION OPTIONS USED ON US 83 IN THE CHILDRESS DISTRICT

The Childress District has approximately 150 miles of US 83 passing through the district. This road was constructed in the 1940s as a 9-6-9 jointed plain concrete pavement (JPCP). In the past 10 years, the district has been rehabilitating sections along this highway. The district has used several treatment types, including HMA overlays, stabilized base overlays, flexible base overlays, crack and seat, and rubblization. The sections that were investigated in this report are shown in Figure 32.

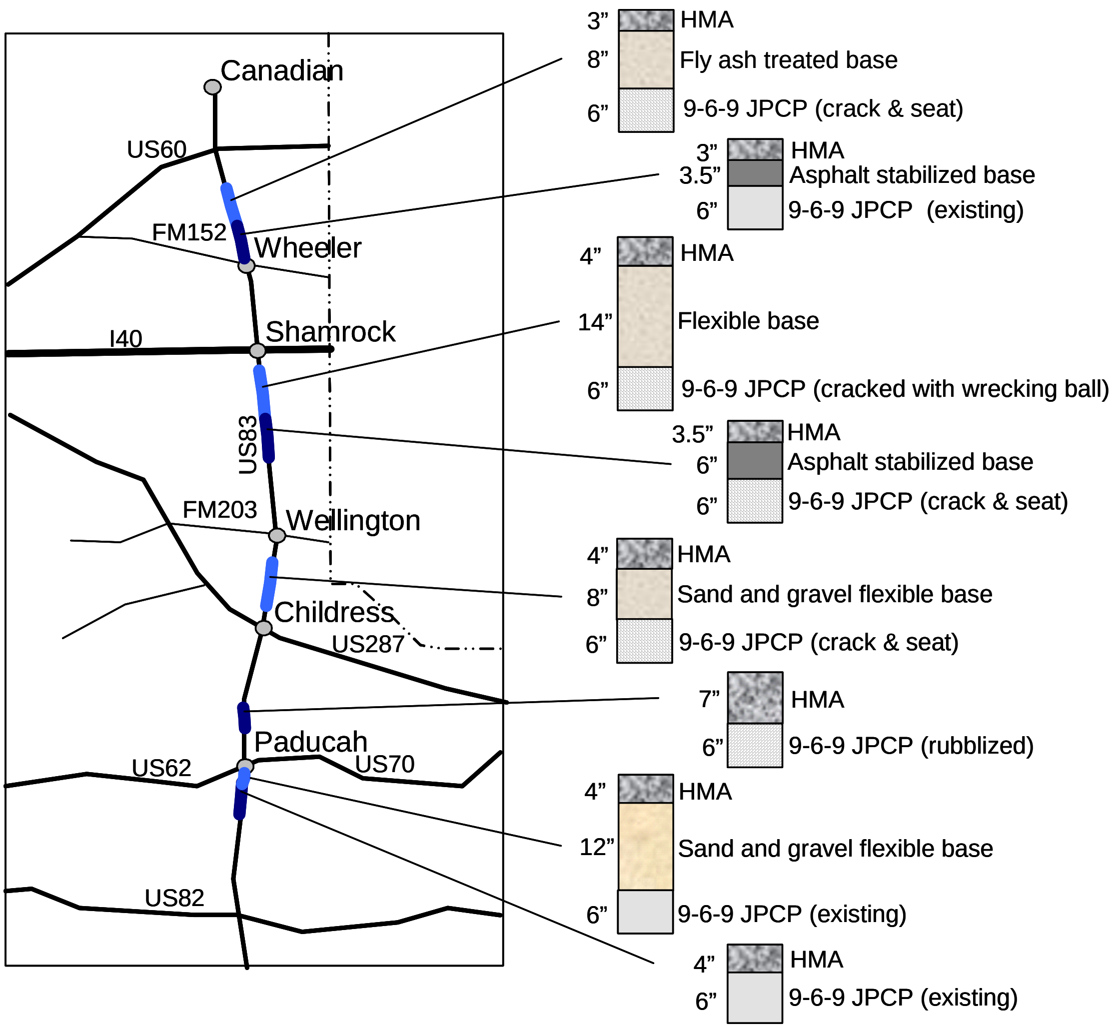


Figure 32. US 83 Sections That Were Investigated.

On US 83, the traffic levels are fairly low with average daily traffic (ADT) in the 1000 to 1500 vehicles per day range. The 20-year design loads are typically less than 1 million equivalent single axle loads. The Childress District in general has good subgrade support conditions with typically sandy/silty well-draining material. Rainfall amounts are low to moderate averaging 15 to 20 inches per year. In the northern counties, the section experiences freeze-thaw cycling in winter.

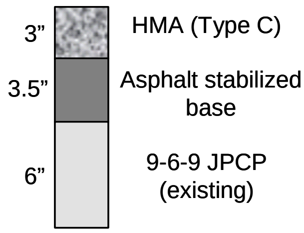
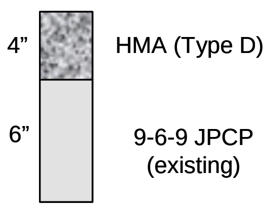
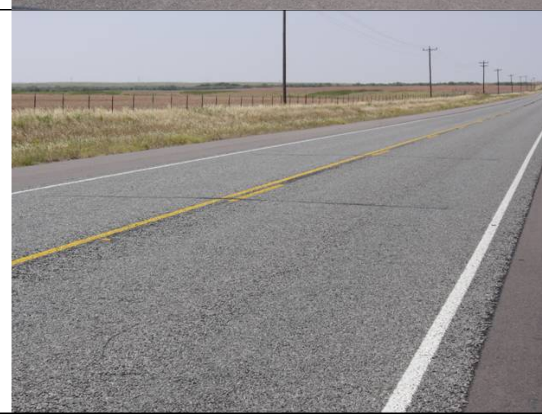
In order to gain an understanding of the effectiveness of the different treatment types used on US 83, each of the rehabilitated sections was visually inspected and tested with both GPR and FWD data. In some cases, samples of the surface or base layers were taken and returned to TTI for lab testing. Results from these investigations are described below.

PERFORMANCE EVALUATIONS OF HMA OVERLAYS

HMA cores were collected from two sections to evaluate the resistance of HMA surface courses placed directly on JCP. The overlays were 2 and 5 years old at the time of testing and both experienced substantial reflection cracking. The first section was between County Roads F and H in Wheeler County, and the second section was between Salt Creek Bridge and County Road 240 in Cottle County. A summary of the sections is provided in Table 1.

The HMA cores were tested for resistance to cracking using the overlay tester (Zhou and Scullion, 2004). Recent work has recommended design criteria for dense-graded mixes. For mixes to perform well at retarding reflection cracking, it has been proposed that they should withstand 300 cycles in this test. The test conditions are test temperature (70 °F) and test opening 0.025 inch. Photographs of the cracked specimens and test results for the mixes from these two projects are shown in Figure 33.

Table 1. Summary of HMA Overlay Sections.

Section	Surface Age	Performance	Photo
	5 years	Very widespread longitudinal and transverse reflection cracks	
	2 years	Widespread transverse reflection cracks	

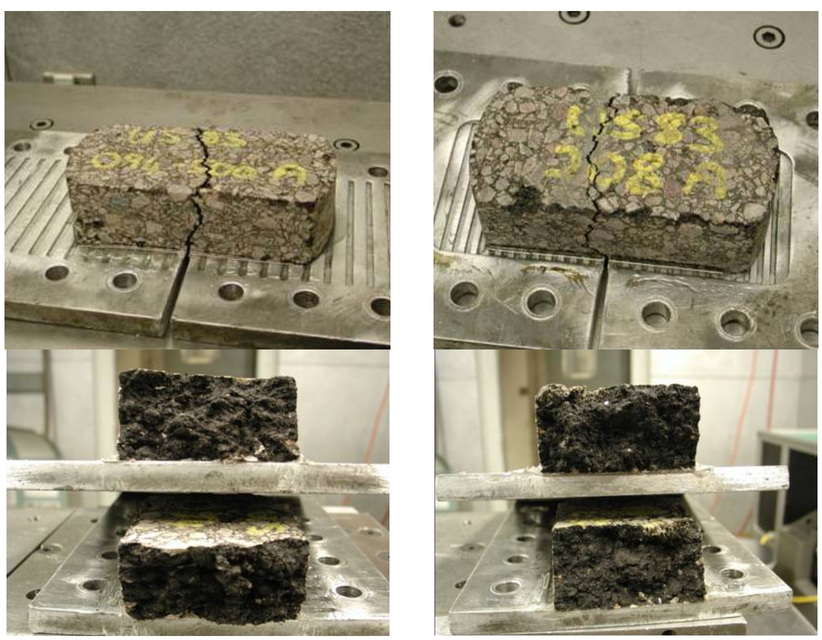


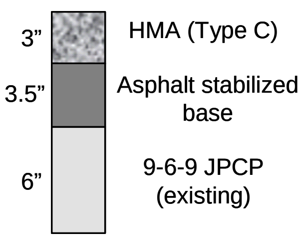
Figure 33. US 83 HMA Specimens Cracked in the Overlay Tester.

The specimens failed within 10 cycles of the overlay tester, which is far below the tentative design criterion of 300 cycles for Types C and D mixes. This early failure indicates that the asphalt mixes that were used on US 83 are susceptible to reflection cracking. A visual inspection of the crack surfaces of these mixes indicated that the mixes were dry and that the asphalt surface was dull.

RESISTANCE OF ASPHALT STABILIZED BASE TO CRACKING

Asphalt stabilized base cores were collected from the section between County Roads F and H in Wheeler County. This is the same section as described above for the HMA surface course evaluation. A summary of the section is provided in Table 2.

Table 2. Summary of Asphalt Stabilized Base Section.

Section	Surface Age	Performance	Photo
	5 years	Very widespread longitudinal and transverse reflection cracks	

The stabilized base cores were also tested for resistance to cracking using the overlay tester (Zhou and Scullion, 2004). Photographs of the specimen are shown in Figure 34.

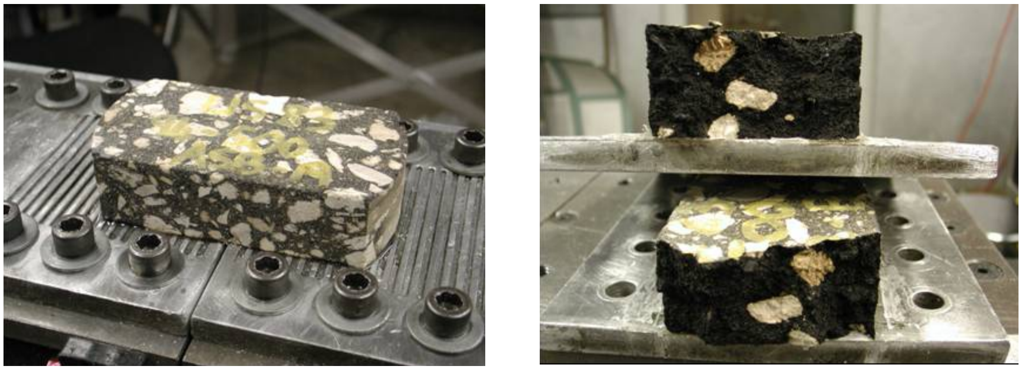


Figure 34. US 83 Stabilized Base Specimens Cracked in the Overlay Tester.

