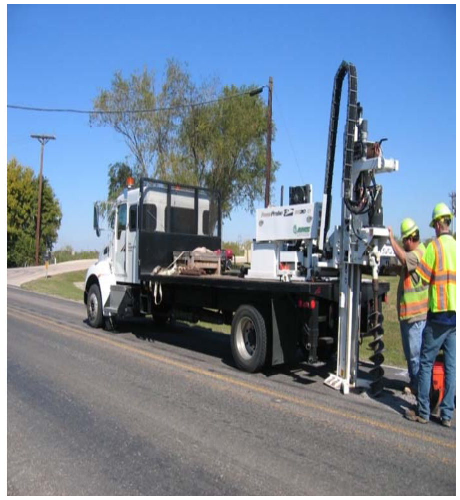

Full Depth Reclamation

Workshop Materials Study 0-6271-P2

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CHAPTER 1 – INTRODUCTION TO FULL DEPTH RECLAMATION

Notes:

When this chapter is over you will be able to:

- Understand the FDR process.
- Be familiar with the steps involved in conducting a comprehensive FDR design.

Section 1.1 Overview of the FDR Process

Rehabilitating an old pavement by pulverizing and stabilizing the existing pavement is a process referred to as Full Depth Reclamation (FDR). This process shows great potential as an economical rehabilitation alternative that provides deep structural benefit, conserves highway construction raw materials, and quickly returns the section to service. The stabilized layer becomes either the base or subbase of the new pavement structure. In the early 1990s, the Bryan and Lubbock Districts constructed their first few projects on low volume roadways. Their initial experiences were positive and both Districts have now recycled close to 1,000 miles of mostly low volume roadways. Although widely used in several Districts there are others that are just getting started with the FDR process. The purpose of this training school is to identify all the key steps in the design, construction, and monitoring of the FDR process so that District just getting started can build upon the lessons learned from earlier projects.

The FDR process generally consists of reclaiming the existing structure by pulverizing and mixing the surface and base materials together as shown in Figure 1.1, applying a stabilizing agent (lime, fly ash, cement, asphalt emulsion, or some combination) then compacting the mixture and applying a riding surface.

Notes:

Figure 1.1. FDR Pulverization Process.

The procedure can be highly cost effective if executed properly. However, lack of guidance in the overall design and construction process, including formulating a mixture design of the reclaimed materials, controlling the construction process, performing quality assurance of the in-place product, and bonding the surface layer to the finished base have led to construction delays and poor performance on several projects. Designing and constructing good performing FDR projects is challenging for several reasons; including:

- The existing pavement hot mix thickness is often very variable, especially if substantial maintenance has been performed.
- Problems have been encountered with pavements build on expansive clays (most of east Texas), edge drying and trees down the sides of roadways are a problem when stabilized layers are placed over them.
- Old base materials are often contaminated and sometimes weak.
- Many low volume roadways are narrow and widening must be part of the FDR process.
- Often the process is conducted on 2 lane highways so traffic handling is a major concern.

The vast majority of the FDR projects in Texas are performing well. But several instances of poor performance have been documented, as shown in Figure 1.2, where a re-occurrence of distresses, particularly longitudinal cracking, has been found not long after construction. The cause and remedy of these performance problems will be discussed during this class.


Figure 1.2. Severe Longitudinal Cracking 12 Months after Construction on an FDR Project.

Notes:

Section 1.2 Key Steps in the FDR Process

The steps recommended in the FDR process are described below:

Step 1: Evaluate Project Soil Conditions and Maintenance History (pre site visit)

- Obtain and review plans for preliminary information on the existing pavement structure, and discuss maintenance history with district personnel.
- Use the Web Soil Survey at http://websoilsurvey.nrcs.usda.gov/app/ to review the subgrade soil types likely to be encountered.

Notes:

Step 2: Characterize Existing Pavement Structure with NDT (site visit)

- Evaluate visually the current pavement condition including types of distresses and likely causes of distresses. Photograph current major problems.
- Evaluate the existing pavement structure, and measure the in-situ materials properties, with upfront non-destructive test surveys including ground-penetrating radar (GPR) and falling weight deflectometer (FWD) surveys.
- Note existing drainage problems, including settling culverts.
- Analyze the GPR and FWD surveys to identify section breaks in the existing pavement and determine the in-situ subgrade modulus values for pavement design purposes.

Figure 1.3. GPR to be Conducted on All Candidate FDR Projects.

Step 3: Verify Pavement Structure and Obtain Material Samples

- Use the GPR and FWD survey analysis as guidance to select focused verification and sampling locations. Sub-divide project at major changes in structure. Select coring and sampling locations.
- Collect material samples to verify the pavement structure down to the subgrade.

- Use an auger or other means to excavate existing materials that will be used in the laboratory mixture design down to the depth of reclamation typically 10 inches.

Notes:

Figure 1.4. Use of Augur to Obtain Samples for Lab Design.

Step 4: Perform Mixture Design

- Considering the preferences of the area office and availability and costs of materials, select preliminary treatment options to perform the mixture design.
- Perform strength and tube suction test if considering options with no stabilization.
- For stabilization options, use appropriate TxDOT Test Procedures to select the optimum stabilizer contents. Supplement the standard strength tests with determination of the seismic modulus, performance in the tube suction test, and retained strength test.
- If the surfacing is to be placed directly on the stabilized layer conduct a study to determine the most appropriate prime material to use to minimize bonding problems.

Notes:

Step 5: Pavement Thickness Design

- Review the thickness variability and with the goal of achieving a standard uniform support layer, decide on milling depth and add rock requirement. Segment the project so details can be included in plan set.
- For pavements without an adequate foundation layer use the stabilized layer as that layer and design a flexible base and HMA surfacing.
- For pavements with adequate structure design an HMA layer to handle the predicted traffic loads.
- Using the materials properties measured in the lab and the traffic information, use FPS 19W to perform pavement design and economic evaluations. Perform the Texas Triaxial design check in FPS to make sure the design adequately protects the subgrade.
- Include in the design recommendations any additional considerations such as pavement widening (based on DCP results), geogrid reinforcement, or specialized materials (such as low-fines bases or crack-attenuating mixes) or construction practices (such as microcracking or delayed placement of final surfacing) that may be needed to minimize the risk of recurring problems.

Step 6: Construction Quality Assurance

- Use field sieve analysis to check that proper gradation has been obtained.
- Use a non-nuclear insertion probe, such as a Vertek probe, to check field moisture prior to compaction. A calibration must be developed for each project.
- Determine section lengths to be treated with each stabilizer load (based upon the treatment width, depth, required treatment level, and weight of stabilizer load). Use visual inspection as the first quality check for stabilizer application rate.
- For cement and lime stabilizers if concerns exist about stabilizer variability pull samples for indicator testing using the techniques developed in study 0-6271.
- As a final check either during construction of shortly after, run the FWD to determine if the pavement is being constructed as designed. Obtain a target deflection based on design assumptions. Investigate all areas where deflections are substantially higher than predicted from design. Modify construction plans for subsequent projects.

Figure 1.5. Use of FWD for Strength Verification.

Notes:

CHAPTER 2 – ONLINE EVALUATION OF PROJECT SOILS CONDITIONS

Notes:

When this chapter is over you will be able to:

- Use the Web Soil Survey at http://websoilsurvey.nrcs.usda.gov/app/ to review the subgrade soil types likely to be encountered.
- Understand the significance of soil properties and how they impact FDR decisions.

Section 2.1 Uses of Soil Survey Data

The soils survey is a good starting point for planning the soil sampling in the field. In most FDR projects one key rule is to avoid, if possible, cutting into the high plastic subgrade soils so it soil survey data can provide upfront information on some of the key design decisions to be made. Also as found in earlier studies, special attention needs to be applied to projects where the existing subgrade soil is clay soil with a PI of greater than 35. These locations are problematic during summer drying where longitudinal cracks have occurred.

Table 2.1 provides an overview of what factors are important when reviewing soils data.

Notes:

Table 2.1. Factors for Reviewing Soil Data.

SOIL PROPERTY	CONSEQUENCE
Plasticity index PI > 15	Avoid if at all possible—do not mix soil into base. If unavoidable; consider lime as stabilizer.
Plasticity index PI > 35	Experience has shown that stabilizer layers built on soils with high shrink swell potential can have problems with severe longitudinal cracks. Consideration should be given to incorporated geogrid into potentially problems sections.
Sulfate Contents > 0.8%	Heaving problems have been documented with the use of cement and lime on sulfate rich soils. Follow TxDOT guidelines on dealing with sulfate, avoid incorporating these soils into bases.
Organic Contents > 2%	Permanent stabilization of these soils is difficult to obtain. Avoid using these soils in FDR designs, follow TxDOT guidelines if these soils are to be treated.

Section 2.2 Using Online Website

United States Department of Agriculture Natural Resources Conservation Service (USDA NRCS) Web Soil Survey database. **http://websoilsurvey.nrcs.usda.gov/app/**

Once the web address is accessed the main screen shown at the bottom of Figure 2.1 appears with 4 pull-down menu options.

Notes:

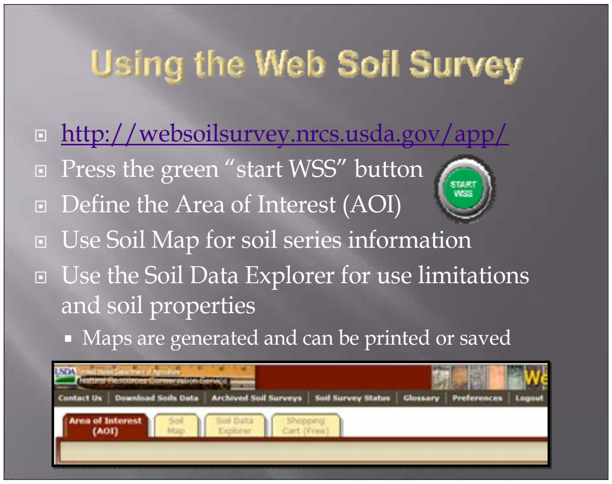

Figure 2.1. Getting Started with the Soil Survey Database.

The use of the soils database required the user to first define an Area of Interest (AOI) this is usually initially a Texas county. Once selected press the "view" button and the county map appears; the user can zoom to the roadway of interest. The AOI buttons are then used to define an area for investigation. The polygon option usually works best.

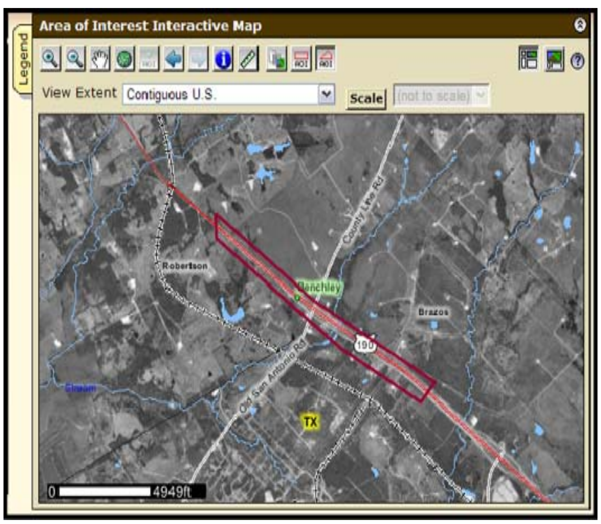

Figure 2.2. Defining an Area of Interest.

The "Soil MAP" pull-down menu is then used to obtain general information about the soil types found in the Area of Interest. Most of the detailed soils information is found under the soils data menu shown below in Figure 2.3. Under the Building Site Development option, select "Local Roads and Streets," then select the soil property to be displayed. One useful feature of this menu is the depth of interest, for example most near surface reclamation jobs will specify 0 to 12 inches of depth; but in the

Notes:

case of a cut on the project depths of 48 to 72 inches can be specified.

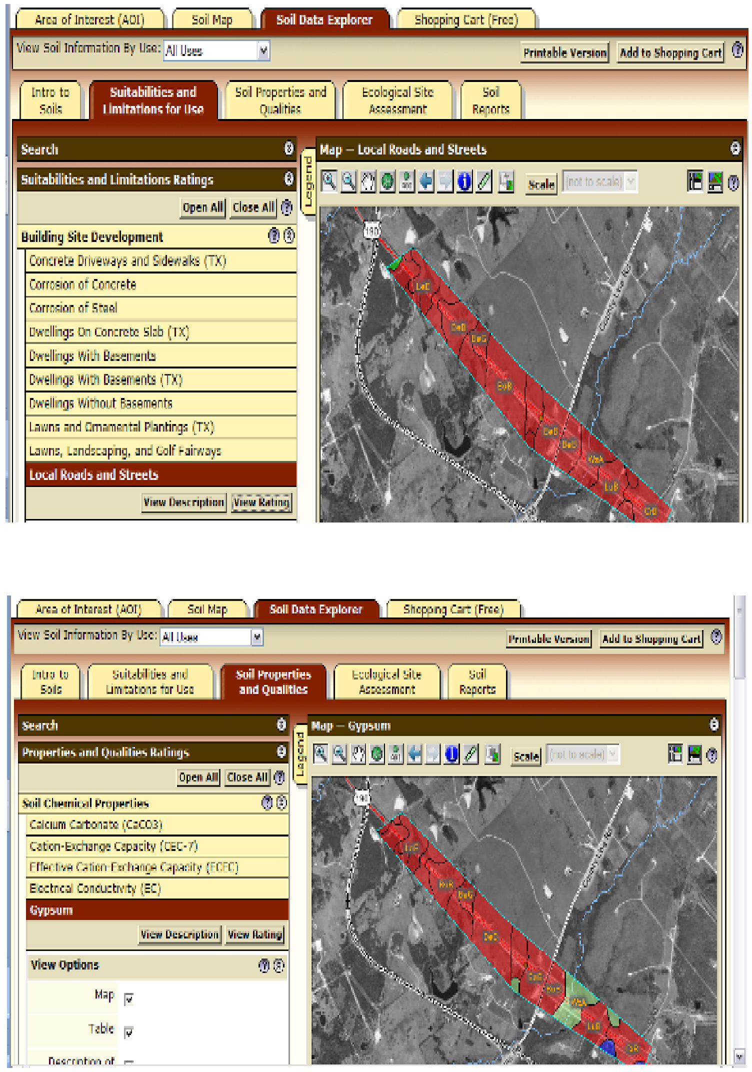

Figure 2.3. Selecting the Soil Properties to be Displayed.

After viewing the data the user must hit the "printable version" button to save the map for printing. The required subtitles can be entered; after pressing "view" and PDF version of the file is generated, which can be saved. An example output is shown in Figure 2.4.

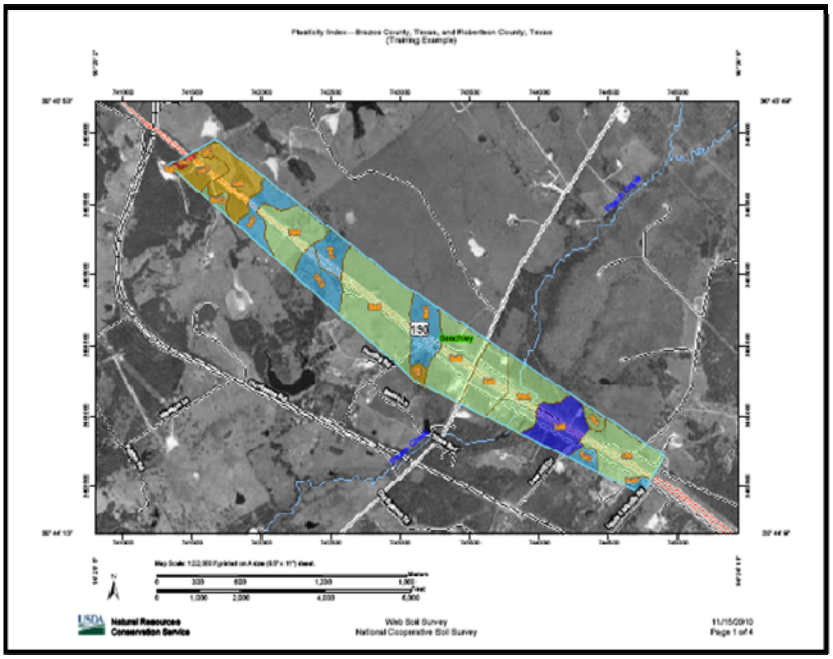

Figure 2.4. Example Map Produced from the Web Soil Survey Site.

Notes:

Section 2.3 Case Study on FM 112 Austin District

In coordination with the Austin District, TTI researchers evaluated FM 112 in Williamson County from US 79 to FM 486. This section of pavement has extensive longitudinal cracking and some faulting occurring. This project would be a good candidate for FDR with widening.

According to NRCS data, the soils in this area are very limited in suitability for roads and streets due to low strength and shrink-swell. Typical surface soil plasticity index values range from 25 to 47, as Figure 2.5 illustrates. Figure 2.5 shows that some pockets of sulfates may also exist, particularly in the middle of the section. In any FDR project the locations of high PI soils are one main interest as these may be areas where performance problems are encountered with longitudinal cracking. As will be described later in these notes, these areas could be considered for additional design attention, some Districts use geogrids on top of the stabilized layer and under the flex base layer in areas of high PI soils to minimize cracking problems with summer drying.

