Pavement Design Guide

Texas Department of Transportation

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Manual Notice 2011-1

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Manual: *Pavement Design Guide*

Effective Date: January 01, 2011

Purpose

This manual notice is intended to transmit changes to the content of the *Pavement Design Guide*.

Changes

Chapter 1, Section 1, Guide Overview

Under subsection, "Purpose," added wording to the end of the first sentence to identify the projects described in this manual as "traditionally-let."

Chapter 2, Section 1, Introduction

- Added information on preliminary pavement design, pavement design concept conference, and pavement design standard operationg procedure.

Chapter 2, Section 2, Pavement Design Standard Operating Procedure (SOP)

- Added new section on pavement design standard operating procedure.

Chapter 2, Section 3, District Pavement Engineer's Role

- Information was moved from Chapter 2, Section 2.

Chapter 2, Section 4, Pavement Types

- Information was moved from Chapter 2, Section 3.
- Figure 2-2. Generalized perpetual pavement design was updated. The "Construction Considerations" document was updated.

Chapter 2, Section 5, Pavement Type Selection

- Information was moved from Chapter 2, Section 4.
- Under subsection, "Principal Factors," added information to the first paragraph describing alternate bidding for higher volume highways and recommended screening criteria for an alternate bid contract.
- The new policy for alternate pavement design was added.

- Under subsection Life-cycle Cost Analysis (LCCA) the term tool is replaced with an analysis in the third paragraph. In the last paragraph is modified to mention LCCA is only one of many processes.

Chapter 2, Section 6, Approved Pavement Design Methods

- Information was moved from Chapter 2, Section 5.
- Under subsection, "Introduction," modified the bulleted list: combined the AASHTO design methods from two items into one and removed the experience bullet.
- Under subsection, "Flexible Pavement System - Window version (FPS-19W)":
  - in the first paragraph, added information about the preferred method for designing flexible pavements on higher volume highways (>10,000 ADT, 5 M ESALs) and added information about the availability of design procedure training
  - in the third bullet, added recommended data value to enter into the program for Central Texas climate
  - in the seventh bullet, added reference to graphs contained in the archived version of "Tex 117-F, Triaxial Compression for Disturbed Soils and Base Materials"
  - added bullet about the use of a mechanistic design check
- Added new subsection, "Other Acceptable Design Procedures":
  - Moved three paragraphs about the FPS-11 and 1993 AASHTO design procedure from "Flexible Pavement System - Window version (FPS-19W)" and created a new section. Added information on Pavement Design staff availability to consult on AASHTO design procedures.
- Under subsection, "Basic Design Types," added information on Type 4 design.
- Under subsection, "AASHTO Design Procedure":
  - title changed to "AASHTO Design Procedure (for flexible and rigid pavement designs)"
  - under second level subsection, "Rigid Design," added projected retirement and scheduled replacement of DARWin™ 3.10
- Added new subsection and corresponding content for, "Experience-Based Design.."

Chapter 2, Section 7, Pavement Design Categories

- Information was moved from Chapter 2, Section 6.
- Updated Figure 2-8. Design process for a new pavement or full reconstruction of pavement
- Updated Figure 2-9. Design process for flexible pavement rehabilitation.
- Updated Figure 2-10. Design process for rigid pavement rehabilitation.

Chapter 2, Section 8, Pavement Design Reports

- Informaton was moved from Chapter 2, Section 7.
- Under subsection, “Traffic Loads”:
  - Under second level subsection, “Tire Loads,” deleted the equation and equation definitions
  - Under second level subsection, “Traffic Projections”:

    added note about 20 yr. traffic projections and rigid structures

    added instructions to use Form 2124, Request for Traffic Data, for design purposes

    added “caution” note to complete Form 2124 for quick turnaround and other data options

    added paragraphs labels, “Units of Measurement” and “Loading Estimates” to existing content

    added new information labeled, “Projections and Special Factors.” This section highlights Form 2124, Request for Traffic Data, and includes instructions to complete the form. Also included, other tools recommended to verify or determine traiffic data.
- Under subsection, “Serviceability Index,” updated Figure 2-12. Pavement Performance Relationships
- Changed the status of “Reliability (confidence level)” from second level subsection to first level subsection.
- Under subsection, “Reliability (confidence level)”:
  - added the types of pavements the department reliability of 90 to 95 % is usually applied
  - added information about the type of pavements lower reliability may be applied
- Under subsection, “Drainage Characteristics”:
  - in the second level subsection, “Internal (Positive) Pavement Drainage,” added a paragraph label, “Department Policy,” to the sentence beginning, “Aspects of the department’s policy are evident. . .”

Chapter 2, Section 9, Pavement Design Reports

- Information was moved from Chapter 2, Section 8
- Under subsection, “Pavement Design Report and Other Documentation”:
  - added new bullet item on PVR mitigation
  - added new bullet item on identification of the base grade chosen
  - added new second level bullet about alternate pavement design to the bullet item, “Design input values and output.”
- Under subsection, “Completing the Pavement Design Report”:
  - In the table, “Completing the Pavement Design Report,” added bullet to step 1 to provide information on District Engineer’s responsibility. In step 2 of this table, actions were

added and defined for district staff, district pavement engineer (DPE), and District Engineer (DE).

Chapter 3, Section 2, Geotechnical Investigation for Pavement Structures

- Under subsection, "Preliminary Investigation":
  - In the second paragraph starting, "Site inspections are frequently conducted. . ." changed text to encourage frequent site inspections in this stage.
  - In Table 3-1: Existing Data Sources, under column "External Data Sources," changed the bullet order from second level to first level for items: Commercial Internet and Topographic. Change the bullet item, "university library files," to "engineering library files" and changed the bullet order for this item from second level to first level.
  - Under Table 3-2: Reconnaissance Areas of Interest:, in the bullet named, "Surface Soil Exploration," after the last second level bullet in the list, added reference to the test procedure required if sulfur-bearing compounds are detected in the soil.
  - In the second level subsection, "Preliminary Evaluation," under bullet, "Structural Support," added information on the condtions to use triaxial classification
- Under subsection, "Subsurface Exploration":
  - in the second level subsection, "Material Evaluation," under bullet, "Swell potential":

    added guidance and reference information on the use of potential vertical rise (PVR)

    added the recommended test procedure for determining PVR

    added recommendations for best conditions to determine PVR

    added specific criteria that determine whether a pavement structure design proposing to include PVR reduction strategies will require Construction Division review and approval or review only

    added a list of items pavement designers should address in their submission for review and approval in conjunction with the pavement design.

    added criteria that would allow designs incorporating PVR mitigation.

Chapter 3, Section 3, Flexible (Unbound) Base Selection

- Added new reference, "Flexible Base Selection Guide," along with corresponding description of the guide and its contents. Added reference to memo, "Use of Grade 1 Flex Base," along with the memo's effective date. Describes conditions when use of Grade 1 flexible base will require justification.
- Added new subsection, "Flexible Base Description."
- Under subsection, "Flexible Base Description":
  - moved information referencing Item 247 from "Flexible (Unbound) Base Selection." The number of different grades changed from four to five. Added Grade 5 to the sentence, "Grade 1 and Grade 2 base materials are the primary base material grades for base course

performing as a structural layer in a pavement structure." Added guidance on use of Grade 4.

- reorganized and formatted information of each grade of flexible base, from 1 through 5, into a bullet list. Added new information describing the characteristics and best use of each grade.

- Added new subsection, "Base Selection."

Chapter 3, Section 5, Hot Mix Asphalt Concrete Pavement Mixtures

- Under subsection, "General":
  - added temperature parameters for creating warm mix asphalt (WMA)
  - added information on benefits of using warm mix asphalt (WMA)
- Under subsection, "Use of Perpetual Pavements":
  - reformatted the information about stone matrix asphalt. Added conditions when RBL should be considered.

Chapter 3, Section 8, Hydraulic Cement Concrete

- Reformatted the information in this section and created four subsections:
  - Primary Ingredients
  - Determining Ingredient Proportions
  - Creating Workability, Durability, and Adequate Strength - Changed wording for the sentence, "For the concrete to have good workability, durability, and adquate strength, two conditions are to be met:" to ". . . two condtions must be met:"
  - Three Concrete Classes - reformatted the information for concrete classes P, K, and HES into a table.

Chapter 3, Section 9, Geosynthetics in Pavement Structures

- Under subsection, "Geosynthetics for Geotechnical Reinforcement," in the third paragraph, removed "and will concluded in late 2006." from the sentence beginning "Addtional research on the contribution of geogrid. . . "

Chapter 5, Section 1, Overview

- Reorganized the information

Chapter 5, Section 2, Types of Flexible Pavements

- Under subsection, "Types of Hot Mix Asphalt-Surfaced Pavements," in the last sentence of the last paragraph, added "cement" to the list of typical stabilizers.
- Under subsection, "Perpetual (HMA) Pavement":
  - removed reference to "heavy duty" mixes and its uses

- added conditions for the use of stone matrix asphalt and the approval required for this substitution.

Chapter 5, Section 3, FPS-19W Design Parameters

- Reorganized the information and created two subections, Program Tools and Data Input Components.
- Under subsection, “Program Tools,” added information to indicate the manual information supersedes the guidance provide on the program “Help” screen.
- Under subsection, “Data Input Components,” added new table, Table 5-1: FPS-19W Design Input Requirements.
- Under subsection, “General Inputs”:
  - in the second level subsection, “Program Main Menu,” the information was reformatted to numbered list. A caution note to prevent program crashing was added
  - in the second level subsection, “Basic Design Criteria (Card #3),” and in the paragraph named, “Analysis Period,” corrected format of the caution note.

    In the paragraph named “Minimum Serviceability Indes,” changed the value of ESALs for highways of higher importance from “exceeds 1,000,000” to “5 M.” For the highways with moderate traffic, changed the ESAL value from “500,000 to 1,000,000” to “1 M to 5 M ESALs.” Changed the definition of low volume highways from “less than 1,000 vehicles per day (vpd)” to “less than 3,000 vehicles per day.” Changed the value of cumulative ESALs from “less than 500,000” to “less than 1,000,000.” Changed the terminal serviceability of low volume highways from “2.5” to “2.0 to 2.5”

    Updated Figure 5-1. Added an interpretation of Figure 5-1. Pavement Performance Relationships. Added information on factors that shorten the serviceability over time and how it can be restored.

    In the paragraph named, “Design Confidence Level,” added confidence level recommendations by ESAL values.
  - Table 5-2. Output Variations was originally Table 5-1. This table’s contents were completely updated. Under column header, “Traffic Volume,” a “High” designation was added and each traffic volume level was defined by an ESAL value. Column headers, “Initial SI” and “Termin SI” were added. Column headers, “Initial Performance Period” and “Overlay” were modified with “T1” in the units of measurement.
  - in the second level subsection, “Program Controls (Card #4),” defined output pages further
  - in the second level subsection, “Construction and Maintenance Data (Card #7),” in the paragraph named, “Initial Serviceability Index,” added recommended index values based on pavement condtions.

In the paragraph named, “Serviceability Index after Overlay,” changed the index value of 2-3 in. overlays from “4.0” to “4.0 - 4.2.” For thick overlays, changed the index value from “a higher value” to “up to 4.5.”

- in the second level subsection, “Detour Design for Overlays (Card #8),” added caution note to prevent program crashing. Added Pavement Design Task Force’s recommendation for removing possible cost bias in accounting for user costs on low- to medium-volume highways.

- Under subsection, “Traffic Inputs”:
  - added reference to internal Form 2124, Request for Traffic Data, for design traffic requests. Added guidance noted by the Pavement Design Task Force (PDTF, 2009) to districts to review traffic analysis for highway design report to verify data reasonableness.
  - in the second level subsection, “20-Yr. 18-kip ESALs (One Direction), added information that describes when lane distribution factors are appropriate
  - in the second level subsections, “Average Speed in the Overlay Direction” and “Average Speed, Non-overlay Direction,” added recommended value to enter to avoid computation of associated user costs in the overall estimate of project costs.
- Under subsection, “Environment and Subgrade”:
  - updated the information on this feature’s effectiveness and added best use of this feature. Added recommendation of the use of Tex 124-E, Determining Potential Vertical Rise, for evaluation of mitigation of soil movement. Added reference to Chapter 3, “Materials Investigation and Selection Information,” for more information on approval requirements for soil movement mitigation.
  - in second level subsection, “District Temperature Constant,” added default value which corresponds to a Central Texas value. Added specific guidance for use of the default value in program FPS-19.
  - in second level subsection, “Swelling Probability,” added default value and provided conditions when the default value should not be used
  - in second level subsection, “Potential Vertical Rise,” explained the consequence of using zero in “Swelling Probability” and added reference to Chapter 3, Section 2, Geotechnical Investigation for Pavement Structures, for designs to mitigate potential rise and provides conditions on how and when it should be used.
  - in the second level subsection, “Swelling Rate Constant,” added the effect of this field when zero is entered for Potential Vertical Rise or for Swelling Probability.
- Under subsection, “Material Parameters”:
  - in the second level subsection, “Modulus, E (ksi),” in Table 5-3: Recommended Material Design Modulus Values, the following changes were made:

    Table 5-3 was moved from Table 5-2.

Material Type name, Dense-graded Hot (Warm) Mix Asphalt, was updated by adding "(Warm)." For the Dense-graded Hot (Warm) Mix Asphalt, the Design Modulus content and values were modified.

Material Type name, Permeable Friction Course, the Comment field was updated.

Material Type, Performance Design Mixtures, the Design Modulus values were modified. The Comment field was modified, included changing the ksi value from 350 ksi to 500 ksi.

Material Type, Stone-Matrix Asphalt, the Design Modulus content was modified.

Material Type, Flexible Base, modified Comment field.

Material Type, Cement Stabilized Base, modified Comment field.

Material Type, Lime or Cement Stabilized Subgrade, modified Comment field.

Material Type, Subgrade, modified Poisson's Ratio and Comment fields.

- in the second level subsections, "Minimum Depth" and "Maximum Depth," modified the last bullet starting, "For subgrade, use the average depth to bedrock. . ."
- in the second level subsection, "Poisson's Ratio," added more guidance for selecting a value to enter
- in the second level subsection, "Check (CHK) Column," changed the designated job title from "responsible engineer" to "project engineer" for the last sentence starting, "Leeway is granted to. . . "

◆ Under subsection, "Modified Texas Triaxial Check":

- part of the information was reformatted into step-action Table 5-4: Texas Triaxial Design Check. Steps 6 and 7 are new content.
- the other part of the information was reformatted into second level subsection, "Subgrade Triaxial Class Number." Added details about specific archived versions of the Tex-117-E reference. A table was created to display information on the design check routine.

Chapter 5, Section 4, Pavement Detours and Pavement Widening

◆ Under subsection, "Pavement Detours":

- content was reformatted, included the addtion of a bullet list for some content. Added information on estimating traffic loading when a detour is in effect and provides guidelines to request traffic data for detours.
- in second level subsection, "Structural Design of Detours," added suggestion to use the falling weight deflectometer (FWD). Added proven design strategies the districts may employ. Added an alternative option of the modified TTC check. Added suggested routine, TRAFFIC6, or similar routine to estimate cumulative ESALs over the detour design life. Parts of this section were used to create "Material and Construction Considerations for Detour Structures."

- in second level subsection, " Material and Construction Considerations for Detour Structures," added details about preparing subgrade and the resulting pavement structure. Added a new construction consideration for detour structures that involve placing 4 in. of flexible base and provides the resulting condition. Added instructions to recycle detour structures when no longer needed.

Chapter 5, Section 5, Perpetual Pavement Design and Mechanistic Design Guidelines

- Under subsection, "Designing a Perpetual Pavement using FPS-19W":
  - reformatted content into Table 5-6: Designing Perpetual Pavement Using FPS-19W and created a list of corresponding guidance for each step

    At step 6, added new information about a minimum "time to first overlay."

    At step 7, added new information about renewing the surface at periodic intervals and added new note "Analysis period is not pertinent to perpetual design."
- Under subsection, "Checking the Proposed Design for Compliance with Limiting Strain Criteria":
  - changed the value for "the maximum tensile strain at the bottom of all HMA layers" from "52 μ-strain (< 70)" to "46 μ-strain (< 70)"
  - changed the value for "the maximum compressive strain at the top of the subgrade" from "124 μ-strain (<200)" to "115 μ-strain (<200)"
  - added recommendation to check strain levels from ATHWLD with computuations outside the current FPS-19W program

Chapter 8, Section 4, Recommended Input Design Values

- Under subsection, "Input Values," removed "subbase" from the third bullet. The item originally read "Effective Modulus of Subbase/Subgrade Reaction, pci"
- Under subsection, "Effective Modulus of Subgrade Reaction: k-value":
  - reformatted the information and created two second level subsections, "TxDOT-Required Base Layer Combination" and "Selecting Appropriate Strength"

    In "TxDOT-Required Base Layer Combination," added an option for designers to use a higher k-value. Added instructions to determine and respond to k-value.
- Under subsection, "Load Transfer Coefficient," in Table 8-1: Load Transfer Coefficients, modified the coefficient value when the answer is "yes" to both criteria.
- Under subsection, "Reliability, %":
  - added design ESALs value after the phrase, "a reliability of 95% should be used for rigid pavement. . ."
  - added information for "a reliability of 90%"

Chapter 8, Section 5, Determining Concrete Pavement Thickness

The first sentence in the first paragraph, changed the phrase from "round the thickness up to the next higher whole inch" to "round the thickness up to the nearest full or half inch." Changed the values in the example.

In the bulleted items, values were changed. These changes are explained in the two paragraphs following the bulleted list.

Chapter 8, Section 6, Concrete Pavement Design Standards

In the first bullet, the beginning value changed from "8" to "6."

Chapter 8, Section 7, Bonded and Unbonded Concrete Overlays

Under the main section, a large part of the content was rewritten and updated.

- The new subsection, "Overlay Thickness Design," was added.
  - The new second level subsection, "Determination of Effective Slab Thickness by Condition Survey Method," was added.
- The new subsection, "Joints and Cracks Adjustment Factor, F_{jc}," was added.
- The new subsection, "Durability Adjustment Factor, F_{dur}," was added.
- The new subsection, "Fatigue Damage Adjustment Factor, F_{fat}," was added.
  - The second level subsection, "Steel Design," was added.
- The new subsection, "Unbonded Concrete Overlays," was added.
  - The second level subsection, "Overlay Thickness Design," was added.
  - The second level subsection, "Determination of Effective Slab Thickness by Condition Survey Method," was added.
  - The second level subsection, "Steel Design," was added.

Chapter 12, Section 6, Super Heavy Load Evaluaton Process

In the first sentence in this section, the value for the "trailer tire load" changed from "exceeding 5,000 lb." to "exceeding 6,000 lb."

Contact

Contact the Construction Division, Materials & Pavements Section (CST-M&P) at (512) 465-3686 for further information regarding the methods in this manual.

Archives

Past manual notices are available in a pdf archive.

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Chapter 1 — Introduction

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Section 1 — Guide Overview

Purpose

The purpose of this guide is to provide the Texas Department of Transportation (TxDOT), consultant firms, and contractors a detailed pavement overview covering all materials, design, construction and maintenance `considerations for traditionally-let TxDOT projects`. The guide:

- serves as an extensive pavement reference that users can browse to look up typical values, methods, practices and resources
- provides approved policies and procedures for pavement design for use on TxDOT projects
- provides pavement engineers with a uniform, streamlined process for designing pavements
- includes a list of available training courses to pavement engineers and links to web-based training for pavement design and analysis programs used by TxDOT
- serves as a guide to pavement engineers for selecting pavement rehabilitation strategies
- addresses related topics that a pavement engineer needs such as procedures for changing load zones, superheavy load analysis and pavement forensics.

Comprehensive Development Agreements

Comprehensive Development Agreements (CDAs) are comprised of contracting methods that include design, construction and maintenance of highways. Under CDAs, there are two major types of projects: concession and non-concession.

Concession projects require a long-term (more than 20 years) maintenance agreement. In this case, performance (ride, rut, distress, etc.) requirements are defined as criteria to be maintained on the project throughout the term of the agreement.

Non-concession projects have a short-term or no maintenance agreements. In this case, design and construction specifications are prescribed for projects.

The scope of this manual includes only traditional TxDOT projects.

Organization

This guide is organized into twelve chapters:

- Chapter 1, Introduction. This chapter provides an overview of the guide, a list of the training courses related to pavements and highway materials and also provides contacts for pavement and materials related topics.
- Chapter 2, Pavement Design Process. This chapter provides definitions of pavement types, pavement type selection, the approved methods for pavement design, pavement design categories, pavement design process, information needed for pavement design, and pavement design reports.
- Chapter 3, Materials Investigation and Selection Information. This chapter covers geotechnical investigations, stabilizations guidelines, aggregate types and classes, performance graded binders (PG binders), asphalt concrete pavement (ACP) mixtures, Portland cement concrete (PCC) and also geosynthetics.
- Chapter 4, Pavement Evaluation. This chapter provides an overview of pavement evaluation and non-destructive testing for pavements.
- Chapter 5, Flexible Pavement Design. This chapter provides an overview of the types of flexible pavements, design parameters and typical ranges, pavement detours and widening.
- Chapter 6, Flexible Pavement Construction. This chapter covers subgrade and base construction, surface preparation, plant operation, mix transport, mix placement, compaction and ride quality.
- Chapter 7, Flexible Pavement Rehabilitation. This chapter covers the different rehabilitation options for flexible pavements including hot in-place recycling, cold in-place recycling, geosynthetics, flexible base thickening, full depth reclamation, hot mix asphalt (HMA) overlays, whitetopping and surface treatments.
- Chapter 8, Rigid Pavement Design. This chapter covers the approved pavement design methods for rigid pavements, rigid pavement design process, recommended typical values, thickness determination, concrete paving standards and thin whitetopping.
- Chapter 9, Rigid Pavement Construction. This chapter covers construction of subgrade and base, surface preparation, concrete mix design, concrete plant operation, delivery of concrete, steel placement, fixed form paving, slip-form paving, joints and texture requirements.
- Chapter 10, Rigid Pavement Rehabilitation. This chapter covers the different rehabilitation options for rigid pavements including full depth repair, partial depth repair, bonded and unbonded concrete overlays, stitching, dowel bar retrofit, joint repairs, diamond grinding, thin ACP overlays, and retrofitting concrete shoulders.
- Chapter 11, Forensics. This chapter provides an overview of the Forensic Team, investigation procedures and archiving forensics.
- Chapter 12, Load Zoning and Super Heavy Load Analysis. This chapter provides an overview of the procedures for changing load zones on highways (including county roads and bridges) and also super heavy load analysis.

Section 2 — Training

Overview

This guide provides general information about pavement design processes and procedures. Information on more comprehensive training courses and materials is available through the district pavement engineer, the Construction Division, Materials & Pavements Section (CST-M&P) and other sources. A partial list of available training courses is as follows:

- National Highway Institute (NHI)/American Society of Civil Engineers (ASCE) Course on Techniques for Pavement Rehabilitation.
- Flexible Pavement System, Version 19 (FPS-19W) and MODULUS Backcalculation Program. Training is presented as a "seminar" by CST-M&P. Contact CST-M&P for schedule (see Contacts section below).
- NHI 6-week Highway Materials Engineering Course (NHI 131023A). Contact NHI.
- TxDOT 5-week and 2-week Highway Materials Engineering Course. The course is listed in TxDOT's Human Resources Training Catalog:
  - CON409, 5-week course (for the first 3 weeks)
  - CON410, 5-week course (for the last 2 weeks)
  - DES114, TxDOT HWY Materials Engineering (2-week course)
  - Contact your training coordinator.
- Basic Geotechnical Engineering for Roadways. There is an NHI course entitled, "Geotechnical Aspects of Pavements" (NHI 132040A). Contact NHI.
- Flexible Pavement Rehabilitation Training CD-ROM. Self-paced training media developed by Texas Transportation Institute (TTI). Contact TxDOT's Research and Technology Implementation Office (RTI) for more information about obtaining this CD (see Section 3 -- "Contacts").
- Rigid Pavement Rehabilitation Training CD-ROM. Self-paced training media developed by TTI. Contact TxDOT's RTI for more information about obtaining this CD (see Section 3 -- "Contacts").
- Web based training for Modulus 6.0, FPS 19W, Modulus Temperature Correction program, Program for Load Zoning Analysis (PLZA), and Super Heavy Load Analysis Procedures (PALS) available on the TxDOT Pavement Design and Analysis Training web site.
- Web based training for CRCP 10, CRCP 9, JRCP 6, Pavers, Rigid Pavement Life Cycle Cost Analysis (RPLCCA), Texas Pavement Type Selection (TxPTS) available through TxDOT Online Training.
- Web based training for Rational Estimation of Pavement Remaining life (REPP2000), Design Modulus Values from Seismic Data Collection (SMART), and Impacts of Construction Quality

Life Cycle of Pavement (RECIPPE) available on CTIS Pavement Design and Analysis Training web site.

Other training and reference material is available from your TxDOT training coordinator.

Section 3 — Contacts

Contacts for Questions and Comments

Contact the following with questions, comments, or issues regarding the particular subject matter:

- Director of CST-M&P at (512) 506-5808 for general questions or questions or comments about this guide
- Flexible Pavements Branch at (512) 506-5836 for Flexible Pavement Design, Construction or Rehabilitation
- Rigid Pavement & Concrete Materials Branch at (512) 506-5858 for Rigid Pavement Design, Construction or Rehabilitation
- Geotechnical, Soils & Aggregates Branch at (512) 506-5907 for issues regarding geotechnical investigations or aggregate
- Asphalt and Chemical Branch at (512) 506-5821 for issues regarding asphalt binder and chemicals modifiers
- Traffic Materials Branch at (512) 506-5889 for issues regarding traffic materials
- Pavements and Materials Systems Branch at (512) 465-3686 for Pavement Management, Pavement Testing and Load Zoning
- Research and Technology Implementation Office at (512) 465-7555 for research projects and their implementations.

More information on the CST-M&P Section can be found at Construction Division, Materials and Pavements Section Information.

Chapter 2 — Pavement Design Process

Contents:

Section 1 — Introduction

Section 2 — Pavement Design Standard Operating Procedure (SOP)

Section 3 — District Pavement Engineer's Role

Section 4 — Pavement Types

Section 5 — Pavement Type Selection

Section 6 — Approved Pavement Design Methods

Section 7 — Pavement Design Categories

Section 8 — Information Needed for Pavement Design

Section 9 — Pavement Design Reports

Section 1 — Introduction

TxDOT spends more than 50% of the annual construction and maintenance budget on pavements. Because of funding limitations, only a portion of pavement-related needs can be addressed. Given these facts, pavements should be designed using the approved pavement design methods specified in Section 6 — Approved Pavement Design Methods. In general, these methods are classified as either:

- an analytical process with accurate design inputs or
- past proven practices (experience-based) that may not meet conventional analytical design standards, but, have a proven performance track record.

The objectives of the pavement design process are to guide the district pavement engineer (DPE) to select a pavement type and design the pavement with an approved method using all the information needed to provide a structure that is capable of carrying traffic loads with minimum physical deterioration, maximum safety, and maximum ride comfort. The designer should document the pavement design process in a report format as discussed in Section 9 — Pavement Design Reports.

Preliminary Pavement Design

A preliminary pavement design needs to be performed during the early phase of project development. This step ensures that a viable design is generated, balancing risk while ensuring adequate funding rather than allowing the project cost to dictate the pavement design. Preliminary design considerations are then discussed at a district level Pavement Design Concept Conference.

Pavement Design Concept Conference

Pavement Design Concept Conferences are used to refine initial considerations and allow development of an approved design and design report. The Pavement Design Concept Conference can be held in conjunction with other early stage project planning meetings that include participation of key district personnel.

Pavement Design Standard Operating Procedure

Each district is required to maintain and update a pavement design standard operating procedure (SOP). The SOP will formalize district communication channels in the pavement design process and document typical design and rehabilitation strategies based on material availability, traffic lev-

els, environmental conditions, and an appropriate level of risk management. The SOP will also establish the final authority for pavement design within the district.

Section 2 — Pavement Design Standard Operating Procedure (SOP)

Communication

Communication between the district pavement engineer, planning staff, maintenance staff, construction staff and area engineers is key to designing, constructing, and maintaining quality pavements.

District Engineers are responsible for ensuring this communication takes place and documenting communication channels in a district pavement design standard operating procedure (SOP). This SOP should be reviewed and updated annually.

One component of this communication process is to hold a Pavement Design Concept Conference. The general approach to pavement design should be refined in a Pavement Design Concept Conference after the project is programmed and in the early stages of plans, specifications and estimates (PS&E) development. The designer/project engineer presents an initial pavement design report and provides any additional information to assist in selection of the final pavement design.

Conference Participants

Depending on the size or sensitivity of the project, all or some of the following individuals should participate in the conference:

- Project Engineer
- Area Engineer
- Pavement Engineer
- Laboratory Engineer
- Maintenance Supervisor
- Lead Construction Inspectors
- Director or Transportation Planning and Development (TP&D)
- Director of Operations
- Director of Construction
- District Engineer

Discussion Items

Factors that should be discussed during pavement design meetings or conferences are outlined in the district standard operating procedure (SOP). They include:

- communication to staff (including maintenance) of this SOP
- communication of the project scope and available funding and pavement design strategies (district-wide policies and project specific considerations)
- existing pavement history and material and structural analysis. Examples include:
  - performance and maintenance history (multi-year trends)
  - district maintenance staff input
  - existing pavement analysis (field and lab testing to characterize functional and structural properties).
- equivalent single axle loads (ESAL) and the average ten heaviest wheel loads daily (ATH-WLD) review and adjustments. For example, Transporation Planning and Programming (TPP) maps and data for ESALs generated through current data systems may not account for specific truck generators or may overestimate truck loads or percent trucks in the traffic stream.
- Modified Texas Triaxial Check process
- plan for use of recycled and existing materials
- material selection (including selection of asphalt concrete pavement [ACP], binder type, type of flexible base, stabilization of bases, etc.)
- pavement design material properties
- Wet Weather Accident Reduction program (WWARP)
- need for and type of subgrade stabilization
- alternate pavement designs (Alternate materials selection - not pavement type. Alternates to limited competition pavement rehabilitation techniques such as Ultra-thin Lift [Novachip] should be addressed.)
- alternate pavement types for new or total reconstruction projects
- pavement design strategies based on ESALs (chart or descriptive)
- special considerations, such as:
  - urban designs
  - constructability (e.g., ACP to match curb depth)
  - lateral support provided by shoulders
  - typical section, front slope, and ditch geometry
  - project drainage
  - 2R, 3R, 4R criteria
  - special haul routes such as logging or aggregate haul roads
  - sulfate-bearing soils
- use and methodology for potential vertical rise (PVR) design consideration

- pavement design development and approval process

Final Authority for Pavement Design

Authority for pavement design approval may not be delegated below the district engineer, except for metropolitan districts.

In metropolitan districts, pavement design approval authority may be delegated to the deputy district engineer, district director of construction, or the district director of transportation planning and development for projects with estimated construction cost of less than $20 million.

Section 3 — District Pavement Engineer's Role

History

The district pavement engineer (DPE) is a licensed professional engineer who serves as the district point of contact for the evaluation, preservation, and structural design of pavements. This position was formalized by the department in 1993 as a district-level staff position.

The DPE serves as the coordinator for district staff. The DPE's responsibilities include planning activities, such as, forensic studies; participating in design concept conferences; reviewing performance histories of materials; studying processes for pavement construction; maintaining databases for subgrade and pavement material stiffness or structural properties; assessing pavement performance with maintenance staff; and coordinating design strategies for pavement rehabilitation with district staff.

The development of the pavement design and rehabilitation strategies should jointly involve material engineers, maintenance engineers, planning engineers, construction engineers, design engineers and area engineer staff.

The DPE should also coordinate and participate in the development of district pavement preservation plans in conjunction with district maintenance plans.

Responsibilities

The DPE is responsible for:

- producing cost-effective district structural pavement designs and reviewing district pavement design reports for technical content
- recommending pavement preservation policies to maximize the condition of district pavements within budget constraints
- identifying pavement related research needs, and
- participating in technology transfer and pavement related training activities (refer to "Job Functions for the District Pavement Engineer").

The DPE is charged with being the district expert on all matters pertaining to:

- the evaluation of functional and structural aspects of existing pavements
- traffic loading characteristics
- prevailing geologic conditions within the district
- suitability of proposed materials (new and recycled), and

- use of structural design software.

The DPE may be asked to direct the activities of the district's pavement data collection efforts (visual distress, rut/ride, deflection surveys). These data collection efforts are integral to maintaining the network level Pavement Management Information System (PMIS) and in evaluating project level structural properties.

Because of the importance of understanding material properties and evaluation of materials used in pavements, some districts have assigned DPE duties to the district lab engineer.

District Pavement Engineer (DPE) Skills

The DPE should attend courses listed in Table 2-1. To develop the final pavement thickness design, courses have been identified to provide the basic skills necessary for engineers to understand the design process and complete viable, cost conscious pavement design.

The DPE or other district staff may develop a design, however, it is suggested that the DPE and other key district staff review pavement design inputs, material requirements, strategies, and thickness prior to the final PS&E submission.

Newly assigned DPEs are encouraged to schedule attendance of the required training sessions for flexible and rigid pavement design as soon as practical.