The specimens failed within 10 cycles of the overlay tester. This indicates that the asphalt stabilized base also has a poor resistance to reflection cracking.

THE USE OF THICK FLEXIBLE BASE OVERLAYS

In Report 0-4517-1, it was found that the best performing section on the Lufkin project was the use of untreated gravel base and a thin asphalt surfacing placed directly over the jointed concrete pavement.   In 1963, a similar rehabilitation strategy was used on a short section of US 83 just south of Paducah. A ¾-mile jointed concrete pavement section was overlaid with 12-inches of flexible base (sand and gravel) and a 2-inch HMA surface course. This section has provided 40+ years of excellent service with no transverse cracks reflecting through from the bottom layers and no other major distresses. Adjacent sections, where the jointed concrete was overlaid with HMA, have experienced reflection cracking problems. A 2-inch HMA overlay and surface seal was applied to the entire road section in 2003, so no distresses were visible on the surface at the time of evaluation.   However, the ride was excellent and maintenance forces had mentioned that the base overlay section had little distress at the time of overlay. A typical pavement section and a photograph of the base retrieved are shown in Figure 35 below.

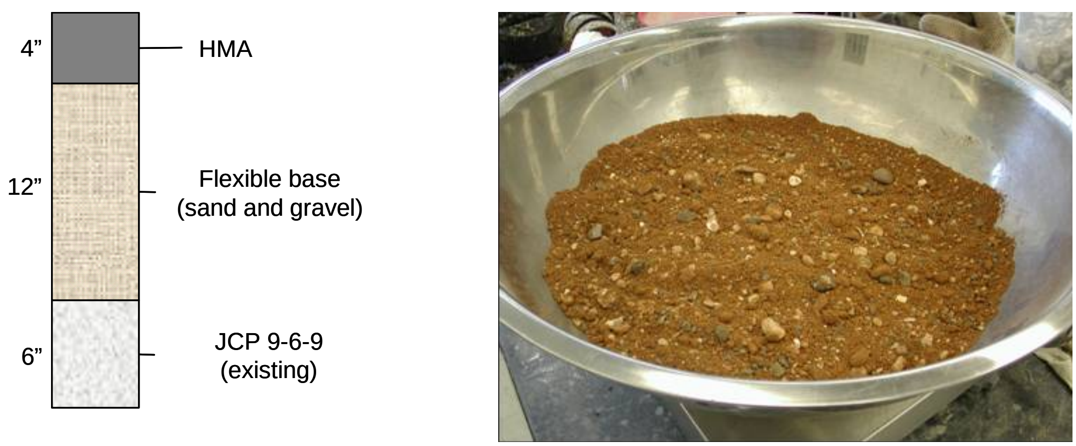


Figure 35. Flexible Base Overlay Section on US 83.

A FWD survey was conducted on this section, and the results are shown in Figure 36. The backcalculated moduli values for the flexible base are low, at around 20 ksi. It must be recalled that the base is well-confined as it is resting on a concrete slab, so the average values are fairly low.

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

District:
County :
Highway/Road:

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	4.00	700,000	1,240,000	H1: v = 0.35
Base:	12.00	10,000	150,000	H2: v = 0.35
Subbase:	6.00	100,000	2,000,000	H3: v = 0.25
Subgrade:	212.97(by DB)	20,000		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	10,423	16.14	10.48	5.67	3.55	3.29	1.72	1.43	1165.3	16.9	1102.9	15.2	6.89	300.0	
120.000	10,069	15.48	8.51	3.75	2.30	1.81	1.38	1.17	721.6	17.9	2000.0	22.9	2.72	163.1	*
240.000	9,994	20.46	11.52	4.48	2.38	1.78	1.44	1.27	700.0	10.0	2000.0	22.8	4.51	78.8	*
360.000	10,483	16.04	9.43	4.78	2.76	2.06	1.50	1.22	864.4	20.7	203.0	20.6	1.59	164.3	
480.000	10,880	16.41	9.86	4.86	3.28	2.33	1.78	1.48	887.8	19.2	1295.6	18.1	1.41	300.0	
600.000	10,574	19.68	11.55	5.45	3.35	2.30	1.64	1.26	751.3	14.0	673.0	18.1	1.18	293.3	
720.000	9,521	21.29	12.95	6.52	3.80	2.33	1.54	1.16	831.8	10.0	188.1	15.9	2.79	140.0	*
840.000	10,510	21.21	12.42	5.67	3.30	1.99	1.31	1.18	819.2	10.4	619.6	20.3	3.22	131.8	
1520.000	10,153	12.65	6.61	2.98	2.00	1.46	1.16	0.99	750.7	25.6	1998.5	27.5	1.62	206.5	*
1640.000	10,308	11.01	6.99	4.08	2.74	1.99	1.52	1.26	1240.0	37.7	461.6	19.8	1.23	300.0	*
1760.000	10,129	16.93	10.38	5.24	3.33	2.43	1.76	1.33	860.1	17.7	514.1	16.6	1.02	300.0	
1880.000	10,467	14.46	8.71	4.38	2.79	1.97	1.46	1.37	1010.8	21.4	649.5	20.8	0.51	300.0	
2000.000	10,669	15.48	9.95	5.22	2.95	2.23	1.64	1.32	1219.0	17.7	473.1	19.3	2.42	143.3	
2120.000	10,590	15.54	9.97	5.02	3.00	2.08	1.50	1.09	1212.4	17.1	383.3	20.2	0.89	217.1	
2240.000	10,598	17.06	10.86	5.58	3.39	2.28	1.67	1.26	1116.2	15.5	420.1	18.1	0.55	242.2	
2360.000	10,304	15.35	9.00	4.41	2.88	1.97	1.44	1.19	872.1	20.1	614.6	20.3	1.33	300.0	
2480.000	10,304	18.75	11.38	5.71	3.47	2.36	1.74	1.39	821.3	15.9	208.8	17.0	0.37	256.6	
2600.000	10,343	17.04	10.35	5.13	3.29	2.23	1.62	1.29	855.6	19.0	188.9	18.2	1.24	259.6	
2720.000	10,475	17.23	10.21	4.84	3.00	2.08	1.58	1.24	780.0	19.1	184.9	19.8	1.25	300.0	
2840.000	10,308	19.35	12.28	6.04	3.44	2.26	1.70	1.41	1029.1	10.3	1233.4	17.8	0.89	149.1	
Mean:		16.88	10.17	4.99	3.05	2.16	1.56	1.27	925.4	17.8	770.7	19.5	1.88	235.0	
Std. Dev:		2.68	1.67	0.83	0.46	0.36	0.16	0.12	178.0	6.3	623.4	2.8	1.57	96.8	
Var Coeff(%):		15.87	16.40	16.60	15.05	16.48	10.40	9.44	19.2	35.2	80.9	14.4	83.33	45.2	

Figure 36. FWD Analysis for Flexible Base Section.

A GPR survey of the outside wheel path of both directions was conducted. Details of GPR data analysis can be found elsewhere (Scullion and Chen, 1999). The purpose of this evaluation was to check the moisture condition in the base layer. High dielectric values are associated with wet base areas. Typical GPR data for this section are shown in Figure 37.

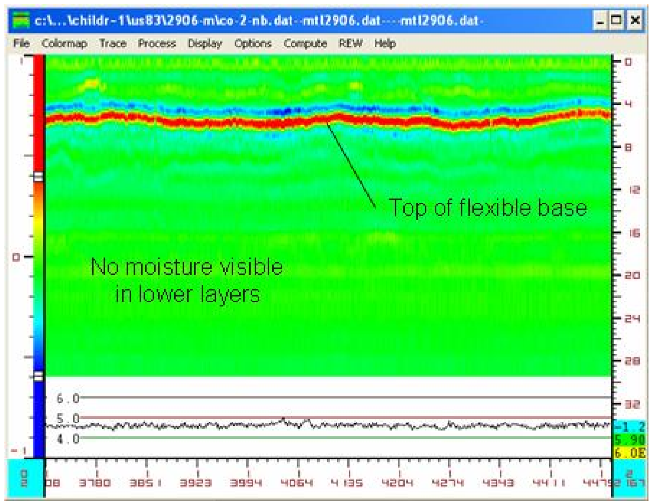


Figure 37. GPR Data on US 83, Thick Flexible Base.

The dielectrics of the flexible base layer for the northbound direction are shown in Figure 38. The average dielectric for this layer is in the 7 to 9 range, which is judged to be low, indicating a dry base. Experience has shown that bases with dielectrics in the 10 to 12 range indicate moderate moisture levels and values above 16 indicate wet/saturated. The base on this section is very dry.

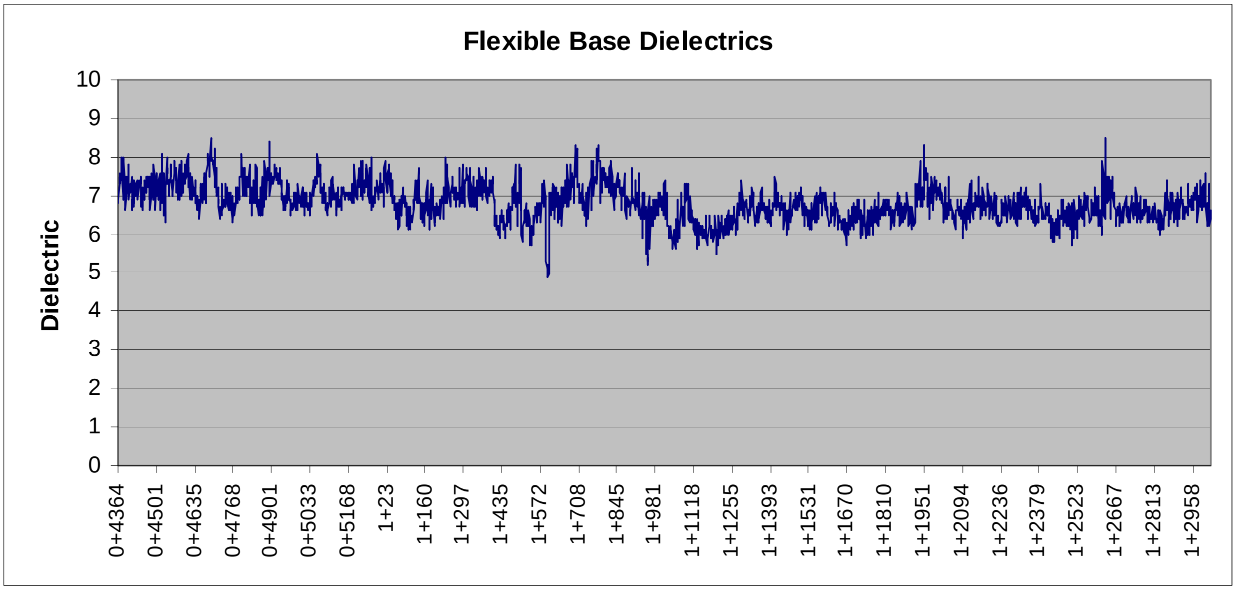


Figure 38. Dielectrics of Flexible Base Overlay on US 83.