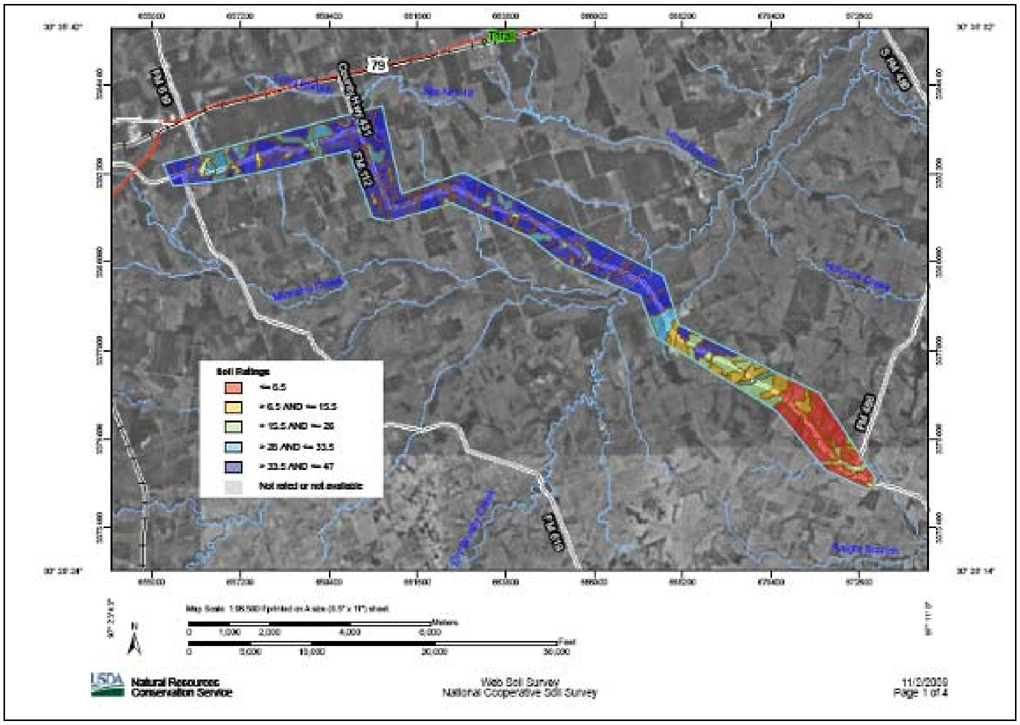

Figure 2.5. Surface Soil Plasticity Index on FM 112.

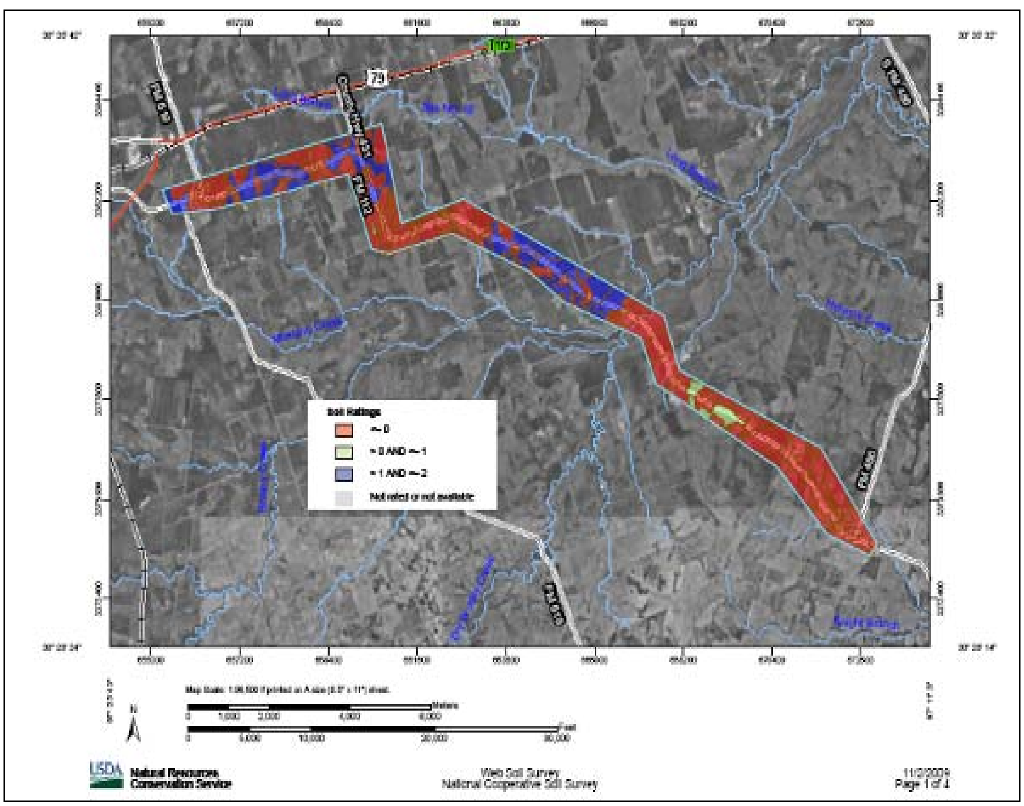

Figure 2.6. Sulfate Soil Content on FM 112.

CHAPTER 3 – CONDITION SURVEY AND NONDESTRUCTIVE TESTING

Notes:

When this chapter is over you should be able to:

- Discuss when a roadway should be considered a candidate for FDR.
- Understand what upfront non-destructive test should be conducted. This includes both Ground-Penetrating Radar (GPR) and Falling Weight Deflectometer (FWD) surveys. One of the major challenges in all FDR projects is to handle the variability that exists in the field. The NDT equipment, especially the GPR, will help substantially in this area.
- Identify other pavement and geometric issues that impact the design of the FDR project.

Section 3.1 What Makes a Good FDR Candidate

In any evaluation the first consideration is to determine if the proposed section is a good candidate for FDR, rather than just a structural HMA overlay. The following are factors involved in making that decision:

- The candidate has a poor support layer as measured by the FWD or Dynamic Cone Penetrometer (DCP).
- The section has multiple load associated failures and is not structurally capable of carrying current traffic.
- The section has severe edge problems and is very narrow.
- The section continues to require extensive maintenance.

Figures 3.1, 3.2, and 3.3 were proposed by TxDOT Districts as potential FDR candidates. Upon investigation each was found to be suitable.

Notes:

Figure 3.1. US 287 Amarillo District.

US 287 has extensive alligator cracking and rutting between 10.5 and 1 inch. The section has very heavy truck traffic. The current pavement has 3 inches of HMA over a thick flexible base. Deflection data indicated that the base was viewed as marginal. US 287 is a good candidate. FDR should be considered as well as a thick structural overlay.

Figure 3.2. FM 1641 Dallas District.

As shown in Figure 3.2 severe rutting and continuing extensive maintenance make FM 1641 a strong candidate for FDR. The HMA layer is 6 to 8 inches thick but the highway still has severe load associated distress. The base and subgrade layers are clearly weak.

This highway most probably needs a foundation layer.

Figure 3.3. FM 429 Dallas District.

Notes:

As shown in Figure 3.3 longitudinal cracks and edge failures are very common in many areas in East Texas. FDR can help these sections as long as it includes some type of widening. The existing roadway is often treated and turned into a subbase foundation layer.

Section 3.2 Use of GPR to Map Subsurface Variability

One of the biggest challenges in successfully designing an FDR project is to be able to document and handle the existing project variability. Several sections in East Texas have received extensive maintenance and have very variable hot mix thicknesses. The following two general guidelines have been used and are recommended for all FDR projects in Texas.

1. Do not allow more than a 50%/50% blend of base to recycled HMA in any stabilized layer. This means that if the reclamation depth is 8 inches then no more than 4 inches of existing HMA can be used.
2. Avoid cutting into the subgrade especially if the subgrade is a plastic clay.

Therefore mapping the existing pavement structure is very important, and TxDOT's existing GPR units can help in this area. Figure 3.4 shows a typical unit. They have an integrated video

Notes:

recording system and processing the data can be performed using either the COLORMAP or PAVECHECK processing packages. Training is available on using these packages. GPR data can be collected and processed for Districts, and the contact person is Phillip Hempel of the Construction Division (512-465-3650).

Figure 3.4. TxDOT's GPR Unit.

All GPR systems send discrete pulses of radar energy into the pavement and capture the reflections from each layer interface within the structure. It is normal to collect between 30 and 50 GPR return signals per second, which for high speed surveys means one trace for every 2 to 3 ft of travel. The captured return signals are often color coded and stacked side by side to provide a profile of subsurface conditions, this is analogous to an "X-Ray" of the pavement structure. The principles of GPR are shown in Figure 3.5 together with the approach of color coding the reflections to make a subsurface image of the pavement structure.

Principles of Ground Penetrating Radar

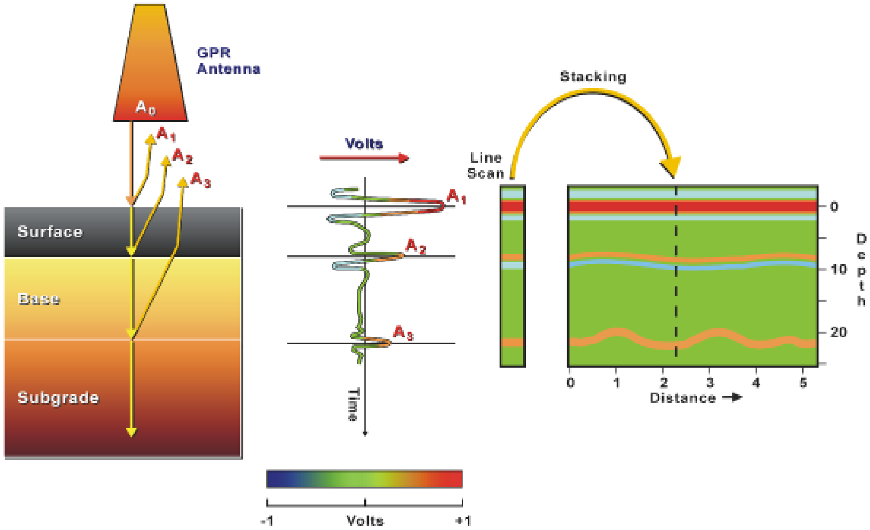

Figure 3.5. Color Coding and Stacking Individual GPR Images.

Notes:

The raw GPR image collection is displayed vertically in the middle of Figure 3.5. The single trace generated is color coded into a line scan using the color scheme in the middle of Figure 3.5. In the current scheme the high positive reflections are colored red and the negatives are colored blue. The green color is used where the reflections are near zero and are of little significance. These individual line scans are stacked so that a display for a length of pavement is developed. Being able to read and interpret these images is critical to effectively using GPR for pavement investigations, to locate section breaks in the pavement structure and to pinpoint the location of subsurface defects.

Figure 3.6 shows an example of a typical GPR display for approximately 1500 ft of a proposed FDR candidate. In all such displays the x axis is distance (in miles and feet) along the section and the y axis is a depth scale in inches, with zero being the surface. The major observation here is that the HMA thickness is very thick at the start and then decreases to around 4 inches for the rest of the section.

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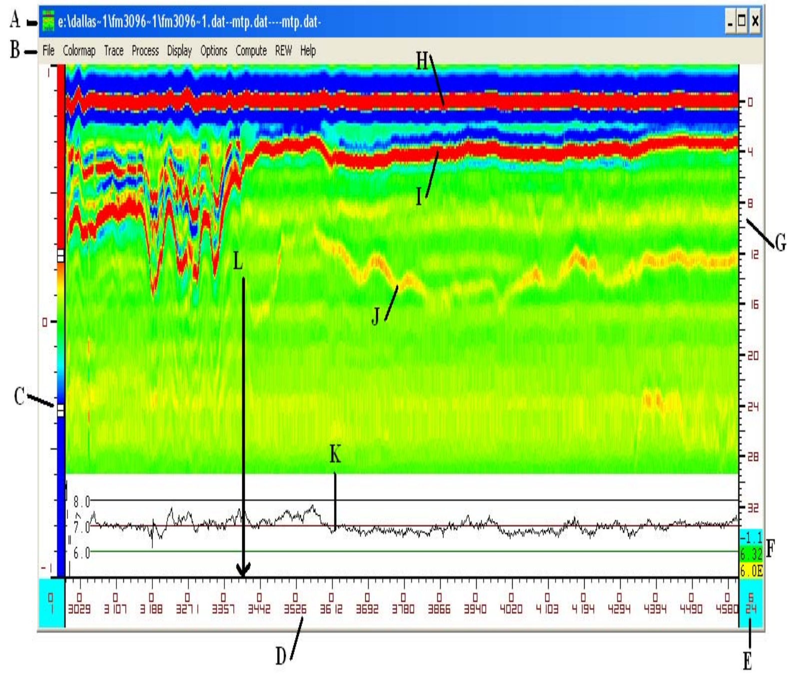

Figure 3.6. Typical Color Coded GPR Data from FDR Project in Austin.

The labels on this figure are as follows:

A. GPR files being used in analysis.
B. Main pull-down menu bar of the software used to process the GPR data.
C. Buttons to define the color coding scheme used to convert the GPR reflections into a color scheme as shown in Figure 3.5.
D. Distance scale in miles and feet.
E. End location of data within the GPR file (6 mile and 24 ft).
F. Depth scale in inches, with the zero (0) being the surface of the pavement.
G. Default dielectric value used to convert the measured time scale into a depth scale, also other calibration factors.
H. Reflection from the surface of the highway. The blue-red-blue is the typical color scheme for the surface reflection.
I. Reflection from the top of the base, the more intense the color the wetter the base layer.
J. Reflection from the bottom of the base top of the subgrade. The stronger (more intense) the reflection the wetter the subgrade material.

Notes:

K. This is the computed surface dielectric for the surface layer. This is a measure of the electric properties of the top 2 inches of the pavement. The amplitude is related to both the moisture content and density of the top layer. Well constructed dry HMA overlays have a very flat line indicating uniform density.

L. This shows the location of a break in the pavement structure at a distance of 0 miles and 3400 ft; the HMA thickness reduces from around 12 inches to 4 inches. Identification of very thick HMA is important in FDR design.

When processing GPR data, the first step is to develop displays such as Figure 3.6 to determine if there are substantially different sections in any FDR project and to identify normal sections where samples should be taken for lab testing. If substantial variations occur then multiple sets of samples may be required for the lab test program.

The most recent GPR processing package is called PAVECHECK, which integrates GPR and video data. Figure 3.7 shows a typical display from an FDR candidate. The color coded GPR images are at the top of the screen and in this case the HMA is very thick and also very variable. The image is displayed is at the location of the vertical line in the color coded display.

Notes:

Figure 3.7. Typical Display from the PAVECHECK GPR Processing System.

The data collected in the GPR survey can be used to select the locations where cores are to be taken. If the pavement section has two distinct pavement structures it can also be used to determine where to take samples for laboratory testing. Figure 3.8 shows a FDR candidate from Dallas where two very distinct pavement structures were found. In this case augur samples were taken from both the thin and thick sections for lab testing.

Notes:

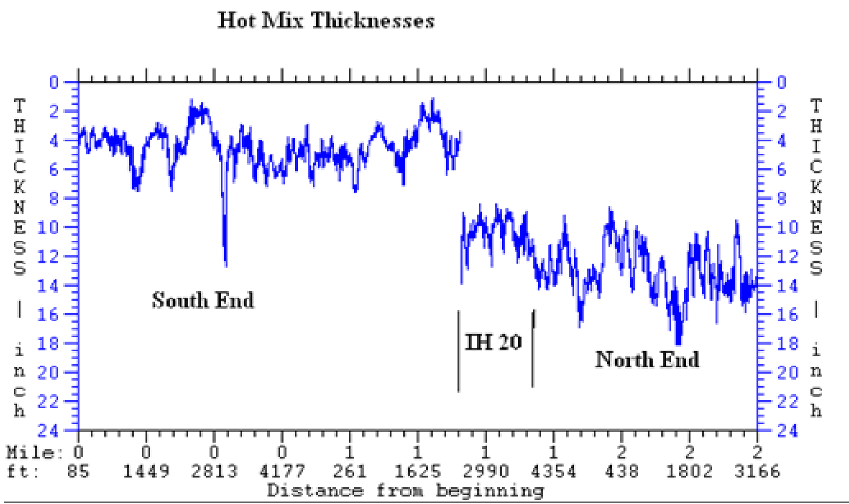

Figure 3.8. GPR Data Showing a Distinct Change in HMA Thickness.

<u>Section 3.3 Using the Falling Weight to Map Subgrade Strength</u>

In any FDR project it is important to get a subgrade modulus to be used in the eventual pavement design. A FWD survey is recommended. Figure 3.9 shows TxDOT's FWD unit, and Figure 3.10 shows typical subgrade modulus data from a FDR candidate. In this case the average subgrade modulus was variable from below 6 ksi to above 10 ksi. Using the MODULUS 6 package the average value can be obtained for the structural design analysis to be described later.

Figure 3.9. One of TxDOT's FWD Units.