Table 2-1: Recommended and Required Training for the District Pavement Engineer

Training	Category	Comment
MODULUS	Flexible Structure Evaluation	Designed as a seminar and is combined with FPS-19W training. Combined training is usually arranged at district request to CST-M&P. Hands-on approach used to emphasize evaluation techniques, capabilities, and limitations of the software. A 2 1/2-day period should be scheduled. **Required for flexible pavement design.**
FPS-19W	Flexible Structural Design	As above. **Required for flexible pavement design.**

Table 2-1: Recommended and Required Training for the District Pavement Engineer

Training	Category	Comment
Visual Distress Rater's Course CON110 CON111	Identification of visual distress: ◆ concrete distress ◆ flexible distress.	Goals are: ◆ understand the Texas Reference Marker System and know how it is used to identify and locate PMIS section in the field ◆ read a PMIS section list and automated rating form to identify the sections ◆ complete an automated rating form ◆ identify the distresses rated for concrete (CON 110) or flexible (CON 111) pavements ◆ conduct visual distress ratings for PMIS.
PMIS Concepts for Administrators CON107	PMIS	The goals are: ◆ identify the types of pavement evaluation data available in PMIS ◆ describe the differences between network-level and project-level pavement management, and explain how PMIS can be used to support both ◆ interpret PMIS data and scores ◆ use PMIS to monitor pavement condition, estimate total pavement needs, and assess the overall level of service provided by pavement maintenance. *Course material required for certification can be found on the Pavement Management Information System webpage.
PMIS Data Interpretation and Analysis CON109	PMIS	The goals are: ◆ use PMIS data to diagnose surface and sub-surface pavement problems ◆ define and interpret the five PMIS scores and ◆ produce three PMIS analysis reports. *The course material description can be found on the Pavement Management Information System webpage.
MapZapper Familiarization*	PMIS	GIS-based software that allows the user to plot various pavement condition indicators on district maps. Formal training is available on request. Self-study instructional material may be found at: http://cst-pmisdata/pmis/PC%20Programs/PC%20Programs%20Frame.htm
Ride Quality Software	PMIS	Software available at http://www.dot.state.tx.us/services/information_systems/engineering_software.htm. The DPE may receive guidance on how to use the Ride Quality evaluation software by contacting the Pavement & Material Systems branch, CST-M&P.
TSLAB86, DARWin™ 3.1	Rigid Structural Design	Designed as a seminar. **Required for rigid pavement design**.

Table 2-1: Recommended and Required Training for the District Pavement Engineer

Training	Category	Comment
2, 5, or 6-week Materials Course	Materials Properties	National Highway Institute (NHI) sponsored courses addressing fundamental properties and testing of highway materials.
Selecting Rehabilitation Strategies for Flexible Pavements	Rehab of existing pavements	There are two options: ◆ Texas Transportation Institute (TTI) administered course taught through an interagency contract (IAC). This course is hands-on oriented – uses department software. ◆ NHI course. This course is more oriented at the planning level.
*Available through the TxDOT Intranet only.		

PMIS and pavement design assistance from CST-M&P will be provided upon request.

CST-M&P will maintain a list of DPEs.

Section 4 — Pavement Types

Flexible Pavement

A flexible pavement structure is typically composed of several layers of material with better quality materials on top where the intensity of stress from traffic loads is high and lower quality materials at the bottom where the stress intensity is low. Flexible pavements can be analyzed as a multilayer system under loading.

A typical flexible pavement structure consists of the surface course and underlying base and sub-base courses. Each of these layers contributes to structural support and drainage.

When hot mix asphalt (HMA) is used as the surface course, it is the stiffest (as measured by resilient modulus) and may contribute the most (depending upon thickness) to pavement strength. The underlying layers are less stiff but are still important to pavement strength as well as drainage and frost protection.

When a seal coat is used as the surface course, the base generally is the layer that contributes most to the structural stiffness. A typical structural design results in a series of layers that gradually decrease in material quality with depth. Figure 2-1 shows a typical section for a flexible pavement.

Figure 2-1. Typical section for a flexible pavement.

Perpetual Pavement

Perpetual pavement is a term used to describe a long-life structural design. It uses premium HMA mixtures, appropriate construction techniques and occasional maintenance to renew the surface. Close attention must be paid to proper construction techniques to avoid problems with permeability, trapping moisture, segregation with depth, and variability of density with depth. A perpetual pavement can last 30 yr. or more if properly maintained.

Structural deterioration typically occurs due to either classical bottom-up fatigue cracking, rutting of the HMA layers, or rutting of the subgrade. Perpetual pavement is designed to withstand almost infinite number of axle loads without structural deterioration by limiting the level of load-induced strain at the bottom of the HMA layers and top of the subgrade and using deformation resistant HMA mixtures. Figure 2-2 shows a generalized perpetual pavement design. Detailed construction considerations are available through this link.

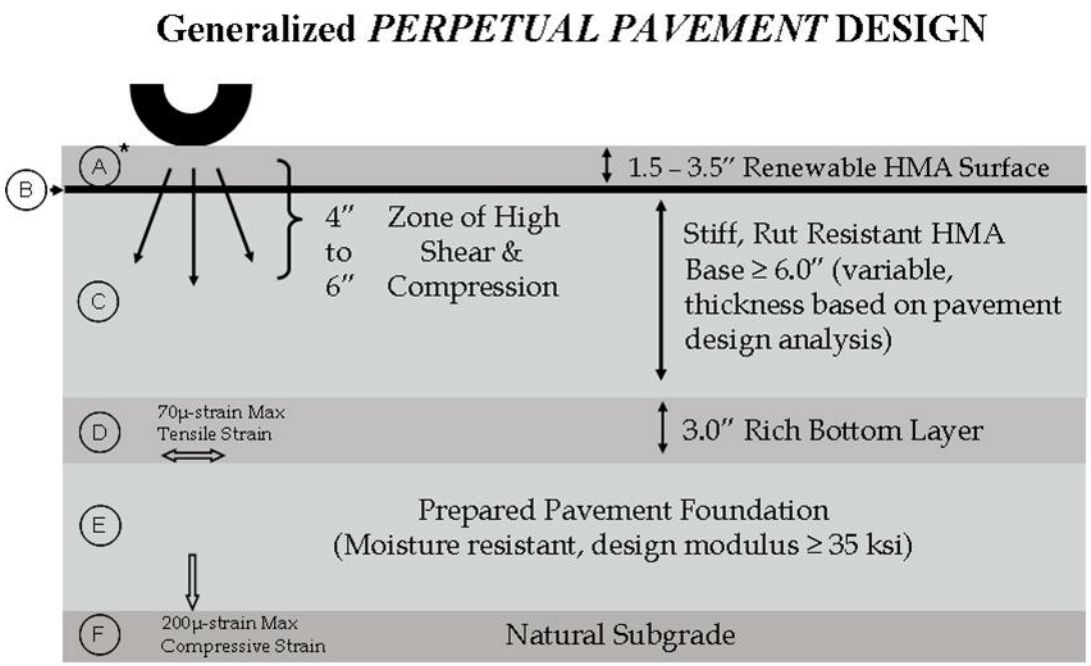


Figure 2-2. Generalized perpetual pavement design.

Rigid Pavement

A rigid pavement structure is composed of a hydraulic cement concrete surface course and underlying base and subbase courses (if used). Another term commonly used is Portland cement concrete (PCC) pavement, although with today's pozzolanic additives, cements may no longer be technically classified as "Portland."

The surface course (concrete slab) is the stiffest layer and provides the majority of strength. The base or subbase layers are orders of magnitude less stiff than the PCC surface but still make important contributions to pavement drainage and frost protection and provide a working platform for construction equipment.

Rigid pavements are substantially 'stiffer' than flexible pavements due to the high modulus of elasticity of the PCC material, resulting in very low deflections under loading. The rigid pavements can be analyzed by the plate theory. Rigid pavements can have reinforcing steel, which is generally used to handle thermal stresses to reduce or eliminate joints and maintain tight crack widths. Figure 2-3 shows a typical section for a rigid pavement.

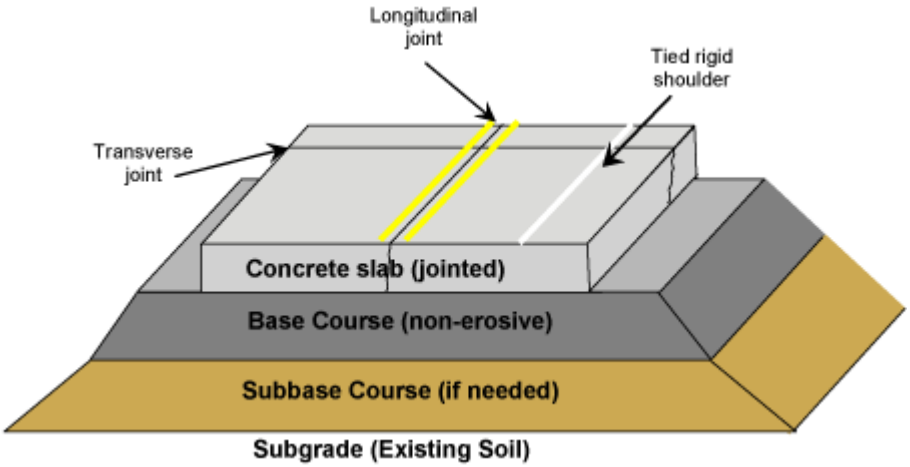


Figure 2-3. Typical section for a rigid pavement.

Continuously Reinforced Concrete Pavement

CRCP provides joint-free design. The formation of transverse cracks at relatively close intervals is a distinctive characteristic of CRCP. These cracks are held tightly by the reinforcement and should be of no concern as long as the cracks are uniformly spaced, do not spall excessively, and a uniform non-erosive base is provided. Figure 2-4. shows a typical section of CRCP.

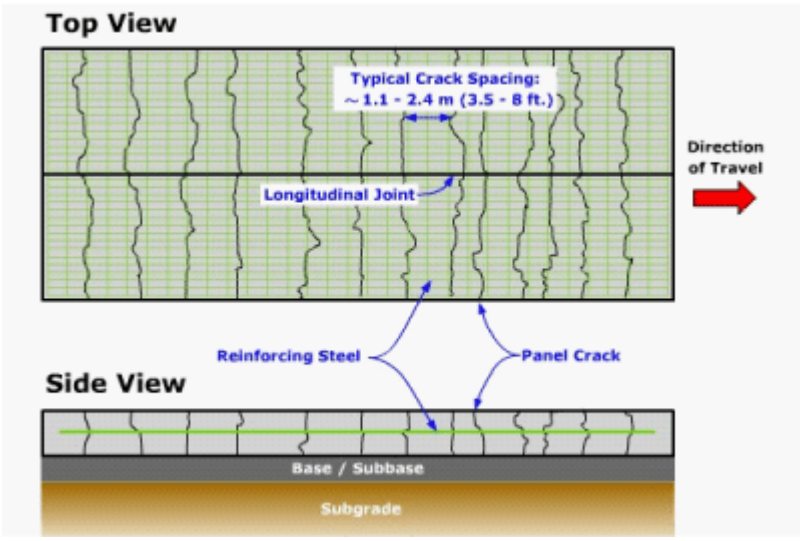


Figure 2-4. Continuously Reinforced Concrete Pavement.

Concrete Pavement Contraction Design (CPCD)

CPCD uses contraction joints to control cracking and does not use any reinforcing steel. An alternative designation used by the industry is jointed concrete pavement (JCP). Transverse joint spacing is selected such that temperature and moisture stresses do not produce intermediate cracking

between joints. Nationally, this results in a spacing no longer than 20 ft. The standard spacing in Texas is 15.0 ft.

Dowel bars are typically used at transverse joints to assist in load transfer. Tie bars are typically used at longitudinal joints. Figure 2-5 shows a typical section of CPCD.

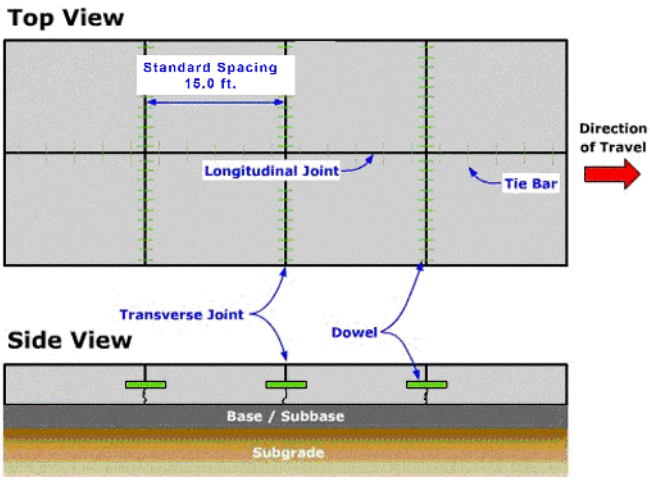


Figure 2-5. Concrete Pavement Contraction Design (CPCD).

Jointed Reinforced Concrete Pavement (JRCP)

JRCP uses contraction joints and reinforcing steel to control cracking. Transverse joint spacing is longer than that for concrete pavement contraction design (CPCD) and, in Texas, it typically ranges from 30 ft. to 60 ft. This rigid pavement design option is no longer endorsed by the department because of past difficulties in selecting effective rehabilitation strategies. However, there are several remaining sections in service. Figure 2-6 shows a typical section of jointed reinforced concrete pavement.

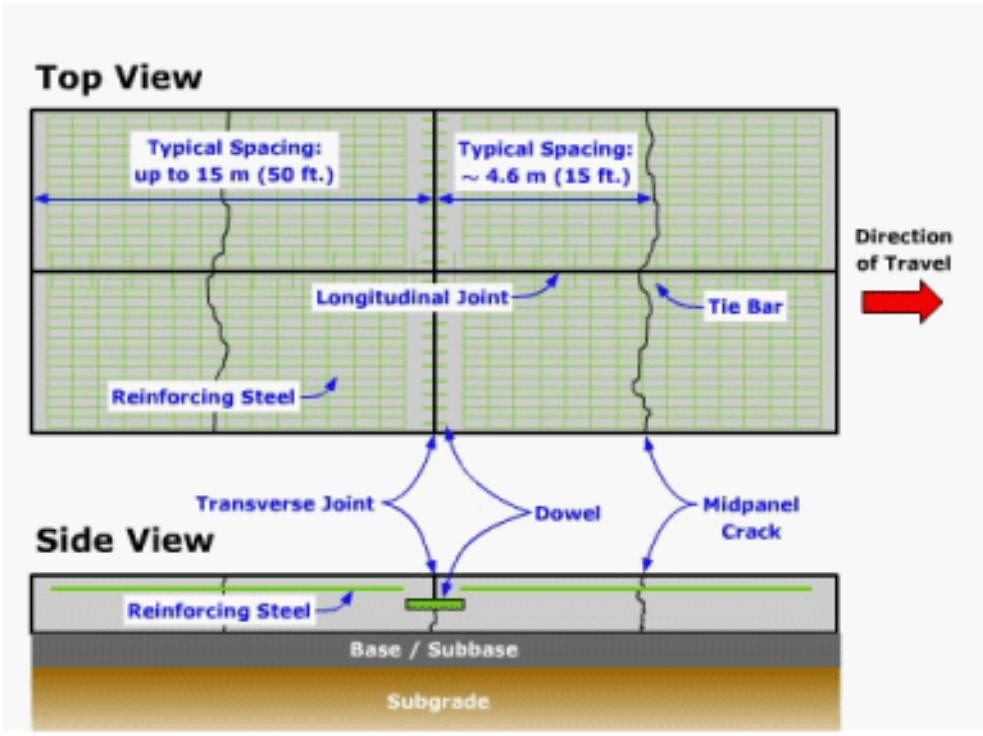


Figure 2-6. Jointed Reinforced Concrete Pavement (JRCP).

Post-tensioned Concrete Pavements

Post-tensioned concrete pavements remain in the experimental stage and their design is primarily based on experience and engineering judgment. Post-tensioned concrete has been used more frequently for airport pavements than for highway pavements because the difference in thickness results in greater savings for airport pavements than for highway pavements.

Composite Pavement

A composite pavement is composed of both hot mix asphalt (HMA) and hydraulic cement concrete. Typically, composite pavements are asphalt overlays on top of concrete pavements. The HMA overlay may have been placed as the final stage of initial construction, or as part of a rehabilitation or safety treatment. Composite pavement behavior under traffic loading is essentially the same as rigid pavement.

Rigid and Flexible Pavement Characteristics

The primary structural difference between a rigid and flexible pavement is the manner in which each type of pavement distributes traffic loads over the subgrade. A rigid pavement has a very high stiffness and distributes loads over a relatively wide area of subgrade – a major portion of the structural capacity is contributed by the slab itself.

The load carrying capacity of a true flexible pavement is derived from the load-distributing characteristics of a layered system (Yoder and Witczak, 1975). Figure 2-7 shows load distribution for a typical flexible pavement and a typical rigid pavement.

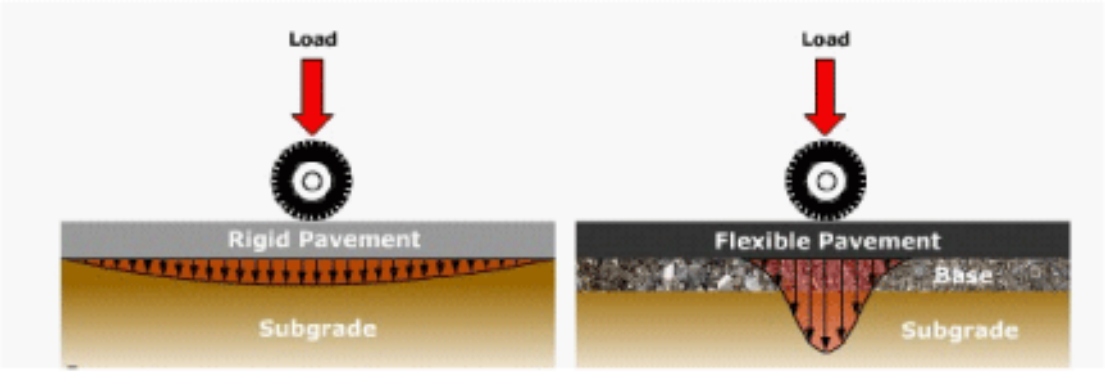


Figure 2-7. Typical stress distribution under a rigid and a flexible pavement.

Section 5 — Pavement Type Selection

Principal Factors

Selecting a pavement type is an important decision. Like other aspects of pavement design, the 1993 *American Association of State Highway and Transportation Officials (AASHTO) Guide* states, "The selection of pavement type is not an exact science but one in which the highway engineer must make a judgment on many varying factors. . ." Appendix B of the *1993 AASHTO Guide* provides a list of principal and secondary factors to consider in the selection process.

The use of alternate bidding has proven to stimulate competition between pavement industries. When this type of bidding is used, the contractor with the lowest bid may ultimately decide the pavement type.

Alternate bidding on Federal-aid National Highway System (NHS) projects is allowed only with prior written approval from the Texas Division Office of FHWA. Failure to obtain approval prior to contract letting will make the projects ineligible for Federal-aid funding.

Alternate bidding on Non-NHS projects does not require FHWA approval. However, the pavement design reports must be coordinated and submitted at an early stage of the PS&E development to TxDOT's Construction Division.

The recommended screening criteria for considering an alternate bid contract include:

- project cost > $5M
- ADT > 10,000 and
- limited constructability constraints such as limitation in width, traffic control, drainage, construction sequencing, etc. that might dictate the selection of a specific payment type.

Project Selection

Chart 1 provides the criteria of project selection for alternate pavement design.

Projects with greater than 30 percent trucks or 100,000 average daily traffic (ADT), or any combination of percent trucks and ADT that falls above the curve in chart 1, will require a Life Cycle

Cost Analysis (LCCA) and a design life of 30 years for both rigid and flexible pavements. These projects are presented in the orange portion of the chart. The flexible pavement will require a perpetual design without staged construction. The LCCA will be used to determine the bid adjustment factors for these projects.

Projects with up to 30 percent trucks and ADT greater than 10,000 but less than 100,000, which are represented by the yellow region in chart 1, will be referred to as "Thickness Design." Both the rigid and flexible alternates will have a 20-year design life. The rigid pavement is further defined by a minimum slab thickness of 6 inches and the flexible alternate will be limited to one structural overlay, maximum 2 inches. LCCA is not required but may be used for these projects at the district discretion. When LCCA is used, the cost of the overlay will included to determine the bid adjustment factors for these projects.

For projects with less than 10,000 ADT, alternate pavement design is not required. These projects are represented by the green portion of the chart.

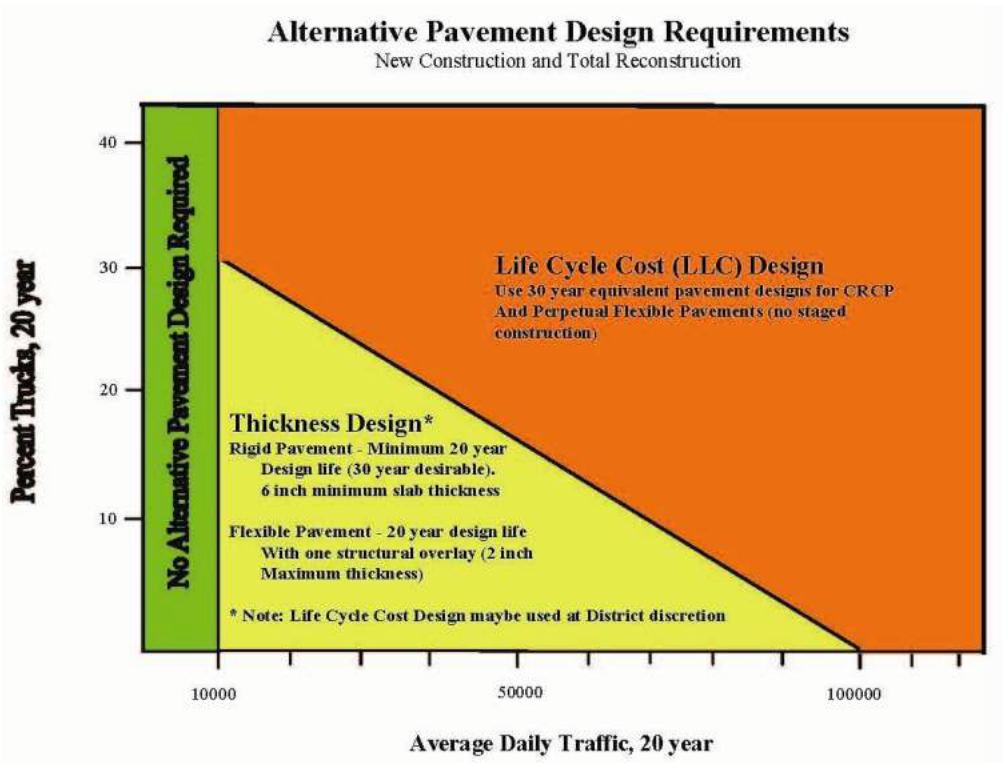


Figure 2-8. Criteria for alternate pavement design.

Exceptions

Several exceptions may justify utilization of district preference in lieu of incorporating an alternate pavement design in the construction plans. These exceptions are:

- Projects with a total pavement cost less than $5 million.
- Projects where it is necessary to match adjacent pavement types (i.e intersections or projects of minimal length) requires specific documentation in the pavement design file; and,
- Projects where constructability constraints such as limitations in width, traffic control, drainage, construction sequencing, etc, dictate a specific pavement type, requires specific documentation in the pavement design file

The decision factors for pavement design type should be included in the pavement design report.

Life-cycle Cost Analysis (LCCA)

LCCA is an engineering economic analysis that allows engineers to quantify the differential costs of alternative investment options for a given project. LCCA can be used to compare alternate pavement types (flexible versus rigid) on new construction projects and rehabilitation projects. LCCA considers all agency expenditures and user costs throughout the life of the facility, not just the initial investment, and allows for cost comparison of options with varying design lives to be compared on an equivalent basis.

More than a simple cost comparison, LCCA offers methods to determine and demonstrate the economic merits of the selected alternative in an analytical and fact-based manner. LCCA helps engineers answer questions like:

- Which design alternative results in the lowest total cost to the agency over the life of the project?
- To what level of detail have the alternatives been investigated?
- What are the user-cost impacts of alternative strategies?

LCCA's structured methodology provides the information and documentation necessary for successful open dialogue. Because of this, LCCA is a valuable analysis to support pavement type selection decisions.

The Federal Highway Administration (FHWA) has developed general guidance for running a detailed life-cycle cost analysis. This website is a good starting point for information on running a life-cycle cost comparison of alternatives.

TxDOT research project 0-1734 (Center for Transportation Research [CTR]) developed a computer program, TxPTS (Texas Pavement Type Selection). More information may be found at TxDOT Online Training. The program evaluates candidate flexible and rigid pavement strategies for pavement rehabilitation, reconstruction and new construction projects. The economic evaluations are based on life cycle cost analysis. Also, CTR developed a life-cycle cost analysis program for rigid pavements under project 0-1739.

LCCA is only one of many processes for selecting a pavement type. The reliability of output from any LCCA is a function of the reliability of the input data. It may be beneficial to evaluate user costs separately from agency costs when looking at different structural options.

Section 6 — Approved Pavement Design Methods

Introduction

Use one of the following analytical methods for designing pavements:

- FPS-19W for flexible pavements (FPS-11 may be used under certain circumstances)
- AASHTO design procedure (1993) for flexible and rigid pavement designs
- Modified Texas Triaxial Design Method for flexible pavements.

Flexible Pavement System – Windows version (FPS-19W)

For most flexible pavement design work, especially higher-volume highways (>10,000 ADT, 5 M ESALs), the Flexible Pavement Design System (FPS-19W) is the **preferred** method for designing flexible pavements. FPS-19W should be used as a check for all flexible designs as described in , "Pavement Design Process."This design procedure training is available to department personnel through CST-M&P.

- FPS-19W provides a methodology for selecting a complete pavement design strategy. Such a strategy calls for action now (initial construction) and for future action (overlays or reconstruction). Depending upon the range of material layer thicknesses the designer is willing to consider, the output will consist of one or more recommended strategies. For a given design analysis, initial construction costs as well as future costs are computed for each design strategy. The engineer selects a design strategy based on a multitude of considerations including past performance, cost, constructability, user delay, adjoining section, etc.
- FPS-19W is a mechanistic-empirical design procedure that uses a performance model based on degradation of the serviceability index as defined in the AASHO Road Test research. Also borrowed from the AASHO Road Test is the standardization of cumulative traffic loading in terms of 18-kip equivalent single axle loads (ESALs). The FPS-19W program assumes that a smaller deflection means smaller stresses or strains and, therefore, longer life.
- The program uses a "District Temperature Constant" that assigns a cold region multiplier to those areas of the state more susceptible to thermal cracking as the only environmental input. However, the current recommendation is to nullify this parameter by assigning a value of "31" corresponding to climate in Central Texas. See , "FPS-19W Design Parameters," for additional discussion.
- All other environmental influences including seasonal changes in material stiffness, frost heave, or moisture susceptibility of materials are not directly considered by the program. Input fields are provided to assess the impact on performance if the project location has swelling

foundation soils. Adding thickness to overcome swelling effects is not encouraged, except in very limited cases. For more information, go to , "Geotechnical Investigation for Pavement Structures."

- The program uses a "confidence level" approach to account for variability in the in-place subgrade stiffness, construction variability, and traffic loading growth. A multiplier is assigned to the cumulative traffic loading as the desired level of confidence or reliability increases.
- The system can generate designs that may fail under occasional heavy wheel loads. This circumstance is particularly acute for designs that have low cumulative loading in regions with poor subgrade. For this reason, designs obtained with the FPS-19W (or the older FPS-11) program must be checked with the "Modified Texas Triaxial Design Method." Considerations for accepting this procedure as the governing method for determining design thickness are described in , "FPS-19W Design Parameters."
- The "Modified Texas Triaxial Design Method" is included in FPS-19W in a post-design check module. It can also be used as a standalone procedure using the graphs contained in the archived versions of "Tex-117-F, Triaxial Compression for Disturbed Soils and Base Materials."
- A mechanistic design check is provided to evaluate expected fatigue life of the HMAC layers and full-depth rut life of the structure with options to use several strain-based performance models.
- FPS-19W uses back-calculated modulus to characterize the pavement layer strength (stiffness) based on Falling Weight Deflectometer (FWD) deflection measurements (see "Pavement Evaluation."). Note that back-calculated modulus used in FPS-19 is not the same as the resilient modulus used in the AASHTO design procedure.
- It is incumbent upon the designer to have a recent set of deflection data for the project under consideration from which moduli can be generated, as well as institutional knowledge of material moduli when virgin or recycled materials are to be incorporated in the design. Each district should develop a database of typical moduli through a routine program of aggressive deflection testing and subsequent back-calculation.

Other Acceptable Design Procedures

Although FPS-19W is the preferred design method for analytical design of flexible pavements, FPS-11 and the 1993 AASHTO design procedure are also acceptable, provided the designer is versed in their respective methodologies. Used extensively by the department until the mid-1990s, FPS-11 is a DOS-based design routine with design inputs that are substantially the same as those in FPS-19.

The two biggest limitations with the FPS-11 system are the use of material stiffness coefficients (versus modulus, also, not the same as AASHTO layer coefficients) to characterize the strength of

the material layers and the required supporting deflection testing device (Dynaflect), which is no longer maintained at the department level. Other than the subgrade stiffness coefficient, layer stiffness coefficients can not be directly derived through the complementary STCOEF2 back-calculation routine, nor is there a robust correlation with back-calculated moduli.

Training for FPS-11 is no longer supported at the department level and related documentation is scarce. Familiarity with the AASHTO procedure must be gained through participation in an NHI class or equivalent or through self-study. Pavement Design staff from the Construction Division can consult with individuals interested in using the AASHTO design procedures.

Basic Design Types

FPS-19W uses a menu of five basic design types:

Table 2-2: FPS-19W Five Basic Design Types

1	2	3	4	5
HMA or surface treatment	HMA	HMA	HMA	HMA Overlay
flexible base	asphalt stabilized base	asphalt stabilized base	flexible base	existing HMA
subgrade	subgrade	flexible base	stabilized subbase/ subgrade	existing base
		subgrade	subgrade	subgrade

In addition, a previously generated design input file can be recalled. **The number of distinct layers that can be evaluated is limited to that shown in the table**. The designer may opt to consolidate two or more layers or ignore a minimal contributing layer (such as select fill) in a design as a work-around if the number of layers in the proposed structure exceeds that available in any design option. In the case of consolidating multiple layers, a combined modulus must be assumed; this procedure would require some discretion to ensure adequacy of the overall design. This philosophy applies to the design of full-depth or "perpetual" type HMA pavements.

There are also unique aspects to the performance algorithms with each type of design. For example, in design **Type 1**, the flexible base modulus is mathematically calculated based on the designer's input modulus for the subgrade and the flexible base thickness. The designer may override this calculated value, but values that are considerably greater than the program-generated value should be carefully weighed; the intent of this design type is to scale the stiffness of the base modulus to the degree of support offered by overall base thickness and subgrade stiffness.

The **Type 3** design also adjusts the flexible base modulus based on the subgrade modulus, but at a fixed ratio of 3:1. Using discretion, this value may be overwritten as described in **Type 1** design.

The Type 4 design has no adjustments associated with it. When the pavement structure is not clearly a Type 1 or Type 2 or Type 3 design. Type 4 is the one recommended. All data in this pavement type may be overwritten to suit the structure being designed within the limitations already described.

The **Type 5** design is intended for overlays of flexible pavements only and cannot be used reliably for overlay of concrete or HMA-surfaced structures on heavily stabilized bases.

For more information, refer to the *Flexible Pavement Design System (FPS) 19: User's Manual* (TTI Research Report TX-02/1869-2). Contact the district pavement engineer (DPE) or CST-M&P to obtain a copy of the latest version of the FPS-19 computer program.

AASHTO Design Procedure (for flexible and rigid pavement designs)

The AASHTO (originally AASHO) pavement design guide was first published as an interim guide in 1972. Updates to the guide were subsequently published in 1986 and 1993. The AASHTO design procedure is based on the results of the AASHO Road Test conducted from 1958-1960 in Ottawa, Illinois.

Approximately 1.2 million axle load repetitions were applied to specially designed test tracks in the most comprehensive pavement test experiment design ever conducted. The original AASHO design process was strictly empirical in nature; subsequent updates have included some mechanistic provisions, such as, classifying the subgrade stiffness in terms of resilient modulus and accounting for seasonal variation in material stiffness.

AASHO design originated the concept of pavement failure based on the deterioration of ride quality as perceived by the user. Thus, performance is related to the deterioration of ride quality or serviceability over time or applications of traffic loading.

Also developed at the AASHO Road Test was the rendering of cumulative traffic loading in terms of a single statistic known as the 18-kip equivalent single axle load (ESAL).

Flexible Design

Flexible design using the AASHTO procedure requires the designer to derive a structural number (SN) that is adequate for the anticipated traffic over the length of a desired performance period. The SN is equivalent to the sum of a layer coefficient (a), layer thickness (D), and drainage coefficient (m) for each layer.

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3, etc.$$

The AASHTO procedure for flexible design is automated in the DARWin™ 3.10 software. One aspect that makes using this design procedure somewhat problematic is that layer coefficients are

not directly correlated to any universal system of measurement. AASHTO does provide some guidelines for correlation to laboratory-derived resilient modulus and one design option using the DARWin software is to input layer resilient moduli instead of layer coefficients.

Rigid Design

The department uses the 1993 AASHTO procedure for rigid pavement design (in 1998, a rigid design supplement was published by AASHTO; the department does not use the 1998 supplement). The design produces a rigid slab thickness in inches required to support the estimated traffic under a selected serviceability interval and estimated support and environmental conditions.

The AASHTO procedure for rigid design has been automated in a DOS routine referred to as TSLAB86 or the designer can use the DARWin™ software. Steel design is reflected in the department's recommended CRCP standards, found under the Pavements section on the Roadway Standards webpage.

DARWin™ 3.10 is the latest automated version of the AASHTO design process. TxDOT has an unlimited license for this software for internal use only. The license cannot be shared with outside agencies. This software is scheduled to be retired by December 2011 and will no longer be supported by AASHTO. DARWinTM 3.10 is scheduled to be replaced by DARWin ME 2.0, AASHTO's mechanistic-empirical design procedure.

