FORENSIC INVESTIGATIONS OF ROADWAY PAVEMENT FAILURES

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A paper submitted for presentation at the TRB 86th Annual Meeting in Washington, DC in January 21-25, 2007, and for publication at a Transportation Research Record: Journal of the Transportation Research Board.
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ABSTRACT

Forensic investigations of pavement failures are critical, as the information gained can be used to identify the underlying cause of the problem, develop an optimal rehabilitation strategy, and to resolve construction disputes. The Texas Department of Transportation has had a formalized forensic team approach for over 10 years. Application of nondestructive testing such as the Ground Penetrating Radar (GPR) and Falling Weight Deflectometer (FWD), as well as field testing such as Dynamic Cone Penetration (DCP), coring, and laboratory testing have been found critical to these forensic investigations on flexible pavements. The root causes of the pavement failures often can be identified through these tools, in conjunction with thorough review of construction records and rehabilitation history.

In this paper three field projects are presented to illustrate the integrated approach used widely in Texas. In each case the combined GPR and FWD data was extremely useful at identifying contributing factors; such as stripping in the hot mix or localized areas of wet or weak base. The DCP is used for validating problems with base and subbase layers. To determine the optimum rehabilitation strategy, it requires knowledge of what is the main cause of the problem. Laboratory tests are often required to complete the investigation especially if the repair strategy calls for in place recycling of the existing structure. The extent of stripping and high porosity that caused delamination for projects 1 and 2 were detected by GPR and verified by core samples. This combination (GPR+coring) was used to map the entire project. GPR, FWD, DCP and field soil samples all showed indications that the existing base on project 1 was wet and the stiffness was only about 1/3 of a typical flexible base in Texas. FWD data demonstrates that the pavement structure for projects 1 and 2 were inadequate, so a rehabilitation strategy was selected that included structural strengthening. In project 3, GPR, lab density, and permeability tests indicate that the dramatic pavement failures were attributed to moisture entering the base through a poorly-compacted AC layer and poor longitudinal joints. The base material was found to be highly susceptible to moisture. It did not meet TxDOT’s compressive strength requirements when subjected to capillary soaking. The repetitive triaxial test results revealed that the stiffness and load carrying capability became inadequate when the base materials were exposed to moisture. The techniques demonstrated in this study are widely used within Texas and are applicable to a wide range of pavement forensic studies.

INTRODUCTION

Despite advancements in pavement technology in past decades, premature failures and chronic pavement distresses continue to occur that cost millions of dollars in repairs. Although improvements have been made to construction specifications, equipment, and construction processes, poor quality construction can occur due to a number of complex and sometimes competing variables such as (1) the low bid process (2) lack of experienced inspectors and project managers (3) poor selection of construction materials (4) lack of knowledge of the existing pavement conditions (5) unfamiliar construction methods and procedure (6) other issues
unforeseen during design and construction phases. To reduce the probability of re-occurring premature pavement failures, the root causes of problems need to be identified and the lessons learnt incorporated into future project designs. This can be challenging, as sometimes the information obtained is incomplete and resources are limited. In conducting forensic studies, a thorough review and analysis of existing construction records and tests is required. Also, nondestructive testing such as Ground Penetrating Radar (GPR) and Falling Weight Deflectometer (FWD) are essential to identify problematic areas [1]. Field tests such as Dynamic Cone Penetration (DCP), coring, trenching, and laboratory testing are also conducted, as needed, to validate/confirm the initial hypothesis. From time to time, the results from forensic studies have been used to either validate or modify the existing design plan, and to resolve disputes involving construction claims.

The GPR, FWD, and DCP employed in this study are shown in Fig. 1. The GPR is normally operated at highway speed and does not require traffic control. This is very important for urban areas in particular where lane closures are difficult and costly. The non-contact GPR unit presented in this paper is equipped with a 1.0 GHz antenna which tests approximately the top 610mm of the pavement structure. Several thousand GPR traces are collected for a typical GPR project. In order to conveniently display this information, a color-coding scheme is used to convert the raw traces into a subsurface image of the pavement structure. To achieve this, the Texas Transportation Institute developed the COLORMAP signal processing package [2], which has been used extensively in Texas for processing non-contact GPR data. A color coding system is included in COLORMAP to graphically display subsurface conditions so that defects can readily be identified. An example of this for thick hot mix pavements is shown in Figure 2. Fig. 2A shows the GPR image for a well-compacted pavement with no defects. The individual GPR reflection shows two reflections from the top and bottom of the asphalt layer. When this is transformed into a color display the surface of the pavement is the red line at the top of the figure, the faint yellow reflection at a depth of 400mm to 425mm (16 to 17 inches) is the top of the base layer. The depth scale is on the right and the distance scale is on the X-axis. This should be contrasted with the upper figure where numerous strong reflections (Blue and red areas) are identified within the pavement layer, these are found to areas of poor compaction at layer interfaces. GPR has become very useful for detecting defects within asphalt layers whether
they are construction related compaction problems or are of subsequent deterioration such as stripping.