Notes:

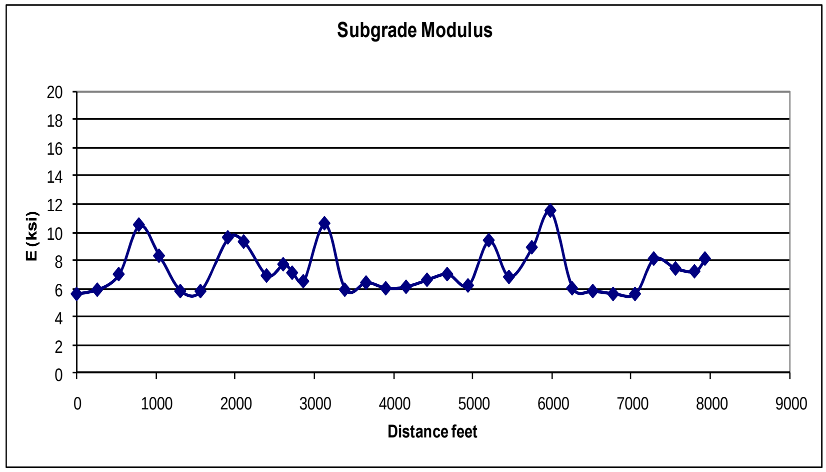

Figure 3.10. Typical Subgrade Modulus Data Computed from FWD Data.

Section 3.4 Identification of Failing Culverts

On several of the FDR candidates tested to date many have areas where the HMA thickness increases substantially in localized areas. These are usually associated with failing or settling culverts. One important feature of any pavement inspection is to identify locations where culvert replacement is required prior to commencing the FDR. GPR can identify locations where substantial thickness of the HMA layer has occurred. Figure 3.11 gives an example below. Many Districts initiate culvert replacements as shown in Figure 3.12 prior to the FDR construction.

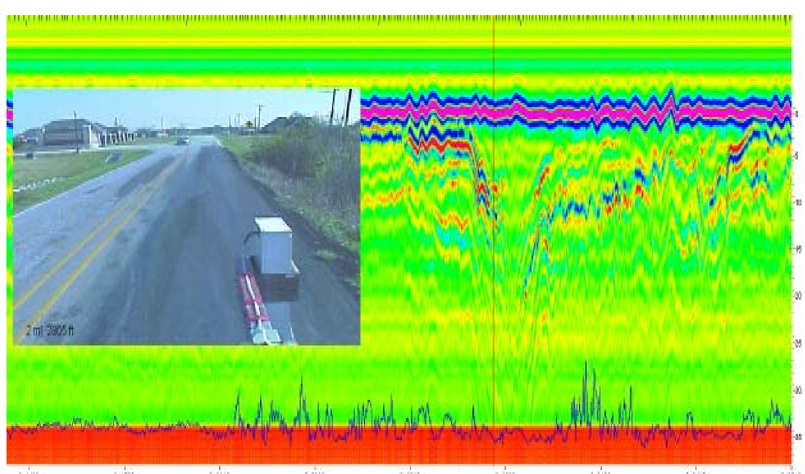

Figure 3.11. Substantial HMA Thickness Increases around Failing Culverts.

Figure 3.12. Culvert Replacement and Widening Prior to FDR.

Notes:

CHAPTER 4 – VERIFICATION CORING AND SAMPLING

Notes:

When this chapter is over you should be able to:

- Understand what field testing is required to verify GPR interpretations.
- Know options needed to take samples for lab testing.
- Understand how to use the DCP to investigate pavement edge failures.

Section 4.1 Thickness Verifications

Verification locations should be selected at locations of typical and non-typical GPR signatures to verify the pavement structure and aid in interpreting the GPR signals. Normally between 2 and 4 locations are selected per project. It is important to verify the thickness of the HMA layers and determine if there are defects in the HMA. In some instances the lower base layers are fine and the surface defects are associated with problems in the HMA layer. In these cases FDR may not be the best strategy for rehabilitating the highway. Figure 4.1 shows verification coring.

Notes:

Figure 4.1. Validation Coring and Location with Severe Moisture Damage to HMA.

- At each verification location perform the following:
    - If sufficient hot mix asphalt (HMA) is present, collect a pavement core to verify the thickness and condition of the HMA.
    - Perform a Dynamic Cone Penetrometer test normally through the core hole; if no cores take drill down to the top of the base and start testing.
    - Send an auger down the core hole to take samples of the base and subgrade; verify base thicknesses.
    - Collect bag samples of the base and subgrade soil for plasticity index, sulfates, and organic tests.

- If the pavement is experiencing major edge stability problems then move approximately 2 to 3 ft off the pavement edge and collect a Dynamic Cone Penetrometer profile to a depth of at least 4 ft to investigate for weak zones or slip planes in the subgrade.

Notes:

Section 4.2 Auguring Samples for Lab Testing

Sampling locations should be selected at locations representative of the typical pavement structure as based on GPR. These locations serve to both verify the pavement structure and generate materials for laboratory testing. Multiple borings take place at sampling locations to generate sufficient quantities of materials for use in laboratory testing. At least one boring at sampling locations should go into the subgrade to fully validate the interpretation of the GPR signal at that location and enable collection of subgrade samples for laboratory testing.

Samples can be taken using a milling machine, field augur, or backhoe. Figure 4.2 shows the field augur operation used by TTI.

Figure 4.2. Sampling Materials for Lab Studies.

- At the sampling location(s) perform the following:
  - If sufficient HMA is present, collect a pavement core to verify the condition of the HMA.
  - Collect a Dynamic Cone Penetrometer profile.
  - If the pavement is experiencing major edge stability problems move approximately 2 to 3 ft off the

Notes:

pavement edge and collect a Dynamic Cone Penetrometer profile to a depth of at least 4 ft to investigate for weak zones or slip planes in the subgrade.

- Collect material samples to verify the pavement structure down to the subgrade.
- Collect subgrade soil samples for plasticity index, sulfates, and organic tests.
- Use an auger to excavate existing materials that will be used in the laboratory mixture design down to the depth of reclamation typically 10 inches. If the project is sub-divided into more than 1 distinct pavement structure than the following sampling requirements should be applied to each one. If the HMA thickness is greater than 4 inches maintain separate samples of recycled asphalt pavement (RAP), flexible base, and subgrade. (For thin pavement structures blend all materials together).
  - Based on district preferences and availability of stabilization agents, most lab tests focus on a cement-based stabilization design. For this series of tests, the amount of material collected for each sampling location should be at least 10 five-gallon buckets of material.
  - An additional 5 five-gallon buckets of material is required to perform a laboratory emulsion-series with two different emulsion levels.
  - If lime or lime-fly ash treatment is being considered, an additional 5 five-gallon buckets of material is required for each level of lime or lime-fly ash (LFA) treatment under consideration.

Testing at TTI has concluded that the augur sample system generates materials with a similar gradation to that obtained under a milling machine. This is demonstrated in Figure 4.3 where a comparison is given of the gradation obtained before construction with the field auger and during actual construction. The differences were found at the large stone size; the large rocks retained on a 1.25 inch sieve were discarded from the augur samples and the two gradation curves were almost identical.

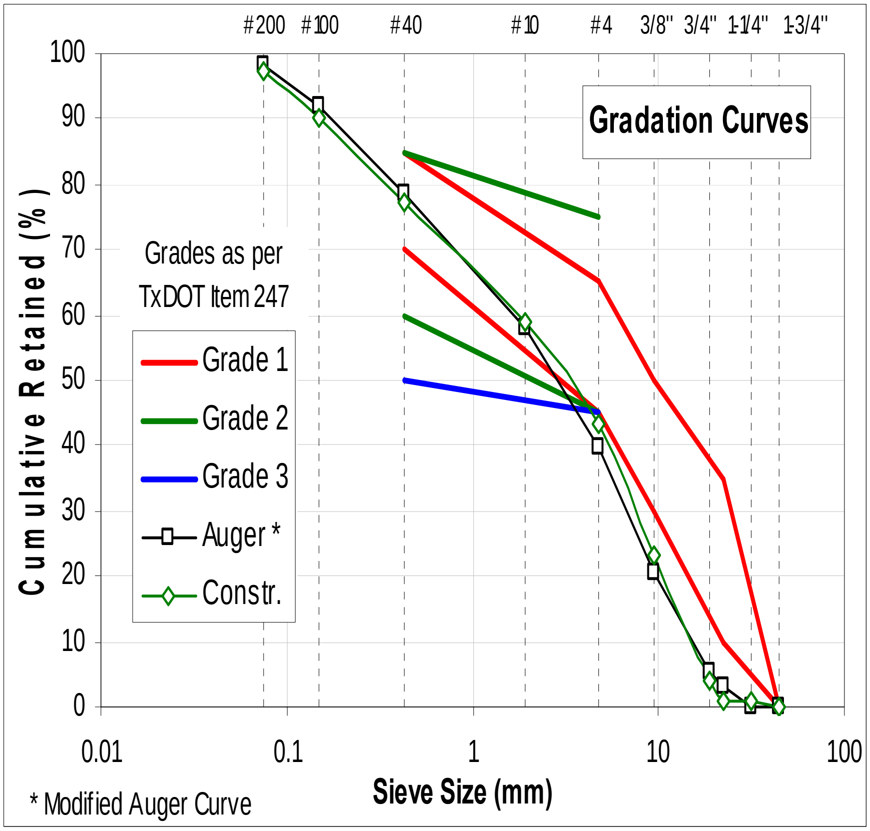

Figure 4.3. Comparison of Augur and Milling Machine Gradations.

Notes:

Section 4.3 DCP Testing on Shoulder

As shown in Figure 4.4 many FDR projects have edge stability problems. These are typically pavements without shoulders with steep side slopes, trees close to the sides of the road, in high rainfall areas with highly plastic soils. In some cases full FDR is not the answer to the pavement problems; often a system must be developed to provide adequate lateral support to the pavement. One useful tool to assist in evaluating this condition is the DCP with testing conducted in the shoulder. If a weak zone is present the rate of penetration will increase substantially. If lateral support in terms of shoulders is to be added, it but deep enough to cut thru this weak zone.

Notes:

Figure 4.4. Edge Stability Problems Common on Many FDR Projects.

Figure 4.5 shows the DCP schematically. The rate of penetration through the pavement layers (PR) is measured in mm/blow. This is used in the standard equation below to obtain a pavement layer strength:

$$CBR = 292 / PR^{1.12}$$

Which is then often converted to layer modulus using the following equation:

$$E\ (ksi) = 2.55\ (CBR)^{0.64}$$

CBR values below 3 are very weak, normal soils will have values between 8 and 12, and a good quality base will have values above 50.

Notes:

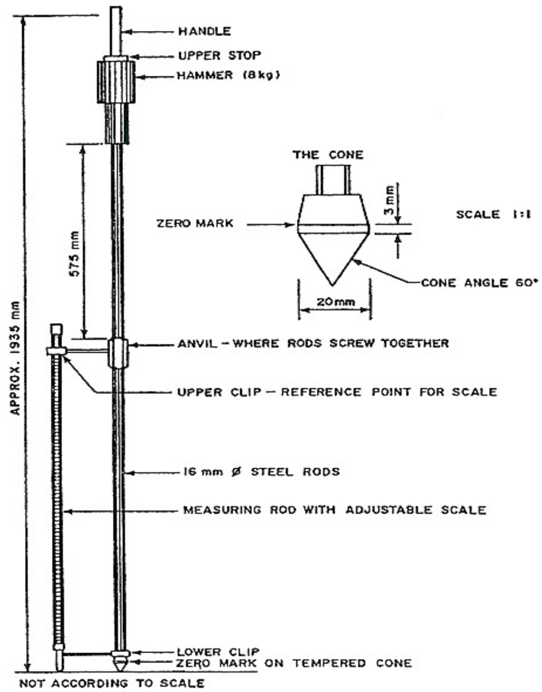

Figure 4.5. DCP Test Equipment and Testing on Shoulder.

The results from the testing being conducted in Figure 4.5 are shown below in Figure 4.6. In this case the very weak zone is located at a depth from 13 to 17 inches below the surface. If lateral support is to be required to adequately cut through this area the top 20 inches of shoulder should be removed and replaced with good quality base material. In all cases the base material should

Notes:

be day-lighted to the ditch. Given the right of way constraints, adding as much shoulder as possible is recommended to minimize the re-occurrence of the edge problems.

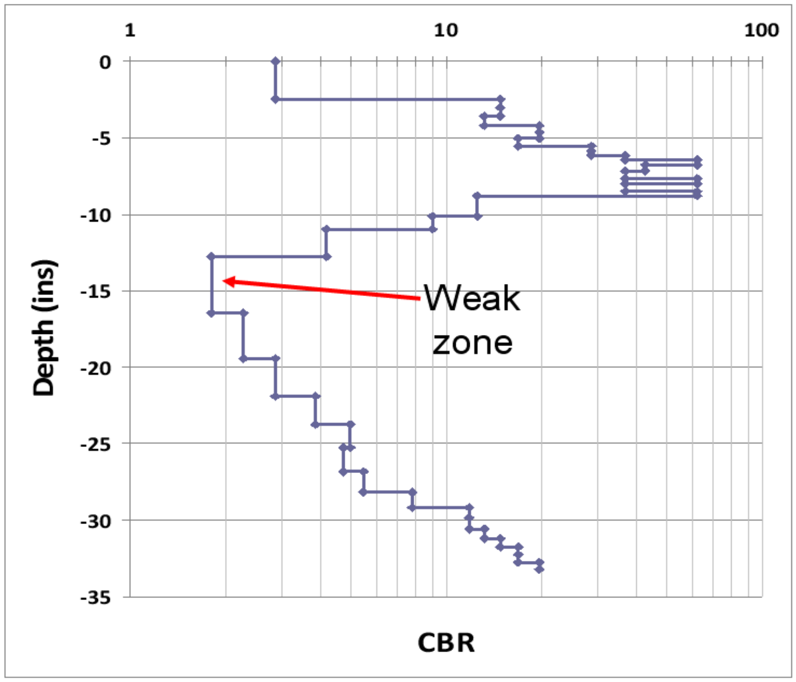

Figure 4.6. DCP Strengths with Depth Showing Weak Zone.

CHAPTER 5 – LABORATORY MIX DESIGN PROCEDURES

Notes:

When this chapter is over you will be able to:

- Understand the guidelines for determining if stabilization is required.
- Understand TxDOT's recommended procedure for selecting stabilizer types.
- Understand the steps in selecting cement and emulsion contents.
- Be familiar with TxDOT's design criteria.
- Be familiar with the new tests proposed to ensure adequate surface bonding.

Section 5.1 When to Add Stabilizers

Texas has a whole range of pavements sections, which are proposed as candidates for Full Depth Reclamation. There is a range of traffic levels, subgrade support conditions, and climatic zones. Figure 5.1 was put together to assist designers with the decision of when to "create a stabilized base," which is the rational FDR application and when can the pavement structural strength be improved by either base thickening or minimal stabilization.

For base thickening projects, the existing pavement structure must be uniform with very few structural defects. The base strengths must be reasonable and the section has medium to low traffic levels < 2000 vpd. Many areas of West Texas have good silt/sand subgrades, thin surfaces, and low traffic levels. If the pavement is in need of structural improvements then simply adding new flexible base to the pavement surface blending the new base and existing surface layers together without stabilization, then compacting, sealing and adding a new surface has proved to be very effective. Blending the old and new pavement together is highly recommended. Placing new base directly on top of old has been problematic with moisture often getting trapped in the upper base layer.

For upgrading Base to Class 1 projects the existing pavement should have reasonable subgrade support (> 10 ksi) from the FWD, the existing traffic is low at less than 2000 vpd and the existing surface layer is thin (less than 2 inches); then a very feasible alternative is to select a low level of stabilizer that will return the base to class 1 requirements in terms of compressive

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strength. This typically requires between 1 and 2% cement or 2 to 3% lime.

Creating a stabilized base is the traditional FDR approach where the existing pavement structure is recycled and treated with a lab designed level of stabilizer (cement, lime, asphalt or fly ash). These highways typically need structural improvement and often widening. If the base is marginal and thin, and the subgrade poor (less than 10 ksi) and the traffic level moderate (> 1000 vpd); then it is recommended that the existing pavement layers be stabilized and used as a foundation layer for a new pavement structure, which will typically consist of a layer of flexible base and a designed riding surface.

The FDR stabilized layer can be used as a base layer when:

a) The pavement is low volume, or
b) The existing pavement has a thick base and reasonable subgrade to provide support to the stabilized layer (this occurs in many areas of West Texas).

Geometric considerations such as the number of driveways and existing drainage features are also important considerations. This table also highlights other important design considerations such as the need to a) avoid cutting into clay subgrade, b) allow no more than a 50/50 RAP/base blend, c) consider geogrids on top of the stabilized layer with difficult soil conditions, and d) consider microcracking to avoid excessive shrinkage cracking.