The moisture susceptibility and strength of the sampled base was evaluated in the laboratory using TxDOT's tube suction test (TxDOT Method 144E). Details of the tube suction procedure can be found elsewhere (Guthrie and Scullion, 2002). The test measures the capillarity of the base compacted at optimum moisture content. The sample is compacted, dried back, and then exposed to moisture at the base of the sample for a period of 10 days. The surface dielectric is measured with a Percometer at the top of the sample, which gives an indication of the amount of unbound moisture reaching the surface of the sample. The asymptotic dielectric is used to evaluate the moisture susceptibility of the base material. Interpretation criteria for this test are shown in Table 3.

Table 3. Classification of Base Materials Based on the Tube Suction Test.

Final Dielectric	Classification
<10	**Excellent** – no moisture-related problems.
10-13	**Good** – typical of 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 for a high-volume roadway.
16+	**Fair-Poor** – moisture susceptible, consider for modification for all applications.

The results of the tube suction test are shown in Figure 39.

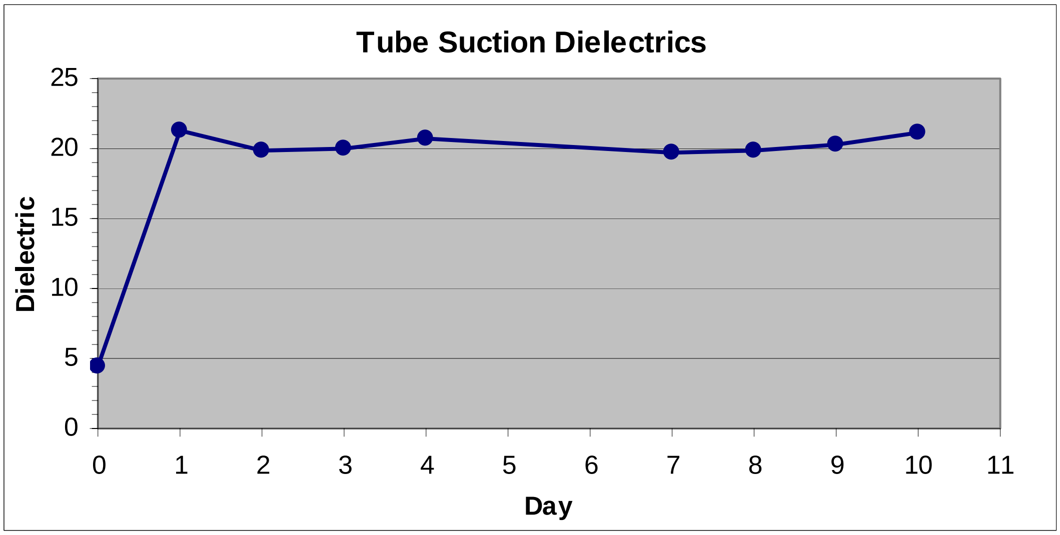


Figure 39. Tube Suction Test Dielectric Results.

The results of the test show a rapid increase in moisture at the surface of the sample and a high final dielectric of 20.5. The final moisture content of the base was 11 percent, which is much higher than the optimum moisture content of 4 percent. This test indicates that this base is very susceptible to moisture.

The retained strength of the soil was tested by comparing the unconfined compressive strength (UCS) after the 10-day capillary soak with the UCS of an unsoaked specimen. The results of the test are shown in Figure 40.

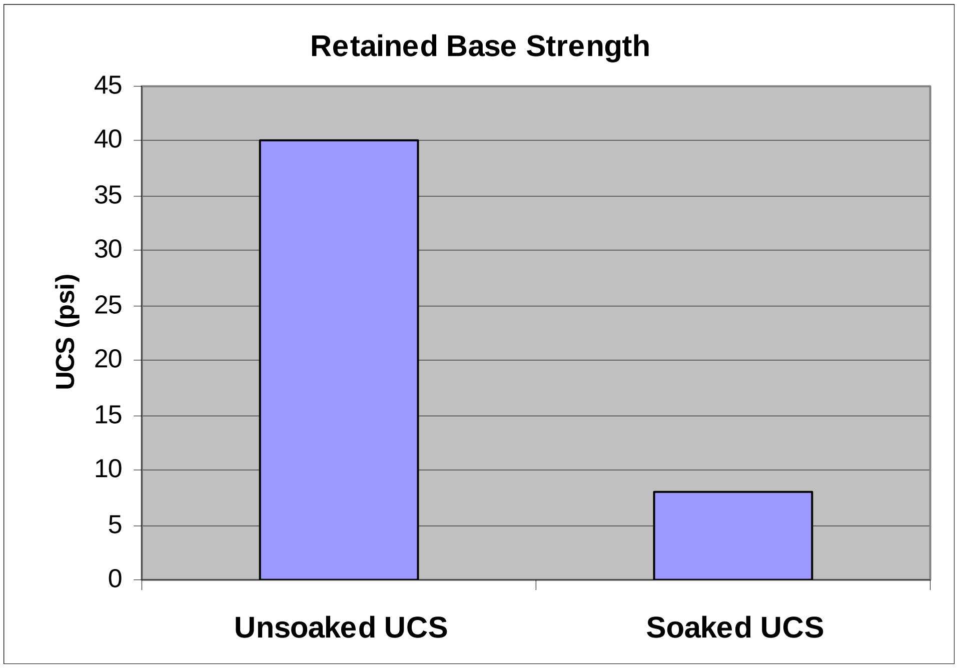


Figure 40. Unconfined Compressive Strength of Base.

The retained strength is, therefore only 20 percent of the unsoaked strength. This is considerably below the requirement of 75 percent retained strength. This indicates that this is a highly moisture susceptible base.

Based on the test results above, it is clear that the use of a thick flexible base can reduce the likelihood of reflection cracking from underlying concrete layers. What is somewhat surprising is that the base used on this project is of very low quality. The field results (FWD) and lab results indicate a low strength, moisture susceptible base; the District Lab Supervisor, Mr. Ron Hatcher, indicates that the base was a Texas Triaxial Class 3.4, which is very marginal. Despite this low base quality, this section has performed excellently. The good performance of this base can probably be ascribed to several factors including:

- the relatively low rainfall in this area location,
- the ability of the uncracked HMA to keep water from entering the base,
- no moisture ingress from below as the base is directly on top of concrete, and
- the low traffic levels on this section.

Based on the experience on US 59 in Lufkin and on the performance of this section, the use of thick flexible base overlays is a feasible rehabilitation option for jointed concrete pavements.

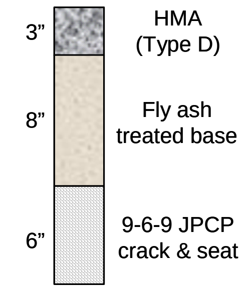
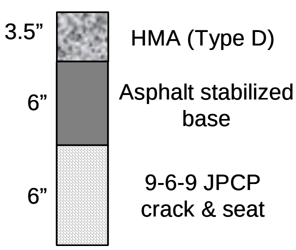
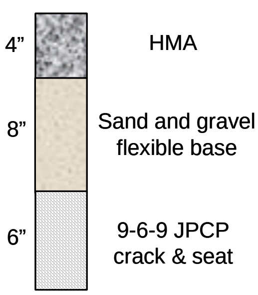
CRACK AND SEAT OF JOINTED CONCRETE PAVEMENTS

The crack and seat procedure performed very poorly in the US 59 Lufkin experiment as reported in Report 0-4517-1. On US 59, 4 inches of HMA was placed directly over the cracked and seated concrete. From discussion with engineers in the Childress District, the placement of hot mix directly over fractured concrete is not recommended because of possible reflection cracking problems. In the Childress District, there are several projects where the existing pavement has been fractured with the crack and seat method and then overlaid with a thick base and an HMA surface course. Different bases have been used including fly ash stabilized bases, flexible bases, and asphalt stabilized bases. The crack and seat method has performed well on all projects to date. The three sections investigated were:

- Hemphill County Line to County Road C in Wheeler County,
- County Road E to FM 1439 in Collingsworth County, and
- Collingsworth County Line to Buck Creek Bridge in Childress County.

A summary of the sections is given in Table 4.

Table 4. Summary of Crack and Seat Sections.

Section	Rehab Age	Performance	Photo
	7 years	Good condition. No structural distresses. Isolated transverse cracks, which may be related to problems with construction at the time.	
	2 years	Excellent condition with no distresses evident.	
	5 years	Excellent condition. This section had recently been overlaid as an extension to a project on an adjacent section. No structural distress was evident at the time of the overlay.	

FWD data were collected on several crack and seat sections to obtain an estimate of the modulus value of the cracked concrete layer. MODULUS 6 software (Liu and Scullion, 2001) was used to backcalculate the layer modulus of the cracked layer. The distribution of backcalculated moduli from 123 FWD drops is shown in Figure 41.

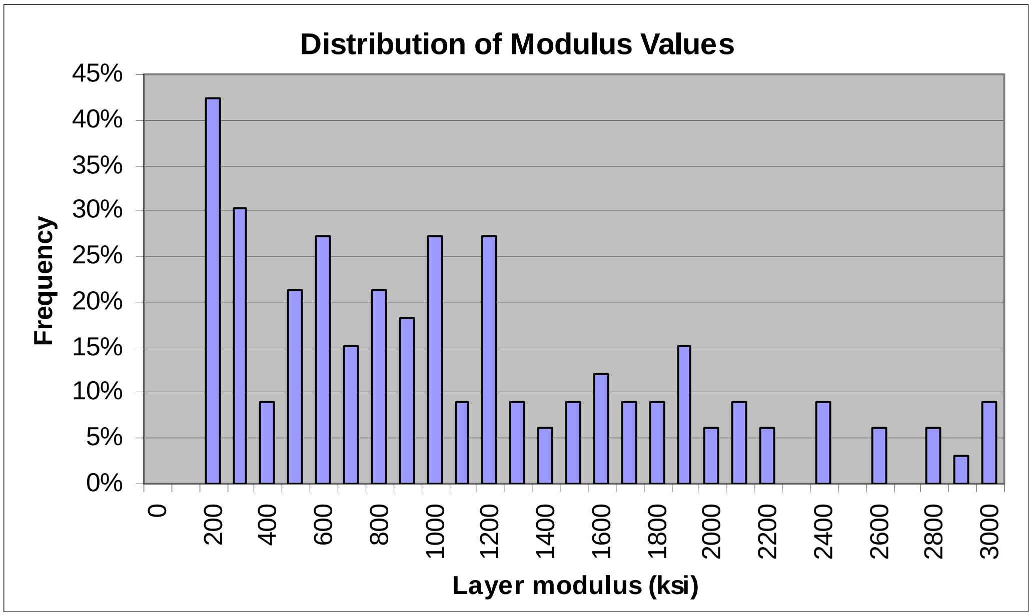


Figure 41. Distribution of Modulus of Cracked Layer.