Additional Information

For more information on using the AASHTO flexible or rigid pavement design procedures, refer to the *1993 AASHTO Guide for Design of Pavement Structures*.

Modified Texas Triaxial Design Method

The Texas Triaxial Classification of soils was developed in the late 1940s and early 1950s by the department as an indexed soil classification system related to soil shear strength. Evaluating a soil for its Texas Triaxial Classification is covered in "Tex-117-E, Triaxial Compression for Disturbed Soils and Base Materials."

When the FPS design system was first developed in the 1970s, solutions produced for some lightly trafficked highways that had an occasional heavy load were found to be under-designed. The Modified Texas Triaxial Design Method was developed to overcome shortcomings of the FPS design procedure by determining the required pavement thickness to ensure protection against shear failure in unbound layers due to heavy wheel loads.

The modified triaxial method requires the use of the subgrade or base Texas Triaxial Class as derived from laboratory test results. Since the testing procedure requires the soil sample or base be

moisture conditioned to establish its triaxial classification (capillary absorption time based on material plasticity), the evaluation represents the soil's strength at a weakened state.

The engineer may determine that this saturation level is not likely for a particular environment (like west Texas) and, therefore, not the overriding design consideration. Additional credit is given for bound materials within the structure that will allow a reduction in the calculated coverage above the evaluated unbound layer. See also , "Pavement Design Process" for additional guidelines.

This method has been automated and is included as a post-design check module in FPS-19W. The method can also be used as a standalone tool for designs where traffic loading can not be easily evaluated in terms of 18-kip ESALs, such as, parking lots, temporary detours, etc. Results of this check may be waived based on local experience. See also , "Pavement Design Process."

When soil testing can not be performed to establish the triaxial classification, soil maps may be used to identify the general soil type and approximation of the soil triaxial classification can be made using historical test results (Soil_Series.xls).

Experienced-Based Design

In 2009, the Pavement Design Task Force (PDTF) emphasized the need to scrutinze the performance of local highways. With a view of reverse engineering, compare the performance of good performing highways against predictions generated through the FPS/Modified Texas Triaxial procedures. A check of structural properties through deflection measurements and an evaluation of material properties (base classification, ACP mixture design, etc.) should be performed.

The potential to conserve limited construction funding can be realized where local experience has shown structural designs to outlast formal predictions. Quantification of their structural attributes can be used to improve prediction models. These proven strategies can supersede analytical design results provided documentation is added to the design report.
For special cases, see "Pavement Design Reports." The pavement design for special cases will typically be based on engineering judgment, historical performance, district policy, and other guidelines (e.g., this guide, industry guidelines).

Section 7 — Pavement Design Categories

There are three pavement structural design categories:

1. "new" pavement design
2. pavement reconstruction design and
3. pavement rehabilitation design.

It is very important that the designer conduct an early investigation to determine which category applies to the project.

Definitions

The following table lists the definitions for each pavement structure design category.

Table 2-3: Pavement Structural Design Category Definitions

Term	Definition
"new" pavement	A combination of a base and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed for flexible pavements or a combination of a base and a PCC slab for rigid pavements.
pavement reconstruction	Construction of a new pavement structure, which usually involves complete removal and replacement of the existing pavement structure including new and/or recycled materials.
pavement rehabilitation	Resurfacing, restoration, and rehabilitation (3R) work undertaken to restore serviceability and to extend the service life of an existing facility. This may include partial recycling of the existing pavement, placement of additional surface materials, or other work necessary to return an existing pavement, including shoulders, to a condition of structural or functional adequacy.
NOTE: These definitions do not necessarily coincide with program definitions.	

Example of Conditions for Each Pavement Design's Usage

"New" Pavement Design. The "new" pavement design category assumes a pavement structure is constructed from point "A" to point "B" and there is no existing pavement along the proposed route. It may include a new parallel roadbed, such as, when a two-lane highway is transformed into a divided four-lane highway.

Pavement Reconstruction Design. Pavement reconstruction design assumes there is an existing pavement along the projected route. It is further assumed the structural condition of existing pavement is in such deteriorated condition, removal of all or part of the existing pavement is necessary.

Also, there is the possibility that adjustments to the vertical and/or horizontal alignment make reconstruction necessary.

Figure 2-8 provides a flow chart for the design process for new pavements or a full reconstruction.

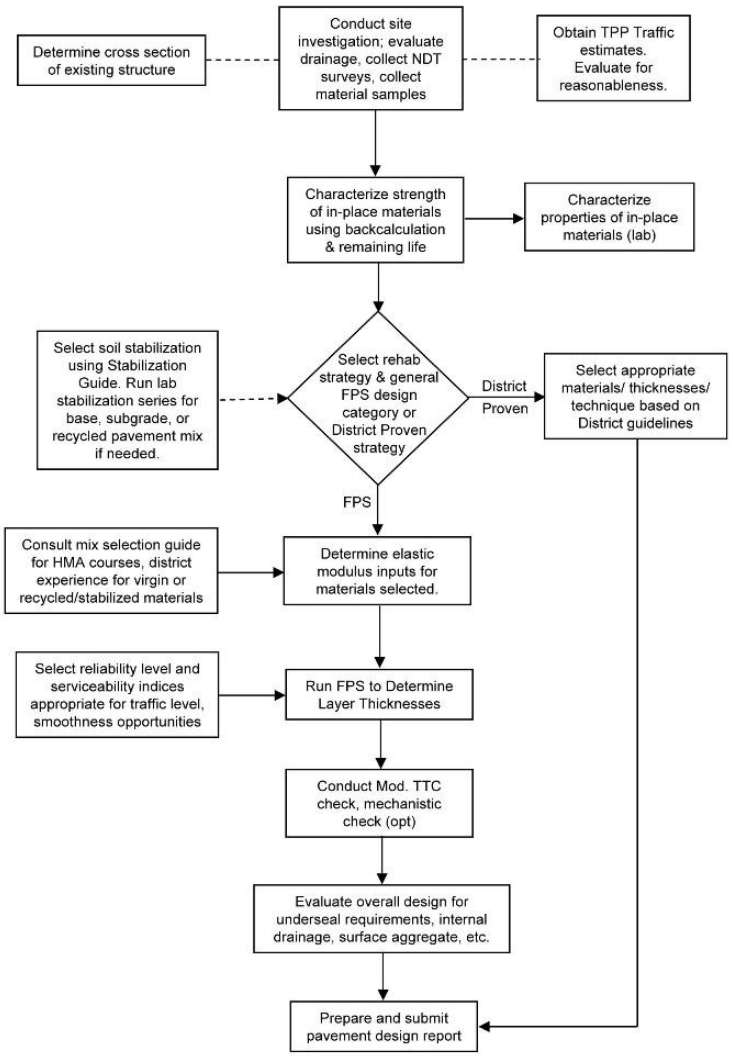


Figure 2-9. Design process for flexible pavement rehabilitation.

Pavement Rehabilitation Design. Pavement rehabilitation design assumes there is an existing pavement structure along the project route. The vertical and/or horizontal alignment will not change significantly. Also, it is assumed the existing pavement structure possesses a degree of remaining life. More information can be found in the Flexible and Rigid Pavement Rehabilitation Training CD/ROMs. Refer to "Training" in Chapter 1, "Introduction," for more information on these CDs.

Figure 2-9 and Figure 2-10 provide flow charts for flexible and rigid pavement rehabilitation design, respectively.

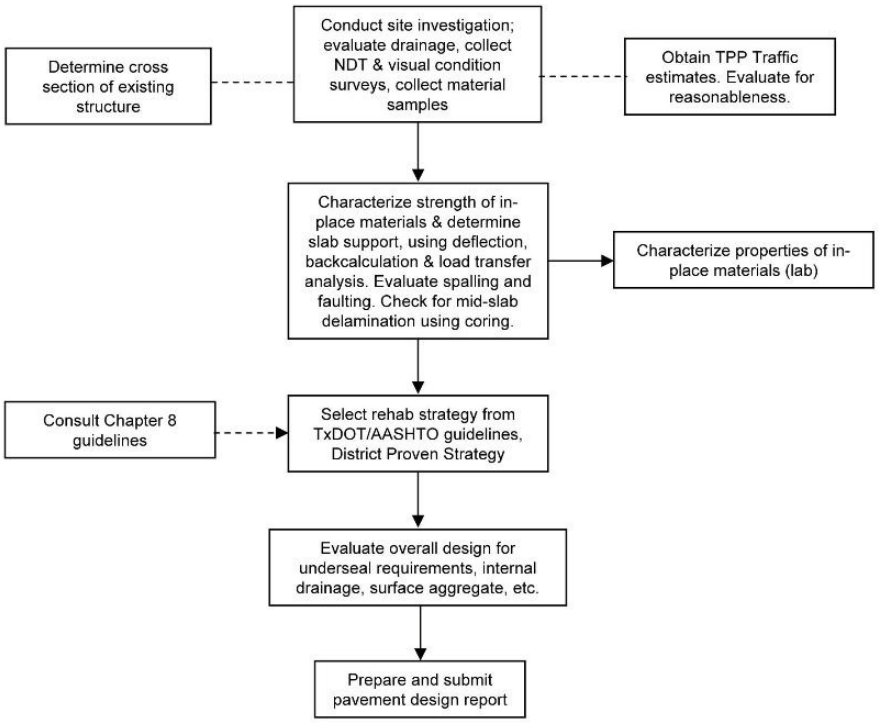


Figure 2-10. Design process for rigid pavement rehabilitation.

Section 8 — Information Needed for Pavement Design

Overview

Specific and accurate information is needed and critical for effective decisions regarding pavement design and rehabilitation. The information will also be included in the pavement design report. This section discusses the major requirements critical to a pavement design:

- traffic loads
- serviceability index
- reliability (confidence level)
- material characterization
- drainage characteristics and
- evaluating existing pavement conditions.

Traffic Loads

One of the primary functions of a pavement is load distribution. Therefore, in order to adequately design a pavement, representative loading characteristics must be presumed about the expected traffic it will encounter. Loads, the vehicle forces exerted on the pavement (e.g., by trucks, heavy machinery, airplanes), can be characterized by the following parameters:

- tire loads
- axle and tire configurations
- typical axle load limits
- repetitions of axle loads
- traffic distribution (by direction and lane)
- traffic projections (growth rate).

Traffic loads, along with environment, damage pavement over time. The simplest pavement structural model asserts that each individual load inflicts a certain amount of unrecoverable damage. This damage is cumulative over the life of the pavement and when it reaches some maximum value, the pavement is considered to have reached the end of its useful service life.

Pavement structural design requires a quantification of all expected loads a pavement will encounter over its design life. This quantification is usually done in one of two ways:

1. **Equivalent single axle loads (ESALs)**. This approach converts axle configurations and axle loads of various magnitudes and repetitions ('mixed traffic') to an equivalent number of 'standard' or 'equivalent' loads.
2. **Load spectra**. This approach characterizes loads directly by number of axles, configuration, and load. It does not involve conversion to equivalent values. Structural design calculations using load spectra are generally more complex than those using ESALs since the impact of each specific axle load is evaluated.

Both approaches use the same type and quality of data, but the load spectra approach has the potential to be more accurate in its load characterization.

Tire Loads

Tire loads are the fundamental loads at the actual tire-pavement interface and are generally assumed to be equal for all tires on any given axle. For most pavement analyses, it is assumed that the tire load is uniformly applied over a circular area. Also, it is generally assumed that tire inflation and contact pressures are the same.

Axle and Tire Configurations

While the tire contact pressure and area is of vital concern in pavement performance, the number of contact points per vehicle and their spacing is also critical. As tire loads get closer together their influence areas on the pavement begin to overlap. At this point, the design characteristic of concern is no longer the single isolated tire load, but the combined effect of all the interacting tire loads. Therefore, axle and tire arrangements are quite important.

Tire-axle combinations (see Figure 2-11) are typically described as:

- single axle – single tire (truck steering axles, etc.)
- single axle – dual tires
- tandem axle – single tires
- tandem axle - dual tires

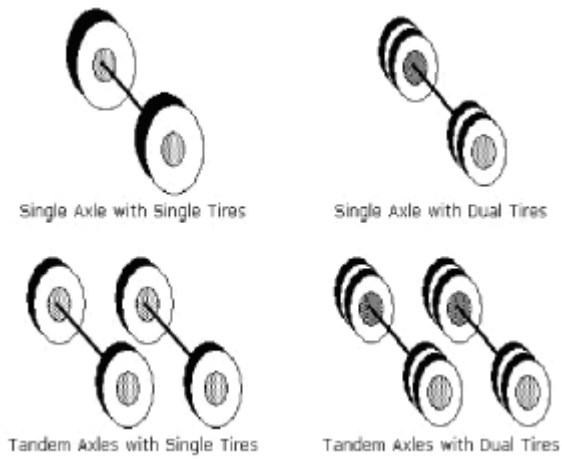


Figure 2-11. Tire axle configurations.

Other axle configurations exist (tridem [or three axle], quad [or four axle]), but generally represent a small fraction of the entire population.

Typical Axle Load Limits

Federal and state laws establish maximum axle and gross vehicle weights to limit pavement damage. The range of weight limits in the U.S. varies, based on federal and state laws.

Table 2-4: Axle Load Limits*

Axle**	Limits (lb.)
Single Axle	20,000
Tandem Axle	34,000
Gross Vehicle Weight	80,000
*Based on various federal and state laws. **Limits for tridem and quad axles are generally governed by the bridge formula:	

$$W = 500\{[LN/(N-1)] + 12N + 36\}$$

Where:

- W = load limit for the axle group
- L = distance in feet between the extreme axles within the group
- N = number of axles in the group.

Repetitions of Axle Loads

Wheel and axle loads for an individual vehicle are not difficult to determine. However, the number and types of wheel/axle loads a particular pavement will be subject to over its entire design life becomes complicated to determine. It is not the wheel load, but the damage to the pavement caused by the wheel load that is the primary concern.

There are two basic methods for characterizing axle load repetitions:

1. **Equivalent single axle load (ESAL)**. Based on AASHO Road Test results, the most common approach is to convert axle configuration and axle loads of various magnitudes and repetitions ('mixed traffic') to an equivalent number of "standard" or "equivalent" loads. The most commonly used equivalent load in the U.S. is the 80 kN (18,000 1b.) equivalent single axle load (normally designated ESAL).
2. **Load spectra**. The proposed *M-E Guide for the Design of New and Rehabilitated Pavement Structures* (NCHRP 1-37A) essentially does away with the ESAL and determines loading effects directly from axle configurations and loads. This is a more precise characterization of traffic but relies on the same input data used to calculate ESALs.

   A typical load spectrum input would be in a form of a table that shows the relative axle load frequencies for each common axle combination (e.g., single axle, tandem axle, tridem axle, quad axle) over a given time period. Load spectra data are commonly obtained from weigh-in-motion stations.

Traffic Distribution

Along with load type and repetitions, the load distributions across a particular pavement must be estimated. For instance, on a six-lane interstate highway (3 lanes in each direction) the total number of loads is probably not distributed exactly equally in both directions. Often one direction carries more loads than the other. Within that one direction, not all lanes carry the same loading. Typically, the outermost carries the most trucks and is subjected to the heaviest loading.

As a result, pavement structural design should account for these types of unequal load distribution. This is usually accounted for by selecting a "design lane" for a particular pavement. The loads expected in the design lane are either a) directly counted or b) calculated from the cumulative two-direction loads by applying factors for directional distribution and lane distribution.

The 1993 AASHTO Guide offers the following basic equation:

$$w_{18} = D_D \times D_L \times \hat{w}_{18}$$

Where:

- w_{18} = traffic (or loads) in the design lane
- D_D = directional distribution factor, expressed as a ratio, that accounts for the distribution of loads by direction (i.e., east-west, north-south).

- Example: One direction may carry a majority of the heavy truck loads; that direction would be designed differently or, at a minimum, control the structural design. Generally taken as 0.5 (50%) for most roadways unless more detailed information is known.

- D_L = lane distribution factor, expressed as a ratio, accounts for the distribution of loads when two or more lanes are available in one direction
- $\hat{w}_{18}$ = the cumulative two-directional 18-kp ESAL units predicted for a specific section of highway during the analysis period.

For instance, on most interstate routes, the outside lane carries a majority of the heavy truck traffic.

The Transportation Planning and Programming Division (TPP) posts the directional distribution in the *Traffic Analysis for Highway Design* report, but this distribution is related to peak ADT distributions (the 30th highest hourly volume) that affect level of service for geometric design versus loading for structural design.

The assumption made in the *Traffic Analysis for Highway Design* report is traffic loading is equivalent in both directions. If the designer anticipates the truck directional distribution to be different from 50/50 or loads to be significantly greater in one direction, then this concern should be indicated in the request submitted to TPP for project level traffic data.

Recommended lane distribution factors for both flexible and rigid pavement design are:

- one or two lanes in one direction, use 100% one-direction ESALs
- three lanes in one direction, use 70% of one-direction ESALs
- four or more lanes one direction, use 60% of one-direction ESALs.

Traffic Projections

TPP provides traffic projections ("Single Source Traffic Data Operating Procedures" from the *Transportation Planning Policy Manual*, Chapter 3, Section 4). The designer must request a 20-yr. traffic projection for flexible pavements and 30-yr. traffic projection for rigid pavements from the Traffic Section of TPP.

NOTE: When it is cost effective for alternate bidding, use a 20-yr. traffic projection for rigid pavement structures with moderate to high traffic volume roads.

For pavement design purposes, external and internal requests for traffic projections should be coordinated with the district director of Transportation Planning and Development (TPD). The district will use Form 2124, Request for Traffic Data.[1] External requests for traffic data for purposes other than pavement design must be immediately referred to the open records

1. Available internally only.

coordinator assigned to the district, division, or office that received the request.

CAUTION: Close coordination is needed with the district TPD to ensure timely turnaround of traffic data for pavement structural design. For quick turnaround, check ONLY box "1" on Form 2124[1]. For check box "1" data, time frame to receive data is 7-10 business days.

If the district has need of other data options on the form (environmental studies, line diagrams, or corridor analyses), the district submitter should make clear on Form 21241 that box "1" requirements should be given priority. The data for check box "1" will be sent to the district in the quick turnaround time frame with the other requirements sent at a later date.

Units of Measurement. Traffic loading is only rendered in terms of ESALs (versus axle load spectra). This is the standard method of evaluating loads for highways designed by the department. ESALs are evaluated for flexible and rigid pavements differently as a result of empirical relationships developed following the AASHO Road Test. Estimates for each type of pavement will be different. ESAL estimates will also vary slightly based on the overall pavement structure (also a result of empirical relationships).

Loading Estimates. The default structure used by the Traffic Section of TPP for traffic loading estimates is an 8 in. rigid slab or a flexible pavement of structural number (SN) 3. These structural classes are for highways with light to moderate traffic, but ESALs generated for these structures result in more conservative structural thicknesses and are preferred for comparative design in alternative pavement type bidding. An option for highways with moderate to heavy traffic is to provide TPP with the closest estimate for the appropriate structural class in terms of SN (flexible pavement) or thickness in inches (rigid slab). For more information, refer to "AASHTO Design Procedure."

Projections and Special Factors. The designer must ensure that all pertinent information regarding characteristics of the project that influence traffic loading are made known to traffic analysts in TPP. Use Form 2124, Request for Traffic Data1, to request traffic data. For structural pavement design, supply administrative data on Form 21241 including:

- District, County, CSJ
- Highway designation (System and Number)
- Limits (beginning and ending geographical/offsets in decimal miles from highway intersection)

1. Available internally only.

- Beginning and ending reference markers and offsets
- Y/N. Is the project in the Unified Transportation Plan (UTP)? The UTP is the 11-yr. statewide plan for transportation project development.
- District Priority, Estimated Letting Date
- Existing Number of Lanes (total)
- Proposed Number of Lanes (total)
- District Contact Person and Phone Number

A location map of the project, noting any existing or proposed development that will be a traffic generator. This is critical. Be as detailed as possible in identifying traffic sources, include whether loading is anticipated to be heavier in one direction (e.g. gravel haulers leaving a pit, etc.). Other examples of special situations:

- a street that is or will be a major arterial route for city or school buses
- a roadway that will carry truck traffic to and from heavily used distribution or freight centers
- a highway that will experience an increase in traffic due to a connecting major high-traffic highway that will be constructed in the near future
- a roadway that will experience a decrease in traffic due to the future opening of a parallel roadway facility

- Check box "1" for "Basic Highway Traffic Data" for pavement design that includes:
  - Base year/Beginning year (year project will be open to traffic following the proposed construction/rehabilitation)
  - Desired forecast interval (20-yr., 30-yr., or both)
  - rigid pavement slab thickness. NOTE: The default is 8.0". If the slab will be thicker than 8.0", consult with CST-M&P before changing. The default value must be used when the alternalte bidding process is used (see "Loading Estimates" above).
  - AASHTO structural number for flexible pavements. NOTE: The default is 3. If the pavement structure will result in a higher structural number, consult with CST-M&P before changing. The default The default value must be used when the

alternalte bidding process is used (see "Loading Estimates" above).

Not specifically included on this form is a field for identifying roadbeds. When multiple roadbeds exist along a corridor, identify them by mainlanes or frontage roads and by direction, if necessary. Make clear for which lanes traffic data is being requested so the design will reflect traffic loading specifically for the intended project scope.

Districts should review the "Traffic Analysis for Highway Design" report to verify reasonableness. Trucks have the largest impact in ESALs and, therefore, the simplest check is to verify the percentage of truck traffic in the average daily traffic (ADT) through observation.

For low to medium volume highways, procedures are being developed to assist designers in deriving traffic data (ESALs, ATHWLD) from TPP mainframe Traffic Log (T-Log) files versus requesting a corridor analysis for design purposes.

For more information on traffic projection, refer to "1430: Obtain traffic data" in the *Project Development Process Manual*.

Serviceability Index

Serviceability is a concept derived during the AASHO Road Test. This concept is related to the primary function of a pavement structure: to provide the traveling public with a smooth, comfortable, and safe ride. A scale ranging from 0 to 5 is used to evaluate a pavement's present serviceability index (PSI); pavement with a rating of zero is impassible and a rating of 5.0 would be perfectly smooth. Figure 2-12 illustrates the concept of serviceability index.

All pavements, when newly constructed or rehabilitated, are expected to begin at a high level of serviceability with a decrease in serviceability over time and traffic loading as the pavement becomes more distressed and rough.

Typically, the initial serviceability of a pavement will be about 4.0 or higher (5.0 is a lofty, but, inaccessible goal and levels below 2.0 are not considered acceptable). Pavement managers strive to keep the terminal serviceability index at a level that will not be indicative of a public safety hazard or discomfort or be excessively costly in rehabilitative effort.

Smooth highways minimize dynamic axle loading; typically, the smoother a pavement begins its life, the longer the time to initiate rehabilitation. The change in serviceability over time is also

known as “performance.” General pavement performance relationships are illustrated in 'Pavement Performance Relationships.'

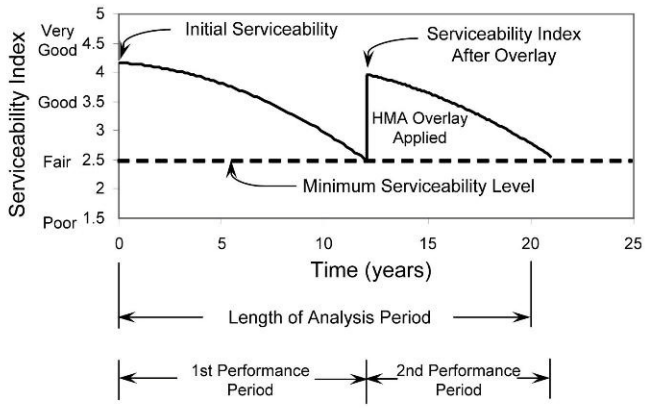


Figure 2-12. Pavement Performance Relationships.

Reliability (confidence level)

The concept of reliability as applied to pavement design can be defined as the probability that the pavement will perform as intended under the design traffic loading, and other crucial design inputs (material properties, environmental factors, etc.).

Reliability for pavements designed using the department’s recommended design programs is related to maintaining serviceability at or above the specified minimum (terminal) serviceability index throughout the desired performance period or design life. The department generally uses a reliability of 90 to 95% for rigid pavements and higher volume flexible pavements. Lower reliability, consistent with managed risks, may be appropriate for low to medium volume flexible pavements. For any given pavement structure, the serviceability at the end of this period is typically assumed to have a normal distribution with a mean and standard deviation.

The critical design inputs will have some variability associated with them; the assigned modulus values for the subgrade and other materials used in the structure, the construction process (in-place densities, layer thickness, etc.), traffic load predictions, and the design equation itself all have variability. Generally, the higher the reliability of a designed structure, the thicker the structure will be. Otherwise, the thickness must be offset using higher quality or stabilized materials. Since the assigned confidence level adjusts the calculations to account for material variability in the agglomeration of all inputs, including traffic, select average moduli for all inputs.

Material Characterization

For structural design of pavements, an accurate characterization and determination of layer moduli is desired, particularly for flexible pavement design. Pavement layers can be characterized with laboratory testing or field testing.

Laboratory tests can be used to determine the parameters that affect properties of materials, such as moisture susceptibility, stress level, strain amplitude and the strain rate. The moduli derived from these laboratory tests (triaxial loading, seismic):

- can be either smaller or larger than moduli determined from in situ testing and
- are not compatible with inputs required in FPS-19W.

Field tests are more practical and more preferred because they can be performed rapidly and may be used to test a large volume of material in its natural state-of-stress. On the whole, it is more likely the project can be better characterized using field tests rather than relying on the evaluation of samples taken from a few discrete locations.

Laboratory Measurements of Moduli

The common way to develop the stress-strain relationship is through laboratory tests. The laboratory method of choice in pavement engineering, especially for flexible pavements, is the resilient modulus (M_r) test.

The testing procedure for the resilient modulus consists of subjecting a specimen to a sequence of confining pressure and cyclic deviatoric stress levels. The load pulse consists of a haversine load with a duration of 0.1 sec. loading followed by a rest period of 0.9 sec. Up to 1,000 load cycles may be applied. The load applied to the specimen is monitored by a load cell. The resilient deformation is measured with Linear Variable Differential Transducer (LVDT).

The resilient modulus is the ratio of the repeated axial deviator stress divided by the recovered axial strain. Research is also exploring seismic tools to evaluate modulus of laboratory-prepared samples.

Base and Subgrade Materials

The base and subgrade materials, depending on their gradation and plasticity can be divided into two groups, fine-grained (cohesive) or coarse-grained (cohesionless or granular). The constitutive properties of both materials are defined based on the state of stress applied to them.

The recommended and simplified constitutive model can be expressed as:

$$M_r = k_1 \sigma_c^{k2} \sigma_d^{k3}$$

Where:

Mr = resilient modulus

S_c and sd = confining and deviatoric pressure, respectively

k_1, k_2, and k_3 - coefficients determined from the results of laboratory tests

The model is universally applicable to fine-grained and coarse-grained base and subgrade materials. For unbound materials, AASHTO T 307 is currently used to determine resilient modulus. Sealed latex membranes are placed around the molded triaxial specimens and subsequently placed in a sealed pressure chamber. The specimen and chamber are placed under a loading mechanism in a load frame. Constant confining pressure is applied to the membrane as axial loading is applied in haversine-shaped load pulses. Similar testing can be conducted on bound materials.

Bituminous Mixtures

Modulus of hot mix asphalt (HMA) can be determined in several ways. The most common laboratory tests are the resilient modulus, creep ("Tex-231-F, Static Creep Test"), and the uniaxial frequency sweep. Research is also exploring seismic tools to evaluate modulus of laboratory prepared samples and highway road cores.

Resilient modulus tests have been used by many researchers to measure the modulus of HMA. These tests can be performed either in compression or diametrically (ASTM D 4123). The following figures illustrate this test method's setup and results. Figure 2-13 shows the resilient modulus testing apparatus. Figure 2-14 shows the stresses applied to a confined specimen. Figure 2-15 shows the resilient modulus test cycle. Figure 2-16 shows the diametrical loading of HMA specimen for modulus testing.

In the creep test, the specimen is subjected to a static load. The displacement of the specimen due to the applied load is measured with time. Using the variation in compliance (ratio of the strain and stress) with time and time temperature superposition principle, the relaxation modulus can be determined.

The uniaxial frequency sweep test is very similar to the cyclic triaxial tests. The stresses and strains under sinusoidal loading and varying temperatures and loading frequency are measured and the dynamic modulus is determined. The dynamic modulus will be an HMA input parameter in the proposed M-E NCHRP 1-37A design process.

Several parameters affect the modulus of bituminous mixtures. The most important parameters are the rate of loading, temperature, and air void content.

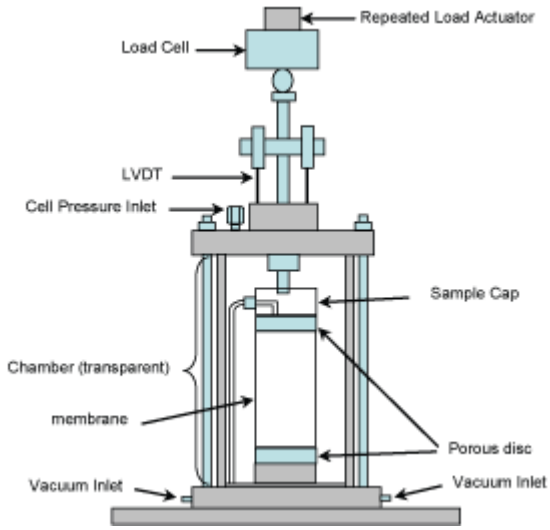
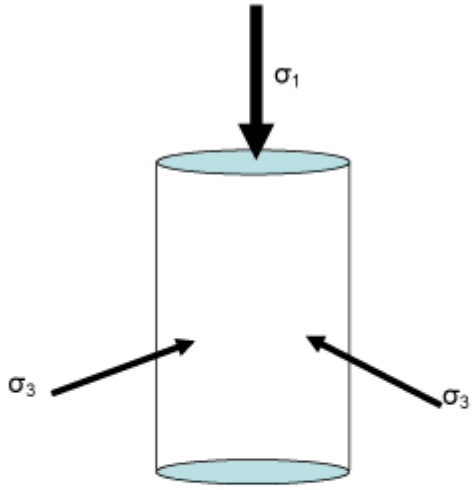


Figure 2-13. Resilient Modulus Testing Apparatus.



$$\sigma_d = \sigma_1 - \sigma_3$$

σ_1 is the total applied axial stress
σ_3 is the confining stress
σ_d is the deviator stress (axial stress in excess of the confining pressure)

Figure 2-14. Stresses acting on a triaxial specimen.

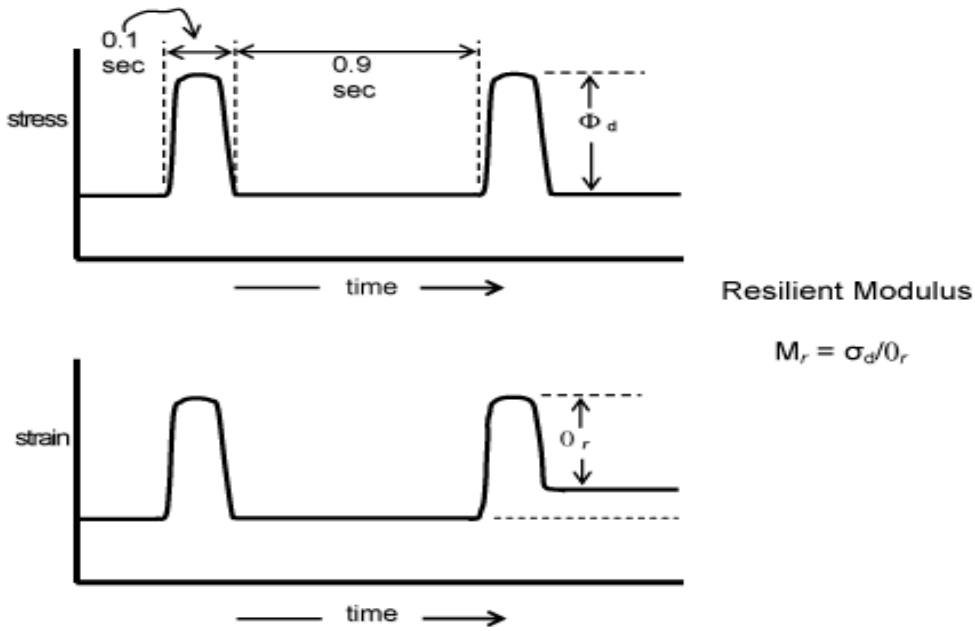


Figure 2-15. Resilient Modulus Test Cycle.

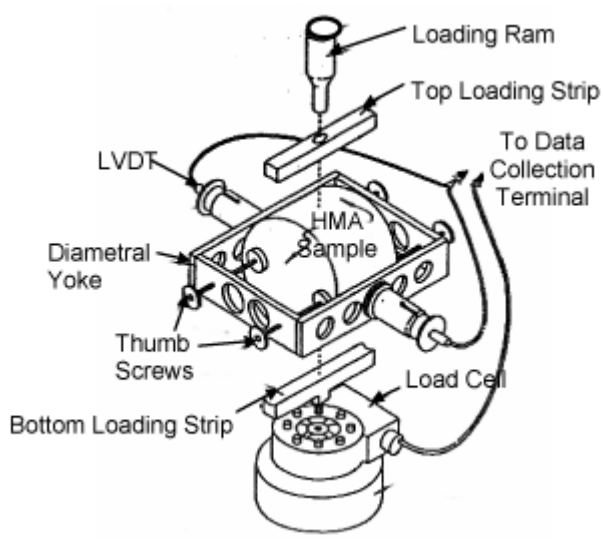


Figure 2-16. Diametrical loading of HMA specimen for modulus testing.