Objective	Base Thickening	Upgrade base to Class 1	Create a Stabilized Base
Used When	• Existing base is uniform • No widespread structural damage • Low to medium traffic • Medium to Good subgrade	• Low – moderate traffic • Subgrade > 10 ksi • Moisture not a concern	• Bridging over poor subgrade • Strengthening required • Low quality variable base/stripped HMA • Higher Rainfall • Early opening to traffic
Selection of Stabilizer	No Stabilizer added to the existing material. This is a base thickening project, where new untreated granular material is placed on top of existing.	Full Texas Triaxial test (117-E), add low levels of stabilizer Criteria after 10 days capillary rise 1) 45 psi at 0 psi confining 2) 175 psi at 15 psi confining	Use Prevailing TxDOT spec and Test Methods (120 E, 121 E, 127E, SS3066) • All tests should include a retained strength on moisture saturation
FPS 19 Moduli	70 ksi	100 ksi	150 ksi
	1) New base should be of higher or equal quality than existing, and 2) Blending of existing and new base strongly recommended to avoid trapping moisture in upper layer		1) Avoid cutting into high PI subgrade, if existing structure is thin then add new base before milling where needed 2) To avoid longitudinal cracking consider grids and flex base overlay where the PI subgrade soils > 35 3) Max RAP 50% 4) If lab strength > 350 psi then consider micro-cracking 5) Max Cement 4%, other stabilizer can be used

Figure 5.1. When to Use Stabilizers.

Notes:

Section 5.2 Soil Properties and Guidelines on Stabilizer Selection

TxDOT’s guidelines for selecting the appropriate stabilizer content are provided in the “Stabilization Guidelines,” which are available online at: http://ftp.txdot.gov/pub/txdot-info/cmd/tech/stabilization.pdf.

Several of the key elements are described below. The first is the proposed chart shown in Figure 5.2 for selecting stabilizers that should be considered in the lab testing sequence. The key parameter is the materials plasticity index. If the base is a blend of RAP and old or new base then the PI measurement should be made on the blend of materials, whatever is going to be treated in the field. The FDR process uses the recommendations at the left of the figure under “Base,” for low PI materials cement, fly ash, and asphalt are recommended. Lime is strongly recommended if the base has a PI of more than 12, which is an indication of substandard materials or clay contamination. Both cement and asphalt have problems stabilizing bases with substantial clay content.

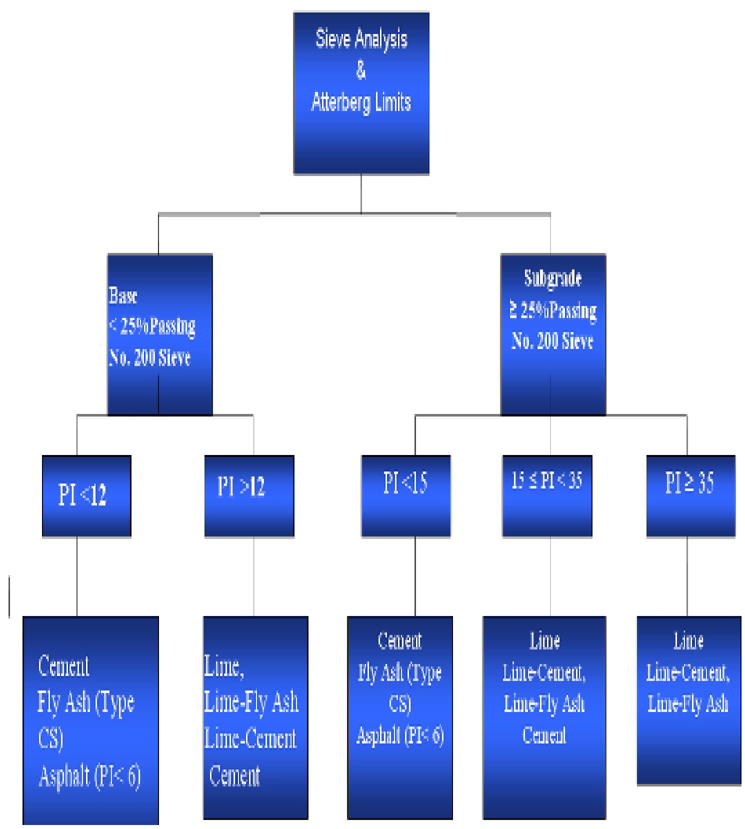

Figure 5.2. Stabilizer Selection Guidelines.

In many of the designs conducted at TTI it is often desirable to consider two stabilizers as alternatives. This could be a cement design versus an asphalt emulsion design. Both are designed according to the criteria presented in the next section. They are then also entered into the pavement design system. Given the vast array of other factors involved in the pavement design process such as quality of subgrade support, environmental factors, existing pavement structure the proposed typical section may be different for each stabilizer. The following criteria are also important when determining which stabilizer to select.

Notes:

When to Use Cement

- Base and subgrade are poor and there is a need to create a foundation layer.
- Low volume roadway with adequate base thickness, with cutting into the clay subgrade (from experience thicker lightly stabilized cement treated base layers perform better than thinner stiffer layers).
- Low PI base materials.

When to Use Asphalt Emulsions

- When the pavement structural problems are base related (below the treated layer is some existing base and a reasonable subgrade).
- Base layer has low fines (PI < 6 from Figure 5.2).
- Can be economical when the depth of treatment is not greater than 6 inches.

When to Use Lime or Fly Ash Blends

- When the base has substantial clay fines (ideal for low volume roadways where the existing material may be clay contaminated).

Section 5.3 Selecting the Optimal Stabilizer Content

The criteria used when selecting stabilizers is taken directly from TxDOT's standard recommendations with several additions. All tests now require a moisture susceptibility indication as measured by the unconfined strength after 10 days capillary rise. There is also a need to collect supplemental information.

Notes:

Cement Design Strength Criteria

Test	Spec Limits
Unconfined Compressive Strength (psi) @ 77ºF (Tex-120-E)	175 min
Retained UCS (psi) @ 77ºF after Tube Suction Test	100% min
Tube Suction Test Final Dielectric (Er) and moisture content (%) (Tex-144-E)	For Information Only
Unconditioned Seismic Modulus (ksi) (Draft TxDOT Method 149E)	For Information Only Tested at 7 days

Figure 5.3. Laboratory Requirements for Cement Treatment.

Figure 5.3 shows the current requirements for cement treatment. The current strength criteria recommend three strength levels for selecting the appropriate cement these being:

- Class L 300 psi.
- Class M 175 psi.
- Class N As shown on plans.

Many Districts still specify the 300 psi level and the current item 275 do not require any moisture susceptibility test. In many of the recent designs the 175 psi 7 day strength has been specified with 100% retained strength on wetting. Performance of these sections to date has been good and the low strength makes it easier to minimize shrinkage cracking. As a rule of thumb, do not use cement content of more than 4% to minimize shrinkage cracking.

Figure 5.3 also recommends additional tests: the Tube suction test and Seismic modulus test are shown in Figure 5.4.

Figure 5.4. Seismic Modulus and Tube Suction Tests Underway.

The upside of the TxDOT procedures is that when they are followed they lead to long lasting durable stabilized layers. The downside of the current procedures is that they a) require too much material and b) take too long. The sample size for all TxDOT work is 6 inches diameter by 8 inches high, given than optimum moisture contents should be determined (theoretically at all stabilizer contents). Then a single design will require almost 300 lb of material; in addition to this as shown in Figure 5.5 after the Optimum Moisture Content is determined it takes a minimum of 19 days to run the engineering strength tests. Future efforts should look at procedures for both reducing the amount of material and time of the test.

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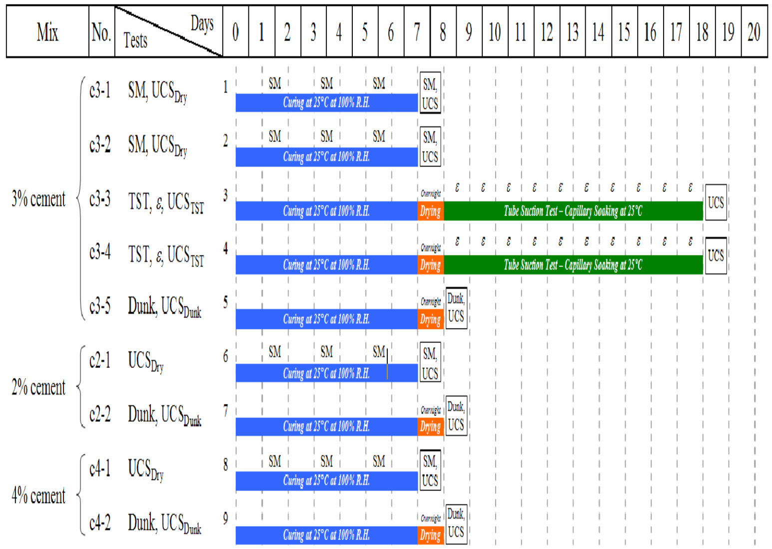

Figure 5.5. Testing Sequence for Cement Treated Materials.

Emulsion Design Criteria

Emulsion treatment, with or without a small percentage of cement, has become a somewhat popular option to provide increased strength while retaining some flexibility. Figure 5.6 shows the current criteria for emulsion treatment.

Test*	Spec Limits
Unconfined Compressive Strength (psi)(SS3066)	150 min
Indirect Tensile Strength (Tex-226-F)	> 50 psi
Tube Suction Test Final Dielectric (Er)(Tex-144-E)	< 10
Unconfined Compressive Strength after the Tube Suction Test	≥ 80% Dry UCS
Seismic Modulus	Report

*All tests are preceded by 2 days curing at 60°C and 1 day cooling

Figure 5.6. Laboratory Requirements for Emulsion Treatment.

The length of time required to run this test is somewhat shorter because of the two day oven curing requirement, shown in Figure 5.7.

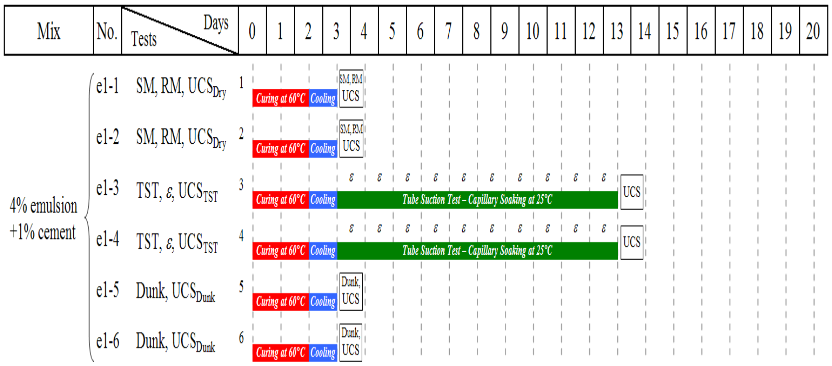

Figure 5.7. Time Required to Run the Emulsion Stabilized Base.

Notes:

The criteria provided in Figure 5.6 have been used on several projects and the following observations were found:

1) It is very difficult to make the 150 psi strength criteria without the addition of small levels of cement (typically 1%); the problem with this is that cement stiffens the treated layer and this has been found to cause performance problems (return of longitudinal cracks).
2) The Indirect Tensile strength requirement is difficult to meet.

TxDOT should encourage the use of emulsions because of their waterproofing ability and because they are less stiff than cement and therefore less prone to cracking from either shrinkage or subsurface ground movements. However the high UCS values work against these requirements. For emulsion-only designs, consideration should be given to:

a) Reducing the UCS requirement to 100 psi.
b) Increasing the retained strength requirement to 100%.
c) Removing the IDT requirement.

<u>Emulsion Criteria for Lime and Fly Ash</u>

Fly ash and lime-fly ash are used in some districts for stabilization particularly on sections with “dirty bases.” Figures 5.8 and 5.9 show the lab requirements for these mixtures.

Notes:

Test	Spec Limits
Unconfined Compressive Strength (psi)* (Tex-127-E)	150 min as subbase; Similar to cement treatment for base course
Unconfined Compressive Strength (psi)**	200 psi

*After conditioning per Tex-127-E over 17 days
**After 6 days bench-top curing per project 0-5223 recommendations

Figure 5.8. Laboratory Requirements for Fly Ash and Lime-Fly Ash Treatment.

Lime-treated mixtures are tested in accordance with Tex-121-E and should achieve strengths after the 17-day conditioning program as shown in Figure 5.9. The strengths below are all for moisture-conditioned samples after 10 days capillary rise.

Test	Spec Limits
Unconfined Compressive Strength (psi) (Tex-121-E)*	50 psi min as subbase; 150 psi for final course of base construction

*After moisture conditioning per Tex-121-E over 10 days

Figure 5.9. Laboratory Requirements for Lime Treatment.

Experience has shown that with FDR projects the 150 psi requirement is often difficult to achieve. Based on limited testing it is proposed that the lime criteria be modified to:

a) Unconfined compressive strength after 7 days cure in moisture room should be a minimum of 100 psi.
b) Retained strength after 10 days capillary rise should be 100%.

Section 5.4 New Bonding Test

One recurring problem with some stabilizers is bonding of the proposed surface material to the stabilized layer. Slippage failures have been reported on several projects. See Chapter 9 for an example.

A new test shown in Figures 5.10 and 5.11 has been developed as part of the research study to measure the bond strength between the treated base and new surfacing layer. The proposed test is widely used for measuring the bond strength of flooring materials to concrete. For the FDR application, it is used on 6-inch diameter, 2-inch high samples of the treated base; the top layer is primed and a grade 5 seal coat is applied. Draft specifications are available. As shown in the figures below, the surface is lightly cored to a depth of 0.25 inches and the steel disk is glued to the surface.

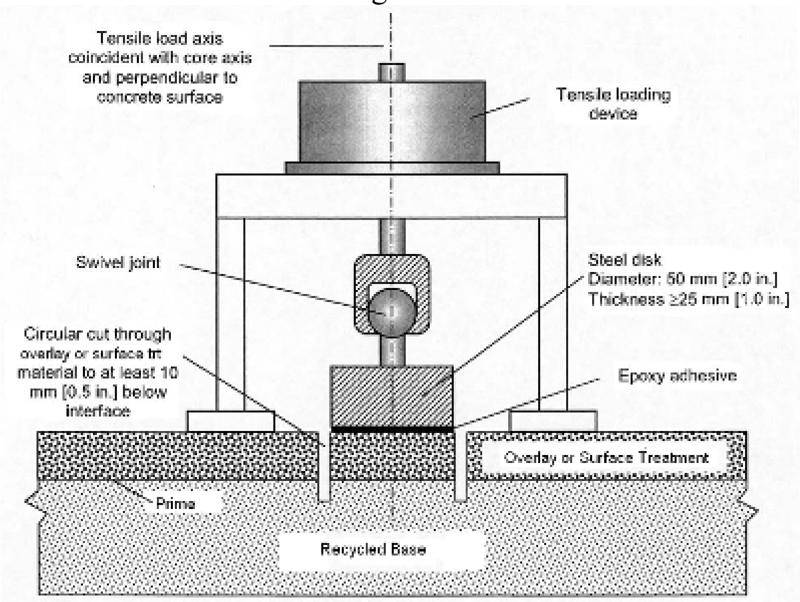

Figure 5.10. ASTM Test Method C -1583 Used for Measuring Bond Strength.

Figure 5.11. New Bond Strength Test Sample after Test.

Preliminary studies have been conducted at TTI and the test does have the ability to discriminate between different prime materials and the different application rates of the same prime. The results in Figure 5.11 show the results from four different products on a

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cement treated base sample. In this case, products P1 and P4 are acceptable, P2 is marginal, and P3 is not recommended. More work is needed but the initial results are encouraging.

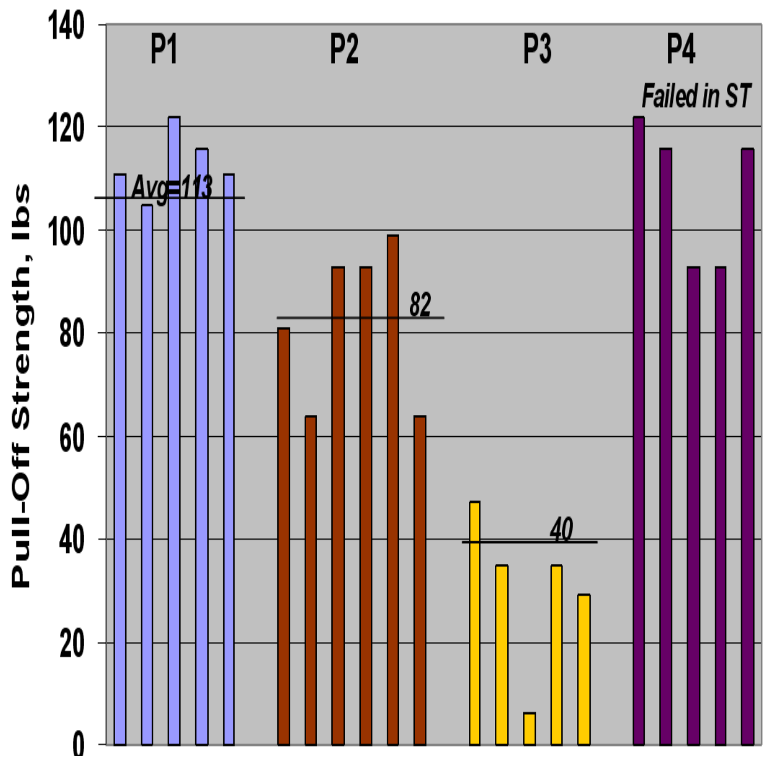

Figure 5.12. Bond Strength Test Results.