From the distribution above, it is evident that the resultant modulus values of the crack and seat layer are highly variable. The values greater than 2000 ksi are suspected to be areas where the concrete has not been cracked. Over 10 percent of the tested locations had modulus values of greater than 2000 ksi. With such highly variable results, it is not possible to recommend a single modulus value for design. Further investigation may be needed to derive a design value. Detailed FWD analysis results can be found in Appendix A.

RUBBLIZATION OF JOINTED CONCRETE PAVEMENTS

The Childress District has rubblized a section of US 83. A continuous section 0.9 mile long, from FM 3256 northward, was evaluated. This section is 2 years old and is still performing excellently. The pavement section and a photograph of the section are shown in Figure 42.

Figure 42. Structure on US 83.

GPR survey was conducted on this section, and no moisture was found in the rubblized layer. Typical GPR data are shown below in Figure 43.

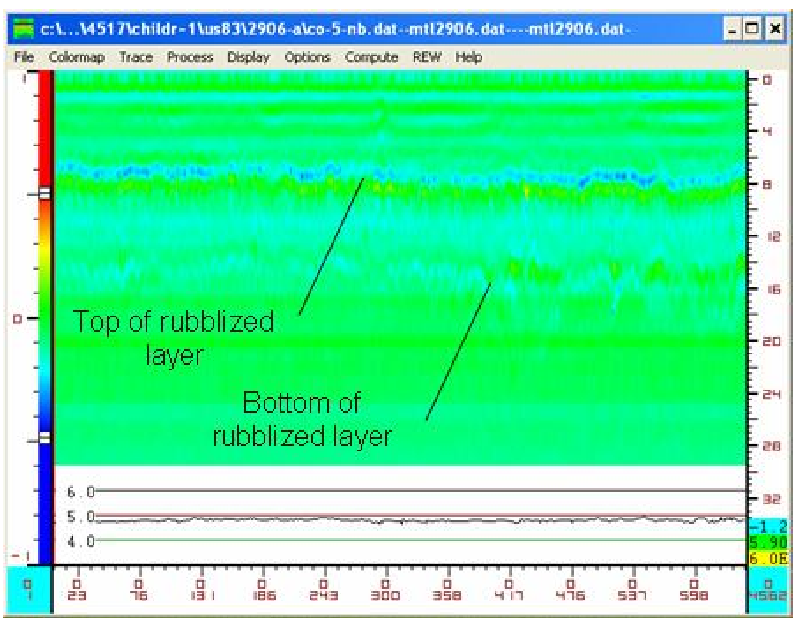


Figure 43. GPR Data on US 83, Rubblized Section.

The dielectric values of the rubblized layer are shown in Figure 44.

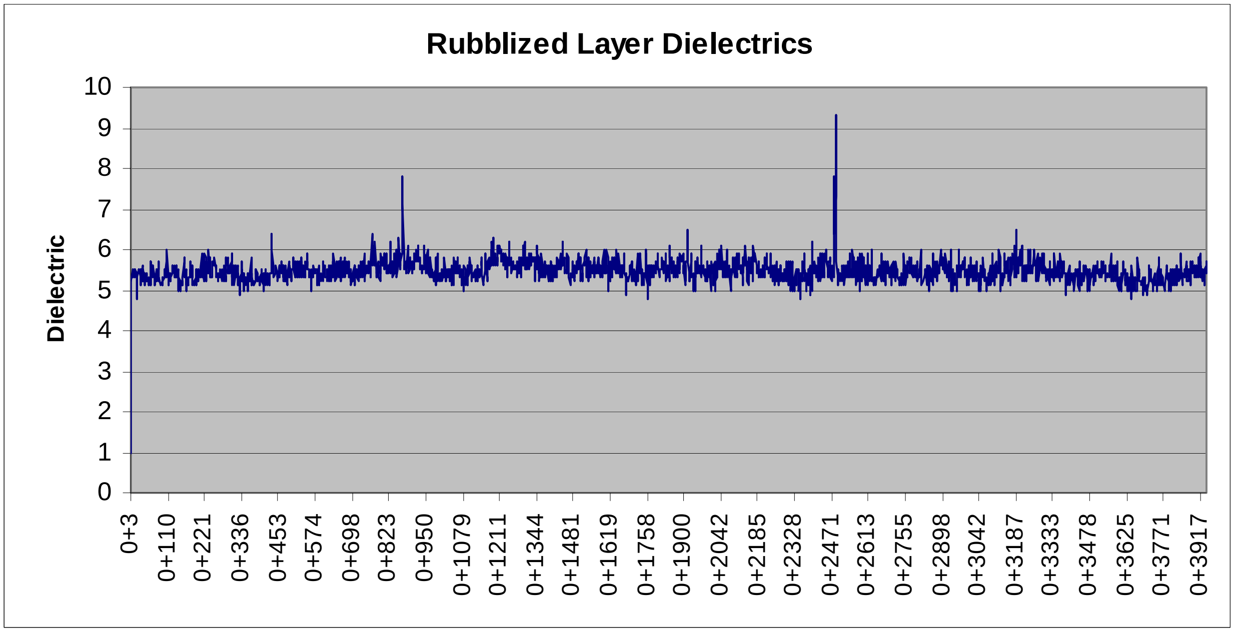


Figure 44. Dielectrics of a Rubblized Section on US 83.

The dielectrics show a consistent low moisture level within the rubblized section. There are two low spikes in the dielectric values, which may reflect moisture or other discontinuities in the layer.

FWD data were collected on this section to obtain an estimate of the modulus value of the rubblized layer. MODULUS 6 software was used to backcalculate the layer modulus of the layer. FWD data were collected before the rubblization project, 6 months after the rubblization, and 18 months after the rubblization. A summary of the layer strength properties is given in Table 5 (all figures are rounded). Detailed FWD results can be found in Appendix B.

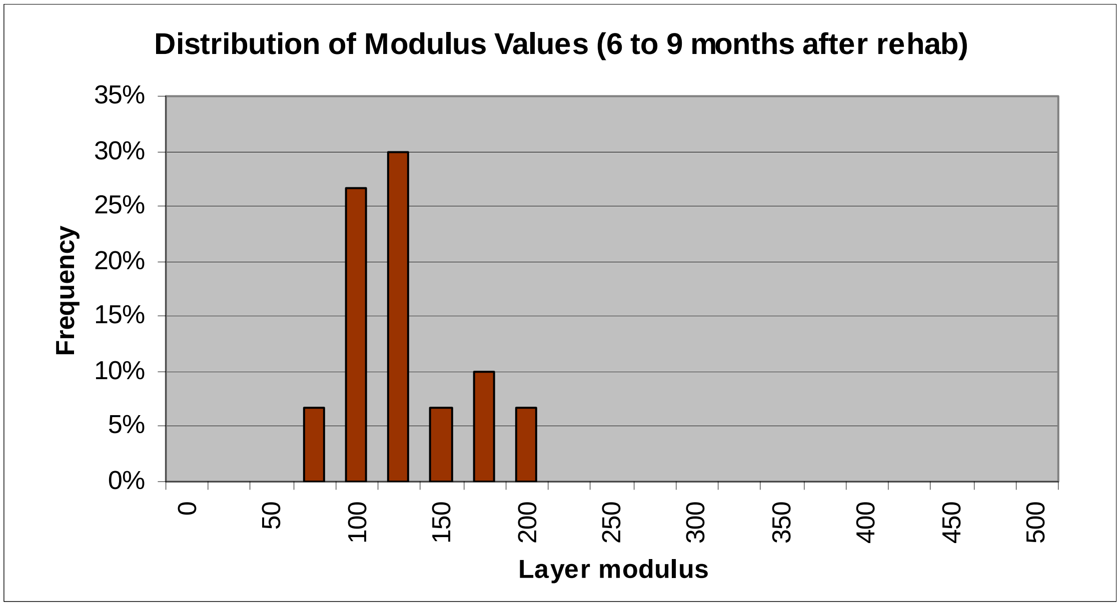
Table 5. Summary of Layer Strength Properties for the Rubblized Section.

Rehabilitation Life Stage	Concrete Layer Modulus			Subgrade Modulus		
	Mean	Std dev	Coeff of Var (%)	Mean	Std Dev	Coeff of Var (%)
Prior to rubblization	2930	890	30	15	2	13
6 months after rubblization	115	35	30	12	1	10
18 months after rubblization	200	60	30	13	2	12

The results show that the concrete layer has increased in stiffness after the rubblization from 115 ksi to 200 ksi. The variance of the layer modulus has remained fairly constant over all stages of the rehabilitation, and the subgrade modulus has also remained fairly constant. The distributions of the modulus values soon after the rubblization and 18 months after the rubblization are shown in Figure 45.

When the rubblized concrete modulus values are compared to the crack and seat modulus distributions, it is clear that the variance of the rubblized modulus values are considerably less than the crack and seat modulus values. There are no tests with modulus values greater than 2000 ksi in the rubblized section. This indicates that the rubblization was effective in breaking up the slabs. It should be noted that the sample size of the rubblization tests is much smaller than the crack and seat tests with 30 drops and 123 drops, respectively. However, the results described above are very similar to the moduli distributions reported by Witczak and Rada (1992).

More FWD data should be collected on the rubblized monitor section on US 83. The increase in base modulus with time should be studied further; if confirmed, it could indicate that the rubblized base is "self-cementing." The Childress District is planning to use rubblization on three more projects in 2006. These projects will also be monitored in the future.