Field-based Measurements of Moduli

Several field-testing methods are available for determining the modulus of a pavement layer. The main methods used are either deflection-based or seismic-based. The falling weight deflectometer (FWD) is the most common field evaluation device used in Texas. The deflection measurements can be used in backcalculation methods to determine pavement structural layer stiffness, the subgrade elastic modulus, and the depth to stiff layer (bedrock). These are required inputs when using FPS-19W design.

'Backcalculation' is a mechanistic evaluation of pavement surface deflection basins generated by various pavement deflection devices. Backcalculation takes a measured surface deflection and attempts to match the value (to within some tolerable error) with a calculated surface deflection generated from an identical pavement structure using assumed layer stiffnesses (moduli).

The assumed layer moduli in the calculated model are adjusted until they produce a surface deflection that closely matches the measured one. The combination of assumed layer stiffnesses that results in this match is then assumed to be near the actual in situ moduli for the various pavement layers.

The backcalculation process is usually iterative and normally run with computer software. Backcalculation guidelines are discussed later (see "Pavement Evaluation") in the guide.

Other Material Characterization Methods

Chapter 4, "Pavement Evaluation" describes additional field and laboratory methods for characterizing material properties.

Drainage Characteristics

Drainage characteristics should be noted during a visit to the project site. Items such as the general terrain drainage, the highway drainage (including ditch depth/capacity), and any existing internal pavement drainage features should be noted.

Another drainage item to consider is bridge-class drainage structures. The number of bridges and how the existing pavement terminates at the bridge ends is important to note. Also, note if the bridges have bridge approach (rigid pavement) slabs. The condition of the bridge end/approach slab and the approach slab/pavement interface conditions are of special interest where concrete pavement is present. These pavement interfaces often provide a location for surface runoff to enter the pavement structure.

Internal (Positive) Pavement Drainage

Moisture intrusion into a pavement structure has been a known source of reduced service life since the earliest roads were constructed. The principles of drainage management for pavement structures have not changed radically since AASHTO published the 1986 *Guide for Design of Pavement Structures*.

In Vol. 2, App. AA of that reference, internal drainage systems are advocated particularly for "problem areas" as determined from experience and a drainage analysis of the particular project right-of-way. Federal-Aid Policy Guide's supplemental materials, 23 CFR Part 626, stresses

> ". . . inadequate subsurface drainage continues to be a significant cause of pavement distress, particularly in Portland cement concrete pavements." and "Where the drainage

analysis or past performance indicates the potential for reduced service life due to saturated structural layers or pumping, the design needs to include positive measures to minimize that potential."

Positive drainage measures are defined as permeable bases and the gathering and discharge system required for these bases. They are generally synonymous with internal or subsurface pavement drainage (underdrain) features. When internal drainage would potentially be used within the pavement structure, since 1994 the department philosophy on conducting a full drainage analysis, has been restricted to:

1. all rigid structures and full-depth HMA pavements that are 8.0 in. thick or greater
2. where rainfall is 20 in./yr. or more and
3. average daily traffic (ADT) exceeds 7500 vpd

The following is a list of exceptions:

- The proposed design has given good performance in the past (under similar soil, environment, and traffic) and this performance can be documented.
- Adequate ditches can not be constructed to collect water from the pavement structure due to right-of-way restrictions. A drainage system should not be constructed if it is susceptible to back flow from ditches or storm sewer during the 10-yr. flood.
- Future maintenance of the longitudinal edge drains can not be accomplished.
- In urban areas where frequent utility work may be needed.
- When movement is expected in full-depth HMA structures due to swelling clay subgrade soils.
- Where fast track construction is required or where the subbase must carry traffic during construction. A discussion must be included to cover the need for fast track construction and traffic handling.

The department's overall philosophy on pavement drainage design has been more focused on minimizing surface moisture infiltration or the effects of surface infiltration through construction and maintenance techniques such as constructing an adequate surface cross slope, maintaining proper ditch depth, using non-moisture susceptible materials, and aggressive use of seal coats and crack sealing, rather than on establishing internal drainage features.

Department Policy. Aspects of the department's policy are evident in many ways, such as, establishing a non-erosive base beneath rigid pavements, establishing QC/QA density requirements and anti-stripping evaluation for HMA. A substantial concern has existed over the maintainability of the internal drainage systems, including clogging of the permeable layer with fines, crushing of the permeable layer during construction, clogging of edge drains through rodent activity or sedimentation, crushing of edge drains, etc. A clogged drainage system is worse than no drainage system; it can keep the pavement in a state of saturation for a prolonged period.

As stated above, there is potential for back-flow if ditches can not be constructed with enough depth. However, there are occasions where positive pavement drainage may be considered a viable alternative, especially in cases where non-uniform cross sections exist or are planned. In particular, sections that have a "bath tub" nature where the outside edge of the structure is fairly impervious and will not allow lateral exodus of trapped moisture, positive drainage systems may be a good solution.

The case of retro-fitting edge drains on old rigid pavements with flexible pavement shoulders is one example. These structures tend to have a highly pervious longitudinal joint at the PCC-HMA interface that resists long-lasting maintenance solutions. An edge drain trenched into the shoulder at the interface with laterals to carry the water away from the structure can be effective in reducing or eliminating pumping under the slab. A similar situation can exist in flexible pavements widened using full-depth HMA.

Typical retro-fitted edge drain and permeable base systems are shown below in Figure 2-17. An example of positive drainage using a drainable base layer is shown in Figure 2-18. Variations can exist where laterals will empty into a storm drainage system.

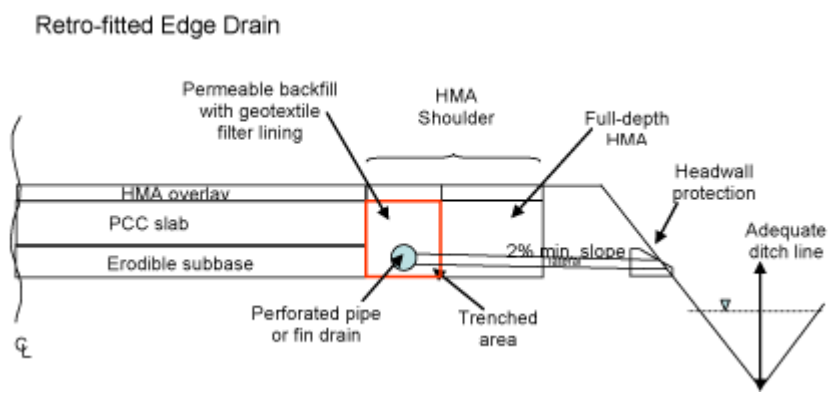


Figure 2-17. Retro-fitted Edge Drain.

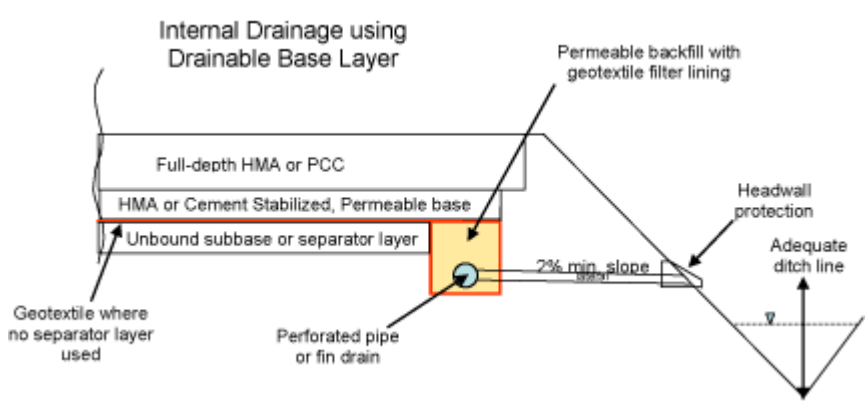


Figure 2-18. Positive drainage using a drainable base layer.

Another major source of free moisture into the pavement structure is ground water. The department's policy has been that ground water should be intercepted outside of the pavement structure to

eliminate the impact of this source. This should be pursued when seepage from higher ground is a problem. Moisture migration from capillary action or vapor movements can be addressed using the drainage options pictured above.

'Appendix AA' of the *1993 AASHTO Guide* and the *Drainable Pavement Systems Participant Notebook* for the FHWA Demonstration Project 87 (1992) are useful references for designing internal pavement drainage systems.

Other references: *1993 AASHTO Guide, NCHRP Synthesis 285* (2000), and the NCHRP Mechanistic-Empirical Pavement Design Guide (NCHRP Project I-37A), 'Appendix SS, Hydraulic Design, Maintenance, and Construction Details of Subsurface Drainage Systems.'

Cited references are available upon request from CST-M&P. There is a NHI training course, 131026, titled 'Pavement Subsurface Drainage Design' that describes pavement drainage techniques.

Evaluating Existing Pavement Condition

Evaluating a pavement's existing condition is necessary for any rehabilitation or reconstruction project as a means to determine the adequacy of past performance, determine failure mechanisms, and determine the extent of rehabilitative effort necessary. Evaluations are conducted to determine both structural and functional characteristics and can involve both destructive and nondestructive tests and surveys.

Destructive testing involves boring or trenching into the existing structure with the goal of obtaining samples for further laboratory evaluation and to observe the in situ condition of the various layers.

Nondestructive tests and surveys are available to acquire data that can be reduced to structural properties, presence of moisture, degree of distress, friction properties, and smoothness. Nondestructive surveys can also serve as a screening tool to identify locations to perform destructive testing. Pavement evaluation is discussed in , "Pavement Evaluation."

Section 9 — Pavement Design Reports

Projects Requiring Pavement Design and Pavement Design Reports

A pavement design and a pavement design report are required for the following projects that are over 500 ft long:

- new location projects (flexible and rigid)
- reconstruction projects (flexible and rigid)
- rehabilitation (3R) projects (flexible and rigid)
- hydraulic cement concrete (rigid) unbonded overlays of existing pavements

Tie-ins, such as bridge approaches, do not require pavement designs when following department or district proven standards.

A new design is not necessary. Previously approved design can be used if through an analysis, considering traffic, environmental and subgrade conditions, the pavement design analysis yields the same thickness. However, adjustments to designs thicknesses and specific conditions, even within a project, should be considered in the design process for economic purposes.

HMA overlays, approximately 2-in. thick and less, are considered a preventative maintenance treatment; therefore, a pavement design report is not required where adequate structural capacity is documented. Considering the significant investment thin overlays represent; these treatments should be taken into account in an overall pavement preservation program. An analysis should be performed that substantiates the appropriateness of this maintenance strategy.

The pavement design for special cases will typically be based on engineering judgment, historical performance, district policy, and other guidelines (e.g., this guide, industry guidelines, and research findings). A design report may be required for documentation purposes.

The following list provides examples of special cases that require documentation of the criteria and rationale for the strategy selected for projects greater than 500 ft long:

- approaches on a bridge replacement
- detours
- pavement widening including shoulders
- HMA overlays of rigid pavements. The only “rational” approach generally available to department engineers for designing a HMA overlay to an existing rigid pavement is the AASHTO Overlay procedure (automated in DARWin™). However, this process is highly subjective.
- bonded rigid overlays on rigid pavements

- thin whitetopping of flexible pavements.

For design categories not covered above, contact the district pavement engineer for guidance about recommended design procedures and documentation requirements.

Pavement Design Report and Other Documentation

A pavement design report is a formal engineering document that presents all analyses, data, policies, and other considerations used to design the structural aspects of a pavement. The pavement design report should include the following:

- Cover sheet showing highway designation, district, county, project CSJ, geographical limits, and signatures of persons involved in the preparation and approval.
- Narrative discussing the overall objective, site particulars (location, facility type, soil conditions and subgrade Texas Triaxial Classification [TTC], drainage considerations), PMIS data analysis/pavement condition surveys for 3-R projects, conclusions, and recommended pavement structure. The narrative should include a discussion of the factors that significantly affect pavement performance, and a summary of laboratory tests conducted on any materials extracted from the existing structure.
- If the pavement structure selected is different from the structure recommended by the design procedure, a discussion of the selection process must be included in the report.
- Location map. Maps should be detailed enough to distinguish urban or rural project locations, and the presence of water features such as lakes, streams, etc.
- Soils map of the project area with a brief description of each type of soil located within the project area. Provide information pertaining to shrink/swell potential and plasticity.
- The study of the presence of sulfate bearing compounds, organic content, and any mitigation technique selected.
- Determination of PVR mitigation requirements, if any. Obtain and provide approval to use PVR mitigation techniques when roadway characteristics do not meet policy criteria (see , Section 2 — Geotechnical Investigation for Pavement Structures)
- Existing and proposed typical sections. For the proposed structure, clearly define the various pavement layers, thickness, and materials with specification item. For the existing structure, sections should be as detailed as possible. Proposed or existing positive drainage systems should be indicated on the typical sections.
- The project specific factors used for selecting the pavement type.
- TPP Traffic Data and any adjustments to the traffic data.

- Identification of the base grade chosen, whether shown on the typical section or in the report text. Provide approvals for use of Grade 1 base, if required (see , Section 3 – Flexible (Unbound) Base Selection, for criteria and requirements).
- Results of NDT to characterize the existing structural condition (including the MODULUS backcalculation summary).
- Design input values and output.
  - FSP-19W summary, modified Texas Triaxial check, mechanistic checks, stress analysis, etc. for flexible pavement.
  - AASHTO (DARWin or TSLAB86) design summary for rigid pavements.
  - Alternate pavement design, if appropriate, using past successful practices.
- Conclusion. The pavement design report will conclude with a recommended pavement design based on the data, analyses, and procedures included in the report. The information included in the report should be a synthesis of all work performed to arrive at the recommended pavement structure.
- Appendices.
  - Surface Aggregate Selection Form 2088 (Wet Weather Accident Reduction program, flexible pavements only)
  - Additional appendices (results of borings, material lab tests, raw PMIS data, life-cycle cost analysis, drainage analysis, design exceptional approvals, etc.), as needed.

For other reporting requirements, contact the DPE for guidance.

Completing the Pavement Design Report

The pavement design report may be prepared by anyone with knowledge of the specific project under development and familiar with the analysis tools used. The first licensed engineer in the chain of responsibility will review and sign the report. After completion of the pavement design

report development phase, please complete the report by using the following procedure:

Table 2-5: Completing Pavement Design Report

Step	Action
1	◆ The District Engineer (DE) provides approval of final pavement design. In metropolitan districts, the DE may delegate final pavement design approval to the District Director of Construction or the District Director of Transportation Planning and Development for projects less than $20 million. ◆ The district pavement engineer (DPE) reviews the technical content of the report and appropriateness of the design for conditions cited within the report.
2	◆ Upon DPE approval of the report, the DPE signs and provides their respective profesional engineer (PE) license number on the report cover. NOTE: If the district does not have an assigned DPE, the report must be forwarded to the director of CST-M&P for review and endorsement. ◆ Forward to the DE for approval. If the report is approved, the approval is indicated by the DE signature and date on the report cover sheet.
3	Add the statement "This document is released for the purpose of interim review and is not intended for bidding, construction, or permitting purposes." after the engineer's license number on the approved pavement design report, in accordance with paragraph 137.33[e] of the Texas Engineering Practice Act.

Pavement Design Report Review and Archive

CST-M&P archives pavement designs for the state for the purposes of forensics and knowledge-based pavement performance. Districts are asked to transmit a scanned copy of completed pavement design reports to CST-M&P (Flexible Pavement and Rigid Pavements Branches).

Archived reports can be viewed on the TxDOT intranet (site not available to internet users) at the "Plans Online" page located at http://iplans/ under the heading, "Pavement Design."

Chapter 3 — Materials Investigation and Selection Information

Contents:

Section 1 — Introduction

Section 2 — Geotechnical Investigation for Pavement Structures

Section 3 — Flexible (Unbound) Base Selection

Section 4 — Performance Graded Binders (PG Binders)

Section 5 — Hot Mix Asphalt Concrete Pavement Mixtures

Section 6 — Hydraulic Cement

Section 7 — Reinforcing Steel

Section 8 — Hydraulic Cement Concrete

Section 9 — Geosynthetics in Pavement Structures

Section 1 — Introduction

Material selection forms one of the three legs of the high performing pavement "stool." Proper selection of materials and an understanding of how they perform in unison within the composite pavement structure must be based on careful consideration of expected traffic loads, the environment, and proven evaluation and construction practices. Other considerations, including availability of materials and economics, will often influence which materials are ultimately selected.

While it would be cost prohibitive to always demand the highest quality materials for every job, materials must be of sufficient uniformity and quality to provide reasonable performance under expected traffic loading and environmental conditions.

Proper use of specifications and test procedures are two ways to ensure the "quality materials" leg is adequate.

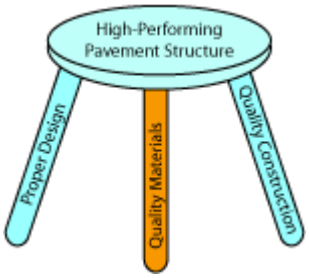


Figure 3-1. High Performing Pavement Stool.

Section 2 — Geotechnical Investigation for Pavement Structures

Introduction

All transportation systems are built either on, in, or with soil and products from the ground. Soil is arguably the most critical component of the transportation system, since most construction is dependent upon project soil properties and characteristics. The characterization and evaluation of soil is critical to the performance of pavement structures. The guidelines provided below will only address geotechnical considerations necessary for the design and evaluation of pavement structures.

This discussion is intended to provide TxDOT personnel, consultants, and contractors with guidance in:

1. **establishing soil properties and characteristics** to be used in pavement design. In particular, determination of soil strength, applicable modulus (its stiffness), and matrix stability descriptive of a pavement project, a portion of a project, or materials respectively, will be the result of the analysis. and
2. determining the **influencing site characteristics** that might require modifications to the pavement structure or adjacent works to accommodate those characteristics.

From this information, a report should be prepared that documents the findings from the geotechnical investigation.

Applicability

The guidance provided is intended for use by TxDOT personnel, consultants, and contractors involved in the planning, designing, evaluating, or construction of soil layers to be used or considered in pavement structures.

Although intended for all levels of involvement, the decisions that are generated from an investigation are critical to the performance of the roadway. Contacting the district pavement engineer and/or materials engineer to assist and review recommendations is advised.

Background

How the level of investigation is determined and when to perform them is a primary focus of this discussion. From project conception to construction and throughout the operation and maintenance phase, geotechnical information is necessary. Geotechnical investigations can be general and cover broad geographic areas, like initial site investigation and determination. They can be very detailed and specific, like identification of properties and characteristics of a single soil, and is often done in forensic studies.

Some of the more frequently asked questions with regard to conducting soil investigations for pavement design are:

- How do I get started?
- What information is needed?
- What test data do I need?
- At what interval do I need to retrieve material for testing?
- What test values are acceptable?

There are a number of variables that must be defined in an attempt to answer these questions. The direction of the investigation often depends on the nature of the project and the engineering properties desired (e.g., projects on new location, reconstruction, reclamation of roadway materials, and resurfacing or overlay, required cuts, or fills).

Scope of Guidelines

This guideline is intended solely for pavement applications.

There are numerous geotechnical and soils investigation guides that are relevant to other construction activities; most pertain to evaluations and analyses applicable to structures, slope and global stability, and retaining walls. For information on these subjects, refer to the Bridge Division's Geotechnical Manual.

Preliminary Investigation

Preliminary investigations are not complicated investigations. They require little time and are frequently the place to start if no other information or knowledge of the planned roadbed is available. A number of resources are available to planners and designers and may be obtained with little effort, such as, documents that reside within the office.

Site inspections are frequently conducted and encouraged in this stage as a means of optimizing the pavement design to reflect site conditions.

The idea is to:

1. determine the soil types cross the roadbed alignment
2. estimate the characteristics and properties of the soils present
3. use estimated soil characteristics, properties and potential project geometrics to predict problematic areas, materials, or conditions and
4. establish a testing plan for roadbed soils.

Project Initiation

Upon approval to proceed with project development, numerous activities begin. Planners and designers should begin their soils investigation at this stage to avert problems associated with poor soils or site conditions. Information of interest includes: alignment, type, and scope of the project. Information that is reviewed coincident with project initiation is existing data as discussed later.

- Alignment
  - At some point, the horizontal and vertical alignment of the proposed roadbed must be selected. Providing input about soil properties and characteristics as early as possible is preferable so informed decisions may be made.
  - Alignment is important because it can be influenced by the characteristics of the soils. When alignment has already been decided, the soils are fairly well defined, subject to verification.
  - Soil morphology, mineralogy, characteristics, and strength will all play a role in what manipulation, modification, or considerations are made in developing the pavement design.
- Project Type
  - The same information will be needed regardless of the project type being planned or designed. The requirements of each project type are differentiated by the scope of the information available and the influence of roadbed soils on pavement performance. A review of existing data can indicate the type of information readily available.
  - In all cases, determining the influence the roadbed soils will have or have had on the performance of the pavement structure is necessary. As a result, preliminary soil data and subsequent subsurface explorations are recommended at all times.

Project Type	Requirement
New Construction	This project type requires the greatest effort and time to establish new data or gather information about a roadbed that has not been compiled previously.
Reconstruction	When a roadway is excavated down to natural subgrade or imported fill material, requirements can be equal to or more than New Construction projects. Efforts will depend on the information compiled during the development of the previous pavement structure and the performance history of the roadbed being reconstructed. If prior severe distress was recorded, a detailed investigation may be necessary to provide an explanation.

Project Type	Requirement
Reclamation of Roadbed Materials	When severe distress or roughness is recorded prior to reclamation, soil investigation requirements can exceed those of New Construction. Where there is little severe distress, and reworking the subgrade is not required, the level of detail is substantially reduced. An evaluation of subgrade soils is warranted to ensure material selection, modification of soils, and structural section are compatible and sufficient.
Resurfacing/Overlay	The information required for this pavement type is minimal, but, often driven by the performance of the pavement. This project type often involves a cursory review of soils data and correlation to roadway roughness characteristics and distress manifestations. Assuming good performance, one may proceed to resurfacing for maintenance requirements or determination of design parameters for structural evaluation. A poorly performing pavement may be the result of roadbed soils; a detailed investigation would be appropriate.

Review of Existing Data

One of the first sources of information that can be easily obtained is data that resides in easily retrievable documents. A significant volume of information has been compiled by numerous organizations and agencies over the years. A significant volume of data and methods of representing the data are also available within TxDOT internal resources.

Table 3-1: Existing Data Sources

External Data Sources	Internal Data Sources
1. USDA, Natural Resources Conservation (absorbed and renamed Soil Conservation Service) 2. Bureau of Economic Geology, University of Texas 3. US Geodetic Survey maps 4. Aerial photographs 5. Maps 6. Commercial Internet 7. Engineering library files 8. Topographic 9. Satellite images	1. Previous project data files ◆ planning document ◆ pavement design ◆ boring data ◆ surveys 2. Maps ◆ official state travel ◆ control-sections ◆ county 3. Pavement Management Information System (PMIS) 4. Map Zapper (PMIS mapping routine) and relevant department databases on the server

Field Reconnaissance

Field reconnaissance, site investigations/inspections/visits, field surveys, and other such terms are commonly used to describe the process of traveling to the physical location of the proposed project. Often, this process identifies features, such as, soils, pavement phenomena, and traffic data either previously unknown or require confirmation.

Table 3-2: Reconnaissance Areas of Interest

Typical interest in field surveys	Inferences
1. Surface soil exploration	1. Soil classification, estimation of characteristics and properties
2. Physical layout and alignment	2. Geometrics to determine drainage characteristics, stability of side slopes and cut/fill requirements, steepness and high fills that can contribute to shrinkage cracking
3. Hydrology	3. Determining drainage conditions and drainage patterns
4. Topography	4. Cut/fill requirements, stability, drainage
5. Vegetation	5. Mitigate shrinkage cracking from vegetation in close proximity to roadway edge
6. Geology	6. Mineralogical, pedological (soils) evaluations

- Surface Soil Exploration
  There are numerous guides on how to perform quick field soil explorations. Many experienced pavement engineers have rules of thumb regarding appearance, consistency, smell, and taste. A number of these guidelines can be used to broadly identify soil types and characteristics.
  - Size and percentage of particles can determine whether material is coarse or fine-grained.
  - Consistency and feel of the soil in a dry state can indicate sand content.
  - The presence of several sizes of particles or whether there are few sizes can indicate a well or poorly graded material.
  - Wet materials that can be rolled into thin ribbons have some plasticity. The effort required to rework and successively roll the material can indicate its toughness. The tougher it is to roll, the higher the plasticity of the material. Generally, the higher the plasticity, the more clay will be present.
  - Wet materials that exhibit hardly any plasticity can be silts or sands or organic materials. Silts can be soft and may roll into a ribbon but will quickly crumble; whereas, sands may not be able to withstand any rolling at all.
  - Organics are normally fibrous, dark grey, and have a musty odor from the decayed matter.
  - Moisture content can give an indication of degree of moisture saturation and propensity for moisture movement within the subgrade.
  - Presence of certain sulfur-bearing compounds such as gypsum or lignite can indicate further testing is required.

TxDOT uses a modified version of the ASTM Unified Soil Classification system that is explained more thoroughly in Tex-142-E,"Laboratory Classification of Soils for Engineering Purposes."

- When two material types are present, it is common for the materials to be given a dual designation (SC-sandy clay, GC-clayey gravel). This information can be valuable in determining general soil properties as identified in many of the available resources and can serve as a source of data to confirm the data gathered.
- Physical layout and alignment
  - Terrain features can help determine whether borrow sources might be required, the potential challenges in providing suitable drainage and the stability of side slopes. Both fill and materials at roadbed grade level should be sampled and tested as described in later sections.
  - The presence of other infrastructure can affect the long term performance of the subgrade. Curbs and gutters adjacent to roadways provide special challenges for retaining strength and support from soils in moist environments.
- Hydrology
  - Water resources are not always evident on every site. This information may come from boring logs while conducting subsurface exploration. Seepage and standing water should be noted as these conditions will have a profound effect on project requirements—both in managing the condition and structural requirements.
  - Pockets of trapped free water may escape detection until construction reveals their location. The situation may dictate removal using positive drainage measures (e.g., French or trench drains, structures) as necessary. Check drainage from existing local pipe underdrains, culverts, or RCP pipes tied into drainage inlets.
- Topography
  - As with physical layout and alignment, cut and fill sections will require additional consideration, whether in terms of revealing existing fill material different than the surrounding subgrade, stabilization, etc., or illuminating a requirement to modify existing or import different materials.
  - Side slope stability in undulating terrain may require special fill materials to ensure pavement stability.
- Vegetation
  - Although vegetation is normally beneficial, having large trees or other vegetation requiring a sustainable water supply if it is close to the roadbed or roots that are close or under the roadbed are most likely detrimental. These conditions create a greater chance of subgrade desiccation leading to soil shrinkage and possible cracking. This phenomenon is most evident in soils with higher plasticity indexes (Tex-106-E, "Cal-

culating the Plasticity Index of Soils") and large shrinkage potential (Tex-107-E, "Determining the Bar Linear Shrinkage of Soils").

- Geology
  - From a surface survey, identifying soil mineralogy, presence of rock, potential for sulfur laden soils, and general support potential is possible. As much as the visible evidence of soil layering is useful, the absence of visual evidence also reveals soil characteristics.
  - The presence of rock at the surface can indicate shallow bedrock conditions. How massive or weathered the rock is can indicate subsurface support characteristics. Visual identification of sulfur bearing minerals is problematic when materials are to be chemically modified. The erosion potential of a soil can also indicate support conditions, existing drainage patterns, and whether specific drainage features will be necessary.

Preliminary Evaluation

Subsequent to the "desktop survey" and site visit, piece together data and information to formulate requirements for structural support, subsurface explorations, non-destructive testing, and unique or problematic materials.

- Structural Support
  - From the information gathered in the preliminary stage, it is entirely possible to develop a trial pavement design. The older Soil Conservation Service (SCS) maps or online USDA, Natural Resources Conservation Service Web Soil Survey may be used to identify the soil series that cross the proposed roadbed alignment. The catalog of soils series data developed jointly between TxDOT and SCS can provide an estimate of the subgrade or natural soil strength, as measured by Texas Triaxial Classification.

    The triaxial classification may be used to estimate subgrade modulus which will be used in pavement design procedures. Additionally, the triaxial classification will allow the estimation of base and surfacing requirements as outlined in Modified Texas Triaxial Design Method and as discussed in the design of flexible pavements in this manual. In the absence of other data to determine modulus of materials, triaxial classification may be used to estimate subgrade modulus, which may be used to deterimine a preliminary pavement design.
  - Performing the initial design is not the end of the preliminary design process. From the trial pavement design, the resulting pavement structure should be evaluated for: 1) stability, 2) constructability, 3) cost, and 4) feasibility. It is possible that political, environmental, cultural, and engineering constraints will require that subgrade layers be modified or manipulated in some way to best achieve overall project objectives. Balancing trial pavement structure requirements and project constraints can assist in this process.
- Sampling Plans

- Sampling will primarily be taken from borings, undisturbed samples (Shelby Tubes), test pits, or hand sampling. The frequency at which samples are taken, depths of soils sampled, and the type of sampling required should be defined. Based on all existing data, locations should be able to be identified with stationing or some other reference system. Further discussion of sampling requirements may be found under "Subsurface Exploration."

◆ Non-Destructive Testing (NDT)

- This testing may proceed at any point in the preliminary or design stages. Although the methods and analyses are discussed in Chapter 4, "Pavement Evaluation," the objective of this testing is to arrive at modulus values representative of the entire or discrete sections of the roadbed.
- Testing devices frequently used in Texas, specifically for pavement investigations, include:

Falling Weight Deflectometer (FWD):	Backcalculation of deflection data may be used to estimate the modulus of the subgrade. Testing should be conducted on surfaced roadways. If testing is for a new location, it is often convenient to estimate the modulus by analyzing data collected on an adjacent roadway or one with a pavement structure that is predicted to be similar to the one planned. Testing is unreliable on unsurfaced materials.
Dynamic Cone Penetrometer (DCP):	It is a stretch to call this testing non-destructive, but there is little disturbance of roadway materials. Several correlations have been made to the rate of driving the rod into subgrade materials. From these correlations, one can estimate the soil stiffness and differentiate layers within 3 ft. of the tested surface, assuming that substantial differences exist. Evaluation of soils containing significant amounts of larger (>1.5 in.) aggregate may be problematic since these aggregates may not be easily "pushed aside," thereby, severely reducing penetration rates.
Ground Penetrating Radar (GPR):	Both ground-coupled and air-coupled units can be used to locate areas of high moisture or differing pavement strata. Since the air-coupled system penetration is limited to a depth of about 24 in., it can be particularly helpful: 1) when investigating shallow subgrade depths and 2) in locations where a significant difference in moisture content exists between the base and subgrade. Ground-coupled units can penetrate to great depths, but, are generally used to investigate unique phenomena, such as, trench settlement.

Other Advanced Devices and Techniques:	The application of these devices for determining material properties is currently being researched. Devices showing potential for implementation include the Seismic Pavement Analyzer (SPA), Portable SPA (PSPA), and Dirt Seismic Pavement Analyzer (DSPA) for soils.

- Non-destructive testing is not a substitute for soil testing, but the data collected from these activities can establish a confidence that the subgrade is being properly characterized. Much data can be collected and analyzed relative to the time requirements and effort expended on laboratory tests for physical samples. Its productivity allows correlation between physical sample characteristics and properties to NDT results for application over a broader coverage area.
- Although the production level is high compared to laboratory sample preparation and testing, the data collected is representative of one moisture condition. To rely completely on a single measurement at one moisture condition may cause serious errors in determining an appropriate design modulus.

Preliminary Investigation Conclusions

The result of a review of the existing documents and information that is readily available can be as varied as the extent of data obtained. Sometimes this process yields large amounts of valuable information and, at other times, nothing. Through this process, however, the goal is to at least obtain some of the information useful in defining subsurface investigation requirements and estimate the level of testing that will be required based on characterization parameters given in the following table.

Table 3-3: Project Characterization Parameters Related to a Geotechnical Investigation

Project Characterizations	Soil Characterizations
1. Proposed alignment	1. Geologic model
2. Project type	2. Soil identification
3. Evaluation of project feasibility	3. Estimation of soil characteristics
4. Position of natural drainage features	4. Estimation of soil properties
5. Hydrologic inferences	5. Preliminary stabilization requirements
6. General terrain and some estimate of cuts and fills required.	6. Guidance for subsurface exploration
.	7. Plan development for non-destructive testing.

Subsurface Exploration

A comprehensive subsurface exploration plan is necessary to communicate the intent and level of testing that may be required. Effectively communicating these requirements not only ensures that required data is obtained, but it serves as a plan to minimize resources expended.

- Proposed Testing
  - Communication with lab personnel can help determine the volume of material that might be required to perform the type and number of tests desired. Since there are limited in-house resources and funding often defines outsourcing, it will be necessary to minimize the number of tests and still obtain the level of data required to fully describe project site characteristics. Costs are typically 0.5%-1.0% of the project estimate.
- Location
  - As simple as it seems and obviously critical, locations for sampling have to be specifically identified and communicated to field personnel. Identify not only the geographical location of samples to be taken, but the depth schedule of sampling at each location as well.
- Sampling Method
  - The two sampling methods most often used are disturbed sampling, sometimes called bulk sampling, and undisturbed sampling. Each has its advantages depending on the tests being performed. Since bulk sampling rapidly provides sufficient material for laboratory testing, it is most commonly used. Undisturbed sampling is most commonly used to identify existing engineering properties and to make recommendations to the designer.
- Frequency of Sampling
  - Sampling frequency depends on the level of investigation, uniformity of soils, and the potential for detrimental reaction from chemical stabilization. General recommendations for various soil conditions are:

Uniform	0.5 to 1.0 mile
Non-uniform	0.25 to 0.5 mile
Highly variable	1,000 ft. to 0.25 mile
Potential sulfate bearing	500 ft.