Three TxDOT projects were selected in this study to demonstrate the application of GPR, FWD, and DCP for pavement forensic evaluation. Two pavements were showing localized problems. The responsible TxDOT districts were charged with developing pavement rehabilitation plans. In this process, it is critical to identify the root causes of the problems. A non-contact high speed GPR survey (in coordination with auguring and coring) can rapidly provide a comprehensive evaluation of subsurface conditions along the entire project. The third project was a detour that carried interstate highway traffic and experienced premature distress after being opened to traffic for approximately two months. The detour had been designed to carry two years of interstate highway traffic. The forensic results were used to resolve a construction claim that involved approximately $600,000 of repair and traffic control costs. Also, the local district personnel urgently wanted to identify the appropriate remedy that would minimize or eliminate the maintenance activities that would significantly disrupt the traffic flow. The techniques demonstrated in this study are widely used within TxDOT and are applicable to a wide range of flexible pavement rehabilitation and forensic studies.

**GROUND PENETRATING RADAR (GPR)**

TxDOT owns five GPR’s that can be operated at highway speeds to collect data for forensic or rehabilitation projects. The main functions of GPR are to measure thickness, identify surface segregation or poor construction joints, locate subsurface defects (stripping) and identify wet area in the lower pavement layers. The specifics of GPR measurements and the associated pavement properties are given as follows:

- GPR works on the principle that part of the incident energy will be reflected when the wave encounters a layer of different electrical properties (dielectric). In pavements this could be a transition from one layer to the next or the occurrence of a discontinuity within one layer (stripping). When testing a homogeneous defect free AC layer the GPR signal will give a reflection at the top and a reflection from the bottom of the layer as shown in Fig. 2A. When there is a defect within the surface AC layer, an additional reflection will be observed between the reflections from the surface and top of base. Common defects in Texas are stripping (separation of asphalt binder and aggregate) and delamination of
the HMA layers. For example, Fig. 2B indicates compaction problems in the HMA layer as evidenced by additional reflections. If the defects have not been identified and removed, they often cause early distress after an overlay. As discussed below Projects 1 (US281) and 2 (US69), as shown in Figs. 3 and Fig. 7, were found to have severe problems in the lower AC layers which were not addressed in earlier rehab efforts.

- When there are changes in either the density or moisture content of the surface layer, there will be a significant change in the surface reflection (or dielectric). A typical surface dielectric plot is presented at the bottom of Fig. 2B. The variation in surface reflection has been frequently used to (1) check for segregation within a new HMA surface layer; and (2) test the quality of longitudinal construction joints. Any significant change in the surface reflection normally indicates poor uniformity of AC density. Sebesta and Scullion [3] found that for dense graded mixes, the surface dielectric should not vary by more than 0.4. In particular reductions in dielectric of more than 0.4 from the mean would identify areas of high air voids. The Project 3 section (Fig. 8) illustrates the poor uniformity in surface AC density that allows water to enter the base layer. The surface dielectric of dry pavements is associated with layer density. Large drops in dielectric indicate areas of low density, and that the longitudinal joints are highly permeable.

- When the thickness of a pavement layer increase, the time interval between layer interfaces also increases. Therefore, by collecting data along a highway, the layer thickness can be determined and any major changes in structure can be identified. Since pavement is built in layers with different properties (AC, granular base, subgrade, etc), strong reflections from those layers allow engineers to determine, among other properties, their thickness. An aged pavement normally accumulates patches and repairs that have not been well documented. The GPR is able to locate the section break and thickness changes. A typical processed GPR image is presented in Fig. 2B with thickness labeled on the right hand side (in inches).