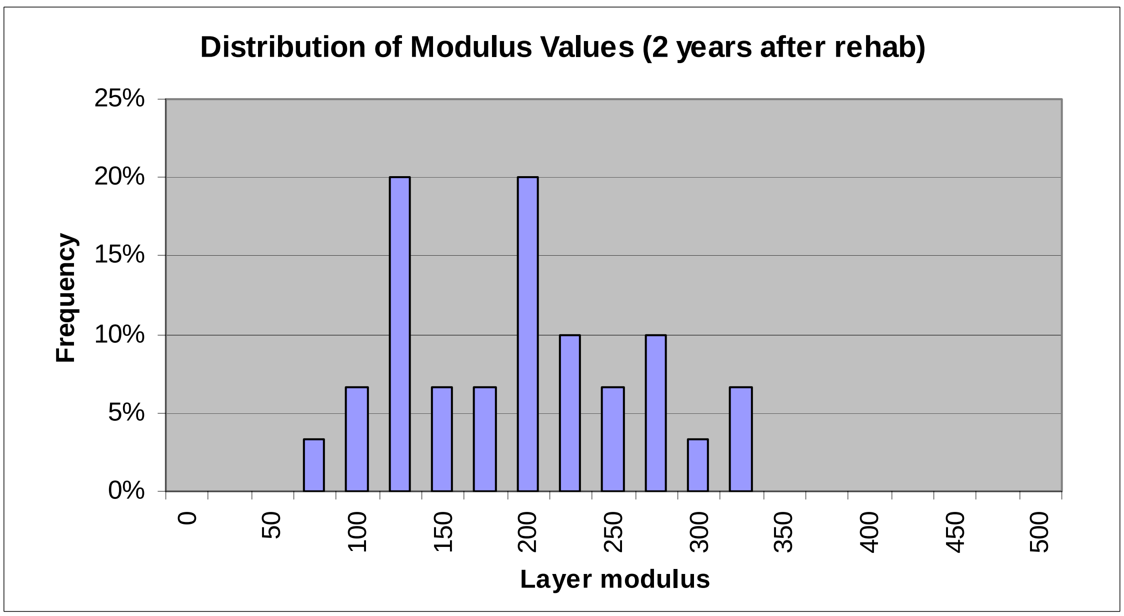


Figure 45. Distributions of Modulus of the Rubblized Layer.

CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

Selecting rehabilitation options for jointed concrete pavements continues to be one of the most challenging tasks for pavement engineers. In Project 0-4517, the performance of numerous treatments were investigated. Reports 0-4517-1 and 0-4517-3 identified treatments that are performing well and those that are not. Report 4517-2 proposed a field investigation plan for testing future candidate projects that combines visual inspections with nondestructive testing. A sequenced approach is proposed for NDT evaluation, which includes GPR and deflection investigations and in some instances dynamic cone penetrometer (DCP) testing. Deflections can be taken with either the FWD or rolling dynamic deflectometer.

For those JCPs judged to be in good condition with few joint failures and reasonable load transfer efficiency, a hot mix asphalt overlay should be applied. As reported, several of the overlays studied such as those in Childress on US 83 are not performing well in terms of reflection cracking. The performance is well-matched to the performance of the mixes found in the laboratory with TTI's overlay tester. However, in Report 0-4517-1, good performance was reported on mixes containing modified binders in both the Houston and Beaumont Districts. The use of the overlay tester to design overlays for cracked pavements including JCPs is currently under study in Project 0-5123, "Development of an Advanced Overlay Design Procedure Incorporating both Rutting and Reflection Cracking Requirements." Until that project is complete, TxDOT should consider the following overlay test mix design criteria:

- for dense graded and performance mixes: 300 cycles to failure; and
- for crack resistant mixes: 750 cycles to failure.

For projects with JCPs with joint failures, very poor load transfer, or sections with severe reflection cracking problems, rubblization is the preferred slab fracturing technique. This is currently under more detailed study in Project 0-4687, "Rubblization and Crack and Seat as Major Rehabilitation Options for Jointed Concrete Pavements." Rubblization will only be possible if the existing slab has reasonable base support (as measured by the DCP) and if any trapped moisture can be effectively removed with retrofitted drains. Actual criteria are currently being proposed in Project 0-4687. Based on the FWD testing performed on the section on US 83, a conservative design modulus of 110 ksi could be used for the rubblized concrete. This design modulus is about twice the value assumed for regular flexible base materials.

For JCPs in poor condition with poor subslab support, a very attractive alternative is a flexible base overlay and thin HMA surfacing. This rehabilitation has performed very well on both US 59 in Lufkin and on US 83 in Childress. This can only be used on rural sections because a thick base overlay of at least 8 inches will be required. The critical issue in this design is that the top of the base must be sealed. The use of an underseal is mandatory. The surface mix should also be a dense-graded mix. If the base overlay is to be used in an area with more than 20 inches of rain per year, then a Class 1 base should be used. For additional insurance, the base should have a dielectric value of less than 12 in the tube suction test. (The good performing base on US 59 had a dielectric of 12.5.) Until further sections have been built and monitored, this treatment is not recommended for very heavily trafficked highways such as Interstate highways. Until further data are available, this strategy should only be considered for sections with an ADT of less than 5000 vehicles per day.

In this project the use of grids within asphalt layers was not found to be a cost-effective alternative. In Report 0-4517-1, delamination problems were reported early in the life of sections, and in this project long-term benefits of grids could not be identified. If grids are to be used in the future, steps must be implemented to ensure that debonding will not occur.

The use of crack and seat of jointed concrete pavements was found to provide much more variable results than rubblization. As found in Lufkin, this treatment will not work if the slab has a weak base. In Childress (and other areas of West Texas), crack and seat appears to be working well when a base material is placed over the fractured concrete. However, as discussed above, flexible base overlays were found to work well without fracturing the slab, so the benefits of crack and seat are not clear.

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6. Liu, W., and Scullion, T., “MODULUS 6 for Windows, User’s Manual,” Report 0-1869-3, Texas Transportation Institute, College Station, Texas, Oct. 2001.
7. Witczak, M.W., and Rada, G.R., “Nationwide Evaluation Study of Asphalt Concrete Overlays Placed on Fractured PCC Pavements,” Transportation Research Record 1374, pp. 19 – 26, Washington, D.C., 1992.

APPENDIX A

MODULUS 6 RESULTS FOR CRACK AND SEAT SECTIONS

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

District:
County :
Highway/Road:

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	3.00	160,000	720,000	H1: v = 0.35
Base:	9.00	30,000	500,000	H2: v = 0.35
Subbase:	8.00	100,000	2,000,000	H3: v = 0.25
Subgrade:	280.00(by DB)	20,000		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
0.159	11,678	11.39	7.89	5.70	4.13	3.08	2.25	1.77	720.0	157.4	100.0	17.0	3.43	300.0 *
0.297	10,709	12.81	8.31	5.37	4.11	3.15	2.51	1.81	720.0	61.9	390.7	14.4	4.03	239.4 *
0.436	10,852	6.54	4.07	3.33	2.83	2.36	1.96	1.64	720.0	207.4	1889.6	18.9	0.33	300.0 *
0.574	11,360	12.01	7.41	5.14	3.69	2.62	1.95	1.53	720.0	108.6	100.0	19.3	3.05	300.0 *
0.712	10,737	11.55	7.55	5.32	4.04	3.16	2.51	2.04	720.0	128.0	116.2	15.5	1.49	300.0 *
0.850	10,701	9.82	6.39	4.98	4.00	3.09	2.37	1.83	218.0	270.5	203.3	16.2	0.62	300.0
1.127	11,051	6.73	4.69	3.92	3.31	2.72	2.24	1.85	342.5	451.7	770.9	17.5	0.39	300.0
1.265	11,039	9.94	6.76	4.39	3.22	2.49	1.96	1.58	720.0	156.8	100.0	20.6	3.07	300.0 *
1.403	11,666	6.59	4.76	4.24	3.67	3.13	2.63	2.10	719.7	413.3	1355.4	14.6	0.76	300.0 *
1.542	10,324	10.57	6.90	4.26	3.35	2.72	2.23	1.87	720.0	78.5	495.6	16.5	4.72	300.0 *
1.680	9,811	7.06	4.88	3.48	2.74	2.21	1.86	1.53	720.0	234.5	242.6	19.8	3.37	300.0 *
1.818	9,855	5.60	3.85	2.96	2.40	1.99	1.67	1.35	720.0	291.7	661.0	21.2	2.31	300.0 *
1.956	9,855	7.89	5.30	3.92	3.13	2.54	2.07	1.72	720.0	192.3	272.1	17.3	1.85	300.0 *
2.095	9,803	7.37	5.96	4.65	3.63	2.74	2.20	1.78	720.0	479.3	102.2	16.0	1.85	300.0 *
2.233	9,958	5.59	3.98	3.15	2.50	1.90	1.54	1.30	663.9	499.1	230.3	23.2	0.87	300.0 *
2.371	9,926	9.14	5.79	3.71	2.80	2.07	1.66	1.43	720.0	99.2	260.4	20.9	3.62	300.0 *
2.509	9,859	9.36	5.74	3.92	3.04	2.41	1.95	1.61	720.0	78.3	1039.6	17.1	4.09	300.0 *
2.648	9,720	12.07	6.88	4.07	3.13	2.46	1.97	1.56	720.0	60.1	171.8	18.1	3.49	300.0 *
2.786	9,688	11.06	6.26	4.37	3.41	2.70	2.15	1.72	720.0	59.2	855.6	15.3	2.17	300.0 *
Mean:		9.11	5.97	4.26	3.32	2.61	2.09	1.69	670.7	212.0	492.5	17.9	2.39	300.0
Std. Dev:		2.37	1.36	0.78	0.53	0.40	0.30	0.21	139.7	149.7	495.5	2.4	1.38	16.7
Var Coeff(%):		26.03	22.76	18.35	16.10	15.26	14.32	12.69	20.8	70.6	100.6	13.7	57.82	5.6

Figure A1. Modulus 6 Results from Hemphill County Line to County Road C in Wheeler County (Southbound).