- Depth of Sampling
  - Sample materials continuously to a depth of at least 15 ft. in areas with high moisture fluctuations. Where excavations will exceed this depth, sampling should be conducted to finished subgrade depth plus 2 additional ft.

    When materials change physical characteristics, a new bulk sample should be started.

Sampling

The two primary sampling techniques used in pavement material analysis are disturbed and undisturbed. Each is descriptive of the amount of disruption of the soil matrix from its natural or in situ state.

- Disturbed
  - Disturbed samples are frequently referred to as bulk samples. The materials are generally collected with a power auger with helical flights that raise the materials to the surface for collection. This method is efficient because a great amount of materials can be collected in a short amount of time.
- Undisturbed
  - Undisturbed samples are not frequently requested. For the most part, these samples are collected by contract geotechnical services. The advantage of having these samples is the ability to test materials with (relatively) little disturbance, at the moisture content and density which it was extracted.

Laboratory Evaluation

TxDOT's testing procedures contain the methods and processing requirements to accomplish each procedure. It is not the intent to repeat those methods in this document, but procedures used frequently are listed below and briefly discussed.

Table 3-4: Typical Laboratory Soil Test Procedures

Test Category	Test	Test Method	Significance
Visual Identification	Soil Classification	Tex-142-E	Use as a check to verify assumed soil properties
Index Properties	Particle Size Analysis	Tex-110-E	A quantitative determination of the distribution of particle sizes
	Moisture Content	Tex-103-E	Determines natural subgrade moisture for use in drainage and soil suitability analyses
	Plasticity Index	Tex-106-E	Defines the amount of moisture a material can hold without turning into a liquid, gives an indication of the potential volume change of the material, assists with classification, potential construction/stabilization characteristics, and a measure that has been correlated to numerous engineering properties
	Potential Vertical Rise	Tex-124-E	Swell potential of subgrade soils

Table 3-4: Typical Laboratory Soil Test Procedures

Test Category	Test	Test Method	Significance
	Moisture Density Relationships	Tex-114-E	Compaction control purposes during construction can provide stronger, more durable materials
Strength Properties	Triaxial Strength	Tex-117-E	Strength of subgrade materials
Chemical Properties	Determining Sulfate Content in Soils	Tex-145-E	Soil analysis to determine the presence and the quantity of soluble sulfates that could have detrimental reactions with chemical (calcium-based) soil modifiers
	Soil Conductivity	Tex-146-E	Field detection of sulfate soils
	Soil pH	Tex-128-E	Determining the alkalinity or corrosivity of soils
	Soil Resistivity	Tex-129-E	Corrosivity of subgrade soils

Material Evaluation

- Suitability
  - It is essential for the design engineer evaluating laboratory data to set minimum acceptable criteria. From a pavement design standpoint, any material in place should be either suitable or modifiable to a suitable state; additional thickness of pavement layers will be able to compensate for most soils. Since there are time constraints, political influences, costs, and other such criteria that often influence the judgment regarding a soil's suitability, this approach is often not feasible. There is not one criterion that can determine what is acceptable. All factors must be weighed and trial designs made with each alternative considered.
- Swell potential
  - Criteria established by the Pavement Design Task Force (PDTF 2009) was established and distributed to all district engineers. The "Guidance on Potential Vertical Rise" memo (available internally only) is intended to encourage cost-saving measures by not treating or replacing soil as a potential vertical rise (PVR) mitigation technique except for roadways where risk and comfort are of the highest importance.
  - Test method, Tex-124-E, "Determining Potential Vertical Rise," is the recommended procedure for determining PVR. A 15-foot soil column is recommended for the analysis to determine PVR. The least amount of PVR for design is 1.5 inches for main lanes (2.0 inches for frontage roads, when

allowed), or as established by the district SOP identifying the requirements. NOTE: The lower the PVR, the more conservative the design.

- Pavement structure design proposing to include PVR reduction strategies will require the review and approval of Construction Division unless the proposal meets all of the following four criteria:

  --Mainlanes

  --high speed facilities (>45mph)

  --average daily traffic (ADT) > 40,000

  --pavement type of continuously reinforced concrete pavement (CRCP) perpetual pavement or hot-mix asphalt concrete (HMAC) > 12 inches, or CRCP with a flexible pavement alternative in the PS&E

  If the proposal meets all of the above four critera or designing PVR is allowed by the district SOP, it will be submitted to the Construction Division for review only.

- In conjunction with the pavement design, designers should address the following in their submission for review and approval:

  --traffic volume

  --operating speed

  --pavement structure

  --historical performance of previous designs and construction

  --type of facility (e.g., freeway, urban arterial)

  --ability to perform corrective maintenance

  --soil strata study, degree of severity, and limits of detrimental soil

  --budget for the project

  --proposed PVR analysis methodology (e.g., Tex 124-E, depth of analysis)

  --proposed treatment strategy

  --presence of sulfates

  --constructibility

- Designs considering or incorporating PVR mitigation will be allowed if it is proposed (optional and not a requirement by

the department) by a design-build or CDA firm to address a pavement maintenance clause.

- Feasibility of chemical modification
  - Recent research has shown the potential for detrimental effects of introducing calcium based modifiers into sulfate laden soils. A protocol has been proposed and is discussed in the 'Guidelines for Modification and Stabilization of Soils and Base for Use in Pavement Structures.' The protocol evaluates the potential for the occurrence of detrimental reactions after the introduction of a calcium based stabilizer. If laboratory mitigation techniques are not successful, the wisest course of action may be to:

◆ replace sulfate bearing soils
◆ leave untreated and modify pavement layers
◆ leave untreated, modify pavement layers, and add geogrid, if necessary, or
◆ dilute problematic soils to a level of acceptability.

Stabilization Guidelines

Satisfactory pavement performance is largely attributed to a good foundation that provides adequate strength and low compressibility. Base and subgrade layers serve as the foundation of pavement structures. Structurally, base and subgrade layers must provide adequate strength and must distribute loads uniformly and effectively. This structural capacity is obtained by optimizing the material and engineering properties of the base and subgrade.

Frequently, in situ soils and local base materials do not meet the material and engineering properties required for good pavement foundation performance. Texas has some of the most expansive soils in the country, which cause distresses in many pavements around the state. Also, a large portion of pavement construction performed today consists of rehabilitating existing roads, which frequently contain reclaimed subgrade and/or base material layers that are inadequate for current or future traffic loading demands. In order to achieve needed engineering properties, subgrade soils and engineered materials (select fill and flexible base) frequently require modification.

Most materials are made suitable by incorporating chemical additives, such as asphalt, cement, fly ash, or lime. Each of these additives is effective when the material is designed and applied properly. Proper design and application of materials with additives will minimize premature failures of the material and pavement structure.

"Guidelines for Modification and Stabilization of Soils and Base for Use in Pavement Structures" is a document outlining the proper methodology of selecting, designing, and evaluating treated soils and base courses for pavement structures. This document also provides some basic knowledge on the various treatment methods, the goals of treatment, and the mechanisms each treatment method employs.

When soils and base contain soluble sulfates, use the "Guidelines for Treatment of Sulfate-Rich Soils and Bases in Pavement Structures" to identify the feasibility for treatment and construction consideration for incorporating chemical additives.

Geotechnical Summary Report for Pavement Design Development

Upon completion of the field investigation and laboratory testing program, the geotechnical engineer will compile, evaluate, and interpret the data and perform engineering analyses for the design of pavement foundation layers. Additionally, the geotechnical engineer will be responsible for producing a report that presents the subsurface information obtained from the site investigations and provides specific technical recommendations. An example Table of Contents for a geotechnical design report is shown below.

Since the scope, site conditions, and design/construction requirements of each project are unique, the specific contents of a geotechnical design report must be tailored for each project. In order to develop this report, the author must possess detailed knowledge of the facility. The report must identify each soil and rock unit of engineering significance and must provide recommended design parameters for each of these units. A summarization and analysis of all factual data is required in the report to justify the recommended index and design properties.

Groundwater conditions are particularly important for both design and construction; these conditions should be carefully assessed and described. For every project, the subsurface conditions encountered in the site investigation should be compared with the geologic setting to better understand the nature of the deposits and to predict the degree of variability between borings.

Table 3-5: Geotechnical Report

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1. Introduction
2. Scope of Work
3. Site Description
4. Field Investigation
5. Discussion of Laboratory Testing and Significance
6. Site Condition and Geologic Setting a. Regional Geology b. Site Geology

Table 3-5: Geotechnical Report

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7. Discussion of Findings
a. Soil and rock properties
b. Ground water conditions and drainage
c. Chemical analysis
d. Swell characteristics
e. Reactivity with chemical modifiers

8. Analyses of Data
a. Soil and rock strengths and moduli
b. Characteristics and properties of chemically modified soils
c. Determination of in situ material properties, if applicable

9. Conclusions and Recommendations
a. Feasibility and use of native materials
b. Recommendations regarding borrow materials
c. Chemical modification of native or borrow materials
d. Modulus values and strengths of native or borrow materials

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Section 3 — Flexible (Unbound) Base Selection

The base course in a pavement structure serves multiple functions, but the primary function is to supply foundational support and capacity to the pavement structure; to provide a stable course to minimize flexural tensile stresses in surface layers; and to dissipate stresses induced by traffic loading to subbases and weaker underlying subgrades. The selection of an appropriate base material for a pavement structure is dependent on the overall interaction of the base course with the entire pavement structure.

The "Flexible Base Selection Guide"[1] was created and distributed to assist TxDOT personnel with the selection of appropriate base grades. The descriptions that follow are contained in this guide; these bases are also most frequently obtained by Item 247. Other information such as a background for establishing the current specification, structural notes, base selection factors, and creation of local base selection decision is also included.

As outlined in the memo, "Use of Grade 1 Flex Base,"1 effective with the November 2009 letting, justification will be required to use Grade 1 flexible base under the following conditions:

- moderate to low traffic (less than 3,000,000 equivalent single axle load [ESAL]), thick surfacing (three or more inches of hot mix asphalt concrete), and without adequate shoulders (<3 feet)
- seal coat surfacing on roadways with moderate to low traffic with adequate shoulders (Š3 feet)
- any traffic loading with thick HMAC surfacing and adequate shoulders

The pavement designer will submit justification for use of Grade 1 base for the review and approval of the district engineer or their deputy in metropolitan districts. (See also , "Pavement Design Standard Operating Procedure (SOP)."))

Flexible Base Description

There are five different grades of flexible base in Item 247, "Flexible Base." Each grade serves a different purpose based on the application for which it will be used.

1. Available internally only.

Grade 1, Grade 2, and Grade 5 are the primary bases to be used as a structural layer in a pavement; Grade 4 may be used when the other grades do not provide the material needed for the specific application intended.

- Grade 1
  - When there is little confinement, a base material must provide its own stability. Grade 1 base is recommended for conditions that do not provide stability from the pavement structure. When there is no lateral support or confinement, the base may become unstable as vertical load is applied. In this case, the base must provide its own cohesion and stability.
  - When there is no lateral confinement, inherent stability is necessary. With an unstable base, the pavement will deflect under the smallest of traffic loads. At up to 3 inches of HMAC, pavement will rut and crack from the deflections.
  - In the case of low confinement or little lateral support, Grade 1 is more likely to protect the HMAC from failure at all levels of traffic. Grade 1 provides an uniform gradation but allows a higher minus #4 fraction than Grade 2.
  - Grade 1 is best intended for used with thin surfaces, little or no confinement (no shoulders), and moderate to high traffic. The gradation for Grade 1 will always meet Grade 2 requirements and will almost always meet the requirements of Grade 5.
- Grade 2
  - Grade 2 material can provide its own stability but to a lesser extent than Grade 1. Grade 2 material is a transitional base, which may or may not have high fines content and soft rock.
  - Grade 2 allows for more gap-graded materials (material passing No. 4 sieve and retained on No. 40 sieve). When comparing the two grades of material, Grade 1 material can lack material in the coarse-sand gradation band while Grade 2 allows a coarser gradation on the coarse aggregate section of the gradation band (material retained on No. 4 sieve and larger). As a result of reduced fines content or inherently less cohesive fines, the UCS may be much lower than other bases.
  - With Grade 2 materials, it is important to select application where the materials will be adequately confined to

perform as a stable, high quality base. If Grade 2 material is adequately confined, it can achieve and possibly exceed the strength levels of Grade 1 material. It may be less expensive than Grade 1. Districts may find that Grade 2 base satisfies a great percentage of its selection needs.

- Grade 3
  - Grade 3 base material is not recommended for base courses in pavement structures. This grade of material is primarily used for subbase courses or maintenance uses, such as back-filling pavement edges, rehabilitation, or shoulder work.
  - Consider using Grade 3 base for the construction of very low volume roadways and with material that has shown adequate historical performance. There have been occasions when local Grade 3 materials have been used on projects due to the lack of availability of other base materials or the expense of other base materials and the material performed satisfactorily.
- Grade 4
  - Grade 4 (properties shown on the plans) presents the flexibility to customize a base specification to address unique pavement and material design situations. Consider adjusting material requirements in Grade 4 for the following reasons:

    • Roadways with low traffic loading (<500,000 ESALs) Surfacing consisting of a seal coat or HMAC three or more inches deep and with or without shoulders can specify Grade 4 material with the gradation, plasticity index, liquid limit, and wet ball mill requirements of Grade 2. A strength requirement is not needed for these situations, as long as the available local sources have history of acceptable performance.

    • To improve the performance of mechanical properties (strength) - An increased demand for performance from a base course may require increased restrictions. Addtionally, Grade 4 specifications can add more stringent gradation, plasticity, or hardness requirements to increase the base strength and durability.

    This scenario can be useful for anticipated significant increases in traffic loading and when encountering weak subgrades. The use of higher quality can also reduce hot mix surface thickness and dissipate stresses more efficiently

than regular bases, creating better protection of soft underlying subgrade soils.

• To design subbase materials in pavement structures for specific applications - Some of these applications can include drainable or permeable subbase layers, separation layers, or PCC (rigid) pavement subbase layers.

- Grade 5
  - Grade 5 material allows harder rock with a lower fines content. Fines may be less sticky than those found in Grades 1 or 2. The material that meets this specification may have difficulty providing its own stability; therefore, it is recommended for conditions where stability is provided from the pavement structure and roadway features (shoulders, surface thickness, or other material placed over it).
  - Grade 5 base is a modification of a Grade 1 base and has most of its characteristics, except it may or may not meet the unconfined compressive strength (UCS), 45 psi, for Grade 1 bases. The ability of the material to meet the UCS requirement is dependent upon the gradation and the constituents of the binder material. Since the Grade 5 base has the potential for having non-cohesive fines but has strengths equivalent to Grade 1 base when confined, the compressive strength of 3 psi confinement is used for Grade 5 base requirements.
  - Unless Grade 5 base is used as a subbase under an appropriate base and with appropriate thickness, it is not recommeded for high traffic roadways with thin surfaces or for roadways with no shoulders.

Base Selection

Many factors are considered beyond material testing and acceptance in accordance with the characteristics identified in Item 247. Selection factors often considered are:

- availability and cost
- surface thickness
- subgrade stiffness and strength
- lateral confinement
- traffic volume and loading

From these factors, a number of different selection matrices may be generated. The applicability of these factors may be weighed quite differently in each district.

An example of a decision matrix for the selection of base grades is contained in Table 3-6. Grades recommended in a chart similar to Table 3-6 should be chosen based on local experience and historical performance.

Table 3-6: Example Flexible Base Selection Chart

Shoulder Width	Asphalt Surface Thickness	Traffic (Design ESALs)	Base Grades
<3’	Surface Treatment	< 500,000	4*
		> 500,000 and £ 3,000,000	2
		³ 3,000,000	1
	HMA < 3”	< 500,000	2
		> 500,000 and £ 3,000,000	1
		³ 3,000,000	1
	HMA ³ 3”	< 500,000	4*
		> 500,000 and £ 3,000,000	2 or 5
		³ 3,000,000	2 or 5
³3’	Surface Treatment	< 500,000	4*
		> 500,000 and £ 3,000,000	2 or 5
		³ 3,000,000	2
	HMA < 3”	< 500,000	4*
		> 500,000 and £ 3,000,000	1
		³ 3,000,000	1
	HMA ³ 3”	< 500,000	4*
		> 500,000 and £ 3,000,000	2 or 5
		³ 3,000,000	2 or 5

*Grade 2 requirements without minimum strengths or classification.

Section 4 — Performance Graded Binders (PG Binders)

General

The Strategic Highway Research Program (SHRP) produced a system of materials selection, testing, and mixture design named Superpave, for Superior Performing Asphalt Pavements. The Superpave binder specifications are performance-based, therefore, these binders are known as performance-graded binders in contrast to the older system of viscosity AC-graded binders. Generally speaking, any mix used hot through a hot mix plant should be specified with a PG binder.

AC (-5, -10) binders are typically used in surface treatment operations and aggregate pre-coating (see Item 300). Engineering properties believed to be related to the expected performance (maximum 7-day pavement temperature, minimum pavement temperature, loading duration based on truck speed, and traffic volume) are featured in the PG binder specifications to allow proper selection for a particular highway application.

Performance graded binder specifications carry a short-hand designation as explained below:

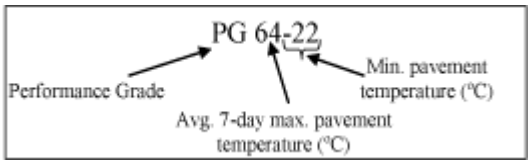

Figure 3-2. Performance Grade Binder.

Selecting a PG Binder

A "base" PG binder can be established for any region within the United States with sufficient climate data. For Texas, at the 95% confidence level, a PG 64-22 base binder can be used in most locations. Initial selection can be made using the computer program (PG Binder Grade Selection), or by using a set of maps (95% confidence level, 98% confidence level).

"Bumping" or increasing the binder high temperature rating by one or even two grades is predicated on building in stiffness to handle slow moving or standing traffic or very high traffic volumes. Stiffer binders may be required for certain mix types (e.g., both SMA and PFC both require a minimum PG 76-XX binder) to insure reliable performance.

Another practice not suggested by the SHRP research, but equally valid, is bumping the low temperature rating downward. This practice could be used to address cracking problems in a specific region by expanding the temperature range of the grade without stiffening the binder.

Where the potential for increased cracking problems exists, such as thin ACP surfaces on resilient bases, use caution when bumping the binder high temperature rat-

ing higher to address traffic issues. These conditions would require a stiffer binder and may cause or worsen cracking.

In addition to the PG requirement, the TxDOT specification contained in Item 300 also requires an elastic recovery test (ASTM 6084, Standard Test Method for Elastic Recovery of Bituminous Materials by Ductilometer) for all binders with a spread between the low and high temperature portions of the grade of 92°C or more. The effect of this requirement is to ensure that a polymer modifier is used in producing these binders. Higher temperature spread binders are generally more costly.

Binder selection also depends on the pavement cross section. Thin-surfaced flexible pavements are designed to deflect and rebound once a load has passed. Because sensitivity to strain levels increases with stiffness, stiff PG binders are not designed to be used under these circumstances. Strain levels are typically maximized by traffic loading at the bottom of a 2- to 3-inch thick hot mix asphalt (HMA) layer. Use of a stiff PG binder in these structures will severely limit the fatigue life of a stiff mix.

Additional guidelines in selecting a PG binder can be found in the document, "Superpave Binder Materials Selection Procedures."

Section 5 — Hot Mix Asphalt Concrete Pavement Mixtures

General

Hot mix asphalt (HMA) is a generic term that includes many different types of mixtures of aggregate and asphalt cement (binder) produced at elevated temperatures (generally between 300-350°F) in an asphalt plant. Typically, HMA mixtures are divided into three mixture categories: dense-graded; open-graded; and gap-graded as a function of the aggregate gradation used in the mix.

A variation on tradtional hot-mix asphalt is warm-mix asphalt (WMA). WMA technologies are processes or additives to HMA that allow mixture production and placement to occur at temperatures (30-100°F) lower than conventional HMA without sacrificing performance. Technology currently used to make the WMA process possible are chemical binder additives, chemical mixture additives, foaming admixtures, and plant modifications.

Additives and processes pre-qualified for use on department projects can be found on the approved WMA list.

Significant benefits derived from using WMA include:

- reduced fuel requirements for mixture production
- extended time available for compaction/workability, particularly for mixtures containing polymer-modified asphalt binders and thin lift/cool weather applications
- ease of incorporating reclaimed asphalt pavement (RAP)
- potential lower oxidation/improved fatigue life

The following information addresses HMA mixtures, but generally do not differ appreciably from WMA. Typically, the same parent test procedures or specification item applies to both types of asphalt.

Dense-graded mixes are produced with well or continuously graded aggregate (gradation curve does not have any abrupt slope change) and are specified under the current Items 340 and 341 (QC/QA). Typically, larger aggregates “float” in a matrix of mastic composed of asphalt cement and screenings/fines (see Figure 3-3).

Open-graded mixes are produced with relatively uniform-sized aggregate typified by an absence of intermediate-sized particles (gradation curve has a nearly vertical drop in intermediate size range). Mixes typical of this structure are the permeable friction course (Item 342) and asphalt-treated per-

meable bases. Because of their open structure, precautions are taken to minimize asphalt drain-down by using modified binders (A-R) "asphalt rubber," or by use of fibers. Stone-on-stone contact with a heavy asphalt cement particle coating typifies these mixes (see Figure 3-4).

Gap-graded mixes use an aggregate gradation with particles ranging from coarse to fine with some intermediate sizes missing or present in small amounts. The gradation curve may have a "flat" region denoting the absence of a particle size or a steep slope denoting small quantities of these intermediate aggregate sizes (see Figure 3-5). These mixes are also typified by stone-on-stone contact and can be more permeable than dense-graded mixes (Item 344, Performance-Designed Mixtures), or highly impermeable (Item 346, Stone-Matrix Asphalt).

Stone-matrix asphalt (SMA) will be missing most intermediate sizes but have a relatively high proportion of fines. Fibers or modified binders (A-R) are combined with these fines to build a rich mastic coating around and between large aggregate particles. Compacting and hand-working these mixes are usually more difficult than with either `dense-graded or open-graded mixes`.

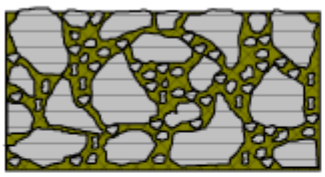

Fig. 3. 3a Dense-graded mix cross section

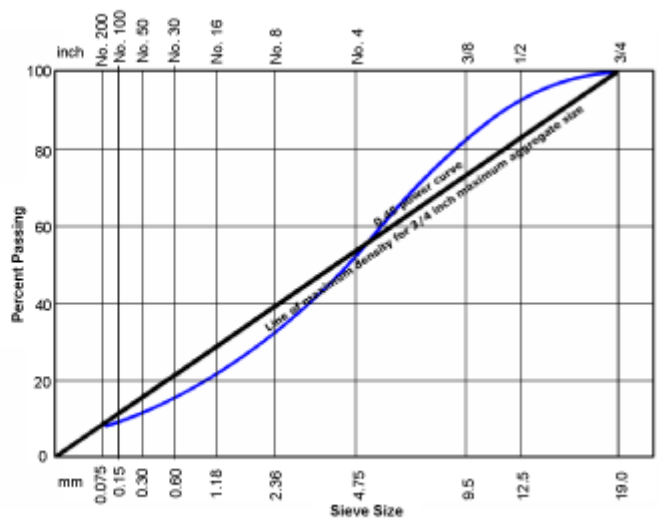

Fig. 3. 3b Typical gradation curve for a dense-graded mix.

Figure 3-3. Dense-graded mix cross-section and typical gradation curve for a dense-graded mix.

Fig. 3.4a Open-graded
(Permeable Friction Course) mix cross section.

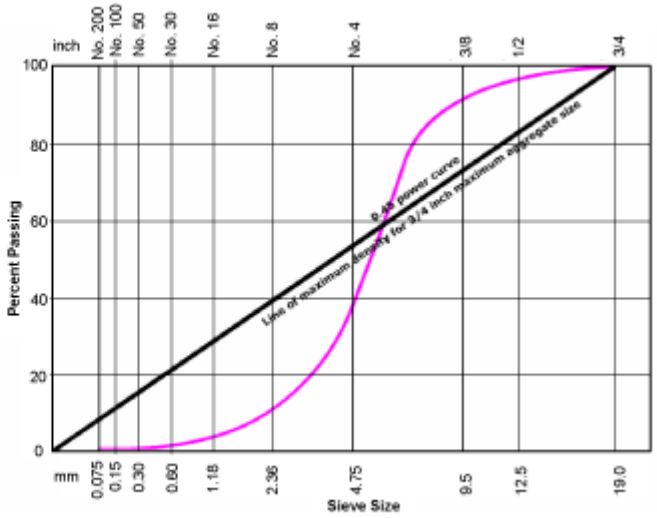

Fig. 3.4b Typical gradation curve for a open-graded mix

Figure 3-4. Open-graded (Permeable Friction Course) mix cross-section and typical gradation curve for open-graded mix.

Fig. 3.5a Gap-graded
(Performance-Designed or SMA) mix

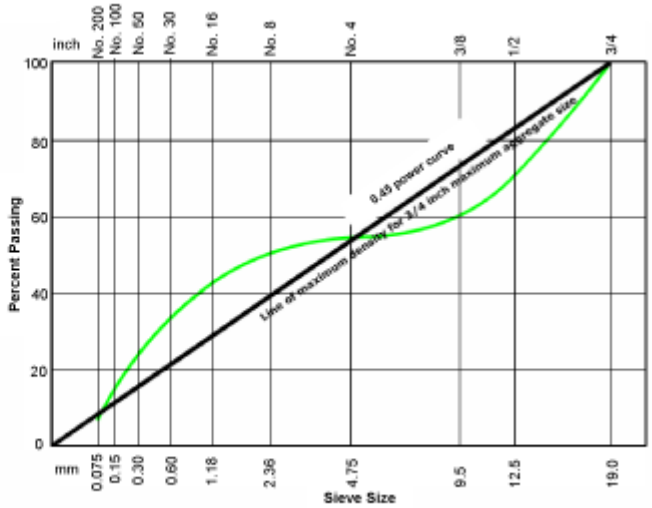

Fig. 3.5b Typical gradation curve for a gap-graded mix.

Figure 3-5. Gap-graded (Performance-Designed or SMA) mix and typical gradation curve for a gap-graded mix.

HMA MIX DESIGN

Mix design is performed in a laboratory using one of the procedures outlined in Tex-204-F, "Design of Bituminous Mixtures," when the applicable procedure varies roughly according to mixture categories outlined above. In addition, material quality, aggregate gradations, and other mixture requirements are given in each of the specific mix standard or special specifications.

Performance Concerns

Mix design seeks to address a number of performance concerns in the finished HMA mat (*Hot Mix Asphalt Materials, Mixture Design, and Construction,* Roberts, et al., 1996). These include:

- **Resistance to Permanent Deformation.** The mix should not distort or displace under traffic loading. The true test will come during high summer temperatures that soften the binder and, as a result, the loads will be predominantly carried by the aggregate structure.

  Resistance to permanent deformation is controlled through improved aggregate properties (crushed faces), proper gradation, and proper asphalt grade and content.

- **Resistance to Fatigue and Reflective Cracking.** Fatigue and reflective cracking resistance is inversely related to the stiffness of the mix. While stiffer mixes are desirable for rut resistance, design for rut resistance alone may be detrimental to the overall performance of the HMA mat if fatiguing or reflective cracking occurs. Stiff mixtures perform well when used in thick HMA pavements and can perform well when used as a thin overlay on a continuously reinforced concrete pavement (CRCP).

  Thin HMA mats placed on an unbound base or on surfaces prone to reflective cracking (e.g., jointed rigid pavements, bound bases subject to shrinkage cracking, etc.) should use a mix that strikes a better balance between rut and crack resistance. Fatigue and reflective crack resistance is primarily controlled by the proper selection of the asphalt binder. Application of a specialty designed crack-resistant interlayer is another option for mitigating cracking

- **Resistance to Low Temperature (Thermal) Cracking.** Cooler regions of Texas are particularly confronted with thermal cracking concerns. Thermal cracking is mitigated by the selection of an asphalt binder with the proper low temperature properties.

- **Durability.** The mix must contain sufficient asphalt cement to ensure an adequate film thickness around the aggregate particles. This helps to minimize the hardening and aging of the asphalt binder during both production and while in service. Sufficient asphalt binder content will also help ensure adequate compaction in the field, keeping air voids within a range that minimizes permeability and aging.

- **Resistance to Moisture Damage (Stripping)**. Loss of adhesion between the aggregate surface and the asphalt binder is often related to properties of the aggregates. The assumption on the part of the mix designer should be that moisture will eventually find its way into the pavement structure; therefore, mixtures used at any level within the pavement structure should be designed to resist stripping by using anti-stripping agents.

- **Workability.** Mixes that can be adequately compacted under laboratory conditions may not be easily compacted in the field. Adjustments may need to be made to the mix design to ensure the mix can be properly placed in the field without sacrificing performance.

- **Skid Resistance.** This is a concern for surface mixtures that must have sufficient resistance to skidding, particularly under wet weather conditions. Aggregate properties such as texture,

shape, size, and resistance to polish are all factors related to skid resistance. Under the department's Wet Weather Accident Reduction Program (WWARP)*, aggregates are classified into four categories (A, B, C, or D) based on a combination of frictional and durability properties. A friction demand assessment is made by the responsible engineer. The proper aggregate or blend (using categories A and B only) to achieve the assessed rating is then selected.

*Available through the TxDOT intranet only.

Design is facilitated by the use of a series of automated mix design programs in Excel format. Mix designs can be generated by either department personnel or by a consultant/contractor who is Level II (HMA mix design specialist) certified by the Texas Asphalt Pavement Association. Raw materials must be furnished by the contractor to the responsible department engineer to allow verification of the mix design.

Texas Gyratory Compactor (TGC). For dense-graded hot mix asphalt (Types A, B, C, D, and F of Items 340 and 341), the Texas gyratory compactor (TGC) is used to compact sample mixtures under Parts I or II of the procedure (see "Tex-206-F, Compacting Specimens Using the Texas Gyratory Compactor [TGC]"). The TGC uses a 4.0-in. diameter mold, with a target specimen height of 2.0 in.

Compactive effort is achieved by a combination of gyratory compactions governed by achieving a low pressure threshold, followed by uniform axial compaction achieving a high pressure threshold. Optimum asphalt binder content is derived by molding specimens at various binder contents, plotting the asphalt vs. density curve and selecting the binder content that corresponds to a lab-molded density of 96%.

Superpave Gyratory Compactor (SGC). Alternatively, larger stone dense-graded mixtures (Types A and B) can be designed using the Superpave Gyratory Compactor (SGC) under Part III of the procedure (see "Tex-241-F, Superpave Gyratory Compacting of Test Specimens of Bituminous Mixtures").

The SGC uses a 6.0-in. diameter mold with a target specimen height of 4.5 in. The larger diameter mold allows retention of material 3/4 in. or larger in the compacted sample whereas Parts I and II require removal of this material because of the smaller mold size. Samples of all remaining mixture types are compacted using the SGC. Samples are again prepared at various asphalt binder contents about the estimated optimum content. Plots are generated within the design software to evaluate optimum binder content at 96.0% density and to ensure other key parameters are met.

Mixture designs using the SGC are also controlled by the number of gyrations (N) required to achieve proper density and are related to the expected traffic loading. Depending upon the mix type, a N design (N_{des}) related to design air voids will be established (see "Tex-241-F" and "Tex-204-F," Part IV). N_{des} can be adjusted to ensure sufficient asphalt cement content and mix workability.

Traditionally, lab-molded specimens have been produced at an AC content that will yield a target density of 96% of the theoretical maximum density. Some variation is allowed to ensure mixes are workable under field compaction conditions thus mitigating tendencies toward very dry mixes and improving field achieved densities.

Voids in the Mineral Aggregate

Another mix design parameter that has significant impact on mix workability and durability is the voids in the mineral aggregate (VMA). Conceptually, this is the volume of space within a mix that is available for asphalt binder to occupy, as a result, this pavement design has a direct impact on the binder film thickness. For this reason, minimum values are placed on this parameter, specific to the mix type and gradation. Related to VMA is the voids filled with asphalt cement (VFA), the percent of the volume of VMA that is filled with asphalt cement. A range of acceptable VFA is a further control placed on Superpave mixtures.

Evaluating Mix Stability

Historically, mix stability for the traditional dense-graded mixes was evaluated using the Hveem stabilometer. The lab-compacted samples were subjected to axial compression and shearing resistance of the mixture was evaluated.

A more comprehensive evaluation of all hot mix asphalt mixtures for problems related to stability and moisture susceptibility (with the exception of permeable friction course and mixtures using asphalt-rubber modified binders) is now accomplished using the Hamburg Wheel Tracking Device, or simply Hamburg (see "Tex-242-F, Hamburg Wheel-tracking Test").

For the case of a dense-graded mixture designed using the Texas Gyratory Compactor (TGC), once optimum asphalt cement is determined, new samples are molded in the Superpave Gyratory Compactor (SGC) using the optimum asphalt content to 93% optimum density (a requirement for all lab-prepared Hamburg test specimens).

The Hamburg test uses a pair of abutting, trimmed samples placed in a 50ºC water bath. A weighted steel wheel passes back and forth across the surface; rut depth is evaluated per number of passes. A minimum threshold of passes resulting in a rut depth no greater than 1/2-in. is established based on the PG binder grade.

Tools to Improve HMA Mixes

Research project 0-5123 developed a methodology to design a balanced HMA mixture, considering both rutting (Hamburg) and fatigue (Overlay Tester) properties.

Recently, a new test device (Overlay Tester) was implemented at the CST-M&P laboratory to test mixes for cracking potential found in Tex-248-F.