- As the moisture content of the base layer increases, the amplitude of reflection (or dielectric) from the top of the base increases. High moisture content in the base layer is indicative of a problematic area. It is often associated with (1) water infiltration through
a porous AC layer or longitudinal joints; or (2) a moisture susceptible base that wicks moisture up from the subgrade. When base dielectric exceeds 16, the base is almost always saturated [4].

PROJECT 1: REHABILITATING A PAVEMENT WITH STRIPPING AC AND WEAK/WET BASE

The distress condition survey indicated a near perfect score in 1997, the pavement deteriorated and received a thin overlay in 2003. However, by 2005, conditions were nearly back to the 2002 levels, as shown in Fig. 4A. It means the rehab done in 2003 was not effective. The section currently has extensive in cracking and rutting making it a rehabilitation candidate.

US281 is a major corridor for the District and the length of the project presented in this study is approximately 18 miles. The typical section consists of approximately 150mm (6 inches) of AC and 300mm (12 inches) of granular caliche base. In early 2006, the District planned to remove the top 150mm AC and replace with 150mm of new AC. The District requested a forensic study to verify this rehab design. Extensive GPR, FWD, DCP and lab testing were performed. The GPR found many locations with stripping problems at various depths. Cores were taken to verify the stripping, as shown in Fig. 3. The cores did confirm that there are stripping and numerous seal coats and overlays had been applied over the years.

FWD tests indicated that numerous locations have very high deflections (50-60 mils), as shown in Fig. 4B. Note that the deflections for load-zoned roads are normally in the range of 30 mils or higher. Thus, the FWD results indicated that the pavement was structurally inadequate. The areas with high deflections were found to have a stripping problem when the corresponding GPR results were studied. Based on the GPR and FWD analyses, US281 sections have stripping problems in the AC, and the bases are wet and very weak. The low stiffness (10-20ksi) of bases was further verified by 20 Dynamic Cone Penetrometer (DCP) tests. The DCP locations were selected based on the GPR and FWD results. The base stiffness of US281 was only about a third of a typical Class I flexible base used in Texas. Class I material is the best quality base material with the following properties:

(1) It exceeds all requirements for the Texas Triaxial Class 1
(2) It exhibits a compressive strength of 315kPa and 1206kPa at 0 and 103kPa, confining pressure, respectively. The Texas Construction Specification [5] provides additional information about Class 1 flexible base material.

Trenching was performed to retrieve AC and base samples. The quality of the base material was very poor, by visual evaluation. The material is shown in Fig. 5. The larger aggregate was soft and some could be broken by hand.

**Rehabilitation**

The three main problems for US281 are (1) stripping within AC layers (2) wet base (3) very low stiffness of asphalt and base layers. On review of the field data he District prefers to address the stripping and low stiffness problems rather than covering them up with an overlay. The original rehab design of replacing the top 150 mm of AC will not address the wet and low stiffness base problem. To have a long-term solution, the base needs to be removed or reworked and strengthened. Full-Depth Recycling (FDR) has performed well in Texas [6] and is recommended. A 50/50 blend of Reclaimed Asphalt Pavement (RAP) and base has performed well in past experience. This was the ratio adopted in the lab design. The goal of the current FDR design is to find the suitable stabilizer and its optimum concentration. Unconfined Compressive Strength (UCS), and moisture susceptibility are all evaluated. This design was based on the following criteria:

- 7-Day cure, dry unconfined compressive strength (UCS) ≥ 2067 MPa.
- 7-Day cure, submerging for 4 hours UCS ≥ 80% of 7-Day cured dry UCS
- Final surface dielectric constant after the 10-day Tube Suction Test (TST) < 16
- UCS after the Tube Suction Test ≥ 80% of 7-Day cured dry UCS

Based on the results obtained for the 50/50 blend of caliche base and RAP, the ideal design was found to be a treatment with 3% cement by dry weight. Although the 7-day dry UCS fell just short of the recommended strength of 300psi, the UCS obtained after the completion of the TST shows that strength continues to increase. Additional information for TST can be found in Barbu and Scullion [7].
It was found that the 7-day dry UCS with 4% cement was lower than with 3% cement thus three percent cement is the optimum content. Normally, higher cement content would yield higher UCS but not in this case. This was attributed to extensive cracking in the lab samples molded and cured at the higher cement content. Meeting the 2067 MPa (300 psi) requirement has usually not been a problem in past experience. The low quality caliche base material is the main contributing factor for the 2067 MPa criteria not being met. So for pavement thickness design, a conservative strength of 551 MPa was used for the FDR layer instead of the usual 1034 MPa.