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

District:
County :
Highway/Road:

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	3.00	160,000	720,000	H1: v = 0.35
Base:	9.00	30,000	500,000	H2: v = 0.35
Subbase:	8.00	100,000	3,000,000	H3: v = 0.25
Subgrade:	280.00(by DB)	20,000		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
0.827	11,182	10.53	5.15	3.92	3.06	2.41	1.93	1.56	234.3	113.3	783.4	23.4	0.78	300.0
1.376	10,181	9.48	6.63	4.67	3.44	2.63	2.07	1.71	720.0	178.3	100.0	17.4	2.06	300.0 *
1.651	10,320	8.74	6.73	4.92	3.74	2.84	2.13	1.69	294.9	431.1	100.0	16.7	2.53	300.0 *
1.788	9,720	6.27	4.54	3.70	2.99	2.36	1.94	1.61	430.8	500.0	303.5	18.3	0.62	300.0 *
1.925	9,736	5.87	4.54	3.82	3.12	2.48	1.98	1.61	720.0	500.0	340.0	17.0	1.59	300.0 *
2.063	9,728	8.23	5.28	3.97	3.05	2.43	1.96	1.62	720.0	164.3	244.1	17.9	1.20	300.0 *
2.200	10,483	8.73	5.74	4.27	3.28	2.52	1.99	1.58	720.0	199.9	149.9	18.9	0.65	300.0 *
2.337	9,799	6.36	5.06	4.14	3.31	2.63	2.10	1.65	720.0	500.0	138.4	17.8	3.70	300.0 *
2.612	9,803	5.37	3.97	3.38	2.91	2.36	2.00	1.66	720.0	500.0	432.3	18.3	3.31	300.0 *
2.749	9,760	7.44	5.44	3.96	2.96	2.23	1.75	1.36	413.2	381.3	100.0	20.0	1.79	300.0 *
2.886	9,676	11.26	8.28	6.17	4.76	3.56	2.69	2.07	194.0	267.0	100.0	12.3	2.60	300.0 *
3.024	9,875	8.50	5.78	4.40	3.41	2.65	2.13	1.67	720.0	212.5	152.3	16.6	0.81	300.0 *
3.162	10,038	7.26	5.26	4.47	3.57	2.62	2.31	1.68	356.7	500.0	229.1	16.7	3.14	300.0 *
3.298	9,767	8.55	5.23	3.79	3.00	2.38	1.90	1.55	202.8	224.3	241.8	20.3	2.02	300.0
Mean:		8.04	5.55	4.26	3.33	2.58	2.06	1.64	511.9	333.7	243.9	18.0	1.91	300.0
Std. Dev:		1.73	1.09	0.69	0.48	0.32	0.22	0.15	226.0	152.0	186.2	2.4	1.03	0.0
Var Coeff(%):		21.50	19.57	16.13	14.49	12.56	10.83	9.14	44.2	45.6	76.3	13.6	54.05	0.0

Figure A2. Modulus 6 Results from Hemphill County Line to County Road C in Wheeler County (Northbound).

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

District:			MODULI RANGE(psi)		
County :		Thickness(in)	Minimum	Maximum	Poisson Ratio Values
Highway/Road:	Pavement:	10.50	340,000	1,040,000	H1: v = 0.35
	Base:	8.00	100,000	3,000,000	H2: v = 0.25
	Subbase:	0.00			H3: v = 0.00
	Subgrade:	127.68(by DB)	20,000		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	
0.387	10,050	5.27	4.22	3.67	3.04	2.44	1.92	1.49	914.3	621.6	0.0	11.8	0.78	125.1	
0.426	9,835	5.26	4.19	3.46	2.94	2.11	1.88	1.38	814.8	533.3	0.0	12.9	2.96	132.6	
0.465	9,930	5.56	4.33	3.82	3.28	2.77	2.24	1.80	582.1	1768.3	0.0	9.1	0.37	144.7	
0.503	9,926	5.16	4.19	3.67	3.15	2.63	2.06	1.78	893.2	951.4	0.0	10.2	1.00	157.5	
0.581	9,843	5.68	3.87	3.09	2.48	1.94	1.51	1.17	394.8	986.2	0.0	16.7	1.04	122.9	
0.620	9,744	5.76	4.29	3.68	3.08	2.48	2.00	1.60	492.6	1197.8	0.0	11.1	0.34	146.4	
0.659	9,903	4.52	3.56	3.03	2.55	2.09	1.70	1.37	805.2	1179.7	0.0	13.0	0.45	141.5	
0.698	9,847	5.46	4.22	3.68	3.09	2.58	2.11	1.74	583.1	1458.8	0.0	10.0	0.34	181.2	
0.736	9,855	4.90	4.12	3.56	2.98	2.42	1.95	1.56	1040.0	727.3	0.0	11.3	0.83	144.4	*
0.775	9,922	5.39	4.19	3.61	2.97	2.45	2.01	1.66	632.2	1087.3	0.0	11.1	0.77	189.4	
0.814	9,843	4.95	3.86	3.37	2.87	2.36	1.94	1.57	678.6	1500.3	0.0	10.8	0.30	147.9	
0.853	9,918	6.15	4.88	4.04	3.26	2.57	2.05	1.61	700.8	423.7	0.0	11.6	0.66	141.7	
0.892	9,918	4.77	4.07	3.63	3.01	2.48	2.02	1.59	1040.0	1014.0	0.0	10.4	1.58	134.7	*
0.930	10,042	6.22	4.69	3.98	3.30	2.63	2.00	1.61	533.5	705.7	0.0	11.5	0.88	139.9	
1.008	9,914	6.19	4.56	3.87	3.27	2.68	2.20	1.77	417.2	1624.5	0.0	10.2	0.78	150.4	
1.047	9,970	4.83	3.80	3.30	2.76	2.31	1.87	1.56	745.7	1306.7	0.0	11.5	0.33	300.0	
1.087	9,910	5.27	4.16	3.71	2.98	2.37	2.03	1.54	779.2	812.5	0.0	11.2	2.02	300.0	
1.163	9,883	4.69	3.52	2.99	2.49	2.02	1.64	1.33	607.5	1527.0	0.0	13.7	0.54	149.5	
1.202	9,847	4.41	3.61	3.16	2.68	2.23	1.81	1.48	1038.5	1146.8	0.0	11.7	0.23	156.6	*
1.241	9,875	4.48	3.61	3.12	2.59	2.14	1.79	1.44	901.1	1156.8	0.0	12.2	0.94	145.1	
1.280	10,193	5.00	3.71	3.21	2.70	2.18	1.78	1.38	574.4	1705.1	0.0	12.9	0.47	116.9	
1.319	10,105	4.77	3.84	3.39	2.85	2.32	1.93	1.57	892.6	1159.0	0.0	11.3	0.73	156.4	
1.474	10,105	4.71	3.94	3.51	2.94	2.40	1.97	1.62	1040.0	1049.1	0.0	11.0	1.18	173.5	*
1.513	9,811	4.06	3.48	2.99	2.51	2.05	1.67	1.37	1040.0	1245.7	0.0	12.5	1.68	160.4	*
Mean:		5.14	4.04	3.48	2.91	2.36	1.92	1.54	755.9	1120.4	0.0	11.6	0.88	146.2	
Std. Dev:		0.58	0.37	0.31	0.26	0.23	0.18	0.16	208.1	365.2	0.0	1.5	0.64	27.3	
Var Coeff(%):		11.28	9.18	9.04	9.02	9.61	9.15	10.16	27.5	32.6	0.0	13.0	72.14	18.7	

Figure A3. Modulus 6 Results from County Road E to County Road G in Collingsworth County (Southbound).

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

District:
County :
Highway/Road:

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	10.50	340,000	1,040,000	H1: v = 0.35
Base:	8.00	100,000	3,000,000	H2: v = 0.25
Subbase:	0.00			H3: v = 0.00
Subgrade:	117.71(by DB)	20,000		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	
0.191	10,101	4.87	4.11	3.38	2.89	2.40	1.92	1.38	1040.0	817.0	0.0	11.2	1.08	90.1	*
0.256	10,061	4.89	3.98	3.45	3.10	2.23	1.96	1.54	1040.0	815.6	0.0	11.1	2.80	119.3	*
0.320	9,823	4.37	3.67	3.04	2.46	1.95	1.58	1.21	1040.0	647.7	0.0	14.3	1.63	115.7	*
0.384	10,053	5.09	4.01	3.52	3.02	2.49	2.07	1.69	737.9	1617.1	0.0	10.0	0.41	155.9	
0.449	10,065	5.65	4.16	3.61	3.04	2.47	2.01	1.60	504.9	1737.7	0.0	10.9	0.38	138.4	
0.513	9,930	5.26	3.91	3.35	2.79	2.20	1.76	1.33	589.4	1141.2	0.0	12.7	0.61	108.8	
0.578	9,819	4.88	3.65	3.17	2.66	2.16	1.77	1.42	598.5	1815.7	0.0	12.1	0.45	141.1	
0.642	9,883	4.69	3.66	3.21	2.72	2.19	1.83	1.39	783.2	1458.9	0.0	11.5	0.85	107.5	
0.706	9,815	5.68	4.93	4.30	3.65	2.94	2.36	1.81	1040.0	532.4	0.0	8.7	1.29	120.3	*
0.771	9,771	5.48	4.49	3.93	3.37	2.82	2.28	1.85	866.9	963.3	0.0	8.6	0.33	156.9	
0.835	10,097	4.83	3.83	3.35	2.84	2.31	1.91	1.59	852.5	1265.2	0.0	11.2	0.60	182.9	
0.900	9,934	4.79	3.67	3.25	2.80	2.29	1.89	1.52	680.1	2121.9	0.0	10.8	0.53	137.1	
0.964	10,026	5.67	4.34	3.74	3.15	2.57	2.15	1.74	565.5	1514.1	0.0	10.2	0.79	155.5	
1.530	10,069	4.58	3.43	2.87	2.33	1.81	1.42	1.08	746.0	866.4	0.0	16.5	0.52	110.0	
1.579	9,990	6.06	4.69	3.72	2.99	2.39	1.93	1.57	584.5	584.2	0.0	12.5	1.97	173.3	
1.625	10,081	6.26	4.96	4.31	3.68	3.03	2.49	2.00	620.7	1128.2	0.0	8.5	0.25	150.8	
1.672	9,986	5.31	4.37	3.76	3.18	2.63	2.14	1.72	903.1	890.1	0.0	9.7	0.25	147.7	
1.721	10,018	5.15	4.12	3.67	3.13	2.61	2.16	1.74	793.0	1500.9	0.0	9.3	0.38	146.8	
1.767	10,010	6.08	4.81	4.20	3.55	2.86	2.34	1.81	644.0	990.2	0.0	9.1	0.61	124.3	
1.814	10,053	5.59	4.84	4.23	3.54	2.85	2.31	1.82	1040.0	566.7	0.0	9.2	1.31	138.9	*
1.862	9,998	6.23	5.54	4.89	4.18	3.43	2.81	2.19	1040.0	584.2	0.0	7.0	1.34	132.4	*
1.909	10,081	5.77	4.81	4.28	3.65	3.02	2.47	1.98	1026.2	751.0	0.0	8.1	0.56	149.6	
1.956	9,934	8.55	7.34	6.08	4.86	3.69	2.82	2.06	994.8	100.0	0.0	8.2	0.80	117.1	*
2.004	10,014	5.72	4.84	4.07	3.39	2.84	2.19	1.78	1040.0	506.7	0.0	9.7	1.02	153.4	*
2.051	9,914	5.93	5.05	4.42	3.74	3.03	2.43	1.92	1040.0	553.2	0.0	8.3	0.92	141.0	*
2.098	9,934	7.09	6.06	5.08	4.16	3.28	2.49	1.98	1040.0	219.7	0.0	8.8	0.80	152.9	*
2.146	9,934	5.60	4.73	4.14	3.52	2.91	2.38	1.88	1040.0	726.1	0.0	8.4	0.32	136.6	*
2.193	9,966	5.04	3.95	3.48	2.98	2.48	2.10	1.67	687.3	2059.1	0.0	9.7	0.58	132.6	
Mean:		5.54	4.50	3.88	3.26	2.64	2.14	1.69	842.1	1016.9	0.0	10.2	0.83	136.2	
Std. Dev:		0.85	0.83	0.69	0.56	0.44	0.33	0.27	192.2	536.4	0.0	2.1	0.58	22.1	
Var Coeff(%):		15.32	18.55	17.89	17.10	16.80	15.61	15.72	22.8	52.7	0.0	20.2	69.27	16.2	

Figure A4. Modulus 6 Results from FM 1036 to FM 1439 in Collingsworth County (Southbound).