CHAPTER 6 – PAVEMENT THICKNESS DESIGN

Notes:__________________

When this chapter is over, you will be able to:

- Understand what design moduli values to use in FPS design.
- Understand the values to be used in the Triaxial check system.
- Be able to describe options available to handle project variability.
- Be familiar with the Microcracking technique used to minimize shrine cracking.
- Understand the cause of longitudinal cracking in new projects and how to minimize its appearance in the design process.
- Understand how to evaluate the need for lateral support and how the DCP can help in making that design decision.

Section 6.1 FPS Design Requirement

As with all pavement designs in Texas, it is important that the FDR projects also be designed using the Flexible Pavement Design system (FPS 19 or 21). This could be to calculate the thickness of flexible base overlay to be placed over the stabilized subbase layer or for heavy trafficked sections the amount of hot mix asphalt required to carry the design traffic loads over the stabilized base layer.

Training on how to use the FPS system is given elsewhere. On any FDR project the FWD must be run first to obtain the modulus value for the existing subgrade. Traffic data and other input requirements are assembled so that a design can be generated with the routine FPS pavement design system. Figure 6.1 gives the recommended design moduli values for FDR projects. These values are thought to be conservative and representative of the continuing long-term support stiffness that can be expected from a stabilized layer.

Materials Description	FPS Design Modulus Values	Poisson Ratio	Cohesiometer Value for Triaxial Check
Existing Material (including subgrade)	Backcalculated from FWD	0.40	na
Existing Pavement Scarified, Reshaped	3 Times Subgrade Modulus	0.35	na
Stabilized Existing/Subgrade a) Most Granular Base (75% more base) b) Blend Subgrade & Base (50–75% base) c) Mostly Subgrade (< 50% base)	a) 100 ksi b) 65 ksi c) 35 ksi	a) 0.30 b) 0.30 c) 0.35	a) 800 b) 650 c) 300
Stabilized RAP/Existing Base; Max 50/50 Blend a) Cement b) Lime c) Emulsion d) Fly Ash	a) 150 ksi b) 75 ksi c) 100 ksi d) 75 ksi	a) 0.25 b) 0.30 c) 0.30 d) 0.30	a) 1000 b) 300 c) 300 d) 300
New Flexible Base over Stabilized Layer	70 ksi	0.35	na

Note: The values should be established by each District for their materials

Figure 6.1. FPS Design Moduli and Cohesiometer Values.

Full details on all aspects of pavement design are provided online at:
http://onlinemanuals.txdot.gov/txdotmanuals/pdm/index.htm

Section 6.2 Handling Project Variability

Notes:

One very unique feature of all FDR designs is the in-situ pavement variability that exists in these FDR candidates. In several parts of the state, sections have received substantial maintenance and very variable pavement thicknesses are found. As described in earlier sections candidates have been shown to have from 3 to 20 inches of HMA surfacing. It is handling this variability that is the major challenge to all FDR project designers. TxDOT has the tools available to document and verify these variations as described in Chapters 3 and 4 of these notes.

When doing the pavement design it is important to keep the following two major recommendations in mind:

1) If possible restrict the FDR blend to no more than 50% recycled asphalt (RAP) (little success has been documented with more than 50% RAP for a variety of reasons).
2) Avoid if possible cutting into any high PI clay subgrades.

Clearly it is important to document the thickness and quality of the existing HMA and base layers in any FDR design. The case study below documents the level of detail that is required to meet the recommendations described above.

Case Study FM 148 in Dallas

FM 148 has a variable HMA pavement structure. The original pavement consists of 5 to 6 inches of HMA over a 6-inch lower quality flexible base. But many areas have received additional HMA overlays and numerous locations have full depth patches. The GPR survey was conducted to determine areas where the HMA layer much thicker than the standard sections. Figure 6.2 shows the pavement thickness versus distance chart. The 12-inch thick designation covers all locations greater than or equal to 12 inches thick; at one location a 15-inch core was found. A fairly uniform grade 2 flexible base was found beneath the HMA ranging from 6 to 8 inches in thickness.

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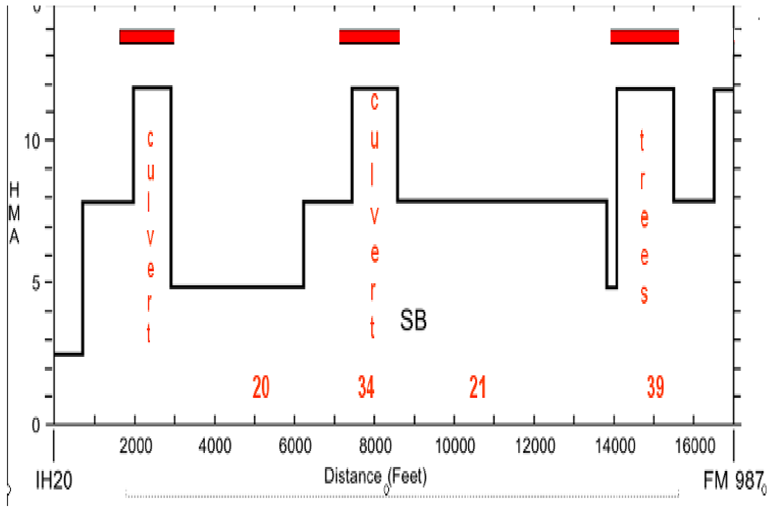

Figure 6.2. Variable HMA Thickness on FM 148.

<u>Pavement Thickness Design Recommendations</u>

Laboratory testing was conducted on the materials obtained from FM 148 and it was determined that 3% cement met all of the strength requirements described in Section 5 of these notes. The Dallas District wanted to use 2 inches of HMA as the final surface with a flexible base overlay and cement stabilized recycled layer. The main design consideration is the required thickness of the granular base overlay.

The FPS system was used to generate this thickness. For the FM 148 analysis the following values were assumed:

- HMA – 500 ksi (Standard TxDOT recommendations).
- Flexible Base – 70 ksi (Good base over CTB).
- Cement Stabilized FDR layer – 100 ksi (Bryan District recommendation for FDR).
- Subgrade – 6 ksi (FWD data).

The traffic levels assumed for this highway are Current Year ADT 1590 vpd with and 20 year 18 kip ESAL estimate of 1.433 million. Pavement Type 4 of the FPS design system was used and the analysis called for the use of 6 inches of flexible base over the stabilized layer to provide a time to first overlay of 15 years. The Triaxial check was also performed on the FPS structure, and Figure 6.3 shows the results. Using the modified Cohesiometer value for a cement treated subbase of 1000, the total design thickness of 15 inches was found to be adequate.

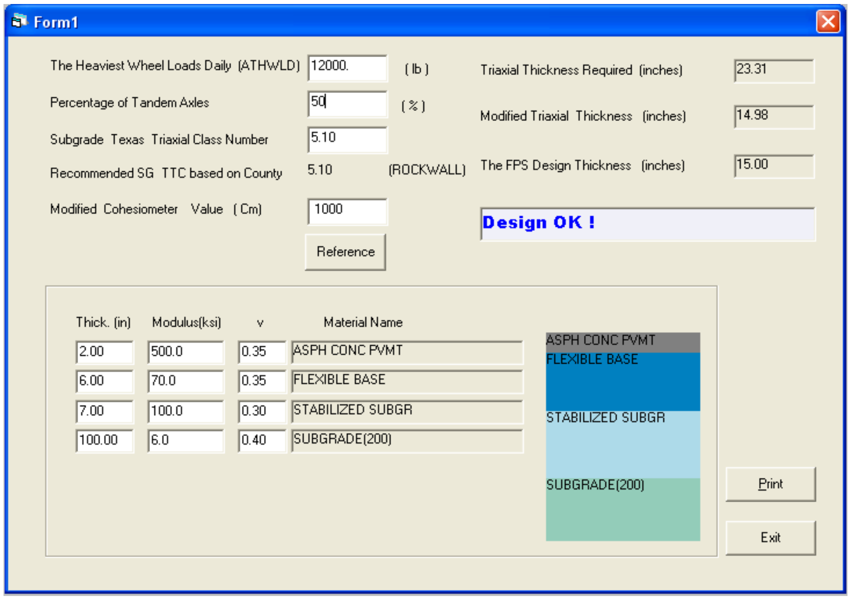

Figure 6.3. Texas Triaxial Design Check for FM 148.

There is a variety of thickness along this section. The most common depth is 8 inches of HMA over 6 inches of granular base. The proposed construction sequence calls for milling 4 inches of HMA and then recycling 8 inches of existing material and treating it with 3% cement. This is followed by a 6-inch flexible base overlay and a two course surface treatment. The first will be CRS 250 with a grade 5 rock followed by an asphalt seal with grade 4 rock. Traffic will be allowed to run on this section for as long as possible before placement of the final HMA surface.

Based on the need for a uniform support layer the recommendations shown in Figure 6.4 are proposed for this project. The normal scenario described above will be used in all areas where the HMA is 8 inches thick. Where the HMA is only 5 inches, no milling will be performed and a total of 8 inches recycled. For very thick HMA sections a mill followed by a new base overlay is proposed.

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From–To (feet)	Treatment
0–700	2"overlay only (new construction)
700–1800	Mill 4" of HMA; the FDR 8" + base overlay
1800–3000	Mill 6" HMA add 4" new base; FDR 8" + base overlay
3000–6000	FDR 8" + base overlay
6300–7200	Mill 4" of HMA; the FDR 8"+ base overlay
7200–8900	Mill 6" HMA add 4" new base; FDR 8" + Geogrid + base overlay
8900–14,000	Mill 4" of HMA; the FDR 8" + base overlay
14,000–15,600	Mill 6" of HMA add 4" new base; FDR 8" + Geogrid + base overlay
15,600–16,700	Mill 4" of HMA; the FDR 8" + base overlay
16,700–end	2" HMA over only (intersection new construction)

Figure 6.4. Detailed Milling and Construction Recommendation to Handle Project Variability.

These recommendations were accepted by the Kaufmann areas office and incorporated into the plan set. This project was constructed in the fall of 2010.

Section 6.3 Microcracking to Minimize Shrinkage Cracking

The development of wide block cracks has long been the major performance concern with using stabilized bases (especially cement). Work conducted at TTI found that early trafficking of stabilized bases actually reduced the amount and severity of cracking in the stabilized layer.

Details on microcracking research can be found at http://tti.tamu.edu/documents/0-4502-S.pdf.

The goal of all microcracking is to introduce a web of fine cracks into the pavement to act as crack relief. If left untreated, CTBs will shrink and a series of block cracks will be found, typically 10 ft by 10 ft. However with the microcracked sections it is hoped that only a series of fine interlocked cracks will be introduced with no reduction in pavement long-term bearing capacity.

Notes:

Microcracking is typically performed 2 to 3 days after placement of the stabilized base. Normally a 12 ton vibratory roller is operated at creep speeds. It is important to design the number of passes required to get adequate breaks. It is recommended that stiffness measurements be made on the unbroken stabilized base with any of the available stiffness measuring systems (FWD, Seismic, Stiffness gauge), then to apply two passes of the roller. The goal is to target a 40% reduction in base stiffness. If this is not achieved then another two passes are applied and the section retested.

Figure 6.5 shows microcracking underway on a Texas project with the typical crack pattern. Often it is very difficult to see cracks in the stabilized base, which is why it is important to use the stiffness measuring system to establish roller requirements for adequate cracking.

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Figure 6.5. Microcracking and Typical Crack Pattern.

On many project construction is performed under traffic so that the traffic loads are applied to the FDR section every night, in that case microcracking may not be required. In Figure 6.6 testing is underway with the FWD and Humboldt stiffness gauge to check to see that the base is adequately fractured as measured by a reduction in base stiffness.

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Figure 6.6. Checking for Reductions in Base Stiffness.

One other issue important in this minimizing shrinkage crack is that the amount and severity of these cracks is also a function of the amount of stabilizer used in the FDR design. This in turn is related to the target strength required by the specifications. In recent years 7 day strengths have reduced form 500 psi to either 300 or 175 psi so cement contents now are typically around 3%, whereas in the past they were 5 to 6%. This reduction in cement content also helps minimizing the amount and severity of shrinkage cracking.

Section 6.4 Minimizing Longitudinal Cracking Problems

By far the biggest performance problem reported by Texas Districts in east Texas has been the appearance of longitudinal cracks shortly after the completion of the FDR project (Figure 6.7).

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Figure 6.7. Longitudinal Cracks 18 Months after Construction.

Studies in the Bryan District found that the following were critical issues in the occurrence of these cracks:

a) Highly plastic subgrade soils (especially soils with PI > 35).
b) Summer drought conditions.
c) Trees close to the edge of the pavement.
d) Deep ditches.
e) Stiff bases (the stiffer the base, the more severe the cracking).

If several of these factors are present on any FDR project, then longitudinal cracking caused by summer drying is a major concern. In the mid 1990s, the Bryan District successfully evaluated the use of geogrids placed over the stabilized layer to minimize this problem. The geogrid is placed to introduce a slip plane into the section so that cracking the lower layers does not reflect through the upper layers. Over the geogrid a layer of flexible base is placed together with a thin surfacing. Figure 6.8 shows a typical Bryan section. Figure 6.9 shows the placement of the flexible base over the geogrid.

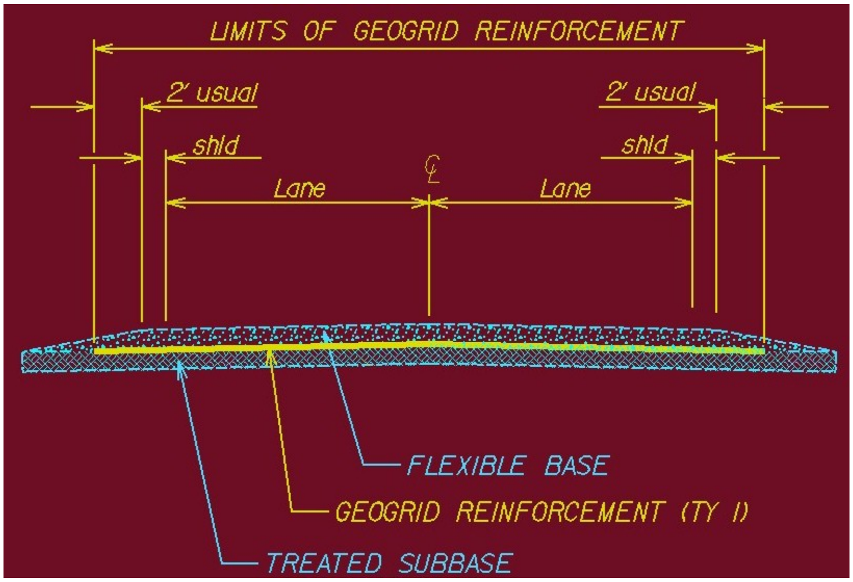

Figure 6.8. Typical Section to Install Geogrid.

Figure 6.9. Covering the Grid with Flexible Base.

Geogrid is not required over 100% of any project, only in locations where the problem conditions exist. This could be typically from 5 to 25% of the sections.

One District proposed that in lieu of using the geogrid that low fines base could be used to retard these reflection cracks. This is a very interesting concept that has not been tried, but it could work.

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Section 6.5 Tools for Designing Lateral Support Requirements

In many failed pavements FDR is not the main answer to solve pavement problems. In many areas the lack of adequate confining to the pavement lanes causes edge stability problems (Figure 6.10). Recycling the main lanes only will not be effective without the addition of lateral support. On major roadways this requires the addition of an adequate shoulder.

Figure 6.10. Pavement with Thick Layers but Continuing Failures with Lack of Edge Support.

The Dynamic Cone Penetrometer will help here; the DCP was described in earlier section of these notes. It is driven thru the shoulder to determine if there is a very weak layer that more than likely is causing the slippage failures shown in Figure 6.10. A typical set of data from the shoulder testing of the pavement shown in Figure 6.10 is shown below in Figure 6.11.

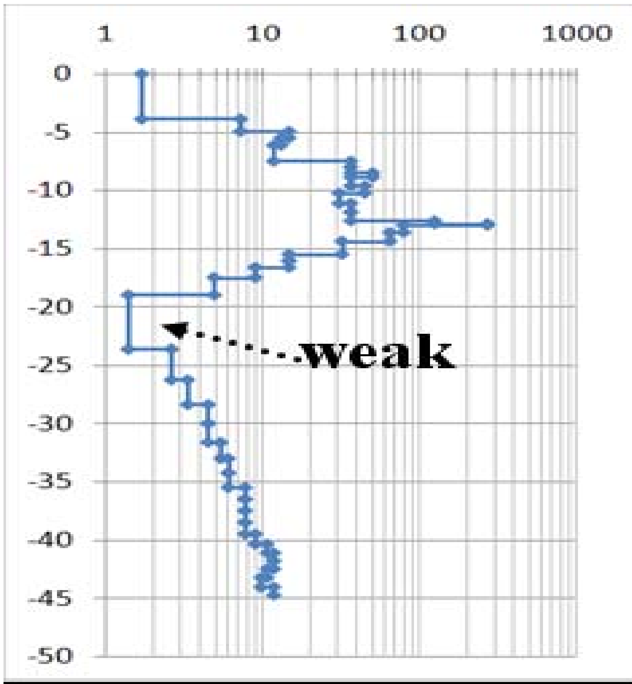

Figure 6.11. DCP Results Showing Strength versus Depth (ins) for the Section Shown Above.