Table 3-6: Tex-204-F Mix Design Options

Part	Type Mix	Compactor Used	Must meet	Mix Evaluation	Comment
I	Dense-graded Types A, B, C, D, F	TGC	Density - 96.0%[1] (Rich Bottom Layer [RBL] – 98%) Min. VMA by mix type	Indirect tensile strength (Tex-226-F), Hamburg (Tex-242-F), both at optimum AC content at 93 ±1% density	Mix designed by weight of constituent materials
II	As above	TGC	As above	As above	Mix designed by volume of constituent materials when aggregate stockpile specific gravities vary by 0.300 or more. Volumes converted to weights.
III	Large stone (Types A, B)	SGC	Density - 97% Min. VMA by mix type	Indirect tensile strength (Tex-226-F) at each asphalt content, Hamburg (Tex-242-F) at optimum AC content at 93 ±1% density.	Mix designed by weight of constituent materials.
IV	Performance-designed (Superpave, Coarse Matrix High Binder) SP-A, B, C, D, CMBH-C, F	SGC	Density - 96%[1] (Rich Bottom Layer [RBL] – 98%) Min. VMA by mix type. Design VFA for SP mixes. By plan note, designate stone on stone contact for SP and CMHB mixes. Density at N_{ini}, N_{des}, N_{max} must all fall in allowable range – SP mixes (summary worksheet of mix design program)	Indirect tensile strength (Tex-226-F), Hamburg (Tex-242-F), both at optimum AC content at 93 ±1% density.	Mix designed by weight of constituent materials

1. Consideration should be given to increasing the lab molded density to 96.5 or 97.0% where achieving field compacted density has been problematic.

Table 3-6: Tex-204-F Mix Design Options

Part	Type Mix	Compactor Used	Must meet	Mix Evaluation	Comment
V	Permeable Friction Course (PFC)	SGC	Min. optimum asphalt content of 6.0%. Lab molded density 78–82%. Max. allowable drain-down < 0.3%. No visible stripping by Tex-530-C.	Cantabro Loss (Tex-245-F) at optimum AC content at 78-82% density.	Mix designed by weight of constituent materials
VI	Stone-matrix Asphalt (SMA)	SGC	Density - 96% Min. VMA Min. AC content 6% for aggregate bulk spec. grav. £ 2.75. Must ensure stone- on-stone contact. Max. allowable drain-down < 0.2%. No visible stripping by Tex-530-C.	Indirect tensile strength (Tex-226-F), Hamburg (Tex-242-F), both at optimum AC content at 93 ±1% density.	Mix designed by weight of constituent materials
VII	Stone-Matrix Asphalt Rubber (SMAR)	SGC	Density - 97% Min. VMA higher for A-R binder Min. crumb rubber modifier content otherwise as above	As above	Mix designed by weight of constituent materials
1. Consideration should be given to increasing the lab molded density to 96.5 or 97.0% where achieving field compacted density has been problematic.					

The result of the mix design process is a job-mix formula (JMF), a starting point for the contractor in producing HMA for the project. The responsible engineer and contractor generally verify the JMF based on plant-produced mixture from a trial batch. The responsible engineer may accept an existing mixture design previously used by the department and may waive the trial batch to verify the JMF.

If the JMF fails the verification check using the trial batch, the JMF is adjusted or the mix may be redesigned. Additional plant-produced trial batches are run until the JMF is verified. During the course of the project, the JMF may be modified without developing a new mix design to achieve specified requirements as long as adjustments do not exceed tolerances established within the applicable mix specification.

Guidelines for Selecting HMA Mixtures

Selection of a HMA mix or combination of mixes to use in a project should be a conscious decision made by the responsible engineer based on mix attributes (suitability as part of the overall pavement design, existing pavement conditions, lift thickness, traffic loading characteristics), environment, past performance, local contractor experience, and economics.

Guidelines have been established to assist in the decision-making process in the form of a Mixture Selection Guide. The Guide provides general descriptions of the various HMA mixes used in the State (typical use, advantages, disadvantages); table ratings (subjective) of mixture characteristics for each of the mixture types; table of typical lift thicknesses; location within a pavement structure for each mixture type; and recommended choices for surface mixtures.

Use of Perpetual Pavements

Perpetual pavements take into account the increased structural demands due to heavy truck traffic, where cumulative one-direction traffic loading of more than 30 million ESALs over a 20-30 year design life is projected. By limiting the strain level at critical locations, these pavements are designed to have a virtually infinite fatigue and full-depth rut life, requiring only periodic surface renewal.

Stone Matrix Asphalt (SMA), Item 346, and Performance-Designed Mixtures, Item 344, should be used in lieu of conventional QC/QA dense graded specifications to ensure rut resistance within HMA layers. In addition, the use of a Rich Bottom Layer (RBL) should be considered when:

- new or "bottom up" reconstruction is anticipated
- full-depth HMA structures have a composite HMA thickness between 8-12 inches or
- there are concerns for bottom up moisture intrusion into less rich (higher air void) bottom HMA layer

Along with establishing an engineered foundation, using RBL is designed to help mitigate conventional bottom up fatigue cracking (see Figure 2-2).

Exceptions to the use of these improved performance mixes for high truck traffic pavements must be granted by the Director of CST-M&P or designated representative. If a district is interested in using these improved performance mixes on routes with less than 30 million ESALs, the district must obtain approval from CST-M&P.

Selecting Surface Aggregates to Comply With the Wet Weather Accident Reduction Program (WWARP)

The department is required to establish and maintain a program to ensure that pavements with good skid resistant characteristics are used. This program is commonly referred to as WWARP Program.

The department is charged with developing and implementing methodologies for the detection and improvement of locations with a significant incident of wet weather accidents using accident record systems and countermeasures to address those locations.

The department is also directed to utilize methods for the analysis of the skid resistant characteristics of selected roadway sections to:

1. ensure that pavements are being constructed that provide adequate skid resistance
2. develop an overview of the skid resistant properties of the highway system
3. provide information for use in developing safety improvement projects and the implementation of cost effective treatments at appropriate locations.

The WWARP allows the department to take advantage of the increased knowledge gained through our research efforts and to more effectively and efficiently address the various regional demands of Texas pavements. WWARP addresses three separate but interrelated phases of pavement friction safety. The three phases are accident analysis, aggregate selection, and skid testing.

- Accident analysis is the first phase and it consists of the identification, evaluation and improvement (as needed) for all wet weather accident locations. This information is contained in the Crash Records Information System. The Traffic Operations Division will furnish reports through the Construction Division (CST) to the districts on an annual basis.
- The second part of the program is aggregate selection. Each bituminous coarse aggregate source is classified into categories based on a combination of the frictional and durability properties of the aggregate. The classifications will be listed in the Bituminous Rated Quality Source Catalog provided (every 6 mos.) by the Geotechnical, Soils & Aggregates Branch of the Materials & Pavements Section of the Construction Division (CST-M&P).
- The third part of the program will consist of skid analysis and will include a mandatory collection of skid data that will become part of the Pavement Management Information System (PMIS).

Although CST-M&P has been delegated responsibility for administering the WWARP program, it is the district's responsibility to manage frictional properties for their pavements through sound engineering judgment and application of the program.

Section 6 — Hydraulic Cement

Hydraulic cements are defined as cements that not only harden by reacting with water but also form a water-resistant product. Hydraulic cements set and harden by reacting chemically with water. During this reaction, called hydration, cement combines with water to form calcium-silica-hydrates, which are the materials providing cementing actions.

The primary constituents of hydraulic cements are calcium and silica. Small amounts of iron, alumina, and sulfates also exist. Sources of raw materials for the manufacture of cements include limestone (for calcium), clay (for silica, alumina, and iron) and gypsum (sulfate). These raw materials are crushed, milled, and proportioned in such a way that the resulting mixture has the desired chemical composition and then fed into a kiln where temperatures of 2,600 to 3,000°F change the raw material chemically into cement clinker, grayish-black pellets about the size of 1/2-in. diameter marbles.

The clinker is cooled and then pulverized, resulting in hydraulic cement. In the pulverization process, small amount of gypsum is added to control the hydration of aluminates.

The four principal compounds of hydraulic cement are:

- tricalcium silicate, 3CaO SiO_2 (alite: C_3S)
- dicalcium silicate 2CaO SiO_2 (belite: C_2S)
- tricalcium aluminate, 3CaO Al_2O_3 (C_3A), and
- tetracalcium aluminoferrite, 4CaO Al_2O_3 Fe_2O_3 (C_4AF).

Tricalcium silicate (alite: C_3S) hydrates and hardens rapidly and is largely responsible for initial set and early strength. In general, the early strength of hydraulic cement concrete is higher with increased percentages of tricalcium silicate.

Dicalcium silicate (belite: C_2S) hydrates and hardens slowly and contributes largely to strength increase at ages beyond one week.

Tricalcium aluminate (C_3A) liberates a large amount of heat during the first few days of hydration and hardening. Gypsum, which is added to cement during final grinding, slows down the hydration to control the heat of hydration. Without gypsum, cement sets rapidly, called flash set. Large amounts of C_3A make cement vulnerable to external sulfate attack and, for sulfate resistant cement, its amount is limited to a maximum of 8% for Type II and 15% for Type III.

Tetracalcium aluminoferrite (C_4AF) reduces the temperature required to change the raw material chemically into cement clinker, thereby making the cement manufacturing process more energy efficient. It hydrates rather rapidly but contributes very little to strength. Most color effects in concrete are due to tetracalcium aluminoferrite and its hydrates.

ASTM C 150, Standard Specifications for Portland Cement, provides for eight types of hydraulic cements as follows:

Type I	normal
Type IA	normal, air-entraining Type I cement
Type II	moderate sulfate resistance
Type IIA	moderate sulfate resistance, air-entraining
Type III	high early strength
Type IIIA	high early strength, air-entraining
Type IV	low heat of hydration
Type V	high sulfate resistance

- Type I: A general-purpose cement suitable for all uses where the special properties of other types are not required. This is the most widely used cement type for pavement concrete.
- Type IA: A type I cement with small quantities of air-entraining material interground with the clinker during cement manufacture.
- Type II: When moderate sulfate resistance or moderate heat of hydration is desired.
- Type IIA: A type II cement with small quantities of air-entraining material.
- Type III: For use when high early strength is desired. For TxDOT paving concrete, this cement type is allowed only for Class HES concrete.
- Type IIIA: Air-entraining Type III cement, where air entraining is desired.
- Type IV: For use when a low heat of hydration is desired.
- Type V: For use when high sulfate resistance is desired.

There is a Type I/II cement that is widely used in TxDOT projects. This cement meets the requirements for both Type I and Type II cements. Therefore, this cement can be used for concrete where either Type I or Type II cement is required.

In addition, there are blended hydraulic cements available. Blended hydraulic cements are produced by intimately and uniformly blending hydraulic cement with other types of fine materials. The primary blending materials are fly ash, ground granulated blast-furnace slag, and other pozzolans. In Item 421, two classes of blended cements are used:

1. Type IP consists essentially of an intimate and uniform blend of Portland cement and Class F fly ash.
2. Type IS consists essentially of an intimate and uniform blend of Portland cement and ground granulated blast furnace slag.

Section 7 — Reinforcing Steel

Reinforcing steel used in concrete pavement includes longitudinal and transverse steel in continuously reinforced concrete pavement (CRCP), dowel bars in concrete pavement contraction design (CPCD), and tie bars for both CRCP and CPCD.

Longitudinal and transverse steel is designed to keep cracks tight to prevent water or foreign materials from penetrating the pavement structure or prevent seepage through the concrete cracks. Dowel bars are used to provide load transfer at the transverse contraction joints. Tie bars are installed at the longitudinal construction joints and maintain the concrete slabs' structure, which prevents lane separation.

For the steel to perform these functions satisfactorily, steel must possess certain engineering properties, which include the modulus of elasticity of 29 million psi and yield strength of 60 ksi. More specific requirements are stipulated in Item 440. The steel should meet the requirements as described in Items 360 and 440 and in the CRCP Design Standards CRCP (1)-03 and CRCP (2)-03.

Section 8 — Hydraulic Cement Concrete

Primary Ingredients

Hydraulic cement concrete is a composite material that consists essentially of cementitious material (Portland cement and supplementary cementitious materials such as fly ash and ground granulated blast furnace slag), aggregates (coarse and fine), and water and/or chemical admixtures.

Determining Ingredient Proportions

The properties of fresh and hardened concrete depend on, among other factors, the proportions of the above ingredients and, to a lesser extent, on the characteristics of coarse and fine aggregates. The process of determining the proportions of each ingredient in consideration of the desired concrete properties is called mix design. The American Concrete Institute (ACI) procedures under ACI 211 provide a process to determine the proportions of the ingredients.

Creating Workability, Durability, and Adequate Strength

In concrete paving operations, good workability and resistance to segregation are important. As for the hardened concrete properties, good durability, adequate strength, and less volume change potential due to temperature variations are characteristics that will provide good performance of concrete pavement. For the concrete to have good workability, durability, and adequate strength, two conditions must be met:

1. each component material should have desirable properties as required in Item 421 and
2. the proportions of the component materials should be optimized.

The current coarse aggregate gradations in Item 421 are more or less gap graded, which has less total coarse aggregate volume in a unit volume of concrete compared with well graded aggregate. TxDOT is currently evaluating concrete with optimized coarse aggregate gradation for paving operations. Concrete with more coarse aggregate volume will have a lower coefficient of thermal expansion (COTE) and heat of hydration, and less drying shrinkage, resulting in less potential for cracking and better pavement performance.

Three Concrete Classes

In the 2004 TxDOT specifications, there are three classes of concrete related to concrete pavement:

1. Class P
2. Class K and

3. Class HES.

Table 3-7: Concrete Classes

Class	Characteristics
P	◆ normal concrete paving
K	◆ once used for projects when high early strength was preferred, such as, full-depth or partial-depth repairs ◆ classified as structural concrete and subject to ASR mitigation options in Item 421.4.6 ◆ when used, strength requirements must be shown on the plans
HES	◆ developed for 2004 Specifications; intended for projects when high early strength is required. ◆ not classified as structural concrete and not subject to ASR mitigation options as long as the cement content does not exceed 520 lb. per cu. yd of concrete ◆ when used, default strength requirements are included in Item 360

Section 9 — Geosynthetics in Pavement Structures

Introduction

A geosynthetic, according to ASTM (1994) is a planar product manufactured from a polymeric material used with soil, rock, earth, or other geotechnical-related material as an integral part of a civil engineering project, structure, or system. Since ASTM provides this definition, the department has had a number of applications in materials other than the traditional "geo-" applications.

For the purposes of this manual, we will define a geosynthetic as "A manmade material that consists of one or more products used to provide added benefit to the infrastructure." In this manual, we intend to address only those roadway applications. Applications of geosynthetics for use in other structures are addressed in the Bridge Division's Geotechnical Manual.

There have been relatively few studies that have offered concrete evidence regarding geosynthetic material performance in the roadway. Although the department has conducted or is a participant in a number of studies, there is plenty of room for improvement in characterizing all geosynthetic materials to quantify their benefit to pavement performance.

Although improper usage or installation has contributed to early failure in some cases, department geosynthetic usage has increased in recent years with good success. Applications have been in asphalt concrete overlays of existing asphalt concrete and hydraulic (Portland) cement concrete surfaces, unbound (flexible) base, soft subgrade, drainage, and encapsulation. Shared terminology exists between these applications; clarification of the terminology will hopefully alleviate the confusion.

This section begins with a description of materials for applications that are most frequently used in the department. Following this, a discussion of the materials frequently used in each application and the expected behavior of the material is presented.

Description of Materials and Applications

Several materials are available for incorporation into pavement structures. The two geosynthetics primarily used are geotextiles and geogrids. Recently, manufacturers have combined the two materials creating a type of geocomposite (combination of a geosynthetic and another product).

Applications

Applications of geosynthetics are typically used by the department within pavement structures. Primarily, there are four applications:

Geosynthetic Applications

Application	Name	Description
1	Pavement Surface Layer Reinforcement	The first application should be separated and not confused with any other application in a pavement structure. This application is specific to hot-mix asphalt concrete.
2	Geotechnical Reinforcement	As it is used in this manual, refers to pavement layers only inclusive of subgrades and subbases.
3	Drainage	Drainage and moisture control are features of the pavement structure, not reinforcements. Their functions are to enhance and lengthen pavement performance by reducing the influence moisture has on pavement materials
4	Moisture Control	

Materials

The term geosynthetic is broad and encompasses numerous materials. A geotextile is a permeable geosynthetic made of textile materials; this material has uses in all applications. Numerous other materials may be used in combination with geotextiles to create a geocomposite—including grids, nets, meshes, and webs; these are most frequently used in HMAC but do have applications with soil moisture barriers and even soil erosion blankets.

Geogrids are primarily used for reinforcement and are formed from integrally connected and attached elements creating apertures in which adjoining material embed and are of sufficient size to interlock with it. The reference to geogrid is primarily reserved for the application in unbound bases and subgrades; although, reinforcing grids are available for use in HMAC. Geomembranes are low-permeability geosynthetics used as moisture barriers.

Geosynthetics for Surface Layer Reinforcement

Geosynthetics used for HMAC applications have been used by the department since the mid-1980s. Since then, there have been numerous products manufactured for various purposes to incorporate in the pavement surfaces. These products are used to:

- reduce reflective cracking from:
  - existing layers of HMAC or
  - cracks and joints in rigid pavements and
- resist moisture intrusion into lower pavement layers. Fabric underseal has worked well in some parts of the state when placed on CRCP and overlaid with HMAC.

Department research project 0-1777 investigated the use of several geosynthetics in HMAC. Researchers developed a guidance document, "Geosynthetic Guidelines," available on TxDOT's

Construction and Materials Tips webpage. This document discusses the advantages and disadvantages of using geosynthetics in HMA applications, guidance on the selection of materials, cost considerations, pavement design, as well as construction considerations.

Geosynthetics for Geotechnical Reinforcement

Several research projects have been conducted in an attempt to quantify the benefits of geosynthetics in unbound pavement layers. Applications range from a passive use of materials creating a separation between pavement layers to an active role when relying on geosynthetics to take on some of the structure's load.

Materials most often used in this application are geogrids and geotextiles. Many geosynthetics have multiple uses and can serve more than one function. For instance, geogrids are often used in a way to restrain base material during compaction or loading, but they also serve as a separation layer to prevent excessive migration and intermingling of pavement layers at interfaces.

Ongoing departmental research will attempt to better describe the interaction between pavement layers and geosynthetics used in pavement layers. Additional research on the contribution of geogrid to pavement performance is being conducted in a pooled fund study. As the findings are made available, this section will be updated.

Current usage in Texas has been for both restraint of pavement materials and for separation of materials. The department acknowledges the benefit of geosynthetics in pavement layers; however, there has been insufficient conclusive research to develop guidance with regard to reinforcement of unbound materials in pavement structures at this time. As a result, usage of geosynthetics is limited to separation and restraint and cannot be accounted for in FPS-19W design.

Separation

Separation of layers is intuitive. This mechanism places a physical barrier between two materials to prevent them from intermixing. Mixing of fine-grained soil particles into the overlying flexible base can create a significantly finer gradation over time, increasing the base suction properties and greatly decreasing its strength. The result of this placement is to maintain a viable structure for a longer period of time.

Another more recent application, and possibly the most quantifiable, is grid used as both a separator layer and a restraint to base movement. It has been used to prevent or reduce reflective cracking due to differential relative movement between pavement layers. An example of this use is between subgrades (or stabilized materials) with high plasticity index (PIs) that exhibit large volumetric shrinkage when moisture is drawn out due to weather, drainage, or vegetation demands and unbound flexible base. The grid holds the unbound material in a tight matrix allowing

the shrinking subgrade (or stabilized material) to move and prevent subgrade cracking from propagating to the pavement surface.

Restraint

Restraint is the mechanism of preventing or reducing the lateral movement of materials. This application is useful when soft subgrades are present and a platform for subsequent construction is required. Both grid and geotextiles have been used in this application. Grid is more likely to be used and can be very effective. When textiles are used, note that textiles must extend much more than geogrids to provide the same level of support.

A grid may be used for:

- mitigation of reflective cracking (see mechanism in separation)
- creation of a working platform on soft subgrades, sacrificial layer to obtain compaction of subsequent layers or
- a substitute in lieu of lime, e.g., sulfate laden soils, where lime is detrimental, or urban areas, where lime may not be tolerated or combinations of these and soft soils.

Geosynthetics for Drainage Applications

Among the products used in drainage applications are:

Table 3-1: Geosynthetics for Drainage

Material	Application
Geotextile	◆ Transmission of moisture to pavement edge. ◆ Trench lining to prevent intrusion of fine soil into drainage layers and structure. ◆ Wrapping drainage pipe to prevent siltation of the drain. ◆ Wrapping aggregate to provide confinement and prevent fine soil intrusion. ◆ Silt fencing. ◆ Erosion control logs.
Geomembrane	◆ Moisture barrier for pavement edges. ◆ Creation of retention/detention ponds. ◆ Encapsulation provision for confining and waterproofing material in the roadbed.
Geowebs	Composite material that provides both a drainage structure and separation with a geotextile.
Vertical Drains	Composite material that transmits water from the roadway, through a geotextile and down to a drainage structure.

Specifications and Testing

The following table shows the current applicable testing specifications and test procedures used to qualify geosynthetics used in the department.

Table 3-2: Departmental Material Specifications

Specification	Title	Geosynthetic	Application
DMS-6200	Filter Fabric	Geotextile	Geotechnical
DMS-6210	Vertical Moisture Barrier	Geomembrane	Geotechnical
DMS-6220	Fabric Underseal	Geotextile	HMAC
DMS-6230	Sediment Control Fence	Geotextile	Drainage
DMS-6240	Geogrid	Geogrid	Geotechnical
DMS-6250	Grid/Fabric Composite for HMAC	Geocomposite	HMAC
DMS-6260	Reinforced Fabric Joint Underseal	Geocomposite	HMAC
DMS-6300	Waterproofing	Geomembrane	Drainage/Waterproofing

Chapter 4 — Pavement Evaluation

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Section 5 — Destructive Evaluation of Pavement Structural Properties

Section 6 — Geotechnical Investigation for Pavement Structures

Section 1 — Overview

Pavement evaluations are conducted to determine functional and structural conditions of a highway section either for purposes of routine monitoring or planned corrective action. Functional condition is primarily concerned with the ride quality or surface texture of a highway section. Structural condition is concerned with the structural capacity of the pavement as measured by deflection, layer thickness, and material properties.

At the network level, routine evaluations can be used to develop performance models and prioritize maintenance or rehabilitation efforts and funding. At the project level, evaluations are more focused on establishing the root causes of existing distress in order to determine the best rehabilitation strategies.

Visual Condition Surveys

Visual condition surveys cover aspects of both functional and structural pavement condition, but generally serve as a qualitative indicator of overall condition. Specialized equipment is used to quantify both functional and structural properties of the pavement structure. Ideally, for any given section of highway, two or more evaluators would arrive at the same assessment of the section's current condition. However, there are still many aspects of pavement evaluation that are highly subjective. For example, in visual condition surveys, the percent of surface area affected by alligator cracking is highly dependent upon the visual acuity of the evaluator. Progress continues in automating the mapping of common surface distress to eventually eliminate this subjectivity.

Non-destructive Testing

In the area of non-destructive testing, data collected in the field are generally objective in nature, but often subjectivity appears in the data analysis and interpretation. Non-destructive testing is the collective term for evaluations conducted on an existing pavement structure that do not require subsequent maintenance work to return the pavement to its pre-testing state. This is generally desirable to minimize disruption to traffic, and is essential as a screening tool to determine locations where selective material sampling should be conducted to evaluate other material properties in the laboratory. As such, its focus is to assess in situ properties that can be used to evaluate the need for further "destructive" testing (i.e., coring, boring, trenching), location of that destructive testing, and the current structural capacity of the highway as related to layer stiffness and strength. Non-destructive testing methods can assess either functional or structural condition.

Destructive Testing

Destructive testing provides more detailed data about the pavement not possible to obtain through non-destructive testing. Such detailed data include:

- laboratory mechanical, physical, and chemical properties (obtained through coring, Shelby tubes, and trenching), and
- visual inspection of pavement layers through coring and trenching.

Section 2 — Visual Pavement Condition Surveys

Overview

This section will review condition categories currently evaluated in visual condition surveys by pavement type. For consistent and reliable survey results, it is important to decrease subjectivity and variability in the identification of distresses typically found on flexible and rigid pavements, and provide instructions in recording these observations in an "orderly and consistent manner." To this end, certification training for visual (human) raters is conducted annually in regional sessions.

The designer/planner is interested in the type, extent, and severity of visible distress or corrective action taken on previous distress (such as number of patches or length of sealed cracks). Depending upon the type of distress, occurrences are recorded in terms of percent area, linear feet per 100-ft. station, number per station, or number per section.

- For network level evaluation, these statistics are typically captured for each 0.1 mi. section and further summarized by 0.5 mi. section when entered into the Pavement Management Information System (PMIS) database.
- For project level evaluation, condition statistics may be summarized over the length of the project or in any other fashion amenable to the district planner/designer. Project level surveys are often conducted on foot in order to map the type and extent of distresses in detail. The root causes of these distresses must be addressed in any comprehensive rehabilitation strategy.

For more information on identification and cataloging of visual distress, refer to the PMIS Rater's Manual or contact the Research & Technology Implementation Office (RTI) at (512) 465-7555 for a copy of the pavement rehabilitation CDs which discuss typical pavement distresses, their causes, and actions required to remedy.

Flexible Pavement Visual Survey Condition Categories

Rutting (Shallow, Deep, and Severe)

Rutting is a surface depression in a wheel path and is a load-associated distress. Contributing factors to the rutting can range from:

- insufficient structure for the traffic loading (compressive shear failure in the subgrade or unbound base)
- unstable mixes due to uncontrolled asphalt cement content that may cause shoving and displacement of hot mix asphaltic concrete (HMAC)
- moisture sensitive materials (including stripped HMAC in non-surface mixes)
- post-construction consolidation of HMAC under traffic loads (air void content too high) or

- temperature-sensitive HMAC mixtures and inappropriate HMAC mix selection for the nature of the traffic loading.

Rutting is rated by area and severity, with the area measured as a percent of the section's total wheel path area. The severity categories are:

- Shallow, 0.25-0.49 in.
- Deep, 0.50-0.99 in.
- Severe, 1.00-1.99 in.

Rutting 2.0 in. or greater is counted as a failure. At the network or project level, rutting is generally measured using an automated rut bar. It can also be measured manually using a 6.0-ft. straight edge and a steel ruler. If the length of the straight edge is such that the entire width of the rut is not spanned, the resulting depth measurement will not represent the maximum rut depth.

Strategies to correct rutting must account for the origin of the rutting, with special attention to the pavement structure. For example, a thin overlay on top of a rut-susceptible or stripped mix would not be a suitable remedy because rutting would likely return in a relatively short period of time.

Patching

Patches are repairs made to previous distress, indicating prior maintenance activity. If done properly, patches can improve the long-term performance of the structure.

Proper patching should always involve saw-cut edges parallel or perpendicular to the direction of traffic, with excavation to the full depth of the weak material. Replacement material must be properly compacted with tacking of all cut HMAC surfaces to improve impermeability. Provided the patch addressed the full depth of the previous distress/weakness, the life of the patch and the surrounding patch/pavement interface will be extended by placing a full lane width seal or overlay. At the network level, this condition is evaluated in terms of feet of full lane-width patching. Improper patching can introduce a degree of roughness, further deterioration at the edges of the patch, or even failure of the patch itself if the underlying problem was not addressed.

Failures

Failures are localized areas of severe distress including erosion of the surface, cracking where blocks of surface material move about freely, or depressions. The pothole is the classic example, but failures can cover much larger areas than the typical pothole. One example of this is edge failure, where the surface has disintegrated perhaps due to heavy wheel loads encroaching on the paved edge where there is a lack of lateral support deeper in the structure. These distresses may pose a safety hazard and are at increased priority for at least temporary repair measures until a long-term fix can be completed.

Failures are typically a load-associated distress, but are often related to poor construction or moisture-susceptible materials (poor in-place HMAC density, surface HMAC lift not bonded to underlying HMAC, moisture-susceptible base). Even for a light rehabilitation strategy, these distresses must be patched prior to resurfacing. At the network level, this condition is evaluated in terms of number per 0.5-mi. section.

Block Cracking

Block cracking is a climate/materials related distress, where shrinkage of the bituminous surface or underlying stabilized base causes interconnected cracks that divide the surface into irregular pieces. The cracking pattern is much larger than alligator cracking, with blocks ranging from 1 ft. to 10 ft. on edge, and is not limited to the wheel paths. Rating is generally in terms of a percentage of the lane's total surface area. This distress is not a structural problem until the effects of traffic and the environment further weaken the pavement by allowing moisture infiltration and raveling/spalling of the crack edges.

Addressing this distress as part of a rehabilitation strategy may be as simple as sealing the cracks prior to placing a surface treatment. Additional considerations will be necessary where the extent of the cracking is severe or where cracks continue to be active during cyclic seasonal temperature changes.

Alligator Cracking

This distress is also known as fatigue cracking and is a traffic loading related distress that is initiated in the wheel paths. Alligator cracking consists of interconnected cracks that form small irregularly-shaped blocks (less than 1 ft. on edge) resembling patterns found on an alligator's skin. Where the appearance of this distress occurs relatively early in the pavement's performance period, its occurrence can also be linked to inadequate structural thickness (including thin HMAC surfacings), surface layer delaminations, construction practices and/or weak materials. Rating is expressed as a percentage of the total wheel path area for the rated lane.

A minimal rehabilitation strategy should include removal of the affected material and proper patching before placing a new surface. Attempting to seal or place an overlay over these cracks without proper patching will result in rapid reappearance of the distress.

Longitudinal Cracking

These cracks or discontinuities may appear anywhere along a shoulder or driving lane and run roughly parallel to the pavement centerline. For purposes of rating, the cracks must be at least 1/8-in. wide, show evidence of spalling or pumping, or have been previously sealed. Measurement is in terms of linear feet per 100-ft. station. Depending on the relative transverse placement in the lane, this distress may be either load associated (a precursor of alligator cracking in the wheel path) or environmentally associated.

Environmentally-induced longitudinal cracking may be the result of desiccated subgrade soils (edge drying) that causes cracks that reflect up through the pavement structure, widening Portland cement concrete (PCC) pavement joints reflecting to the surface, or poorly constructed longitudinal mat joints (with high permeability). How active these cracks are and their origin must be considered in any rehabilitation strategy. Crack activity can be observed by selecting a few representative cracked locations, and measuring the crack width during different seasons. The range of crack width can also be found by taking measurements following known wet and dry periods.

Transverse Cracking

These cracks or discontinuities travel at right angles to the pavement centerline. Minimum eligibility for rating criteria are as defined for longitudinal cracking. Measurement is in terms of number of transverse cracks per 100-ft. station, where cracks that do not extend across the full lane width are counted as a fractional (partial) crack.

Transverse cracking is almost always an environmental associated distress that can deteriorate further under traffic effects and surface moisture infiltration. This cracking generally results from natural shrinkage of chemically stabilized base and subbase materials, thermal cycling of the bituminous/HMAC surfaces, or reflection cracking from underlying PCC joints and active cracks. Material properties can also be a contributing factor, including asphalt concrete (AC) binder aging, stiffness of the HMAC, or percent stabilization used in the lower layers. The activity of the cracks and their origin must be considered in choosing a rehabilitation strategy.

Raveling

Raveling is the progressive loss of surface aggregates caused by weathering, traffic, or a combination of the two. In Texas, raveling is mainly associated with seal coats; however, it can occur in HMAC as well. Measurement is in terms of percent of the total lane area, and by degree of severity (low, medium, high). Contributing causes can be linked to excessive AC binder oxidation, low AC binder content, stripping of the binder, and HMAC segregation/high air voids. Corrective action can include applying a fog seal (short term), seal coat, hot-in-place recycling or conventional thin overlay.

Flushing

This distress is also known as bleeding and is described as the presence of excess asphalt on the surface of the pavement. The condition is generally more prevalent in the wheel paths and can be present in both HMAC and seal coat surfaces. Flushing can reduce surface friction and may contribute to a traffic safety hazard. Measurement is in terms of percent of the lane's total wheel path affected and by degree of severity (low, medium, high). Underlying causes can include high asphalt cement content, excessive densification (low air voids) of the surface mix, temperature susceptibility of the AC binder, soft AC binder, excessive tack, or even migration of AC binder from mixes in lower layers that are moisture susceptible. Corrective actions can include application of microsurfacing, a conventional seal coat (using stringent field control to monitor the asphalt application

rate), cold milling with subsequent seal or thin overlay, a permeable friction course, or a conventional thin overlay.

Rigid Pavement Visual Survey Condition Categories

Spalled Cracks

This distress is the loss of material at the edges of cracks in concrete pavement. In the TxDOT PMIS database, this term is applied to continuously reinforced concrete pavement (CRCP). When occurring in jointed concrete pavement (concrete pavement, contraction design [CPCD]), this distress is known as failed joints and cracks since it may occur at either an intentional joint or shrinkage crack (includes asphalt-patched spalls).

Rating is by number of cracks that show spalling at least 1-in. wide covering more than 1 ft. of the crack length. Spalling on CRCP longitudinal cracks is considered a rarity and is not rated. A spalled longitudinal crack on a CPCD slab is rated as one longitudinal crack if it travels from one transverse joint to the next transverse joint or from a transverse joint to an edge joint and measures more than half the slab length.

Crack spalling is due to excessive local pressure at the joint. This pressure may be due to a combination of traffic action, thermal expansion, and/or steel corrosion. The occurrence may be exacerbated by over-finishing/ excessive consolidation, where excessive surface paste exists, or improper curing.