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

District:
County :
Highway/Road:

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	5.00	160,000	720,000	H1: v = 0.35
Base:	8.00	10,000	150,000	H2: v = 0.35
Subbase:	8.00	100,000	3,000,000	H3: v = 0.25
Subgrade:	121.03(by DB)	20,000		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
0.120	10,371	6.29	3.98	3.33	2.77	2.16	1.75	1.36	720.0	150.0	868.1	13.5	4.26	102.6 *
0.234	9,907	10.13	6.09	4.39	3.54	2.82	2.32	1.93	499.7	45.1	2535.3	8.9	0.94	210.6
0.384	10,006	10.54	6.12	4.60	3.87	3.06	2.59	2.04	349.6	56.5	2753.0	8.2	0.96	300.0
0.534	10,014	8.00	5.02	4.09	3.45	2.77	2.20	1.80	336.9	148.8	1430.1	9.5	0.36	152.7
0.682	9,875	11.04	6.57	4.86	3.93	3.15	2.55	2.03	378.4	50.6	1862.6	8.3	0.65	136.3
0.831	9,859	9.75	6.34	4.80	3.89	3.11	2.44	1.91	452.3	82.5	755.7	8.5	0.25	121.4
0.982	9,875	9.97	6.38	4.85	3.91	3.11	2.59	2.04	449.9	71.2	1131.5	8.1	1.10	127.6
1.131	10,089	9.30	5.38	3.79	2.98	2.36	1.97	1.52	517.9	47.8	2832.8	11.3	1.65	108.5
1.280	9,676	11.17	6.93	4.78	3.60	2.74	2.12	1.74	453.6	50.6	477.4	10.4	0.74	160.6
1.430	9,601	15.70	8.17	4.65	3.33	2.49	1.98	1.62	326.1	16.6	1805.2	11.4	2.04	191.2
1.580	10,002	9.59	5.92	4.16	3.31	2.57	2.04	1.56	610.3	45.7	1927.4	10.3	0.54	105.1
1.730	9,895	10.68	6.62	4.91	3.98	3.12	2.40	1.80	349.5	75.5	690.0	9.1	0.46	101.3
1.880	9,605	11.94	6.80	5.04	3.97	3.07	2.44	1.90	258.8	58.2	902.9	9.0	0.77	126.6
2.029	9,914	9.80	5.98	4.30	3.50	2.80	2.32	1.91	551.2	44.5	2955.1	8.7	0.85	174.4
2.178	9,672	12.07	7.15	5.09	3.96	3.02	2.31	1.79	327.0	51.7	625.2	9.4	0.18	122.9
2.328	9,930	9.38	6.50	5.22	4.35	3.31	2.75	2.17	412.2	150.0	429.8	7.9	1.25	300.0 *
2.478	9,803	9.35	5.86	4.74	3.89	3.11	2.57	2.08	323.3	107.2	1223.3	8.1	0.79	155.3
2.627	9,942	9.48	5.89	4.25	3.34	2.61	2.06	1.65	489.5	67.0	906.9	10.7	0.60	140.3
2.777	10,014	9.47	6.34	4.85	3.75	2.85	2.26	1.77	471.1	117.0	283.7	10.2	0.92	129.6
2.926	9,871	8.04	4.70	3.55	2.74	2.05	1.63	1.32	327.4	130.2	573.4	14.6	1.12	151.9
Mean:		10.08	6.14	4.51	3.60	2.81	2.26	1.80	430.2	78.3	1348.5	9.8	1.02	142.0
Std. Dev:		1.88	0.91	0.52	0.43	0.35	0.30	0.24	114.2	40.8	867.9	1.8	0.88	37.7
Var Coeff(%):		18.67	14.80	11.51	11.94	12.41	13.06	13.11	26.5	52.1	64.4	18.5	86.63	26.6

Figure A5. Modulus 6 Results from Collingsworth County Line to Buck Creek Bridge in Childress County (Southbound).

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

District:
County :
Highway/Road:

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	5.00	160,000	720,000	H1: v = 0.35
Base:	8.00	10,000	150,000	H2: v = 0.35
Subbase:	8.00	100,000	3,000,000	H3: v = 0.25
Subgrade:	106.80(by DB)	20,000		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	
0.271	9,791	10.75	6.30	4.39	3.39	2.60	2.09	1.70	465.6	39.6	1809.9	9.3	1.38	168.0	
0.421	9,803	13.10	7.20	5.52	4.44	3.21	2.45	1.94	160.0	81.9	520.6	8.5	1.03	142.4	*
0.571	9,799	10.15	5.78	4.52	3.87	3.14	2.68	2.11	268.1	74.2	3000.0	6.9	1.24	115.3	*
0.722	9,748	13.01	6.87	5.56	4.59	3.63	2.98	2.34	160.0	66.1	2019.8	6.5	0.98	133.0	*
0.872	9,811	8.81	5.24	3.99	3.22	2.51	1.99	1.50	341.9	95.9	1162.6	10.0	0.53	99.6	
1.023	9,903	8.66	5.06	4.03	3.33	2.63	2.09	1.71	289.1	112.6	1671.6	9.3	0.29	159.9	
1.173	9,767	11.46	6.07	4.82	4.01	3.07	2.63	2.02	191.9	70.0	2361.0	7.6	1.62	300.0	
1.324	9,835	11.10	6.41	4.33	3.44	2.66	2.11	1.65	454.6	35.1	2318.3	9.1	0.66	117.9	
1.474	9,815	9.72	5.52	4.17	3.31	2.45	1.82	1.44	200.5	138.1	438.1	11.6	0.53	111.5	
1.625	9,867	9.04	5.81	4.41	3.54	2.78	2.21	1.71	505.2	81.2	987.4	8.4	0.42	113.4	
1.776	9,656	10.87	6.77	4.57	3.44	2.53	2.09	1.63	720.0	24.8	2532.8	8.8	2.30	181.9	*
1.926	9,887	10.20	6.13	4.39	3.44	2.64	2.06	1.63	417.9	59.2	997.5	9.6	0.47	130.9	
2.077	9,728	10.99	6.57	4.65	3.64	2.87	2.33	1.79	464.7	39.1	2049.4	8.0	1.28	113.7	
2.227	9,859	11.31	6.36	4.62	3.80	2.96	2.37	1.88	313.5	49.3	2102.7	8.2	0.57	127.5	
2.378	9,760	9.96	5.67	4.37	3.65	2.94	2.35	1.87	286.1	72.3	2357.8	7.9	0.26	127.1	
2.528	9,807	9.06	4.91	3.85	3.23	2.55	2.07	1.63	257.5	88.5	2776.1	9.4	0.56	114.6	
2.679	9,708	10.81	6.12	4.65	3.80	3.04	2.41	1.93	275.9	62.7	1897.7	7.9	0.34	141.1	
2.980	9,847	7.95	3.76	2.54	2.19	1.82	1.51	1.26	253.5	90.6	3000.0	15.8	4.88	300.0	*
Mean:		10.39	5.92	4.41	3.57	2.78	2.24	1.76	334.8	71.2	1889.1	9.0	1.07	127.8	
Std. Dev:		1.40	0.82	0.65	0.52	0.39	0.34	0.26	145.1	28.6	787.9	2.1	1.09	31.5	
Var Coeff(%):		13.49	13.90	14.67	14.47	13.97	15.04	14.47	43.3	40.2	41.7	22.7	101.93	24.6	

Figure A6. Modulus 6 Results from Collingsworth County Line to Buck Creek Bridge in Childress County (Northbound).

APPENDIX B

MODULUS 6 RESULTS FOR THE RUBBLIZED SECTION

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

District:
County :
Highway/Road:

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	3.00	20,000	460,000	H1: v = 0.35
Base:	8.00	1,500,000	5,000,000	H2: v = 0.20
Subbase:	0.00			H3: v = 0.00
Subgrade:	265.26(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
1.000	10,117	7.31	6.39	5.57	4.62	3.55	2.82	2.12	460.0	3456.8	0.0	13.1	1.57	300.0 *
2.000	10,085	6.98	5.43	5.30	4.33	3.42	2.48	1.85	267.3	4836.8	0.0	13.9	4.39	300.0
3.000	10,014	7.04	5.99	5.13	4.18	3.26	2.54	1.92	459.2	3154.9	0.0	14.6	0.74	300.0 *
4.000	9,942	8.44	6.56	6.00	4.77	3.55	2.75	2.07	208.0	2952.0	0.0	13.3	2.91	300.0
5.000	9,815	7.44	6.13	5.17	4.12	3.30	2.35	1.57	333.1	2659.1	0.0	15.0	1.64	181.2
6.000	9,958	7.89	5.81	5.20	3.98	2.84	2.08	1.54	187.6	2340.9	0.0	17.2	3.60	283.6
7.000	10,061	6.92	5.44	4.45	3.53	2.75	2.09	1.62	228.0	3111.1	0.0	18.2	0.18	300.0
8.000	10,010	7.20	5.91	4.93	3.77	2.81	2.03	1.49	460.0	1862.1	0.0	17.6	1.72	267.6 *
9.000	9,907	7.70	4.91	4.75	3.66	2.65	2.00	1.50	100.7	3795.6	0.0	18.3	4.85	300.0
11.000	9,962	6.40	5.66	5.02	4.12	3.08	2.43	1.82	460.0	3888.4	0.0	14.7	2.92	300.0 *
12.000	9,883	8.04	6.86	5.89	4.65	3.34	2.37	1.74	460.0	1719.3	0.0	14.4	3.74	253.3 *
13.000	9,954	8.53	6.56	6.14	5.09	3.89	2.93	2.11	183.9	3717.4	0.0	12.0	3.30	241.5
14.000	9,958	6.92	6.11	5.35	4.40	3.41	2.59	1.86	460.0	3439.2	0.0	13.8	2.11	237.6 *
15.000	9,930	6.64	5.69	4.93	4.05	3.19	2.42	1.77	460.0	3583.0	0.0	14.8	1.29	262.4 *
16.000	9,871	6.98	5.50	4.50	3.54	2.67	2.01	1.52	252.9	2608.3	0.0	18.4	0.50	300.0
17.000	9,803	8.06	6.58	5.60	4.63	3.61	2.82	2.13	242.3	3221.4	0.0	13.0	0.61	300.0
18.000	9,907	9.08	6.91	5.58	4.37	3.25	2.42	1.80	156.8	1951.0	0.0	15.2	0.55	291.0
19.000	9,855	8.46	6.53	5.15	4.09	2.99	2.17	1.47	193.3	1858.0	0.0	16.5	1.13	188.3
20.000	9,835	9.33	7.09	5.25	4.04	2.80	1.93	1.23	129.4	1500.0	0.0	17.2	2.99	161.1 *
Mean:		7.65	6.11	5.26	4.21	3.18	2.38	1.74	300.1	2929.2	0.0	15.3	2.14	276.3
Std. Dev:		0.84	0.59	0.46	0.42	0.36	0.31	0.26	134.8	892.5	0.0	2.0	1.42	57.4
Var Coeff(%):		10.93	9.68	8.80	10.10	11.24	13.05	14.79	44.9	30.5	0.0	13.0	66.21	21.6

Figure B1. Modulus 6 Results from the Rubblized Section before Rubblization.