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In this case the weak zone is located 19 to 24 inches below the surface of the shoulder. Any improved lateral support will need to cut thru this layer. Figure 6.12 shows the existing pavement structure (very thick) with the proposed widening scenario. In this case the outside edge of the pavement will need to be milled and removed; the next 10 inches can then be pulverized and removed to be returned once the widening is complete. The deep widened support layer is then added to the pavement edge and day lighted to the ditch to ensure that water is not trapped. The reclaimed base can then be returned with a designed HMA surfacing.

This is a lot of work, but it is the only way to address this pavement problem. Without adding adequate lateral support the pavement problems will quickly reappear.

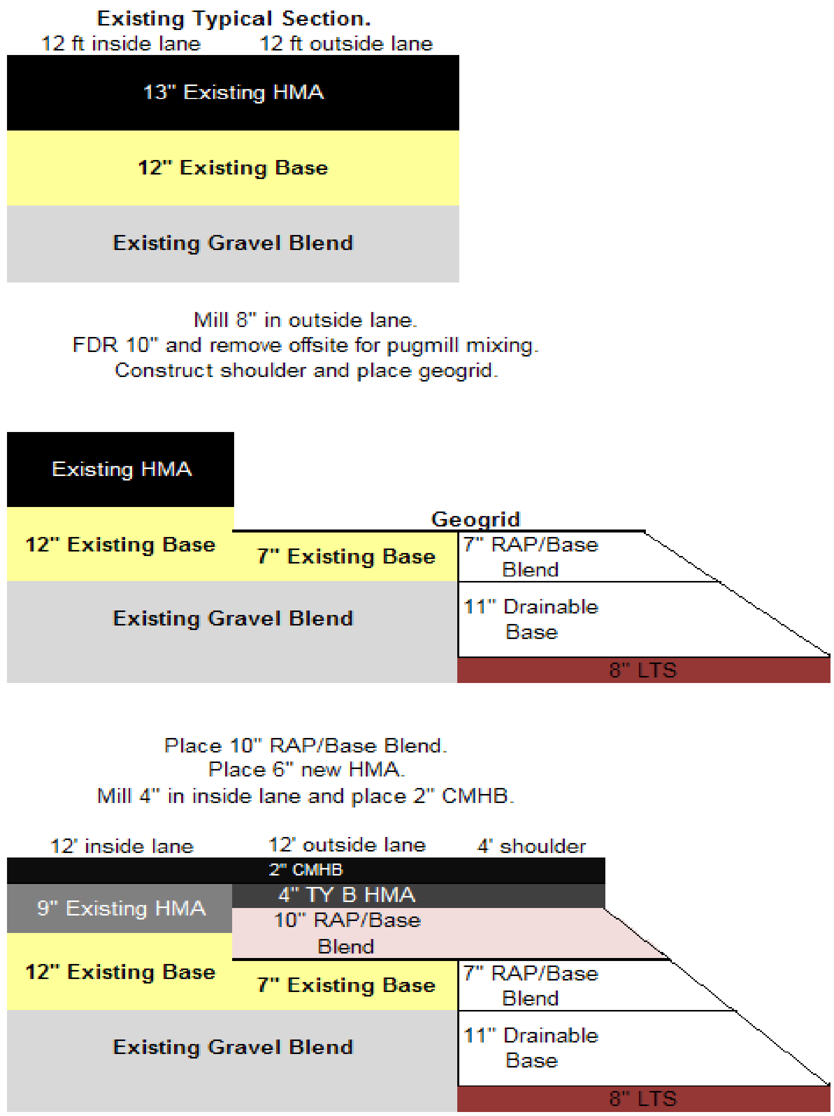

Figure 6.12. Widening Required to Address the Weak Layer Found by the DCP.

CHAPTER 7 – CONSTRUCTION SPECIFICATIONS

Notes:

When this chapter is over you will be able to:

- Be familiar with current TxDOT specifications.
- Understand the testing that needs to be done on a typical FDR project.
- Be familiar with NDT tools available for both QA/QC testing.
- Understand how to certify that the FDR project is being built as designed.

Section 7.1 Existing Construction Specifications

Currently FDR construction is performed under one of the prevailing specifications shown in Figure 7.1. Details of these will be discussed in this chapter.

Figure 7.1. TxDOT Specifications.

Overview of Construction Steps

The steps in a typical FDR sequence with cement/lime or fly ash are shown in Figure 7.2. Each of the steps are also shown in photos in Figures 7.2 a, b, and c. It is recommended that at the start of any project a test strip be built and each step in the process evaluated to ensure its conformity with the prevailing specification.

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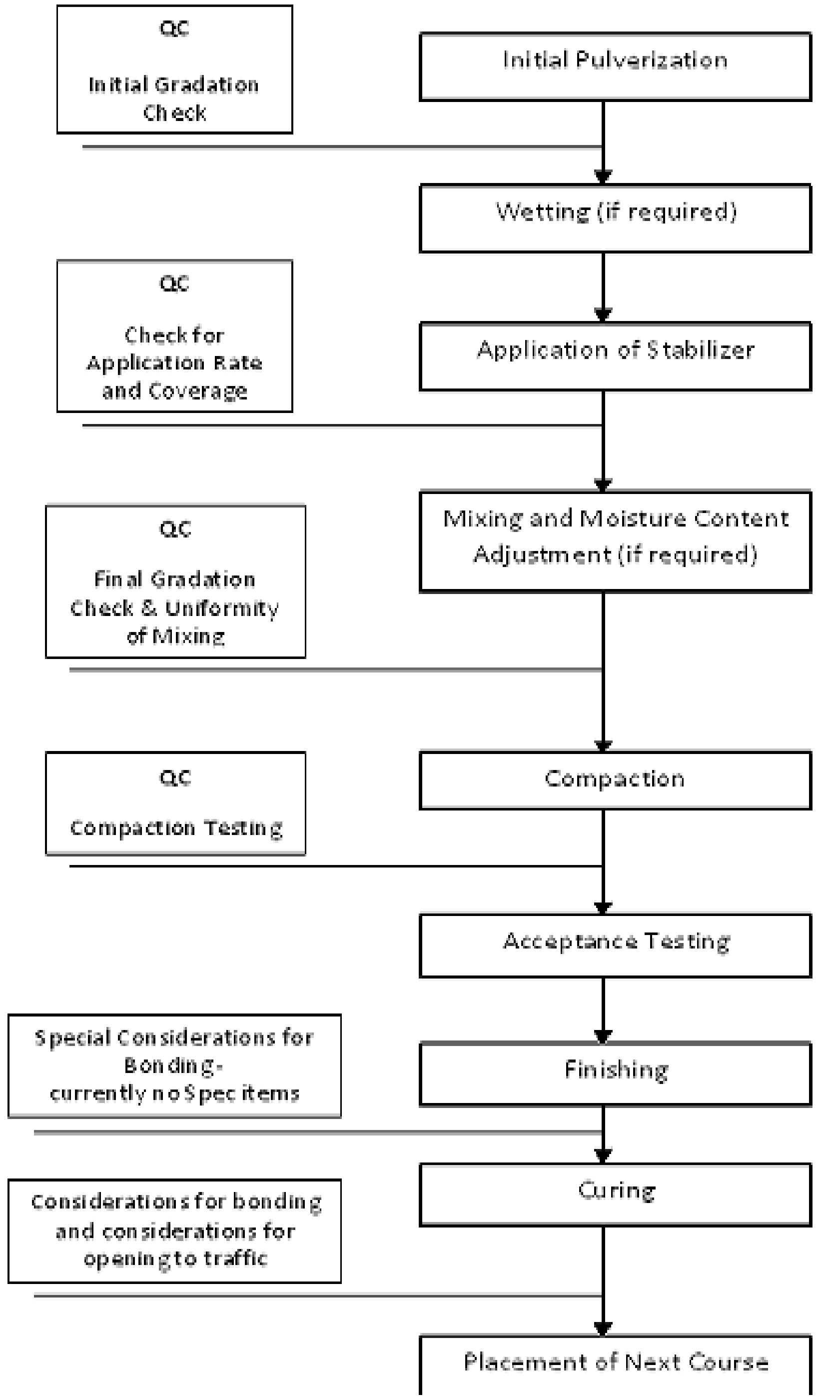

Figure 7.2. Steps in the Construction Process with the Appropriate QC Testing.

Figure 7.2a. Initial Rip, Shaping, and Wetting.

After pulverization the section is worked with a blade and small berms are made along the pavement edge to help keep the stabilizer in place. During these operations the initial gradation is checked, together with moisture content. For dry placement the moisture content is targeted to be close to optimum as determined by Method 113E. Final moisture content to get adequate compaction is targeted to be ±2% of optimum. If the stabilizer is to be placed in slurry form then the field moisture is typically selected to be 50 to 60% of optimum.

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Notes:

Figure 7.2b. Placement of Stabilizer and First Pass Mixing.

It is very important to ensure that the correct amount of stabilizer is placed. Simple spreadsheets are available to calculate the length that can be treated with a typical transport. Full depth mixing is then performed; this is typically 8 to 10 inches. To process an entire lane typically two passes of the recycler will be required. A 12-inch overlap between passes is recommended.

Figure 7.2c. Compaction Typically Initially with Sheep's-Foot Roller Followed by a Steel Wheel to Get Good Finish.

Notes:

<u>Pulverization Requirements</u>

<u>Initial</u> *100% passing 2.5 inch sieve*
<u>After Mixing</u> *100% passing 1.75 inch sieve; 85% passing ¾ inch sieve*

The ability to achieve this level of pulverization is a function of many factors including:

a) The thickness of the HMA layer.
b) The temperature of the HMA layer.
c) The type of base layer.
d) The variability of the pavement structure.

The contractor must select adequate equipment to obtain the required level of pulverization. The upfront testing with GPR and field verification proposed earlier in the class can really provide useful information to the contractor to plan his pulverization work.

Notes:

Temperature Requirements

Most specifications state that the temperature at the start of construction should be 35F and rising. However these are not thought to be adequate. Major finishing and bonding problems have been encountered when the temperature on night 1 and 2 falls below freezing.
Districts are encouraged to put these requirements into future plan notes.

Application Rates

Given the known project dimensions and required application rate the length of coverage is computed. In the example below 1 ton of stabilizer would cover about 53 ft of pavement. A typical 20 ton shipment would cover approximately 1000 ft. It is important to check that the material is being applied at approximately the same depth throughout the marked section.

	A	B	C	D	E	F	G	H	I	J
1	Unit weight		lb/cuft		125		Max Dry density from OMC curve			
2	% Stabilizer		%		3					
3	Depth of treatment		ins		10					
4	Tanker size		tons		1					
5	width of treatement		ft		12					
6	Lbs/ft				3.125					
7	Tons/sq yd				0.014063					
8	Length per tanker		feet		53.33333					
9										

Figure 7.3. Calculation of Application Length.

Compaction Requirements

The different stabilizers have different mellow and compaction time requirements as shown below in Figure 7.4. The density requirements are also shown. In general it is not recommended to place stabilized layers in more than one lift, especially with cement as many bonding problems have been reported between the two treated layers.

	Mellowing Time	Compaction Time	Density 115-E
Cement	None	2 hours after application	95%
Lime (Hydrated)	1 – 4 days	After mellow	95% first 98% next
Lime (Quick)	2 – 4 days	After mellow	95% first 98% next
Fly Ash	None	6 hours after application	95% first 98% next

Figure 7.4. Compaction Requirements for Different Stabilizers.

It is the contractor's choice on how to achieve density of a FDR project. However the sheep's foot type roller is often the first roller used and passes are made until the roller "walks out of the layer being compacted as shown in Figure 7.5.

Figure 7.5. Initial and Final Pass of the Sheep's Foot Roller.

Section 7.2 FDR with Asphalt Emulsions or Foam

Asphalt Emulsions and foam have not been widely used in Texas but they have been used elsewhere with reported success. Excellent references are available from Wirtgen Inc. on how to design and construct sections with either emulsion or foam (TG 2 Technical Guidelines for Bitumen Stabilized Materials, 2009).
The strength and weakness of the use of asphalt stabilization are as follows.

Notes:

Notes:

Strengths

- Can be performed in a one pass operation.
- Excellent for waterproofing existing materials.
- Good structural strengths can be obtained.
- Can be performed in thin lifts (say 4 inches deep) to upgrade low volume roadways.

Weakness

- Typically more expensive than traditional stabilizers.
- Requires low PI materials (PI < 6); difficult to find in Texas.
- Curing of the emulsions is sometimes a problem in high humidity or high rainfall areas.
- The use of foam requires specialized equipment, which often is not widely available.

The District with the most experience with emulsions is Amarillo; however most of their projects have included treatments of new base layers, where the top 8 inches is waterproofed with an emulsion treatment. Other Districts such as Dallas, San Antonio, and Beaumont have also constructed emulsion stabilized FDR sections. A summary of the performance of these sections can be found in the following TTI report (Report 401741-1 Hilbrich and Scullion. Dec 2008).

The use of foam stabilization has not been tried in Texas since the initial failure of the section on US 82 in the Wichita Falls District in 2002. That failure was caused by localized milling too deep and incorporating black clay into the base. However other DOTs have used foamed asphalt with reported success using the new generation of foaming equipment. Figure 7.6 shows a project from California. Two recyclers were working in tandem to get a single full roadway treatment. Twenty miles were reconstructed and repaved in 20 days, 6 year performance is reported to be very good.

Figure 7.6. Foamed Asphalt FDR by Caltrans (Highway 20; 2001).

Notes:

Foaming should be tried in Texas, but clearly sufficient upfront testing must be under taken to avoid the problems that occurred on earlier efforts.

Section 7.2 Quality Control Testing with Stiffness Devices

With the current specifications only nuclear density equipment is required for acceptance. Substantial research has been conducted in Texas on the use of stiffness measuring devices to check the quality of the treated base. These NDT devices are shown in Figures 7.7 thru 7.9, they include:

a) Dynamic Cone Penetrometer (rate of penetration of cone related to stiffness).

Notes: ____________________

Figure 7.7. DCP.

Device	Benefits	Drawbacks
DCP	• Simple, rugged and portable • Already adopted for acceptance testing by some agencies • Inexpensive	• Requires supplementary moisture content test • Selection of target value may require calibration strip

The DCP is the only device that gives direct readings with depth and it can also be used to get the layer thickness. However it is very difficult to penetrate cement stabilized layers once they are more than a few days old.

b) Portable Pavement Analyzer (P-SPA).

Notes:

Figure 7.8. P-SPA.

Device	Benefits	Drawbacks
PSPA	• Linkable to laboratory test results and design values • Portable • Provides rapid results	• Load impulse very small • Susceptible to errors if surface cracks exist • Requires supplementary moisture content test

Notes:

c) Portable FWD.

Figure 7.9. Portable FWD.

Device	Benefits	Drawbacks
PFWD	• Portable • Linkable to design values • Provides rapid results • Already adopted for acceptance testing by some agencies	• May not correlate 1:1 with FWD • Selection of target value may require calibration strip • Requires supplementary moisture content test

These NDT devices have been around for several years and they are very useful for project and forensic testing if concerns are raised about the uniformity or the effectiveness of stabilization. However they have not made it into mainstream specifications because of the following issues:

- The stiffness of stabilized layers increases substantially with time especially in the first few days after treatment, which is the time when QC measurements will need to be made.
- Different stabilizers have different rates of stiffness gain so it is difficult to set targets.
- The weather conditions impact rate of stiffness gain.
- Seating the gauges on the finished base sometimes gives repeatability problems with rough surfaces.
- DCP is not appropriate for cement treated materials more than 1 day old.
- It is difficult to define target values (as currently done for density). There is only a poor correlation between

lab and field stiffness. The lab stiffness are always substantially higher than field values.

d) Intelligent Compaction with instrumented Rollers.

In recent years a push has been made to use roller instrumented with sensors to monitor drum movement to get an indication of the in-place stiffness values. Figure 7.10 shows such a system.

Figure 7.10. Prototype Instrumented Roller.

The pavement layers under test have a "pogo-stick" influence on the roller drum in that the stiffer the support layers the more the movement of the drum; in fact in extreme cases the drum can bounce off the layer. In soft areas the drum imparts most of its energy into compaction, so its movements are less. Typically accelerometers are attached and the average drum movements over a certain length are computed and plotted. Figure 7.11 shows drum movements for 40-ft increments along a project. The areas of low drum movement are weaker areas, which may need other investigations.

One possible application of this technology may be in defining when to stop rolling of a section being compacted. If little or no change is noticed in the drum movement, then no further compaction of the layer is being achieved.