The projected 20yr traffic is 11 million 18-kip ESALs. The following layer moduli were used in the design: AC=3445 MPa, FDR=551 MPa, base=103 MPa and subgrade=103 MPa. The final design calls for 300mm of FDR with 3% cement and 175mm of new AC. The 300mm FDR consists of 150mm of AC and 150mm of base. The cost per mile of this option (on a 38ft wide pavement) is $621,960. This consists of a 4ft inside shoulder, two 12ft travel lanes and a 10ft outside shoulder. The price is based on a state average over last 12 months. Although there are different rehab options available, FDR is believed to be the most cost effective treatment to address all of the pavement deficiencies identified in the field investigation.

The original District design would have cost $463,085 (to remove 150mm of AC and replace it with 150mm of new AC) and would have yielded an much under-designed pavement. It was found that when the poor quality caliche base left in place without any treatment would require 240mm of AC to meet the design requirement.

**PROJECT 2 - CHRONIC PAVEMENT DISTRESS AND ITS REHABILITATION**

The repeated pavement distresses that occurred on US69 prompted the District to initiate a forensic study to identify the root cause(s) of the recurring problem, and to develop an optimal rehabilitation strategy. The District had performed two mill-and-overlay treatments on this pavement in 1996 and 2001. By 2004, severe alligator cracking reappeared on the same route, as shown in Fig. 6. Both earlier treatments had consisted of removing the top 50mm of AC and overlaying with 50mm of AC. The existing section 2067 MPa consists of 115 to 200mm AC (depending on the location), 280mm sand shell base, 200mm of select material, and 150mm of lime treated subgrade. The projected 20-year traffic (2004-2024) is approximately 20 million ESALs.
GPR, DCP, FWD, coring and trenching were performed to determine the cause of the distress and to characterize the pavement conditions for rehab strategy selection. Through GPR, coring, DCP, and FWD testing, it was found that the chronic pavement distress on US69 has two major causes. First is the presence of a porous layer at 50-100mm depth that has caused debonding. Fig. 7A shows the porous layer at a depth of 75mm from the surface and also shows the debonding at the layer interface. In the GPR survey, high reflections were observed at a depth of 75 to 100mm. These are shown in Fig. 7B as a strong blue/red reflection. There is a strong correlation between the surface distresses and presence of the porous layer detected by the GPR; that is, when there is the porous layer, there is surface distress. The second problem identified was that most pavement sections were structurally inadequate. Many locations were found to have high FWD deflections (for this structure, greater than 30 mils of deflection at a 40kN or 9000lb load). This structural problem was clear on several bridge approaches that had thinner structures, higher deflections and many areas of surface cracking. DCP tests showed that the pavement did not have an effective foundation layer. In these locations, it appeared that lime had leached out of the stabilized subgrade. The loss in foundation supports the localized high surface deflections.

Based on the findings from this study, the District has removed the top 100mm to ensure that the porous layer was completely removed from all locations. Thicker AC (225mm) was placed on areas where have higher FWD deflections (e.g. near bridge approaches). To meet the height requirement, 175mm of existing pavement was milled in those thicker AC section. The pavement was rehabilitated in 2005 and has had good performance so far. A condition survey will be done periodically to monitor the performance.