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

		Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
District:					
County :					
Highway/Road:	Pavement:	8.00	160,000	720,000	H1: v = 0.35
	Base:	8.00	30,000	300,000	H2: v = 0.35
	Subbase:	0.00			H3: v = 0.00
	Subgrade:	121.01(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	9,577	10.41	7.90	5.37	3.84	2.68	1.91	1.52	461.9	108.0	0.0	12.2	2.19	121.7
120.000	9,541	10.25	7.64	5.10	3.57	2.47	1.74	1.43	450.0	98.1	0.0	13.3	2.27	115.8
240.000	9,589	9.22	7.09	4.99	3.59	2.48	1.77	1.39	601.4	114.0	0.0	13.1	1.52	120.9
360.000	9,501	10.51	8.23	5.74	4.22	3.00	2.20	1.68	514.5	117.4	0.0	10.7	2.32	140.8
480.000	9,505	10.11	7.90	5.70	4.29	3.15	2.34	1.86	473.8	178.4	0.0	10.2	2.08	149.8
600.000	9,470	10.70	8.33	5.74	4.20	3.01	2.23	1.83	472.9	120.2	0.0	10.7	2.80	149.9
720.000	9,481	10.07	7.71	5.34	3.90	2.81	2.11	1.75	444.6	150.2	0.0	11.5	3.00	161.2
840.000	9,497	9.59	7.40	5.26	3.91	2.87	2.19	1.77	448.9	198.8	0.0	11.1	2.96	179.0
960.000	9,485	10.61	8.16	5.80	4.31	3.14	2.31	1.83	412.0	167.1	0.0	10.3	2.17	143.2
10800.000	9,521	10.10	7.74	5.14	3.52	2.44	1.74	1.41	537.6	76.9	0.0	13.4	2.78	124.0
12000.000	9,450	11.96	8.87	5.78	3.99	2.80	2.01	1.65	362.6	83.3	0.0	11.7	3.27	130.0
1320.000	9,505	10.16	7.66	5.12	3.55	2.43	1.73	1.30	499.9	84.2	0.0	13.4	2.43	122.2
1440.000	9,382	11.31	8.36	5.46	3.86	2.70	2.25	1.76	327.5	126.4	0.0	11.6	5.85	152.3
1560.000	9,470	10.93	8.38	5.77	4.19	3.02	2.19	1.77	422.1	125.4	0.0	10.8	2.51	133.7
1680.000	9,438	12.06	9.03	5.94	4.04	2.80	2.07	1.69	388.6	74.1	0.0	11.5	3.57	150.2
1800.000	9,481	10.16	7.82	5.46	4.04	2.91	2.20	1.96	432.1	164.1	0.0	11.0	2.95	168.5
1920.000	9,477	10.81	8.20	5.49	3.84	2.67	1.93	1.55	459.1	87.9	0.0	12.1	2.78	132.8
2040.000	9,434	10.50	7.81	5.22	3.61	2.53	1.87	1.55	409.2	105.7	0.0	12.7	3.32	150.3
2160.000	9,438	10.22	7.61	5.04	3.52	2.44	1.72	1.43	439.2	97.0	0.0	13.3	2.44	116.1
2280.000	9,466	10.63	8.01	5.41	3.76	2.63	1.87	1.51	451.2	92.9	0.0	12.4	2.38	123.2
2400.000	9,402	10.57	7.89	5.19	3.64	2.57	1.91	1.55	395.9	109.5	0.0	12.5	3.69	152.9
2520.000	9,418	10.76	7.80	5.10	3.56	2.44	1.91	1.44	342.6	117.4	0.0	13.0	4.52	131.8
2640.000	9,386	11.35	8.40	5.37	3.58	2.40	1.69	1.38	434.2	56.1	0.0	13.3	2.89	119.5
2760.000	9,410	10.95	8.02	5.21	3.61	2.50	1.83	1.44	370.0	96.3	0.0	12.9	3.47	140.5
2880.000	9,442	9.81	7.15	4.65	3.23	2.14	1.73	1.35	402.3	112.2	0.0	14.5	4.96	107.8
3000.000	9,446	10.25	7.59	4.98	3.46	2.40	1.86	1.35	400.3	112.5	0.0	13.3	4.40	142.2
Mean:		10.54	7.95	5.36	3.80	2.67	1.97	1.58	436.7	114.4	0.0	12.2	3.06	137.0
Std. Dev:		0.65	0.46	0.32	0.29	0.26	0.21	0.19	60.2	33.8	0.0	1.2	0.98	16.9
Var Coeff(%):		6.12	5.73	5.93	7.68	9.85	10.62	11.94	13.8	29.6	0.0	9.5	32.12	12.3

Figure B2. Modulus 6 Results from the Rubblized Section 6 to 9 Months after Rubblization.

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

District:
County :
Highway/Road:

	Thickness(in)	MODULI RANGE(psi) Minimum	Maximum	Poisson Ratio Values
Pavement:	8.00	200,000	750,000	H1: v = 0.35
Base:	8.00	30,000	300,000	H2: v = 0.35
Subbase:	0.00			H3: v = 0.00
Subgrade:	132.40(by DB)	10,000		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	10,181	8.58	7.03	5.43	4.13	3.10	2.35	1.81	750.0	250.1	0.0	11.5	1.29	146.9 *
0.025	10,121	8.81	6.80	5.04	3.72	2.70	2.04	1.56	581.8	217.2	0.0	13.3	1.79	141.7
0.049	10,177	8.30	6.67	4.98	3.65	2.62	1.96	1.48	750.0	176.1	0.0	14.0	1.65	134.0 *
0.075	10,085	8.04	6.41	5.03	3.87	2.95	2.30	1.79	750.0	300.0	0.0	12.0	1.38	151.0 *
0.098	10,101	8.50	6.46	5.07	3.94	3.07	2.33	1.82	684.8	300.0	0.0	11.8	2.47	152.6 *
0.124	9,978	9.47	7.71	5.96	4.40	3.30	2.57	1.88	750.0	171.4	0.0	10.7	1.99	124.4 *
0.149	10,034	8.50	6.78	5.17	3.89	2.95	2.29	1.78	663.2	273.3	0.0	11.9	2.01	154.3
0.172	9,990	8.87	7.22	5.59	4.26	3.23	2.47	1.94	750.0	181.3	0.0	11.4	2.93	167.2 *
0.197	9,978	8.76	6.91	5.29	4.00	3.07	2.35	1.87	587.1	291.6	0.0	11.5	1.74	181.3
0.222	10,034	8.73	6.76	4.96	3.58	2.59	1.98	1.50	605.1	191.9	0.0	13.7	2.31	137.7
0.246	9,986	8.59	6.57	4.84	3.59	2.59	2.13	1.57	504.4	272.4	0.0	13.3	3.49	183.5
0.271	10,014	9.13	7.20	5.34	3.91	2.86	2.17	1.70	630.3	187.2	0.0	12.4	2.00	167.6
0.295	9,907	8.66	6.69	4.99	3.69	2.77	2.13	1.72	539.3	260.5	0.0	12.7	2.25	188.9
0.320	10,018	8.67	6.91	5.19	3.87	2.91	2.27	1.85	666.6	239.0	0.0	12.1	2.37	216.7
0.345	9,879	9.33	7.52	5.72	4.28	3.17	2.43	1.88	716.2	186.7	0.0	11.0	1.67	156.6
0.369	9,922	8.89	7.05	5.39	4.03	2.99	2.26	1.73	674.1	217.2	0.0	11.7	1.31	145.3
0.394	10,093	8.85	6.96	5.31	3.87	2.81	2.05	1.54	748.6	168.4	0.0	12.8	0.84	134.8 *
0.419	9,891	8.54	7.07	5.23	3.78	2.69	1.95	1.54	750.0	121.1	0.0	13.6	2.81	131.8 *
0.443	9,954	9.05	7.33	5.45	3.94	2.85	2.11	1.63	750.0	143.3	0.0	12.5	1.66	150.2 *
0.468	10,030	8.76	7.08	5.39	3.98	2.76	2.04	1.53	750.0	119.8	0.0	13.4	3.17	140.8 *
0.492	9,716	9.48	7.31	5.30	3.77	2.70	1.97	1.57	587.0	132.3	0.0	13.0	1.64	138.7
0.517	10,042	7.44	5.74	4.15	2.95	2.11	1.54	1.20	750.0	178.6	0.0	17.1	1.71	129.2 *
0.543	9,998	8.53	6.74	4.87	3.49	2.47	1.90	1.39	734.1	146.9	0.0	14.3	2.67	161.3
0.568	9,946	10.43	8.22	6.00	4.39	2.91	2.15	1.65	706.3	84.9	0.0	12.0	1.72	111.4
0.591	9,974	7.68	6.04	4.36	3.13	2.23	1.68	1.28	750.0	177.9	0.0	15.9	2.34	136.9 *
Mean:		8.74	6.93	5.20	3.84	2.82	2.14	1.65	685.2	199.6	0.0	12.8	2.05	148.4
Std. Dev:		0.59	0.51	0.42	0.34	0.29	0.24	0.19	77.5	60.5	0.0	1.5	0.63	20.6
Var Coeff(%):		6.74	7.34	7.99	8.90	10.19	11.16	11.76	11.3	30.3	0.0	11.5	30.91	13.9

Figure B3. Modulus 6 Results from the Rubblized Section 2 Years after Rubblization.