Notes:

Notes:

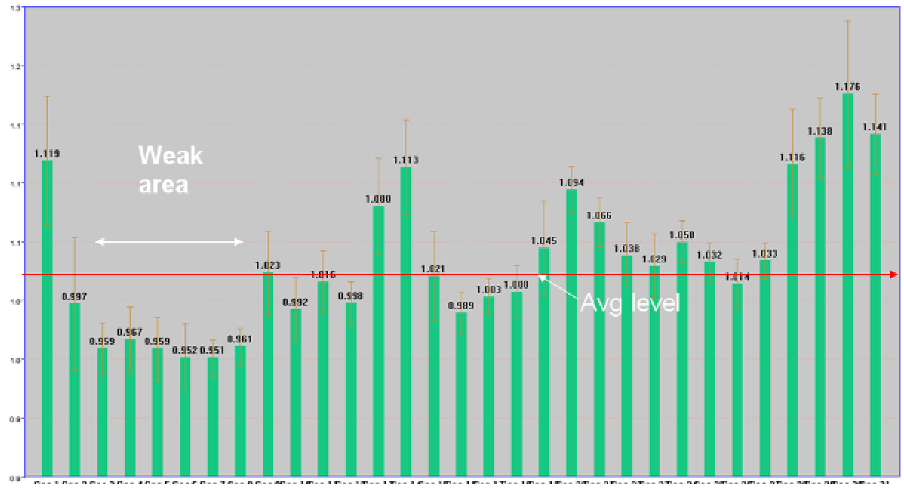


Figure 7.11. Output From IC Roller, Strength Estimated for Every 40 Ft of a Project.

The strength of IC is that it has the potential to:

- Provide 100% coverage.
- Tell the operator when no more compaction is being achieved.
- Identify localized weak spots.

The weakness of this system is that:

- The drum movements are strongly influenced by the strength of the underlying layers; therefore drum movements may tell little about the density or stiffness of the stabilized layer being compacted.

Section 7.3 Quality Assurance Testing with The FWD

The Falling Weight Deflectometer device is recommended to be used to check structural strength of FDR layers after compaction. This will provide TxDOT with the following critical information:

- Does the section have the same structural strength as that assumed in the structural design?
- Is the structural strength uniform along the project?

Figure 7.12 shows one of TxDOT's FWD units.

Figure 7.12. FWD Unit Testing on Top of a Base Layer.

Notes:

All pavement designs in Texas require the designer to specify a modulus for each layer in the pavement structure. The typical assumed design moduli values for stabilized layers were described in earlier sections of these notes. Upfront testing of the existing pavement before FDR is recommended to obtain an existing subgrade modulus value. Software is available within the Flexible Pavement Design system to compute the target deflection bowl for the as designed pavement, and Figure 7.13 shows an example of this software.

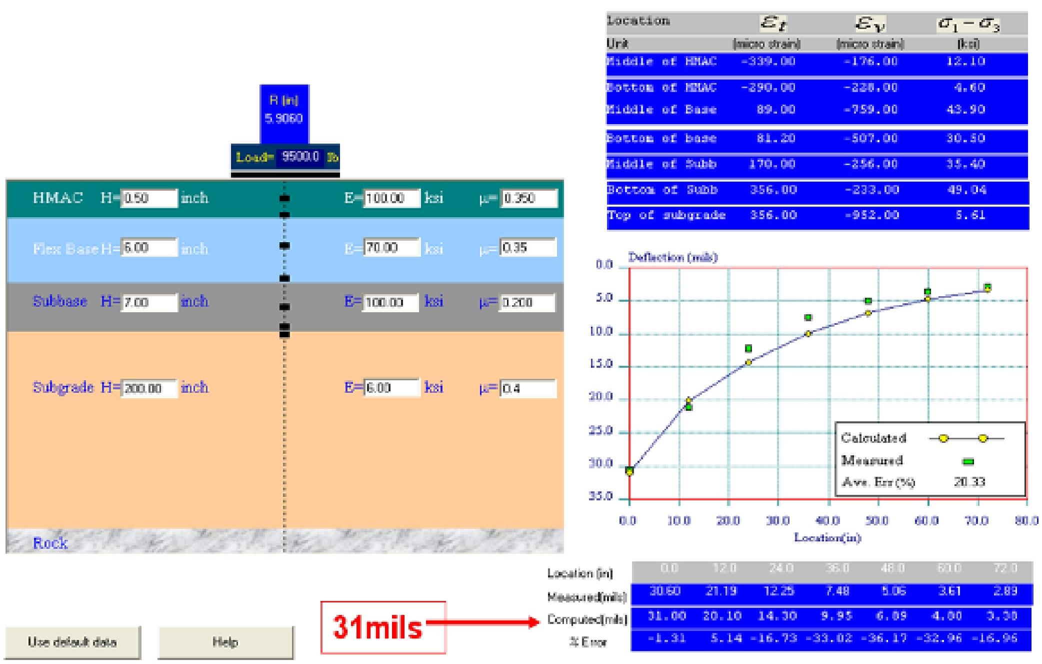


Figure 7.13. Software to Determine Target Modulus Values for FDR Section.

Notes: ________________

Structural testing of a new FDR project should be performed a minimum of one week after stabilization. Testing can be conducted on top of the layer or under a thin surfacing. Testing should be conducted on the first section completed on a project to ensure that no problems exist.

Figure 7.14 shows data from an ideal FDR project where the design maximum deflection at 9000 lb load was predicted to be 31 mils. FWD testing was conducted in the field at 200 ft intervals on top of the underseal placed over the treated layer. The measured values are shown as the blue line in Figure 7.14.

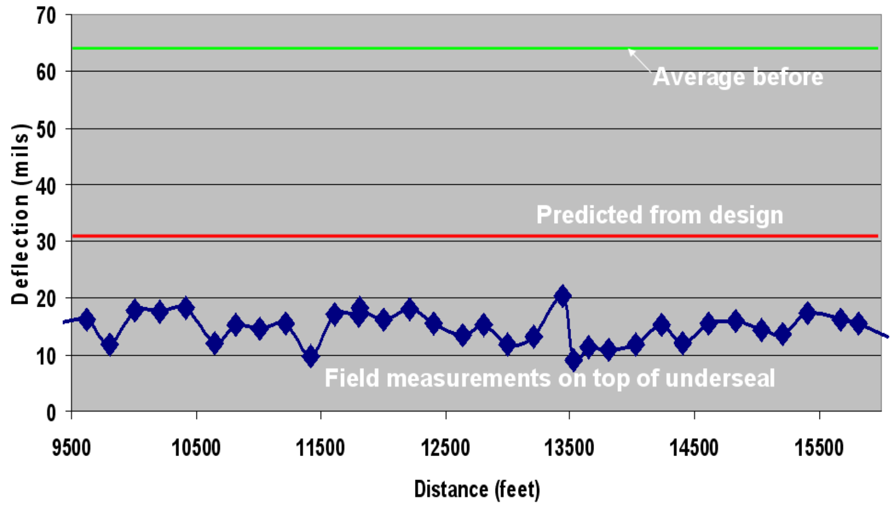


Figure 7.14. Deflection Patterns from an Ideal Case (Measured Deflections Less than Those Predicted Using Design Values).

In this project the design goals are clearly being met. The stabilizer is providing a stiff layer and very little variation in stiffness is being observed along the section. These results should be compared with the results shown below in Figure 7.15 for a different FDR project. In this case the FDR treatment is not stiffening the base layer. This is a case of either using the wrong stabilizer and/or poor construction practices. Obtaining data like this earlier in the project should call for construction to be suspended until the cause is identified and correction actions taken.

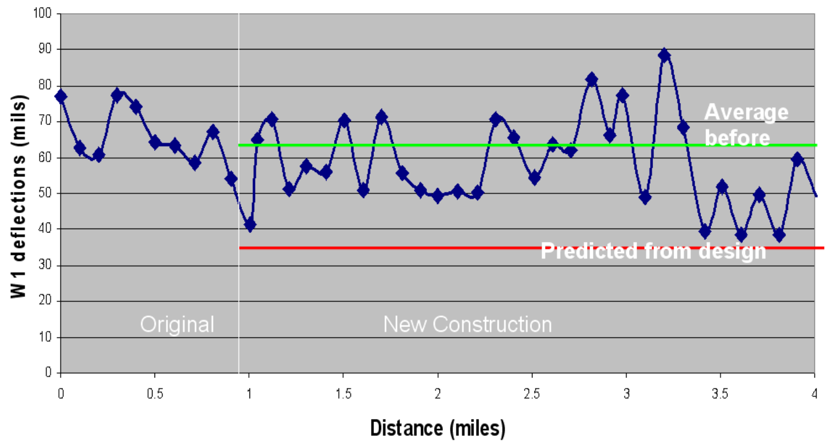


Figure 7.15. Deflection Patterns from a Problem Section 2 Months after Construction.

This Quality Assurance testing is recommended on all projects. It should be performed as soon as sections are completed to tell the agency and contractor if the treatment is providing a structurally sound pavement.

Notes:

CHAPTER 8 – EXAMPLE OF DESIGN REPORT

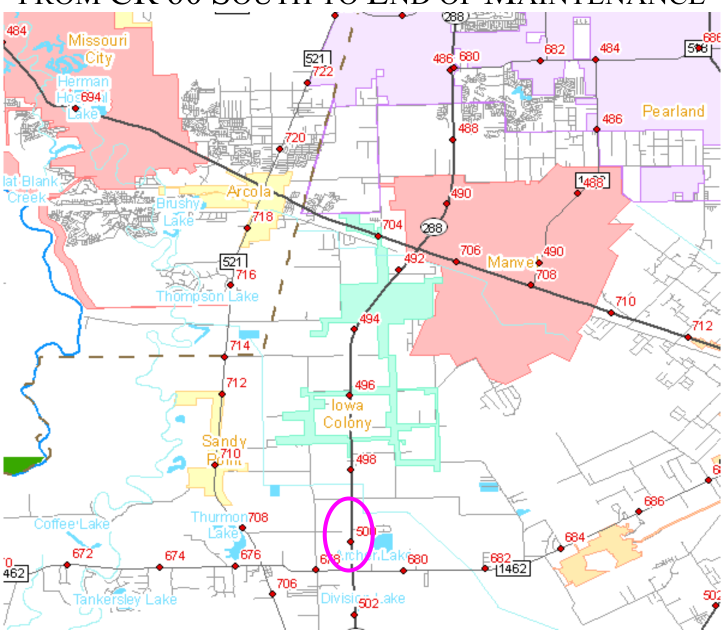
Texas Department of Transportation

At the end of this chapter you should be able to understand what factors must be included in typical FDR design Report.

The case study on the following pages was developed by Darlene Goehl, P.E., of the Bryan District. The final design thickness and recommended pavement structure are presented together with details of the proposed final surfacing. The laboratory test results are summarized. It also includes recommendations to aid in construction such as length of section that can be treated by a ton of cement.

Pavement Design Report			
Highway:	SH288 EFR	CSJ:	Maintenance
County	Brazoria	Limits:	CR 60 South to End Maintenance

PAVEMENT DESIGN REPORT
FOR
SH 288 EAST FRONTAGE ROAD
MAINTENANCE PROJECT
BRAZORIA COUNTY
FROM CR 60 SOUTH TO END OF MAINTENANCE



PROPOSED PAVEMENT DESIGN:

2nd course
- Asphalt – AC12-5TR or AC20-5TR or AC20-XP estimated at 0.42 gal/sy
- Aggregate – Ty PL or Ty PB, GR4 estimated at 1cy/125sy

1st course (directly on cement treated base layer)
- Asphalt – CRS-2 or RC 250 estimated at 0.25 gal/sy (only use CRS-2 during warm/hot weather)
- Aggregate – Ty L or Ty B, GR5 estimated at 1cy/135sy

10" Cement Treat (estimated at 3.0% by weight) Existing Pavement blended with new material
- blend 4" additional base, either GR 2 crushed limestone or recycled crushed concrete with existing.

RECOMMENDED FOR APPROVAL: APPROVED:

DARLENE C. GOEHL, P.E. DATE	MICHAEL W. ALFORD, P.E. DATE
TRANS ENGR SUPVR (PE SERIAL NUMBER: 80195)	DEPUTY DISTRICT ENGINEER

Pavement Design Report			
Highway:	SH288 EFR	CSJ:	Maintenance
County	Brazoria	Limits:	CR 60 South to End Maintenance

Proposed Pavement Design:

Design Information - Modified Triaxial Design						
	Traffic Data					
	Current ADT 2008	20 yr adt	% Trucks	18k ESAL Flex	ATHWLD	
	270	380	3.2		10000	
Triaxial Class estimated from Soil Data – Worst Case 5.6 -- Usual 5.0	Thickness of Better Material (in)	Total Thickness Existing Material (thinnest) (in)	**Total Needed w/ (cement trt existing) (in)**	Estimated Depth of Reworked Existing material (in)	New Base Req'd – No treatment (in)	**New Base Req'd – Cement trt existing (in)**
	(SOP Design Method for Construction Contract)					
Worst Case	21.2	6.5	**14.3**	6.5	14.7	**7.8**
Usual	17.9	6.5	**12.4**	6.5	11.4	**5.9**
	(SOP Design Method for Maintenance)					
Worst Case	14.7	6.5	**10.7**	6.5	8.2	**4.2**
Usual	12.3	6.5	**9.6**	6.5	5.8	**3.1**

TTI performed the laboratory tests for this project. The existing material was blended with three types of material.

1. GR 2 crushed limestone Flexible Base from Colorado materials.
2. RAP supplied from a stockpile in the Houston District.
3. Stockpiled crushed concrete base from Houston District.

All three materials will work when blended with the existing pavement; however there are locations on the existing roadway with thick ACP patches. Additional RAP should not be used in these locations. Use either GR 2 crushed limestone or crushed concrete to blend with the existing material.

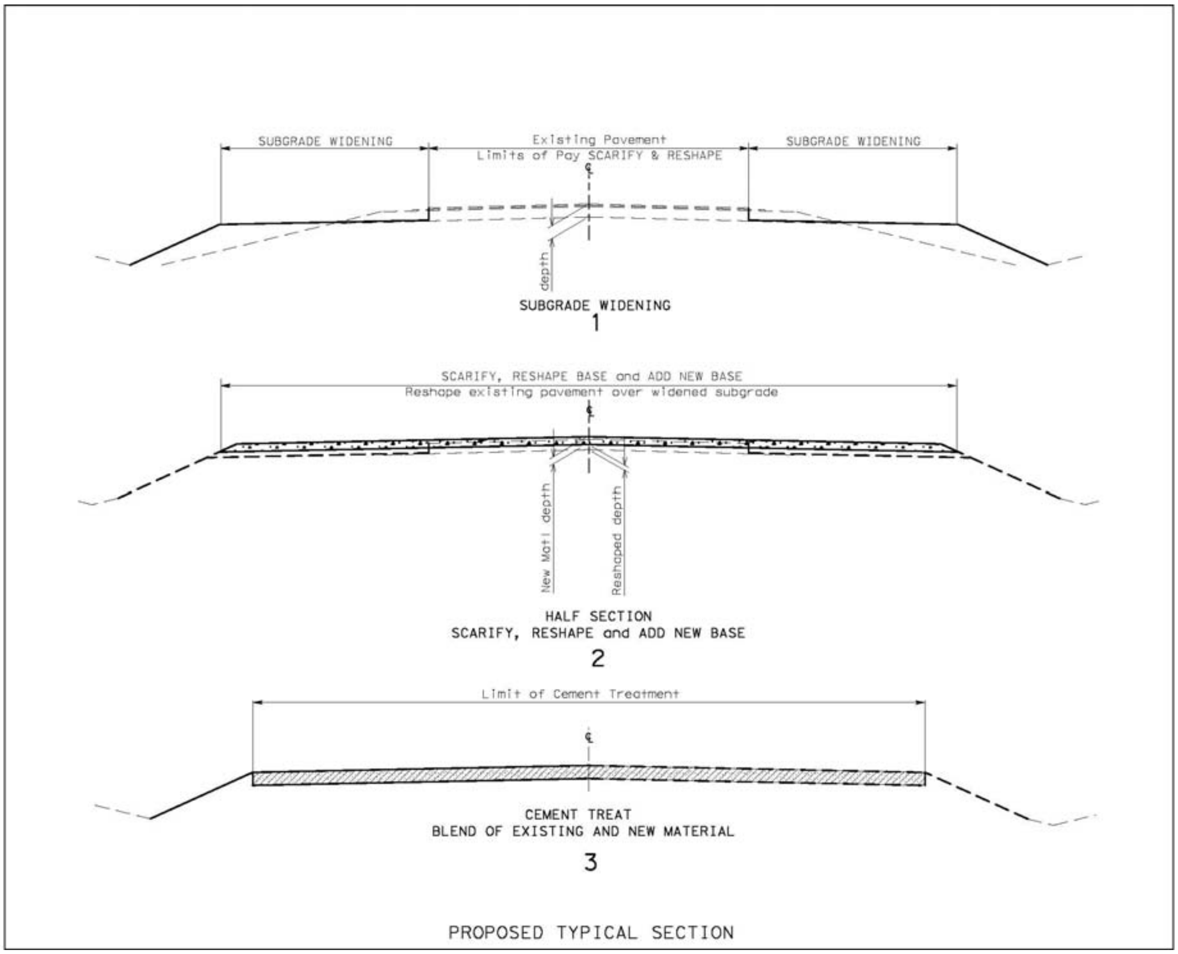
The usual thickness of existing material is 6.5" and ranges from 6.5" to 13.5". The subgrade is a mildly expansive black clay with PIs ranging from 23 to 33. I recommend reworking the existing material and widening the existing pavement to at least 24 ft, then adding enough additional material to treat a 10" thick blend of existing and new material with 3% cement by weight.

Cement $/ton	Cement $/sy	Treat $/sy	**Cement trt total $/sy**	Hot Mix $/ton	**2" lift $/sy**
$ 110.00	$ 1.55	$ 3.30	**$ 4.84**	$ 61.00	**$ 6.71**

Note: Cost is based on Houston District 12 month average low bids for Construction.