If cracks become wide enough to allow the introduction of incompressible materials, subsequent expansion by the slab in warmer weather will cause stress concentrations where these materials are found, resulting in spalling. Also, in areas subject to freeze-cycles, water entering the crack may saturate the concrete around the crack. Freezing temperatures may result in freezing and thawing damage, such as spalling, around the crack. Excessive deflection at the crack from traffic loading may also result in spalling. Finally, if the slab is faulted at the crack, the spalling may be a result of traffic striking the raised edge of the slab.

As a minimum, corrective action will almost always involve cleaning and sealing cracks/joints, but may also include sawing stress relief joints, slab-jacking, patching using bituminous or polymer patching materials, or a thin bonded Portland cement concrete (PCC) or hot mix asphalt concrete (HMAC) overlay.

Punchouts

A punchout is a full-depth block of pavement, formed at the pavement edge, when one short longitudinal crack forms between two existing transverse cracks. The existing cracks are closely spaced, usually less than 4 ft. apart. The punchout is often rectangular, but some may appear in other shapes. Punchouts are most common in continuously reinforced concrete pavements. Where pun-

chouts occur in CPCD pavements, the distress is recorded as a failure. The boundary of the punchout will be severely spalled or faulted.

Formation is usually related to surface moisture infiltrating into the base through the closely spaced transverse cracks and the adjacent longitudinal joint, followed by erosion/pumping of the base and cantilevering of the small slab. Heavy load applications will connect the transverse cracks with a short longitudinal crack. The punchout progresses with spalling of the cracks, possible rupturing of the reinforcing steel, and eventually settlement of the punchout below the original surface of the pavement. In some cases the punchout may become dislodged and present a traffic hazard.

Incorporating a non-erodible base and tied concrete shoulders has been the preferred preventative design strategy. Full-depth PCC patching is generally the preferred method of repair, although asphalt concrete (AC) patches may be used temporarily to address safety issues. Rating is in terms of number of occurrences per mile.

Asphalt Patches

In rigid pavements, full-depth repairs to localized distress are often made using asphalt concrete as an expeditious, temporary fix. As with patching in a flexible pavement, proper patching should always involve saw-cut edges parallel or perpendicular to the direction of traffic, with excavation to the full depth of the slab. Patches should be maintained until such time that a full-depth PCC patch can be placed. Where base erosion was a contributing cause of the original failure, permanent repair measures must address the viability of support under the patch.

Asphalt patches are rated in terms of total number observed, with patches longer than 10 ft. counting as two patches. In the PMIS system, asphalt patches are a rated category for CRCP pavements only; when these patches occur in a CPCD pavement they are rated as a failure. Proper patch construction is discussed in Chapter 10, "Rigid Pavement Rehabilitation.".

Concrete Patches

Concrete patches are intended as "permanent" repairs of localized distress and as such should be properly tied in to the existing structure to insure load transfer and longevity. An area sufficiently large enough to include the entire distress-affected area must be removed, leaving clean, square edges prior to patching. Failure to do this will often result in propagation of distress immediately adjacent to the patch.

Current TxDOT standards require patches to be full-depth, full-lane width, and a minimum of 6 ft. in the direction of travel. Patches placed at joints require a minimum of 38 in. on either side of the joint. Concrete patches are evaluated in terms of total number observed. Proper patch construction is discussed in Chapter 10, "Rigid Pavement Rehabilitation."

Average Crack Spacing

This evaluation is made on CRCP pavements and is used as a method to obtain the percentage of transverse cracks that are spalled, and to determine whether the slab is behaving as designed. Very small crack spacing (<2.0 ft.) may be a precursor to punchouts, whereas large crack spacing (>10.0 ft.) may mean wider crack widths that allow non-compressibles in and may have poor load transfer. A recommended technique to evaluate this condition is to count the total number of transverse cracks in two 200-ft. sections (beginning and end of a typical 0.5-mi. section), and then average the results.

Failures

This is a CPCD condition category. Failures are localized areas where traffic loads do not appear to be transferred across joints or cracks. Failures are typically areas of surface distortion or disintegration.

Failures are evaluated in terms of total number observed and include the following distresses: **corner breaks, punchouts** (previously discussed), **asphalt patches** (previously discussed), **concrete patches** (previously discussed), **severe faulting, D-cracking, spalls** (asphalt filled or not – previously discussed), and **popouts** (>12 in. wide or long, >3 in. deep).

- A **corner break** is a crack (which may or may not be spalled or faulted) that travels from a joint to a slab edge. To be rated as a failure, the crack must intersect between 1 ft. and half way across each edge. Concrete patches that are spalled and/or faulted around all edges are rated as failures, not as patches.
- **Failed concrete patches** are simply patches where severe distress has reappeared following the patch repair.
- **Faulting** means that one edge of the pavement on one side of a crack is at least 1/4-in. higher than on the other side. **Severe faulting** is defined as an elevation difference greater than 2 in.
- **D-cracking** is a series of closely-spaced cracks that curve around a slab corner. They are concave away from the slab corner and concentric toward the center of the slab. The cracking will parallel the transverse joint and curve around to parallel the longitudinal joint. As D-cracking progresses, the cracks radiate outward from the intersection of the joints.

D-cracking is believed to be an environmentally induced distress. As water permeates through the slab, the bottom of the slab tends to be saturated. Freezing and thawing cycles will deteriorate the saturated aggregate near the bottom of the slab. The deterioration starts at the bottom of the slab and eventually progresses to the surface of the pavement.

- A **popout** is an aggregate or piece of pavement missing; forming a hole in the surface of concrete pavement. It can be round or oblong in shape. A weakened plane exists at the depth of the popout which may be the result of improper curing. Popouts usually occur with absorptive aggregates subjected to freeze-thaw temperature cycles.

Shattered Slabs

This is a CPCD condition category. Shattered slabs are formed when a series of cracks intersect to divide the slab into four or more pieces. Although the pieces still remain in their original position, they may settle below the original elevation of the pavement. Also, the intersecting cracks can be accompanied by severe spalling. The slab is so badly cracked that it warrants complete replacement. When five or more failures exist on a single slab, or one or more failures encompass more than half the slab's area, the slab is rated as a shattered slab.

Slab shattering is primarily due to a lack of subgrade support. The base materials may settle, contain voids, or may be susceptible to erosion, resulting in loss of support. Overloading the concrete slab will create excessive bending stresses where the slab has no support. The result is severe cracking, possible spalling, and settlement. Current TxDOT policy requires a non-erodible base beneath all rigid pavements.

Slabs with Longitudinal Cracks

This is a CPCD condition category. Longitudinal cracks are cracks that follow a course approximately parallel to the centerline of the pavement. These cracks are generally straight but they may curve slightly back and forth across the length of the pavement. Slabs with cracks that are over half the slab in length and show severe spalling or are faulted, are counted regardless of the number of such cracks in the slab.

Causes of longitudinal cracking can be related to environmental conditions or improper construction. One possible cause of such cracks is that the joints were not sawed quickly enough during construction (ideally, such joints should be sawed as soon as the concrete can support the sawing equipment). If the slab is too wide or longitudinal joints were not cut deep enough (refer to "Joints" in Chapter 9) plastic shrinkage and lateral thermal contraction can cause a longitudinal crack to appear. Also, swelling soils or loss of foundation support may cause excessive bending stresses in the slab. Finally, warping and curling stresses may be sufficient to initiate longitudinal cracking.

Working cracks (cracks that open and close due to temperature variations and/or traffic loading) are a structural concern – the origin must be addressed to allow for proper remedy.

Apparent Joint Spacing

This is a CPCD condition category. Some transverse cracks may become so wide that they look and act like joints. The crack must be greater than 0.5 in. wide across the complete width of the lane. These 'apparent' joints are important to monitor because they do not have load transfer capability outside of frictional contact and potentially become additional traps for incompressible debris that can cause further damage to the slab. At the network level, the minimum value recordable in the PMIS database is 15.0-ft. Rating should be accomplished by evaluating a 200-ft. section at the beginning and end of each 0.5-mi. section and averaging the results.

Apparent joints may form for a number of reasons ranging from excessive loading to poor construction practices to environmental reasons. Construction practice shortcomings can include

improperly sawn joints, poor dowel bar alignment, poor paste bond on dirty aggregates, and poor curing practices. Environmental contributors include swelling soils, loss of foundation support, and warping and curling stresses.

Section 3 — Non-Destructive Evaluation of Pavement Functional Properties

Non-destructive testing is used to test functional and structural properties of the pavement. This section discusses the use of non-destructive testing to evaluate the functional properties and the next section discusses its use for structural properties. Functional properties of pavements include:

- roughness, and
- skid resistance.

Roughness

Roughness is the absence or lack of smoothness in the longitudinal or transverse profile of the highway surface that causes poor ride quality. Roughness can be generated by localized surface or subsurface distress, poor construction quality (compaction, grade control, poor bonding of the surface HMA layer to the underlying HMA layer, etc.), or shrink-swell activity in the subgrade or fill soils. Proper diagnosis of the source of roughness must be made to insure that any corrective action addresses the source of the problem and will perform to expectations.

Pavements should begin their life (new or post-rehabilitation/reconstruction) in as smooth a condition as practical. Roughness begets additional roughness – more severe dynamic traffic loads are incurred which will accelerate deterioration at the weakest locations. As roughness increases, so do user costs and potential safety problems.

Longitudinal and transverse roughness statistics are measured by the profiler/rutbar vehicle at highway speeds. There are 16 of these vehicles assigned to the districts. For the longitudinal direction, the profiler uses a combination of two lasers, two accelerometers, and a distance signal to measure the inertial profile of each wheel path. Measurements are processed on board to compute serviceability index (SI) and the International Roughness Index (IRI). The SI scale of measurement is 0.1 – 5.0 with an SI of 5.0 being perfectly smooth. The IRI units are in./mi. with summary statistics every 0.1 mi. For more information, refer to the 'Little Book of Profiling.'

Transverse profile is evaluated in terms of rut depth and is measured by an 8-ft. wide van-mounted rut-bar with ultrasonic sensors that measure the distance from the bar to the pavement surface at five locations across the lane. Sensors are located in each wheel path, between the wheel path and 18 in. outside of the wheel paths. Average rut depth is computed on board with summary statistics every 0.1 mi.

Figure 4-1. Profiler/Rutbar Vehicle

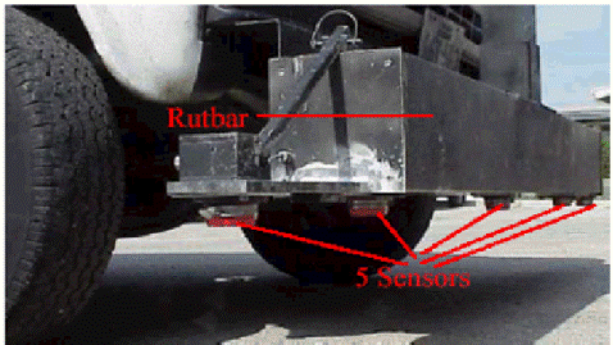

Figure 4-2. Detail of Profiler/Rutbar.

Skid Resistance

Skid resistance is a function of the micro and macro texture of the pavement surface. An index value is assigned (skid score) based on measurements taken with a locked-wheel skid trailer towed behind a specially designed truck. The trailer administers a water spray to the pavement in front of the left tire. Smooth-treaded trailer tires are used. Testing is accomplished at 50 mph by locking the trailer's left wheel at periodic intervals while a metered amount of water is sprayed on the pavement surface. Scores generally range from 10 to 40, with the higher number indicating greater skid resistance. There are seven of these systems assigned to the districts.

Figure 4-3. Skid Truck

Figure 4-4. Detail of skid trailer.

Section 4 — Non-Destructive Evaluation of Pavement Structural Properties

Urgency of the design, importance of the highway, value of the project, and availability of non-destructive tools usually influence what testing is performed. However, proper evaluation is essential to ensure performance; there is no substitute for comprehensive planning that includes reserving the equipment when needed.

The most common structural-based non-destructive tools used in TxDOT are discussed in some detail in the *Flexible Pavement Rehabilitation Training* (CD/ROM). Contact the Research & Technology Implementation Office (RTI) at (512) 465-7555 to request a copy of this CD. Practical exercises in the use of analyses software for data collected by these systems are also included with this training disc.

List of Non-Destructive Tools in Order of Availability

The general order of availability for non-destructive tools is as follows:

- **Falling Weight Deflectometer (FWD).** One device usually shared between two districts; there are 15 of these devices assigned to the districts. This device is essential in establishing the in situ stiffness properties of the pavement layers through analysis of the deflection data by backcalculation of layer moduli values using MODULUS. Moduli can then be used as design inputs to FPS-19.
- **Dynamic Cone Penetrometer (DCP).** Many districts have purchased their own (roughly $2500 per unit), two units are available at CST-M&P. This portable device is a secondary tool used to verify unbound pavement layer thicknesses, confirm the presence of lime stabilized subgrade, and evaluate the relative stiffness of unbound layers. It can prove useful in verifying MODULUS layer inputs and moduli outputs. A conversion equation developed by the Army Corps of Engineers is used to convert DCP data to moduli values. The DCP requires the boring of a small pilot hole through bound materials.
- **Air-coupled Ground Penetrating Radar (GPR).** There are currently four systems located throughout the state. The systems' locations are:
  - Fort Worth District
  - Odessa District
  - CST-M&P in Austin
  - through an interagency contract (IAC) with the Texas Transportation Institute (TTI) that is managed by CST-M&P, also in Austin, and
  - One additional system is scheduled for completion in FY 07.

This is a van-mounted system that provides a nearly continuous profile of layer thicknesses and dielectric variations to a maximum depth of about 24 in. beneath the pavement surface. Dielectric

properties are correlated to material density and moisture content so subsurface problems (stripping, trapped moisture) in the existing HMAC and excessive moisture in base layers can be detected.

GPR can also be used to identify segregation and low density in existing HMAs. GPR testing of PCC pavements has not been as successful as testing on HMAC and surface treated pavements due to the GPR signal interference by the reinforcing steel and the attenuation of the signal through PCC materials.

- **Ground-coupled Penetrating Radar (GPR)** testing is available through an interagency contract with TTI that is managed by CST-M&P. GPR testing involves towing one or more GPR antennas along the ground at walking speeds. Some antennas operate at a lower frequency than the air-coupled GPR units and therefore penetrate much deeper into the pavement and underlying layers. Ground-coupled GPR has been used to locate sink holes; search for buried underground objects such as abandoned tanks or test for water damage or other anomalies that are deeper than an air-coupled GPR unit can sense.
- **Seismic-based tools (Seismic Pavement Analyzer [SPA], Portable SPA [PSPA], Dirt SPA [DSPA], V-meter, Free-free Resonant Column).** There is one SPA system headquartered at CST-M&P. Three of each PSPA and DSPA systems are fielded at the district level. There is one of each system headquartered at CST-M&P.

The V-meter and Resonant Column have roughly the same distribution as the PSPA and DSPA. Seismic devices are slowly gaining acceptance as evaluation tools, as familiarity with their operation increases and improvements to the analysis software have been completed.

The SPA series of devices are used in the field to measure in situ properties. These devices are used to generate waveforms in the material being tested. The elastic modulus at small strain is proportional to the velocity of the wave propagation.

The V-meter and Resonant Column are laboratory instruments used to measure properties of samples collected in the field or molded in the lab. These devices use a pin or hammer “source” to impact the pavement or sample surface. Wave propagation speed and analysis of the wave dispersion curves can be used to determine layer thickness, stiffness with depth, and presence of discontinuities (cracks, delaminations). Seismically-derived stiffness values are evaluated at very low strain and require adjustments to be used for determining values comparable to ones estimated at higher strains, for example, truck wheel loads.

- **Rolling Dynamic Deflectometer (RDD)**. The RDD provides the capability of generating a continuous deflection profile, rather than generating deflections at discrete points as does the falling weight deflectometer (FWD). The device can be instrumental in evaluating older rigid and flexible pavements to determine the relative level of underlying support, load transfer at joints and cracks, and overall pavement stiffness.

Currently, there is only one of these devices and is available for use through an interagency contract (IAC) with the Center for Transportation Research (CTR) at UT Austin. This IAC is managed by CST-M&P.

Falling Weight Deflectometer (FWD)

The falling weight deflectometer (FWD) is a trailer mounted device that places an 11.8 in. (300 mm) diameter load plate in contact with the highway at each test location. The testing interval is set at 0.1 mi. (maximum) or 30 locations per project. A load column above the load plate carries a stack of weights that are dropped to impart a load to the pavement similar to that imparted by a passing dual truck tire set. A series of seven geophones spaced away from the load plate at 12 in. increments measure the surface deflection, generating a "deflection bowl." Measurements are generally acquired in the right wheel path of the outside lane.

Figure 4-5. Falling Weight Deflectometer (FWD).

Figure 4-6. Falling Weight Deflectometer, detail view.

Where a divided roadbed exists, surveys should be taken in both directions if the project will include improvements in both directions. Some care in the placement of the load plate and sensors is required by the survey crew, especially where the highway surface is rutted or cracked. The load plate should lay on a flat surface; the load plate and all geophones should lie on the same side of any visible cracks. A human spotter should be used to ensure placement in these case.

Temperature data collected at the time of testing is necessary for all flexible pavements since the modulus of bituminous materials is temperature-dependent. The FWD has a built-in thermocouple to measure ambient air temperature and records this temperature at each drop location. In addition, for pavements with bituminous surfaces at least 3.0 in. thick, in-pavement temperature measurements should be made at the beginning and end of each survey, at a minimum. Longer surveys or surveys made on days where rapid temperature changes occur will require temperature checks at additional intermediate locations. A portable drill with masonry bit is provided to accomplish this; holes are drilled to mid-depth or 3.0 in. maximum. Alternatively, surface temperatures measured by infrared devices can be made with correlations available for mid-depth temperature estimation.

Liberal use of comments placed in the FWD data file at the time of data collection is encouraged. Comments pertaining to proximity to reference markers, bridge abutments, patches, cracks, etc., are all important documentation for the individual evaluating the data.

Description of the FWD with conceptual discussion on basic operation is given in the *Flexible Pavement Rehabilitation Training* (CD/ROM). Contact the Research & Technology Implementation Office (RTI) at (512) 465-7555 to request a copy of this CD.

Backcalculation of Deflection Data

TxDOT currently uses version 6.0 of the MODULUS software for backcalculation of deflection data collected by the FWD. Version 6.0 is a Windows compatible program with many added analyses and graphing features compared to the previous version. The software and user's manual in PDF document format is available from the TTI on-line pavement design training site. Also, basic operation and discussion of inputs, cautions, and example problems is presented in the *Rehabilitation Strategies for Flexible Pavements* (CD/ROM). Contact the Research & Technology Implementation Office (RTI) at (512) 465-7555 to request a copy of this CD.

The raw deflection file, pavement layer thicknesses, layer Poisson ratios, probable layer moduli ranges, and asphalt temperatures at the time of testing are all required inputs to perform backcalculation. The backcalculation process works on the assumption that the pavement structure can be modeled as a linear-elastic layered system. If the parameters of layer thickness, deflection, and Poisson ratio are known, the modulus can be approximated. A likely range of "probable" layer moduli provided by the program user facilitates the process by forming the basis of a small internal database against which mathematically generated deflection bowls are compared to the actual measured deflection bowl by the software. Once a reasonable match is made, the moduli that allow this match are reported as the individual layer moduli. In addition, the program reports a depth to stiff layer or bedrock.

There are precautions and limitations to the backcalculation procedure that the user must consider. In the end, engineering judgment will be needed to decide on the viability of solutions generated. The following are some pointers when using MODULUS 6.0:

- The modulus for layers thinner than 3.0 in. cannot be backcalculated. This situation arises most often for thin-surfaced flexible pavements. The user must assign a reasonable modulus to this layer (minimum and maximum are input as the same value in the program) based on thickness, level of distress, temperature, etc.
- The surface layer is always the layer that the load plate is in contact with, so a thickness must be entered. Where the surface is a bituminous surface treatment, it is allowable to use a nominal thickness such as 0.5 in. and assign a nominal modulus such as 250 ksi. Alternatively, the surface treatment can be combined with the underlying layer as the "surface" reducing the total number of layers by one.
- The maximum number of layers for which the modulus can be backcalculated is four (one of which is always the natural subgrade) in MODULUS 6.0. For circumstances where more layers are known to exist, the user must either consolidate or ignore layers. Consolidation is recommended for materials that are more likely to have a similar modulus and shear strength properties (i.e., different types of HMAC or flex base). Ignoring layers may be reasonable in certain cases where the material's contribution to the overall stiffness of the structure is minimal (i.e., "foundation course," or lime treated subgrade – constructed as a working platform).
- There are times when a more reasonable solution is obtained modeling your pavement structure as a 3-layer system even if you know there are four layers present. This situation may develop for a number of reasons, such as variable stabilization (leaching), variable depth to bedrock, etc.

  Reasonableness is related to the in-place stiffness characteristics of the layers being modeled and not necessarily to the size of the average errors reported by the software in comparing the mathematically generated bowls to the measured bowls. While the 4-layer solution will generally give lower overall errors, the backcalculated material moduli may be unrealistic with respect to the in-place material. When there is doubt of reasonableness, the user should perform backcalculation runs using both 3- and 4-layer solutions (employing guidelines given in bullet 3). Additional field testing such as with the dynamic cone penetrometer (DCP) along with engineering judgment is necessary to ensure a valid, reliable solution.
- A check of the MODULUS summary table should be made to detect outliers that skew the average value reported. Outliers may be the result of full-depth patches (different pavement structure) or very weak areas.

  For the purpose of using MODULUS-reported values as input to FPS-19, adjustment of the average modulus should be considered; otherwise the performance of any pavement design solution based on these inputs could be jeopardized. As a rule-of-thumb, consider removing values that exceed one standard deviation from the unadjusted average, and then re-average. This should always be done for modulus values that are much higher than values that are more typical for the section. Consideration can be given to eliminating very low values **only if** the

intention is to include a bid item for repair of weak areas (i.e., Item 351 "Flexible Pavement Structure Repair") as part of the job.

- Shallow bedrock (typically less than 60 in. deep) will almost always result in underestimation of the subgrade modulus and over-estimation of the flexible base modulus. The recommended workaround is to fix the depth to bedrock at 120 in. or alternatively 240 in. if the solution using the program-generated depth to bedrock produces suspect subgrade/base moduli. Another clue that the default solution is suspect would be if the ratio of the flexible base modulus (unstablized layers only) to the subgrade modulus is very high (>5).

  MODULUS can perceive a shallow depth to bedrock in high water table situations (water is incompressible) as can be the case in east Texas. It may be beneficial to override the program-generated depth-to bedrock value, by using a fixed value of 120 in.

- Soft upper subgrade can also lead to high errors in the backcalculation process. In these cases, the use of a 4-layer solution where the soft portion of the subgrade is modeled as the subbase layer (fix depth at 12 in.) may provide better fit and more realistic backcalculated values for the base and deep subgrade. Verification with a dynamic cone penetrometer (DCP) may be warranted.

Dynamic Cone Penetrometer (DCP)

This portable device complies with ASTM D 6951 – 03 and consists of a 2-piece rod; the lower rod is about 39 in. long (1 m) and is fitted with a replaceable cone tip at the penetration end and an anvil at the upper end. The anvil houses a threaded receptacle for attaching the upper rod. The upper rod carries the 17.6-lb. sliding hammer and has a handle for steadying the device during testing. The hammer free-fall distance on the upper rod is 22.6 in.

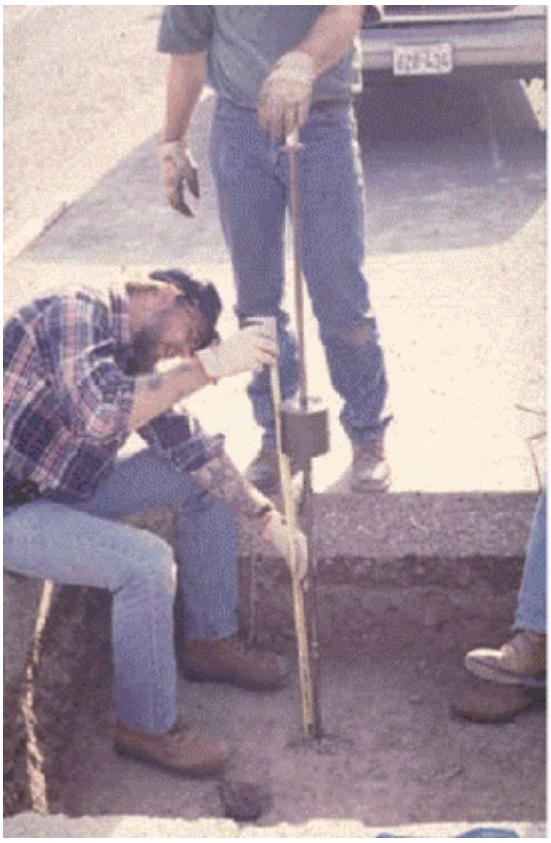

Figure 4-7. Dynamic Cone Penetrometer (DCP).

DCP Operation Instructions

The operator drives the DCP tip into the soil by lifting the sliding mass (hammer) with one hand to the handle then releasing it. The other hand holds the instrument by the handle to maintain an approximate vertical position. If a bound layer (HMAC, PCC, cement-stabilized base) is present above the non-bound layers, a 7/8 in. hole should be first bored through the bound layer(s).

DCP Readings and Data Collection

An initial depth reading is made using a measuring stick between the bottom of the sliding mass and a stationary surface (pavement surface, ground level, etc.). The total penetration for a set of blows is measured and recorded by an assistant. A plot of depth vs. cumulative blows is generated and a trend line is fitted to the data points for each tested layer. The penetration rate in mm/blow is determined as the slope of the trend line.

A separate trend line should be generated for each layer containing only data points measured in that layer. Dividing the data into discrete segments in this manner ensures that only valid data points can influence the trend line's slope.

The number of blows in a set can vary; for softer soils 1-3 blows may be a set, whereas for stiffer soils 5-10 blows may be a set. Some soils may be so stiff that little to no penetration is recorded in a given set. If this circumstance persists, it is better to halt testing and bore down to a lower level, then restart the test. Any time drilling is done before the start of a DCP test, the first data point should be at the correct depth and the person processing the data will not include (0,0) as a data point, which would taint the penetration rate (mm/blow) calculation.

The instrument is extracted by tapping the mass against the handle or by using a jack attached beneath the anvil. Both methods have some risk of damage to the instrument; in softer soils using a disposable cone, tapping the mass against the handle is the most expeditious method.

The Army Corps of Engineers has developed a number of correlations relating rate of penetration to soil stiffness in terms of California Bearing Ratio (CBR). For most applications, the following relationship is adequate for approximating in situ stiffness.

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

Where:

- CBR = California Bearing Ratio
- PR = penetration rate, mm/blow.

This relationship has been further correlated to elastic modulus (E) using the relationship:

$$E = 2550 \times CBR^{0.44}$$

Where:

- E = elastic modulus.

Plots can be made to show the relationship of stiffness with depth. This procedure is useful for determining the effective depth of a granular base, the effectiveness of lime stabilized subgrade, or whether there are soft pockets of material with depth. Visual examination of the DCP shaft once extracted can also reveal the presence of free moisture. Basic operation and example problems are presented in the *Flexible Pavement Rehabilitation Training* (CD/ROM). Contact the Research & Technology Implementation Office (RTI) at (512) 465-7555 to request a copy of this CD.

Air-coupled Ground Penetrating Radar (GPR)

This van-mounted self-contained system carries a data acquisition boom-mounted antenna off the front of the vehicle, suspended above the roadway. This allows for uninterrupted data collection at near-highway speeds. The system sends pulses of radar energy into the pavement and captures returned reflections from each perceived layer interface within the structure. A survey is generally run in the right wheel path with data summarized at 10-ft. intervals; the effective width of the scan-

ning is about 8.0 in. If data is needed at other transverse locations along the lane, additional parallel passes must be made.

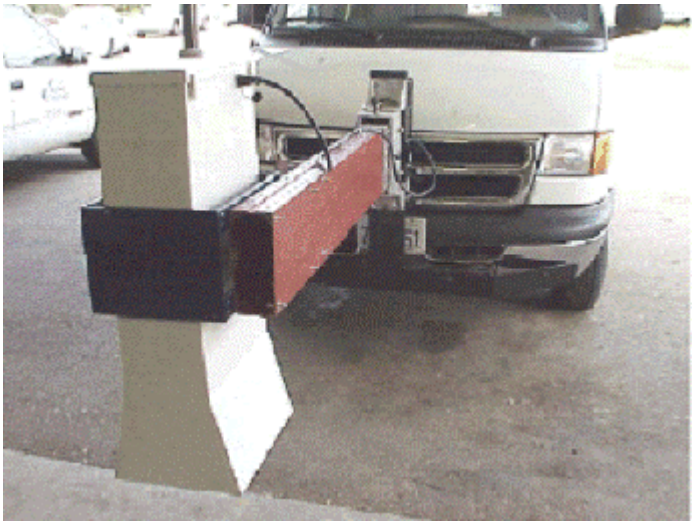

Figure 4-8. Air-coupled Ground Penetrating Radar (GPR).

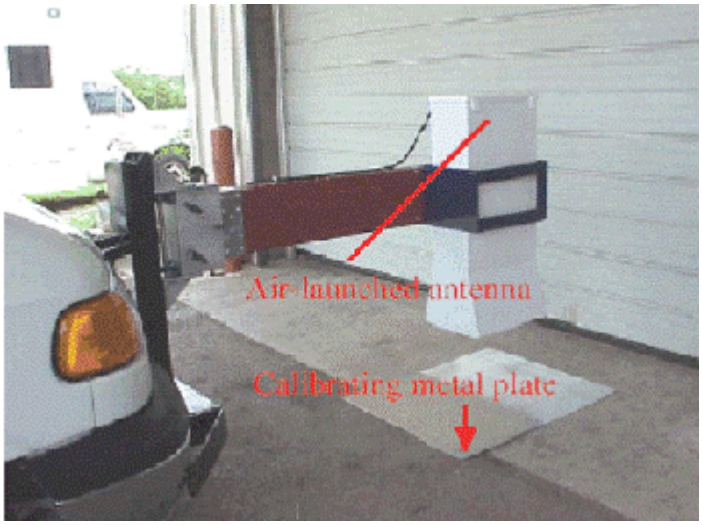

Figure 4-9. Detail view during calibration.

The 1-GHz antenna has a maximum penetration depth of about 24 in. The amount of energy returned and the time delay between reflections are used to calculate layer dielectrics and thickness. The dielectric constant of a material is an electrical property that is most influenced by moisture content and density. As the density and moisture content go up, the amount of energy reflected increases (and the penetrating ability decreases). Conversely, if the air voids increase, the amount of energy reflected decreases. Typical dielectric values for various pavement materials are given in Table 4-1, below:

Table 4-1: Typical Dielectric Values for Various Pavement Materials

Material	Dielectric	
	Typical Range	Signal Attenuation
HMAC	5 - 7	Low
HMAC - stripped	< 4	Low
AC w/light weight aggregate	3 - 4	Low
AC – wet (surface or voids)	> 8	Mod
Open Graded Friction Course (OGFC) (Plant Mix Seal [PMS], Permeable Friction Course [PFC])	3.5 - 4.5	Low
OGFC w/light weight aggregate	2 - 4	Low
Microsurfacing	3.5 - 4.5	Low
Flex Base – dry	< 8	Low
Flex Base at optimum moisture content (OMC)	8 - 12	Low
Flex Base – saturated	> 16	High
Cement Treated Base	7 - 9	Low
Clay – wet	> 20	High
Concrete – old	8	Mod
Concrete – new	10 - 20	High
Air	1	Low
Water	81	High
Ice	3	Low

Analysis of GPR Data

Analysis of raw GPR data is accomplished using the software COLORMAP. A user's manual is available in TTI report 1702-4, "*Colormap Version 2 User's Guide with Help Menus.*" To obtain the Colormap User's Guide, contact CST-M&P at (512) 506-5808. Also, basic operation and discussion of inputs, cautions, and example problems is presented in the *Flexible Pavement Rehabilitation Training* (CD/ROM). Contact TxDOT's Research & Technology Implementation Office (RTI) at (512) 465 -7555 for information about obtaining a copies of each of these.

Because of the capabilities to compute material dielectric properties and layer thicknesses, GPR has become a valuable tool to the pavement engineer to assist in determining the following:

- layer thicknesses, section changes, full-depth patches
- location and extent of potential problems such as elevated moisture levels or stripping in the HMAC and
- location/extent of segregation and low joint density.

Verification of suspected problem areas should be made with targeted coring and sampling. These sites should be selected based upon the analysis results from COLORMAP. Conclusions drawn from this limited sampling can then be applied with more confidence to the remainder of the survey.

Ground-coupled Penetrating Radar (GPR)

GPR systems using lower-frequency antennas are able to discern anomalies deeper with the pavement structure. Resolution is improved when the systems are placed in contact with the surface, so the antenna is dragged behind a van which houses the data acquisition system.

Maximum penetration will vary with antenna frequency, soil type and degree of saturation. Under the right conditions, the 100 MHz antenna can penetrate up to 30 ft.; however, there is poor near surface resolution of layers. These systems have been used successfully in locating buried fuel tanks, archeological burial sites, sink holes, and springs within the right-of-way. The systems and the required data analysis remain specialized; services are available through an interagency contract with the Texas Transportation Institute (TTI).

Figure 4-10. Ground-coupled Penetrating Radar.

Seismic Evaluation Tools

Seismic tools can be divided into two groups: those used in the field for evaluation of in situ pavement properties and those used to evaluate samples in the laboratory.

Field Seismic Tools

Seismic Pavement Analyzer (SPA). The SPA is a trailer-mounted device with an appearance similar to the FWD. The device uses a combination of two impact hammers, three geophones and five accelerometers to measure the velocity of shock waves that pass through the pavement. Data reduction is accomplished by using specialized software developed at the University of Texas El Paso (UTEP).