**PROJECT 3 - FORENSIC STUDY TO RESOLVE CONSTRUCTION DISPUTE**

The main objective of this investigation was to determine (1) the cause(s) of pavement failure on the temporary detour of an interstate highway (2) to identify a short-term remedy that would carry traffic for two years. The forensic results were used to also resolve a construction claim that involved approximately $600,000 in repairs and traffic control costs. The temporary detour was constructed and opened to traffic in May 2002. The detour was intended to carry two years of a major interstate traffic, then be removed and replaced by a permanent structure. After approximately 2 months of traffic on the temporary detour, several repairs were needed to fix
premature failures. The failures included deep ruts and potholes. The original pavement structure consisted of 50mm Type C AC, 100mm Type B AC, 350mm of Class 1 flexible base, and Class B embankment. The definitions of Type C, Type B, Class 1 flexible base, and Class B embankment can be found in reference [5]. Numerous full-depth repairs were made, and more failures were occurring almost on a weekly basis. Although distress was observed on inner and outer lanes, the most severe distress was observed on the center lanes, as most truck traffic traveled in the center lanes. Concrete barriers were placed on both sides of each 3-lane set, which tends to cause motorists to prefer the center lane. Nondestructive testing and field sampling was conducted in Dec. 2002 (when the failures first occurred to identify the causes and recommend a repair strategy) and again in March 2004 (after repairs had been made just prior to the structure was replaced). Based on the preliminary results from the Dec. 2002 testing, recommendations were made and an emergency repair was done on the middle lanes, and then a seal coat and 50mm overlay was placed over the entire section. The emergency repair consisted of removing the top 275mm from the middle lanes and placing 225mm of Type A AC and 50mm of Type C AC. Thus, the final AC thickness (after the 50mm overlay) was approximately 325mm for the middle lanes, and 200mm for inside and outside lanes. After the Dec. 2002 seal coat and 50mm overlay, no additional failures occurred, and the pavement performed very well until the pavement structure was replaced in 2004. It means the short-term remedy was working and the deterioration has been curtailed and the pavement life was extended.

One of main functions for the seal coat and 50mm overlay was to prevent water infiltration through the longitudinal joints between lanes and to increase structural capacity, as will be discussed later.

GPR tests were conducted in Dec. 2002 (just before the repair) to determine the uniformity of the HMA layer across the pavement, as shown in Fig. 8. Each time the GPR crossed a longitudinal construction joint, a marker was placed in the data. The surface dielectric plot at the bottom of the figure is highly irregular. Almost all longitudinal construction joints are associated with large drops in dielectric that indicate areas of low density. In some areas, drops on the order of 1.5 are noted. A 1-unit drop in dielectric is typically associated with an 8 to 10% increase in air voids (Sebesta and Scullion, 2002).
It is likely that the longitudinal joints in the top HMA layer are highly permeable. From NCHRP 9-27, Brown et al [8] found that the in-place air voids content as the most significant factor impacting permeability of HMA mixtures, followed by coarse aggregate ratio and voids in mineral aggregate. The Type B bottom layer was found to be problematic, as it is anywhere from 10 to 30 times more permeable than the typical dense grade mixes. The permeability of Type B mix was found to be around $2.5 \times 10^{-3} \text{ cm/sec}$ that is close to a porous friction course (PFC). Note that Florida DOT has a limit of $125 \times 10^{-5} \text{ cm/sec}$ [9] on their surface mix.

It is believed that moisture entered this pavement primarily through poorly compacted AC layers and longitudinal joints [10]. Sixteen cores taken in March 2004 from the original Type B and C layers confirmed that the majority of cores have air voids exceeding 9% (refer to Fig. 9B). The lower Type B layer was also badly segregated and delaminated from the upper Type C layer at some locations, as shown in Fig. 9A.

**Base Moisture Susceptibility Testing**

Since it is believed that water was entering the base through the porous AC layers and longitudinal joints, moisture susceptibility tests were also performed on the base samples retrieved from the job site, as shown in Fig. 10A. Tube suction (or dielectric) testing has been used successfully to determine the moisture susceptibility of pavement materials [4]. High dielectric values indicate a high potential for moisture susceptibility and likelihood of reduced strength and stiffness. Dielectric and modulus testing was performed by TxDOT on 9 samples to evaluate the behavior of the base material under different degrees of saturation. All 9 samples were molded at optimum moisture and dried back in a 60°C oven for 48 hours. The optimal moisture content determined by standard TxDOT procedure Tex 113-E is 7.7%. The dried back specimens were then capillary soaked for 10 days, as shown in Fig. 10B. The average results from the 9 samples are illustrated in Fig. 10C. Each day in capillary soak, the dielectric and modulus were measured. The free-free column test, a nondestructive/non-intrusive test, was performed to determine the modulus values. Within the first day of soaking, the average modulus was reduced by 60% of its initial value. Over a 10 day period, the average modulus reduced by 95%. The dielectric value increased in line with the reduction in modulus. After 10 days of soaking, the dielectric value at the top of the sample was 17.1 ksi, which means this material can
be classified as highly susceptible to moisture. When the dielectric value is higher than 15 after the soak period, the base material is highly susceptible to moisture.