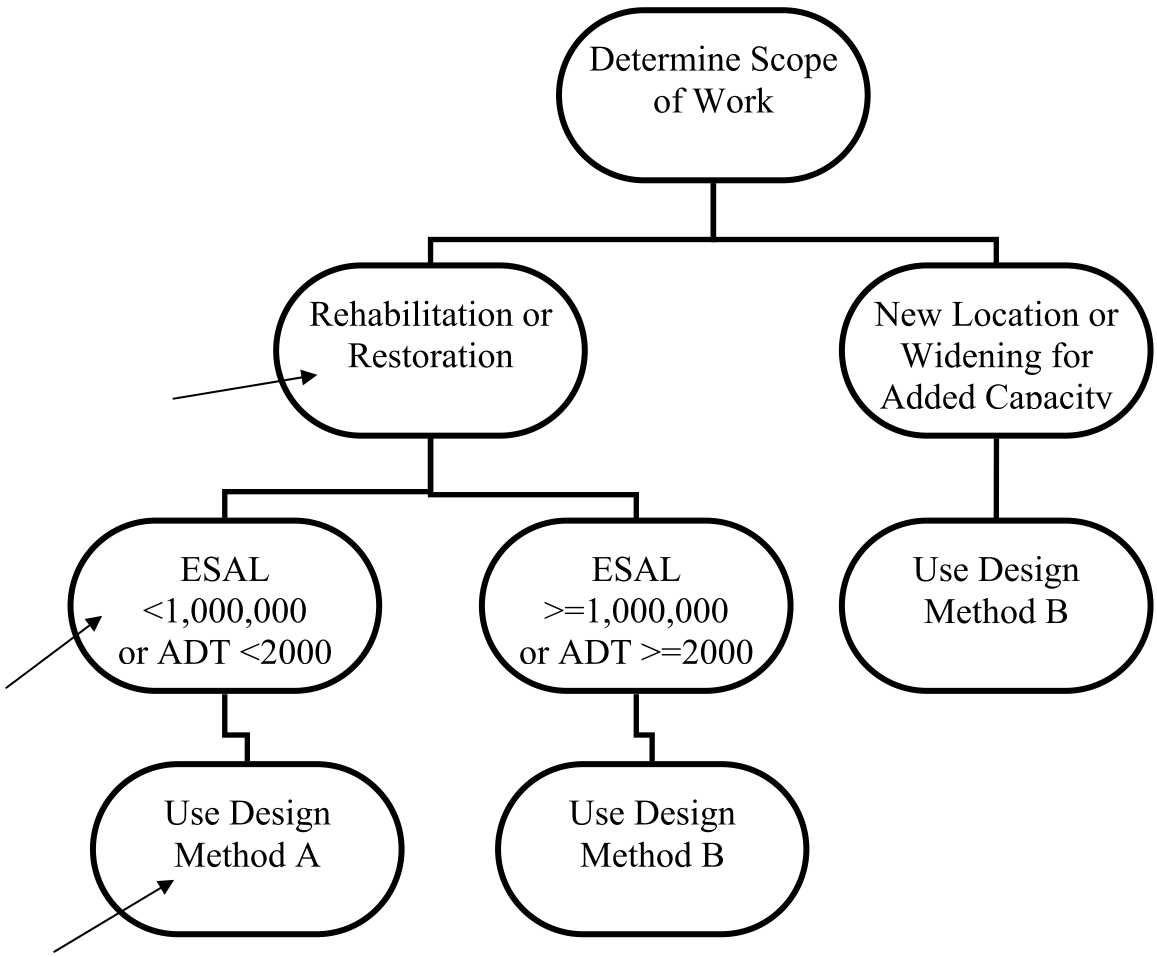
Est. Unit Weight	125	pounds per cubic foot	rate placed	3.125	pounds/sf
Percent cement	3	percent	rate placed	0.0141	tons/sy
Treated Width	12	feet	Length per ton	53	feet

Pavement Design Report			
Highway:	SH288 EFR	CSJ:	Maintenance
County	Brazoria	Limits:	CR 60 South to End Maintenance



Pavement Design Report			
Highway:	SH288 EFR	CSJ:	Maintenance
County	Brazoria	Limits:	CR 60 South to End Maintenance

Bryan District SOP 03-09 Pavement Design Criteria:



Use design Method A.

Use FPS19 and the Load Zone/10 year Modified Triaxial Check (or use ½ of ATHWLD in FPS program automated modified triaxial check). For the Modified Triaxial Check, do not use the 1.3 load adjustment factor based on greater than 50% tandem axles in the ATHWLD (based on TxDOT Research Report 0-4519-1). Refer to Table A1 for typical inputs for these programs.

Texas Department of Transportation

Pavement Design Report			
Highway:	SH288 EFR	CSJ:	Maintenance
County	Brazoria	Limits:	CR 60 South to End Maintenance

Design Method A

Table A1 - Design Method A

Parameter	Range	Usual Input FPS19w	Comments
Time to 1st Overlay (years)		10	May be lower for maintenance projects
Initial Serviceability Index (SI)	3.8–4.0	3.8	
Future Overlay – Initial SI	4.2–4.5	4.2	Future Overlays are not anticipated therefore use the conservative value
Minimum SI	2.0–2.5	2.5	
Design Confidence Level	A (85%)–B (90%)	B (90%)	
District Temperature Constant	30–31	30–31	Use default value in FPS program.
Selling Potential, PVR swelling rate	0–100%	0%	Do not use swelling potential as an input to FPS.
Detour (Road User Cost)	Posted speed and expected speed during overlay	Use same speed for all traffic speed entries and detour Model 3	Does not affect the pavement structure. Eliminates user costs associated with traffic delays for future overlays.
Material Cost per Cy		Use District Specific costs.	Monitor Bid Tabs and adjust accordingly

Material Description	Modulus Value	Poisson's Ratio	Cohesiometer Value for MT check
Existing Material (including Subgrade)	Modulus Back-calculated from FWD data	0.35	na
Existing Pavement – Scarified, Reshaped and Compacted	Approximately 3 times the subgrade modulus	0.35	na
Stabilize Exist Pav/Subgrade **a) mostly granular base (75% or more base)** **b) blend subgrade & base (50% to 75% base)** **c) mostly subgrade (<50% base)**	a) 100 ksi b) 65 ksi c) 35 ksi	a) 0.3 b) 0.3 c) 0.35	a) 800 b) 650 c) 300
New Flexible Base	GR 2 = 50 ksi	0.35	na
Cement Treated Base UCS>210, with 85% retained strength	150 ksi	0.25	1000

Note: the design Modulus values are for materials typically used in the Bryan District. These values may changed with future testing and changes in material suppliers. The range for the stabilized subbase and flexible base over stabilized subbase is dependent upon the amount of existing base/rap material in the stabilized section.

CHAPTER 9 – TROUBLE SHOOTING FDR PROJECTS

Notes:

The vast majority of the FDR projects are performing well and the process is being used by more and more Districts. However as with all paving projects performance problems can occur when they do it is recommended that a forensic study be initiated to identify the cause, required corrective action, and what steps are needed to minimize the re-occurrence on future projects.

The six examples shown below highlight what can go wrong and what recommendations are made to avoid this in the future.

1) Longitudinal Cracking

By far the most comment problem associated with FDR projects in East Texas is longitudinal cracking.

Figure 9.1. Longitudinal Cracking on FDR Projects.

The causes of this distress are associated with:

- Highly plastic subgrade soils (PI > 35), which shrink excessively during summers.
- Steep side slopes.
- Trees down the sides of road, which cause additional soil drying.
- Stiff stabilized bases—stiffer the base, the more severe the cracking.

In the late 1990s, the Bryan District initiated a design medication in these problematic areas by incorporating a layer of geogrid over

Notes:

the treated base, on top of which was placed at least 6 inches of flexible base and a thin surface layer. The geogrid acting as a slip plane has greatly minimized this problem.

This problem was most severe when higher cement contents were used. However the current specs permit much lower 7 day design strengths such as 175 psi. These lower values have performed well and certainly have less shrinkage cracks than earlier higher strength designs.

2) Inadequate Stabilization

The failure below is a result of adding insufficient cement during construction, the design called for 3% but upon checking only 1% was added. Compounding this problem is the presence of a longitudinal joint close to the edge of the wheel path, which is probably weeping water into the base layer.

Figure 9.2. Under-Stabilization.

It is critical to have construction personnel check the application rate of stabilizers. Simple spreadsheets are available based on unit weights, application rates, etc., to compute the length of application for 1 ton of cement. This is a construction problem that is fairly simple to solve.

3) Bonding Failure

This failure is found with all stabilizer types but it is most commonly reported with fly-ash stabilization.

Figure 9.3. HMA Bonding Problems over Stabilizer Base.

Notes:

This problem is often related to:

- In-effective prime coat, which lacks penetration.
- Dirty or unstable stabilized layer surface.

If this is a persistent problem then a lab study using the pull-off device described in Chapter 5 is recommended to select the best prime coat material and the optimal amount.

4) <u>Shrinkage Cracking from CTB Layers</u>

This was a common problem several years ago and it is caused by adding too much cement; under curing recommendations this shrinks and cracks in typically block patterns.

This problem has been greatly reduced in recent years with the changes to the specifications and construction practices. Several years ago the target CTB strength was 500 psi; in recent years this has reduced to 300 psi or 175 psi with the 2004 specifications book. In addition the early application of traffic, as many of the FWD sections are opened to traffic early, or microcracking.

Notes:

Figure 9.4. Block Cracking.

5) <u>Non Uniform Distribution of Stabilizers</u>

Extreme failures of the type shown in Figure 9.5 are very rare but they have occurred. This is largely to do with constructing FDR projects on narrow roadways under traffic, where one lane has to continually be in operation and barriers are used to protect construction workers. In these rare instances a strip of roadway never gets full treatment with the stabilizer of choice. The situation is compounded by having a longitudinal construction joint in the HMA layer directly over the untreated base.

Figure 9.5. Failure in One Wheel-Path Only.

6) Very Early Load Associated Distresses

The situation shown in Figure 9.6 can occur if the guidelines provided in this workshop are not followed. In some cases the existing materials are clay contaminated and if this is the case some type of lime treatment will be required to attain the required laboratory strengths. In the case shown an asphalt emulsion treatment was applied to a roadway with little base. The FWD deflections were found to be very high and roadway cores disintegrated.

Figure 9.6. Alligator Cracking and Rutting a Few Months after Construction.

Notes:

APPENDIX:
DETAILS OF LAB TEST PROCEDURES
ON SAMPLE PREPARATION

A. Preparation of the Base Material for Testing

Day 1	1. Thoroughly mix the material originating from a single sampling location, spread it out on the floor, and let air-dry overnight.		
Day 2	2. Collect representative samples of the air-dried material to determine: • The baseline (air-dried) moisture content of the virgin material; • The particle size gradation of the virgin material; • Plasticity index of the virgin material. 3. Prepare material batches (~ 8,000 g) by adding the desired amount of water and thoroughly mixing.		
Day 3	4. Cover and seal each batch with foil. 5. Weigh each covered batch and record the mass in order to monitor the weight loss due to involuntary water evaporation. 6. Let the batches sit overnight (12 hours). 7. Weigh each batch to check for the possible water loss. Replenish the evaporated moisture.		

TABLE A. Preparation of Materials for Testing.

B. Compaction of the Base Material Specimens

Day 1	1. Prepare the base material according to **(A)**. 2. If necessary, mix additives into the batches, following the additive-specific mixing procedures. 3. Set-up lab equipment to compact the base specimens according to the Tex-113-E procedure. 4. Weigh an empty 6" × 8.5" mold; record its mass.		
–	5. Compact the 6" × 8" specimens in 4 layers using the standard compaction effort (Tex-113-E): 10-lb hammer, 18-in drop, 50 blows/layer. 6. Scarify the surface of each internal layer with a spatula to facilitate bonding between the compacted layers. 7. Finish off the final surface of each specimen using 10 firm blows of a rawhide hammer.		
–	8. Weigh the compacted specimen in the mold and record their combined mass. 9. Extrude the compacted specimen from the mold using the hydraulic press. 10. Determine the height of each specimen using a ruler to the nearest 0.05 inch.		

TABLE B. Compaction of Base Samples.

C. Determination of the Optimum Moisture Content of the Base Material

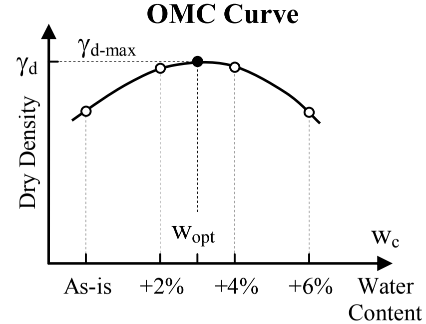
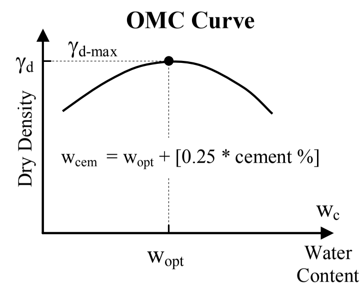
Day 1	1. Prepare the base material according to **(A)**. Namely, prepare four batches (8,000 g) by adding the following amounts of water to the base material: • As-is (air-dried, no additional water); • +2% (160 g) tap water; • +4% (320 g) tap water; • +6% (480 g) tap water. 2. Compact the specimens according to **(B)**.		
–	3. Label and weigh the empty drying bowls. 4. Place the compacted specimens into the corresponding labeled drying bowls, break them to promote drying, and weigh along with the bowls. 5. Place the bowls holding the wet broken-up specimens into the oven adjusted at 85°C (185°F). The lower than recommended drying temperature of 110°C (Tex-113-E) is suggested due to the presence of the RAP in the base material.		
Day 4	6. After approx. three days of drying (weekend), record the stabilized weight of each drying bowl. 7. Use the collected data to construct the OMC curve and determine: • Optimum moisture content; • Maximum dry density; • Original (as-is) moisture content of the base material after air-drying.		<u>Determine:</u> OMC (w_{opt}) % $\gamma_{dry\text{-max}}$ lb/ft^3 Original water %

TABLE C. Determination of Optimum Moisture Content.

D. Preparation of the Cement Stabilized Base Specimens

–	1. Determine the OMC of the base material according to **(C)**. 2. Adjust the OMC value: • For the original (as-is) water content; • For inclusion of cement. 3. Prepare the base material batches according to **(A)**, using the adjusted optimum moisture content.		
Day 1	4. Calculate the desired amount of cement, defined as a percent of the total dry solids. 5. Weigh out cement and thoroughly mix it into the wetted base material. 6. Compact the cement-base mix according to **(B)**.		
Day 8	7. Place the compacted specimen on the porous stone, wrap into a plastic bag, and cover with another porous stone. 8. Move the specimens into a climate chamber set at 25°C (77°F) and 100% relative humidity. 9. Cure the specimens in the chamber for 7 days.		

Texas Transportation Institute

Full-Depth Base Reclamation and Rehabilitation for TxDOT

1

TABLE D. Preparing Cement Treated Base Samples.

E. Preparation of the Base Specimens Stabilized with the Emulsion-Cement Mix

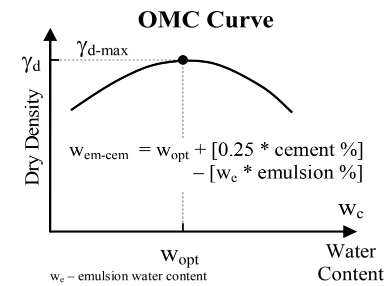
–	1. Determine the OMC of the base material according to **(C)**. 2. Adjust the OMC value: • For the original (as-is) water content; • For inclusion of cement; • For water contained in emulsion. 3. Prepare the base material batches according to **(A)**, using the adjusted optimum moisture content.		
Day 1	4. Transfer the prepared base material into the bucket of an electrical mixer. 5. Calculate and weigh an appropriate amount of cement, defined as a percent by mass of the total dry solids. 6. Add the weighed cement to the base material in the mixer and mix thoroughly.		
–	7. Shake the bottle containing emulsion first. 8. Calculate and weigh an appropriate amount of emulsion, defined as a percent by mass <u>in addition</u> to the total dry solids. 9. Pour the weighed emulsion into the mixer in addition to the blend of the base material and cement.		

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TABLE E. Preparing Emulsion Treated Base Samples (Page 1 of 2).

E. Preparation of the Base Specimens Stabilized with the Emulsion-Cement Mix - CONTINUED

–	10. Mix for no more than 60 ± 10 seconds. 11. Place the loose mixture into a bowl. 12. Move the blended specimens into an oven and cure at 60°C (140°F) for 30 minutes. Do not mix during curing.		
–	13. Compact the cured mixtures according to **(B)**. 14. Place the compacted specimens on the porous stones.		
  Day 3 Day 4	15. Move the specimens into a climate chamber set at 60°C (140°F). 16. Cure the specimens in the chamber for 48 hours (2 days). 17. Remove the specimens from the hot chamber and cool them at 25°C (77°F) for 24 hours (1 day), but not more than 48 hours (2 days).		

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TABLE F. Preparing Emulsion Treated Base Samples (Page 2 of 2).