**Triaxial Testing**

Triaxial tests were conducted to determine the resistance of base material to permanent deformation under repetitive loading at three different moisture levels (optimum, optimum -1%, optimum +1%). A haversine cyclic stress of 0.21MPa with a confining pressure of 0.10MPa was applied to the sample. Test samples were compacted to maximum density at three different moisture contents: 6.7, 7.7 and 8.7%, with 7.7% being the optimal moisture content. The test results indicated that the presence of moisture has a large impact on the resistance of this base material to permanent deformation. The traditional failure level is set at 2% strain. The sample failed after 10 load cycles at 1% above optimum moisture and at 850 cycles at optimum moisture content. At 1% below optimal moisture content, the sample would not fail under these stress conditions. Clearly, if this base is kept dry it will resist permanent deformation, but if water gets to it, then the strength will be reduced drastically. Permanent deformation of the base under saturated conditions is the main cause of failure of this pavement. Although attempts were made to conduct the resilient modulus tests at optimum moisture content and at optimum +/- 1%, it was impossible to get data at the +1% moisture content. The sample was too weak to endure the resilient modulus test.

In addition, it was determined by the Soil and Aggregate Branch of TxDOT that the base has a Texas Triaxial class of 2.3 and the compressive strengths were 0.28MPa and 1.13MPa at confining pressures of 0kPa and 103kPa, respectively.

**FWD-Surface Deflection**

Falling Weight Deflectometer (FWD) tests were conducted on each southbound and northbound lane for a total of six test lanes. Note that only four test lanes (two lanes in each direction) were conducted in Dec. 2002. A comparison of 40kN (9000lb) normalized deflections is presented in Fig. 11A. The deflection denoted “W1” is the maximum deflection that occurs at the center of the load. The test pavement temperatures were approximately 10°C and 27°C for Dec. 2002 and March 2004, respectively. No temperature correction factors were applied to the data in Fig. 11A. The W1 deflections were generally higher in 2002 than 2004. The low deflections in 2004 indicate a stronger pavement structure that correlated well with the lack of distress. It is believed
that two main factors contributed to the low deflection in 2004: (1) the seal coat and 50mm overlay was able to prevent water from entering the pavement system (2) the repair in the middle lane and 50mm overlay over the entire section increased the structural capacity. A linear multilayer analysis program, was used to compute the reduction of deflections when 50mm of AC was added in 2002 during the repair. As shown in Fig. 11A, the contribution of 50mm is labeled 2002+50mm. The ability to prevent water from entering the base layers played an important role in lowering the deflections, and reaffirmed the lab moisture susceptibility results provided above.

**Pavement Design Check**

Efforts were made to determine if this temporary detour was originally under-designed. It was designed to accommodate an estimated traffic of 49 million ESALs (2002-2022) per direction. Although a lane distribution factor could be used in this case, it was assumed that the design lane would carry the 49 million ESALs alone. Thus, the design was very conservative. Using existing TxDOT design procedures and the program FPS19 [11], the original pavement structure of 150mm of AC and 350mm of Class 1 base was found to be adequate to carry the traffic loads for the anticipated 2-year design life of this detour. The layer moduli inputs were: (1) AC = 3447MPa (2) Base = 345MPa (3) Subgrade = 110Mpa. Note that 345MPa is a conservative estimate for Class 1 base. Values around 483MPa are normally used by TxDOT engineers. 110MPa is also a conservative value for the select fill material, as 152MPa is normally used in this district.

Questions were raised about the actual traffic vs. the estimated traffic used in the design. There were traffic counters installed just before (station H92) and after (station H94) the project limits. A comparison between measured and average 20-yr estimated traffic is presented in Fig. 11B. In view of Fig. 11B, the averaged estimated traffic was in the approximate range of the actual measurements.

It was concluded that the design thickness was sufficient to carry traffic load, and the main cause of the premature failure was attributed to material and construction practices. The surface layer was not adequately compacted and it let moisture into the moisture susceptible base layer. The base material used on this project did not meet the Triaxial Class 1 requirement; it was tested to be a Class 2.3 material. The base material was found to be highly moisture susceptible, it did not
meet TxDOT’s compressive strength requirements, when subjected to capillary soaking. In addition, the repetitive triaxial test results revealed that the stiffness and load carrying capability and resistance to permanent deformation became inadequate when the base material was exposed to moisture.

Results and analyses have been presented to the contractor. Their main concern was the base sample collection procedure. In particular, if the base material is collected from a stockpile, the granular aggregates can easily become segregated and the results would be skewed. Fig. 10A shows how the base materials were retrieved and sampled directly from the base layer. The finding from this study was used to resolve the dispute over the cost of repairs and traffic control.

CONCLUSIONS

Verifying the presence and extent of subsurface defects is a critical part of any forensics investigation. A combination of field non-destructive testing, trenching, coring, lab testing, and thorough review of construction records and rehabilitation history enable engineers to determine the root causes of the pavement failures. Observations from testing and evaluating the three field projects are given as follows:

(1) The original overlay strategies used on projects 1 and 2 were not effective because the projects required further rehabilitation in 3-5 yrs. The rehab proposed by the District for project 1 did not consider the base condition and would have yielded an under-designed pavement. The extent of stripping and porosity that caused delamination for projects 1 and 2 were determined by GPR and verified by core samples. This combination (GPR+coring) was used to map the entire project. GPR is an excellent tool for detecting buried defects in the upper layers of flexible pavements.

(2) GPR, FWD, DCP and soil samples all gave indications that the caliche base for project 1 was wet, and the stiffness was only about 1/3 of a typical flexible base in Texas. FWD data demonstrates that the pavement structure for projects 1 and 2 are inadequate, so the rehabilitation strategies have been aimed at strengthening the pavement sections.

(3) On project 3, GPR, lab density, and permeability test results gave indications that moisture entered the base through the poorly-compacted AC layer and longitudinal joints. This accelerated structural deterioration, as the base material was found to be highly
moisture-susceptible. The base did not meet TxDOT’s compressive strength requirements when subjected to capillary soaking. The repetitive triaxial test results revealed that the stiffness and load carrying capability and resistance to permanent deformation became inadequate when the base materials were exposed to moisture. After the Dec. 2002 failure investigation it was recommended that localized base repairs be made followed by a seal coat and 50mm overlay. After these repairs no additional failures occurred, and the pavement performed very well until it was replaced in 2004. The short-term remedy curtailed the deterioration mainly by preventing water from entering the base layer.

ACKNOWLEDGEMENTS

The supports and assistances from John Bilyeu, Susan Chu, Pete Stricker, Don Nyland, Miguel Arellano, Richard Izzo, and Dr. Mike Murphy of Texas Department of Transportation are much appreciated.

REFERENCES


Fig. 1 -- Tools Used in Forensic and Rehab Projects: Ground Penetrating Radar (GPR), Falling Weight Deflectometer (FWD), and Dynamic Cone Penetration (DCP)
Fig. 2 -- Typical Processed GPR Images from Thick Asphalt pavements. (A) 450 mm (18 inch) AC Without Problem in AC Layers, and (B) 500mm (20 inch) AC with Delamination and High Air Void Problem in AC Layers
Fig. 3  -- Stripping Found in Many Locations of US281. Cores Taken to Verify the Stripping
**Fig. 4 -- Pavement Conditions for US281 (A) Distress Scores for US281 Southbound Lane. There is a treatment in 2003. However, by 2005, conditions are back to near the 2002 levels. (B) High FWD deflections correspond to the stripping as found in GPR results**
Fig. 5 -- Caliche Base and FDR Design (A) Poor Quality Base with Large Particles that can be Broken by Hand Easily (B) Sample Condition at 4% cement (Cracking)
Fig. 6 -- Pavement Condition on US69 that Has Chronic Distress.
Fig. 7 -- Core and GPR Conditions (A) Trench Wall and Coring Sidewall Revealing Layer Separation Caused by the Porous Layer (US69) (B) Strong Correlation Between the Surface Distress and Porous Layer Detected by GPR on US69
Fig. 8 -- GPR Results Indicated Delamination Between Layers and the Longitudinal Joints Are Highly Permeable
Fig. 9 -- Core Condition and Air Void Test Results (A) Delamination and Voids Observed on the Cores (B) More than 50% of Cores Exceed 9% Air Void that Allows in Specification
Fig. 10 -- Base Characterization (A) Base Sample Collection (2) Base Moisture Susceptibility Tests with Capillary Soak as Water is Wicking Up the Specimens (C) Modulus and Dielectric Values under Capillary Soak
Fig. 11 -- Site Characterizations (A) FWD W1 (Maximum) Deflections (B) Estimated vs. Measured Traffic at Two Adjacent Sites