Figure 4-11. Seismic Pavement Analyzer (SPA).

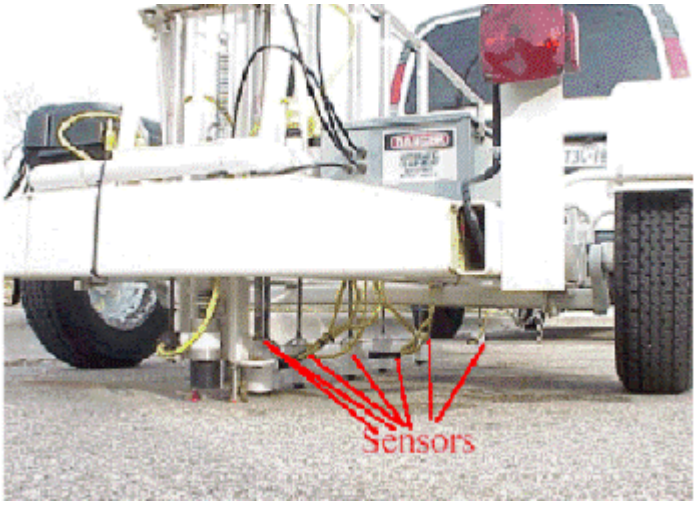

Figure 4-12. Seismic Pavement Analyzer, detail view.

Portable and Dirt Seismic Pavement Analyzer (PSPA, DSPA). The PSPA was developed as a supplemental tool to the SPA, with an enhanced capability to evaluate the upper strata (i.e., HMA

lift) of the pavement structure. The DSPA is similar in configuration, but has a broader hammer surface for use on unbound surfaces. These devices have been fielded in three districts in a trial quality control/assurance capacity by measuring material stiffness. The piece that contacts the surface is an inactive anvil that is struck with an internal striker; the contact piece acts as a wave transmitter.

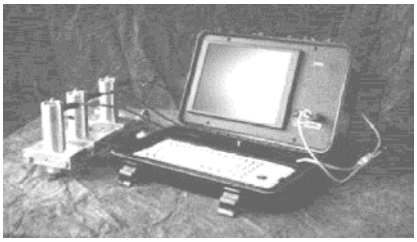

Figure 4-13. Portable Seismic Pavement Analyzer.

Laboratory Seismic Tools

V-meter. The V-meter is an ultrasonic laboratory device that is particularly useful for testing AC briquettes (lab-compacted specimens or field cores). In this device, a transmitting transducer is securely placed on the top face of the specimen. The transducer is connected to the built-in high-voltage electrical pulse generator of the device. The electric pulse is transformed into a mechanical vibration which is applied to the specimen. A receiving transducer is securely placed on the bottom face of the specimen, opposite the transmitting transducer. The receiving transducer, which senses the propagating waves, is connected to an internal clock of the device. The clock automatically displays the travel time of compression wave. By dividing the length of the specimen by the travel time, the compression wave velocity and as such modulus of the material is determined.

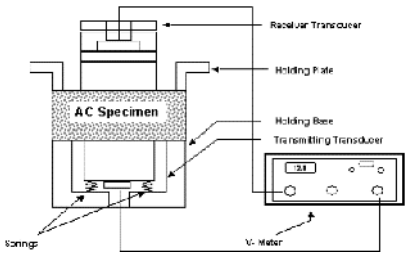

Figure 4-14. V-Meter.

Free-free Resonant Column. The resonant column device uses laboratory-prepared soil specimens that may be prepared using the Proctor (ASTM D-698), modified Proctor (ASTM D-1557) or any other procedure adopted by the agency. Since the test is non-destructive, a membrane can be placed around the specimen so that the specimen can be tested later for strength (static triaxial test) or stiffness (resilient modulus or cyclic triaxial test). An accelerometer is securely placed on one end of the specimen, and the other end is impacted with a hammer instrumented with a load cell. In less than 3 min., a specimen can be tested, and the test result can be obtained. The process has been automated and simplified so that a technician can perform the test, interpret the results, and generate a report almost immediately.

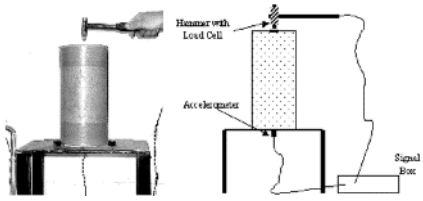

Figure 4-15. Resonant Column.

Rolling Dynamic Deflectometer (RDD)

The Rolling Dynamic Deflectometer (RDD) is a truck-mounted deflectometer that applies large cyclic loads to the pavement and measures the induced cyclic deflections with up to four rolling deflection sensors as it moves along the pavement. These sensors consist of a lightweight, rigid, three-wheeled cart supporting a vertically-sensitive velocity transducer (geophone). Speed while collecting data is 1-2 mph.

The condition and load transfer capacity of all transverse joints and cracks in rigid pavements can be determined by observing the measured deflections as the RDD approaches and crosses each joint or crack. The RDD has also been used successfully to evaluate flexible pavement structural condition. The RDD is a valuable tool for rapidly identifying regions of pavement requiring rehabilitation.

Critical regions of pavement identified with the RDD for possible rehabilitation can be studied further using traditional, discrete testing methods (FWD, DCP, seismic). Continuous deflection profiles provide valuable information about the pavement condition without backcalculating pavement moduli. For example, the relative deflections of heavily-trafficked pavement regions such as the outside wheel path, outside lane and lightly-trafficked regions (center slab, inside lane) give a very good indication of pavement degradation.

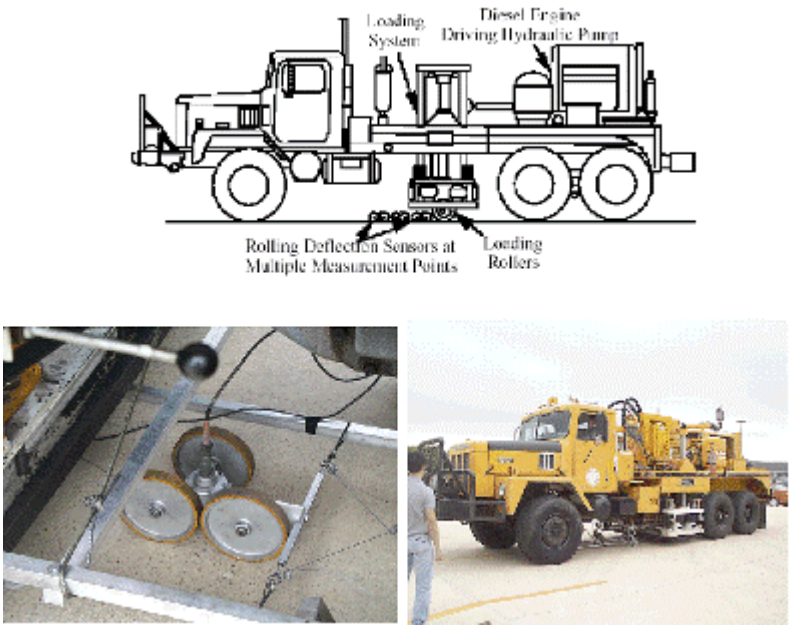

Figure 4-16. Rolling Dynamic Deflectometer (RDD), drawing with labeled parts, loading rollers, and photograph.

Section 5 — Destructive Evaluation of Pavement Structural Properties

Trenching and Coring

Trenching and coring have been used in forensic and routine evaluation to determine the source of the problematic layer or layers. For example, there was severe rutting on US 281 and the district expressed a need to determine the source of the rutting. Although FWD and GPR tests were performed, evaluation of the data could not differentiate from which layer(s) the rutting came. Trenching provides a viable option. For example, Figure 4-17 illustrates the pavement section profiles on US 281. Chalk and stringlines were used to differentiate different pavement layers, as shown in Figure 4-17 and Figure 4-18. In view of Figure 4-17 and Figure 4-18, the rutting was found to be from the surface AC layer.

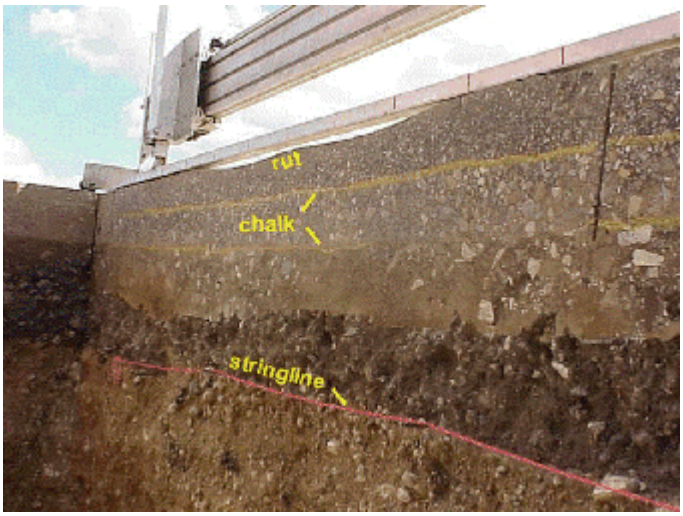

Figure 4-17. Trench sidewalls showing the pavement layer profile.

Figure 4-18. Trench sidewalls showing severe rutting.

Trenching Procedure

A trenching procedure has been developed to run tests and collect samples while minimizing disturbance to the base material. The saw cut pattern is shown in Figure 4-19. The size of each trench is 3 ft. × 12 ft. Each saw cut is deep enough to penetrate all the way through the AC layer(s). A strip about 6 in. wide is cut and removed by hand from one end of the trench, as shown in Figure 4-20. This creates a slot for the backhoe to reach into and lift the AC block without major disturbance to the base layer as shown in Figure 4-21. A saw cut is also made across the center of the trench so the AC blocks can be removed without breaking.

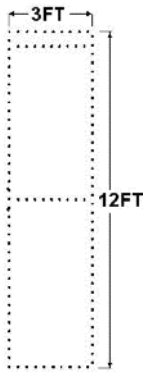

Figure 4-19. Saw cuts for trenching.

Figure 4-20. Removing asphalt concrete (AC) block with backhoe.

Figure 4-21. Flipping asphalt concrete (AC) block out with backhoe.

After the asphalt layer is removed and samples are collected, tests can be run, and samples collected on the base layer.

The backhoe then digs out the base layer and a few inches of subgrade. One of the transverse trench walls (at base and subgrade levels) is then smoothed using shovels, chisels, and brooms. Once a clean trench wall is achieved, the asphalt layers are highlighted with chalk and the base/subgrade layers are highlighted with string lines held with small nails. The thickness of each layer is then measured at regular intervals to determine its contribution to rutting.

Coring Procedure

To extract a core sample, the coring rig is placed at the desired location and the barrel-cooling water is turned on. Note that dry ice can be used to cool the core barrel if samples at in situ moisture content are desired, as show in Figure 4-22.

Figure 4-22. Coring with dry ice.

The barrel is spun at about 500 rpm and gradually lowered through the asphalt layer. If the base is stabilized, the operator may want to cut through it as well to obtain an intact sample.

After the barrel has cut to the desired depth, it is retracted while still spinning. Then the core barrel is stopped and the location of the core is observed. If the core has twisted off and is now lodged in the barrel, the barrel is struck lightly with a mallet to loosen the core. If the core is still in its original position, thin pieces of metal (or screwdrivers) are used to rock the core back and forth until it breaks free from the base or subgrade. Then the core can be lifted out with bent welding rods or loops of thin wire.

Normally, the core diameter is either 4 in. or 6 in. Figure 4-23 shows the extracted core samples with a 4-in. diameter.

Figure 4-23. Extracted core samples.

Shelby Tube

Shelby tube samples have been used to determine the in situ density, moisture content, plasticity index (PI), potential vertical rise (PVR), sulfate content, optimum soil stabilizer, and modulus of the subgrade soil.

The Shelby tube is a sharpened pipe that is pushed into soil by a hydraulic ram on a truck-mounted boom. An auger can be used to remove the HMAC and base layers and collect Shelby tube samples without introducing water in the process. After the HMAC and base layers are removed, the Shelby tube is positioned on the subgrade and pushed roughly 2 ft. (0.6 m) into the soil, as shown in Figure 4-24. The pipe is then pulled out and placed on a rack. A hydraulic ram then pushes the soil out of the pipe in (hopefully) one cylindrical piece. Aluminum foil and cardboard tubes are used to protect the Shelby tube samples, as shown in Figure 4-25. The tube is labeled with information including the location, orientation and depth of the sample.

If deeper samples are desired, the Shelby tube can be extended and pushed into the bottom of the same hole. That sample is then labeled as coming from a lower depth at the same location as the first. In this way, several samples can be collected at any location, then pieced together later for a complete soil profile.

Figure 4-24. Extracting subgrade samples with Shelby tube.

Figure 4-25. Aluminum foil cardboard tube to protect Shelby tube samples.

Section 6 — Geotechnical Investigation for Pavement Structures

See Chapter 3, Section 2 -"Geotechnical Investigation for Pavement Structures"

Chapter 5 — Flexible Pavement Design

Contents:

Section 1 — Overview

Section 2 — Types of Flexible Pavements

Section 3 — FPS-19W Design Parameters

Section 4 — Pavement Detours and Pavement Widening

Section 5 — Perpetual Pavement Design and Mechanistic Design Guidelines

Section 1 — Overview

This chapter discusses the following topics:

- types of flexible pavements
- design parameters of FPS-19W (analytical flexible pavement design procedure)
- pavement detours and widening
- design of perpetual pavements.

The preferred analytical method of flexible pavement design, FPS-19W, uses a systems approach.

The design system is centered upon deflection measurements from the Falling Weight Deflectometer (FWD). These measurements are then used by the MODULUS computer program to determine the material properties of the pavement layers. The material properties of the pavement layers are then used to compute alternative structural designs using FPS-19W.

Designs derived by the FPS design process are always checked for sheer strength adequacy using the Modified Texas Triaxial Class (TTC) design method. There are currently two alternative analytical design procedures allowed by the department for flexible design:

- FPS-11 and
- AASHTO (1993).

These pavement design procedures were briefly described in Chapter 2 and will not be further addressed here.

Section 2 — Types of Flexible Pavements

Definition of Flexible Pavement

A true flexible pavement yields "elastically" to traffic loading. It is constructed with a bituminous-treated surface or a relatively thin surface of hot-mix asphalt (HMA) over one or more unbound base courses resting on a subgrade. Its strength is derived from the load-distributing characteristics of a layered system designed to ultimately protect each underlying layer including the subgrade from compressive shear failure.

Progressively better materials are used in the upper structure to resist higher near-surface stress conditions caused by traffic wheel loads. These materials include an all-weather surface that is resistant to erosion by the environment and traffic action. The bituminous surface layer must also be resistant to fatigue damage and stable under traffic loads when pavement temperatures are in excess of 150ºF.

In this guide, hot-mix asphalt-surfaced pavements is a more generalized term used to describe any "black-topped" structure outside of HMA-overlaid concrete. These pavements are categorized as being flexible or semi-rigid (to include the full-depth or **perpetual** design). This chapter addresses the design of these types of structures.

The fundamental difference between a flexible, semi-rigid, and rigid pavement is the load distribution over the subgrade. The semi-rigid pavement has a higher composite modulus of elasticity than a flexible pavement and begins to resemble the rigid structure in terms of how the traffic loads are distributed over the subgrade. The elements contributing to the higher modulus may be:

1. increased thickness in asphalt concrete
2. chemical stabilization of the base, subbase, and/or subgrade layers
3. asphalt stabilization of the base course.

The higher modulus adds to the structural capacity of the pavement layers. As a result, the load is distributed over a wider area of the subgrade.

Types of Hot Mix Asphalt-Surfaced Pavements

These pavements may generally be placed into one of the following categories:

- surface-treatment on a granular base
- thin hot mix asphalt concrete (< 2 in.) on a granular base
- intermediate hot mix asphalt concrete (2 - 5 in.) on a granular base
- thick hot mix asphalt concrete (> 5 in.) (semi-rigid)

- thin hot mix asphalt concrete on a chemically stabilized base or subbase (semi-rigid)
- thin hot mix asphalt on an asphaltic bound base (semi-rigid).

Stabilization of the subgrade layer can apply to any of the above pavement types. Typical stabilizers include asphalt cement (for base only), lime, cement, fly ash, or lime-fly ash combinations.

Perpetual (HMA) Pavements

In 2001, the Flexible Pavement Design Task Force (FPDTF) studied structurally designed deep HMA pavement; a type of pavement typically associated with high assurance of long pavement life. The intent of the FPDTF study was to address the increased structural demands on heavy truck traffic facilities. The task force was composed of pavement and construction experts from TxDOT and industry.

As a result of the 2001 study, the following guidelines were established:

- department guidelines for materials to be used
- the general ("conceptual") structural design format and
- the locations where these structures should be considered (2001).

Current guidelines (2006) have taken into consideration design and constructability issues experienced in the structures designed under the original guidelines. Recommended structural layer composition (see Figure 2-2) for facilities with a projected 20-yr. one direction cumulative loading of at least 30 million ESALs was clarified.

When 30 million ESALs are exceeded, use Stone Matrix Asphalt (SMA), Item 346, and Performance-Designed Mixtures, Item 344, in lieu of conventional quality control/quality assurance dense-graded HMA specifications. When Items 346 and 344 are used with stiffer modified performance graded binders, these mixtures provide excellent resistance to permanent deformation. In addition, when Items 346 and 344 are used in a structure of at least 12" combined HMA thickness, or in conjunction with a high density rich bottom layer, the structure will be resistant to conventional bottom up fatigue cracking.

For the fatigue resistance characteristic to be effective, all HMA layers must be fully bonded to allow the system to act as a composite mass. The SMA and performance-designed mixes are specifically engineered to offer exceptional performance under heavy traffic loads. Dense-graded mixes under Item 341 may be used as an alternative to the stiff, rut-resistant HMA base (layer 'C'). Prior

approval by the Construction Division Director or designee is required for this substitution.

Special attention is required in designing a durable foundation by investigating the underlying soils to determine the appropriate type and level of stabilization needed. In lieu of subgrade stabilization, a high quality granular base, cement-treated base, or other engineered foundation should be used.

See Section 5 — Perpetual Pavement Design and Mechanistic Design Guidelines for using FPS-19W.

Section 3 — FPS-19W Design Parameters

The Flexible Pavement Design System (FPS) 19: User's Manual was developed as a report under project 0-1869 (TTI Research Report TX-02/1869-2) and serves as a preliminary reference. More definitive guidance pertaining to recommended inputs and functionality is given in this section.

Program Tools

FPS-19W is equipped with built-in help screens activated by pressing the 'F1' key while the cursor is in the applicable field for pages 2 and 3 of the data inputs. More specific guidance is provided here and supersedes the guidance given in the program "Help" screens.

Data Input Components

Data input for FPS-19W consists of a Main Menu screen and three pages of inputs. The current screen layout for most design inputs is historical in nature; its origins date back to the FPS-11 system, predecessor to the FPS-19W. When the FPS-11 system was in use, required data input to the program resided on the department mainframe. Note, references in the FPS-19W program to "card" number and the input order were retained to minimize the learning curve for veteran users of earlier versions of the program.

Pages

Page 1 of the inputs covers the administrative aspects of the design such as job identification and pavement design type selection.

General Inputs (page 1; cards 3 and 4 on page 2; and cards 7 and 8 on page 3) will be addressed first, followed by Traffic Parameters (card 3), Environmental Parameters (card 4), and Material Parameters (page 3).

See Table 5-1 for a summary of recommended inputs that will have a direct impact on the designed pavement thickness. For low- or medium-volume highways, these recommendations are targeted at accepting higher risk in an environment of increased funding limitations. Exercise caution in selecting input levels at the extreme range of the recommendations for multiple parameters. This strategy may yield structures too thin to perform well and should be weighed against:

- proven district design strategies and

- results of the Modified Triaxial design procedure for cited traffic levels

Table 5-1: FPS-19W Design Input Requirements

Parameter	Condition	Current Value	Suggested Value	Example Impact to Pavement Design	Comments
Initial SI[1]	Surface Treatment	3.8	4.0	Reduction in flexible base thickness is 1.0”	The lower ISI has the effect of thickening base.
	Thin ACP (1.5” to 4”)	4.2	4.5	3-5” reduction in flexible base thickness or about 1.5” HMAC	Actual ride is better than previous values. The lower ISI has the effect of thickening base. A 3-4” surface can have two smoothness opportunities.
	Thick (>4”) ACP	4.5	4.8	Reduces HMAC by 1”	Should be reduced to 4.5 if the minimum service-ability value is set at 2.5 or less
Minimum SI[1]	£1 M ESALs	2.0 - 2.5	2.0 - 2.5	Reduces flexible base by about 1” for lower ESALs up to 1.5 inches at the 1 M ESAL level.	Allowing a lower Min. SI has the effect of thin-ning the pavement structure. Risk is accepted requiring addi-tional maintenance before termination of pavement life.
	Between 1 M and 5 M ESALs	2.5 - 3.0	2.5	Reduces flexible base by about 2” for lower ESALs and as much as 3” for higher levels	*May be reduced to 2.5 for thick HMAC when initial SI is 4.5 or less or HMAC thickness exceeds 8”
	> 5 M ESALs	3.0	2.5 - 3.0*		
Confidence Level[2]	£1 M ESALs	B (90%)	A (80%)	Reduces flexible base thickness by 1”	Recognizes that the department will take more risk on low volume roads and provide maintenance.
	Between 1 M and 5 M ESAL	C (95%)	B (90%)	Reduces flexible base thickness by 2”	Reduction in structure will increase risk and will be mitigated by maintenance.
	> 5 M ESALs	C (95%) to D (99%)	C (95%)	Minimal, rare use of D	Confidence level D adds too much conservatism to normal pavement design.

1. Increased difference between initial SI and minimum SI reduces pavement thickness.
2. Confidence is increased when field stiffness test are conducted.

Table 5-1: FPS-19W Design Input Requirements

Parameter	Condition	Current Value	Suggested Value	Example Impact to Pavement Design	Comments
District Temperature Constant	All	District spcific 16 - 38	31	Reduction of 4" of flexible base for northern districts but adds an additional 1" for southern districts	Temperature constant is a known variable having an undesirable effect on thickness design. This variable is being made a constant in future versions of FPS.
Swelling Potential, PVR, Swelling Rate	High PI Soils	Suggested value is 0	Use 0	Same	Follow existing guidance to not include additional structural thickness as a result of swelling calculated by FPS, but the effects of swelling on performance for the structure designed may be checked. Consider stabilization of the subgrade to address swelling.
Overlay Cost	Future Cost	Use current design practice	Use current design practice		Review the "Detail Cost" sheet in the FPS report and determine if the overlay cost is driving a design for thicker pavements unnecessarily. The district may wish to consider pavement design selection without overlay cost included.
Detour (Road User Cost)	Cost for Detour Non-cash Future Cost	Use the estimated speed through and the appropriate model for the detour	Use approach speed for all traffic speed entries and Model 3 for the detour	Eliminates user costs associated with traffic delays for overlay paving operations.	Does not change thickness but re-sorts so that thinner pavements are brought into considerations for design selection.
Cost per CY	All	District-specific cost should be used. When the costs exceed the maximum input value, cost of materials should be scaled, proportionally, to each other.			
1. Increased difference between initial SI and minimum SI reduces pavement thickness. 2. Confidence is increased when field stiffness test are conducted.					

General Inputs

PROGRAM MAIN MENU

Access to the FPS design modules by clicking on the "FPS Pavement Design" button on the main menu. Page 1 of the program allows the designer to enter administrative data on the project and select the design type (combination of layers).

1. An administrative number may be assigned to each "run" in the "problem" field.
2. When the designer clicks on the "District" field, a menu appears that will allow the designer to select the district and county number.
3. The current date is automatically entered in the "date" field.
4. The applicable highway name is placed by the designer in the "highway" field.
5. The control-section-job numbers are placed in the appropriate fields.

CAUTION: Do not leave the control-section-job fields blank as this may cause the program to crash.

6. Comments may be entered in a five-line field by overwriting any existing comments (no text wrap-around).

The designer must now select which design type will be used for the project. Design types were discussed in "Pavement Design Process." If more than one type of design is being considered, then each type of design must be run separately and the respective program outputs subsequently compared.

BASIC DESIGN CRITERIA (Card #3)

Analysis Period. An analysis period is defined as the interval of time between reconstruction or major pavement rehabilitation efforts. This term is sometimes used synonymously with the pavement **design life.**

Normally a 20-yr. analysis period is used in flexible pavement design. A 30-yr. analysis period is possible but the designer must input the projected 20-yr. cumulative traffic in the FPS-19W computer program. Adjustments to the traffic are then made internally by the computer program. Similarly, when a very short analysis period (1-2 yrs.) is considered for design of short-term detours, the 20-yr. traffic (ESALs) must be used as the traffic input in FPS-19W.

CAUTION: When using FPS-19W, traffic loading must be entered as the 20-yr. cumulative ESALs. It is only the analysis period that is adjusted to reflect the expected duration. Refer to 20-Yr. 18-kip ESALs (One Direction) design criteria in the Traffic Inputs section for additional guidance.

Minimum Time to First Overlay and Between Overlays. These time intervals are commonly referred to as performance periods and are based on district guidelines, historical trends, and former federal policy. Considerations for performance periods beyond the minimum include minimizing interruption to traffic and avoiding the necessity for mill and inlay operations where constant profile must be maintained.

For flexible pavements, the selected design strategy should provide a minimum initial performance period of 8 yr. before an overlay is required.

Minimum Serviceability Index. This input is also known as the terminal serviceability or serviceability at the end of a performance period. On the indexed scale of pavement performance (0-5), this is the lowest desirable condition before rehabilitative effort is required.

- For highways of higher importance (cumulative ESALs exceed 5 M), a minimum serviceability index of 3.0 is recommended. A minimum serviceability index of 2.5 can be considered for thick HMAC pavements where the initial SI is 4.5 or less or when the total HMAC thickness exceeds 8.0" (Table 5-1).
- For highways with moderate traffic (1 M to 5 M ESALs) a minimum serviceability of 2.5 is recommended.
- For low volume highways where average daily traffic (ADT) is less than 3,000 vehicles per day (vpd) and cumulative ESALs are less than 1,000,000, a terminal serviceability of 2.0 to 2.5 may be used, but should be carefully weighed (proper risk management) where heavy loads or weak soils exist, or speed limits exceed 50 mph.

The minimum serviceability index and other performance related concepts are shown in Figure 5-1. Here performance is defined as a decrease in serviceability over time or traffic loading. Higher initial serviceability may result in longer performance periods, however, the desire (or necessity) to maintain a higher level of minimum serviceability will shorten the performance period. The rate of deterioration in the serviceability index is affected by the overall structural capacity and environment, including severe climatic events. Serviceability can be restored only by performing preventive or standard maintenance, HMAC overlays, or reconstruction.

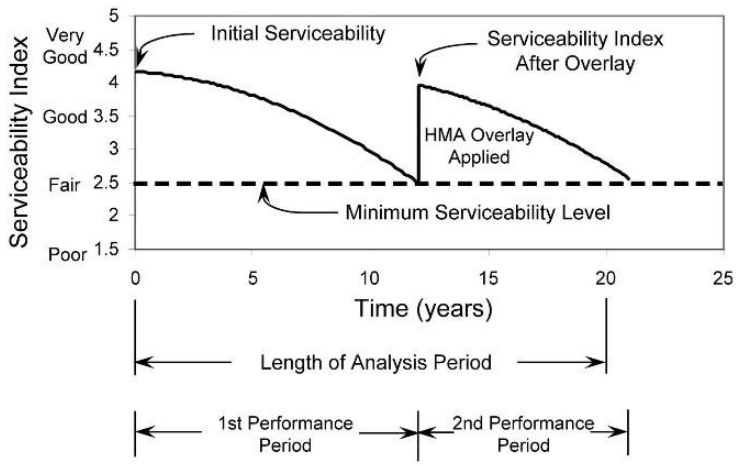

Figure 5-1. Pavement Performance Relationships.

Design Confidence Level. This parameter is meant to address variability in material quality, construction processes, and forecasted traffic as a means of assuring the structure performs as desired. It does not account for defective materials, poor construction, or poor assumptions on material properties. An overall multiplier to the cumulative traffic loading is applied, increasing as the desired level of confidence increases. FPS-19W uses an alphabetic code that is tied to a reliability or confidence level as follows:

A 80%

B 90%

C 95%

D 99%

E 99.9%

A confidence level as low as level 'A' can be considered for designs below 1 M ESALs; level 'B' is recommended for 1 to 5 M ESALs; and level 'C' for above 5 M ESALs.

Examples of designs generated using confidence level A and B on low load level highways and levels B, C, and D on higher load level highways illustrate the impact on initial perfor-

mance periods and structural layer thicknesses when using alternate confidence levels. Output variation for these sample designs is summarized in Table 5-2.

Table 5-2: Output Variations

Traffic Volume	Confidence Level	Initial SI	Termin SI	Surface	Base	Subbase	Initial Performance Period (T1 yr.)	Overlay (in. @ T1)
Low (750,000 ESALs)	A	4.0	2.0	two-course surface treatment (2CST)	6.0″ flex	6.0″ Lime Stabilized Subgrade (LSS)	26	---
	B	4.0	2.5	2CST	6.0″ flex	6.0″ LSS	18	2.5
Medium (2.5 M ESALs)	B	4.5	2.5	2CST	8.0″ flex	8.0″ LSS	13	2.5
	C	4.5	2.5	2CST	11.0″ flex	8.0″ LSS	9	2.5
High (7.0 M ESALs)	B	4.5	2.5	3.0″ HMA	10.0″ CSB	8.0″ LSS	20	---
	C	4.5	2.5	3.0″ HMA	10.0″ CSB	8.0″ LSS	11	2.5
	D	4.8	3.0	5.5″ HMA	10.0″ CSB	8.0″ LSS	8	4.5

Interest Rate (%). This parameter is used in the life cycle cost analysis to discount future expenditures for overlay and maintenance costs. A value of 7% is recommended.

PROGRAM CONTROLS (Card #4)

Number of Output Pages. This field controls how many pages of designs will be generated. There is a limit of eight designs per page. For 24 designs, up to three pages will be printed. If only 2 or 3 unique designs are generated, then only one page will be printed.

Maximum Funds/SY for Initial Construction. This field can be used to constrain design strategies where funding may be restricted. Generally, this parameter is left at a sufficiently high value ($99.00/SY) to facilitate or encourage maximum program output.

Maximum Thickness of Initial Construction. This field can be used to constrain design strategies to meet profile limitations or limit the number of total designs in the output.

Maximum Thickness of All Overlays. This field can be used to constrain design strategies to meet profile limitations.

CONSTRUCTION AND MAINTENANCE DATA (Card #7)

Initial Serviceability Index. On the indexed performance scale (0-5), this is the condition of the pavement immediately after construction or rehabilitation. Historically, the statewide average has been about 4.2. With the introduction of ride specifications, this value has been increasing.

Because surface treated pavements tend to be rougher; a value of 4.0 is recommended. For thin-surfaced ACP pavements (1.5 to 4.0") a value of up to 4.5 may be recommended. For thicker HMAC structures (>4.0" ACP) a value of up to 4.8 may be considered.

Use higher values in these ranges where more smoothness opportunities (e.g., multiple HMA lifts) exist (see Figure 5-1 and Table 5-1).

Serviceability Index after Overlay. This field is intended as a measure of the pavement condition following an overlay predicted by FPS-19W, projected after the initial or subsequent performance period.

Typically, these overlays are thin (2-3 in.) and placed in one lift. Therefore, ride improvement may not necessarily return smoothness to "original" levels; a value of 4.0 - 4.2 is recommended.

If the predicted overlay is thick enough to require more than one lift or district experience dictates otherwise, a value up to 4.5 may be considered, depending on the anticipated final ride condition (Figure 5-1).

Minimum Overlay Thickness. This parameter is dictated by the nominal maximum aggregate size of the mix typically used for overlays following the initial performance period. A 1/2 in. level-up is automatically included for cost purposes in the program, but does not count for toward an increase in structure.

Overlay Construction Time, Hours/Day. This input is used to evaluate traffic delay costs as a result of overlay operations required at the end of a performance period. Daily construction time typically ranges from 8-12 hrs.

***ACP Compacted Density, Tons/CY**. See NOTE for parameter influences. Typically the value ranges between 1.90–2.00.

***ACP Production Rate, Tons/Hour**. See NOTE for parameter influences. Typically the value ranges between 150–300 tons/hr.

***Width of Each Lane, Feet**. See NOTE for parameter influences. This value should be equal to the typical lane width.

First Year Cost for Routine Maintenance ($/lane-mile). This is a parameter that will affect life-cycle costs. The average cost of routine maintenance for the first year following initial or structural overlay (design option 5) construction should be tracked at the district level. Values have typically ranged from $50-$200/lane-mi.

Annual Incremental Increase in Maintenance Cost. This parameter is an adjustment to the base-line first-year routine maintenance cost where a uniform rate of increase is assumed. This value should again be tracked at the district level. Values have typically ranged from $10–$50/lane-mi.

NOTE: This parameter influences the time required to place the overlay and, as a result, affects the traffic delay costs.

DETOUR DESIGN FOR OVERLAYS (Card #8)

Detour Model during Overlays. There are five different models in the program for handling traffic during overlay operations, each one generating a unique user-delay related cost. Unfortunately, the built-in help screen only addresses three of the five models. The model number (1-5) is entered in this field.

CAUTION: Use of the incorrect detour model can result in excessive user delay costs or cause the program to crash, particularly when insufficient lanes are allotted for very high ADT inputs.

A short description of each model is given here.

- Model 1. Highway cross section consists of two driving lanes (one each direction) with wide (8-10 ft.) shoulders. Paving operations will block one lane at a time, with traffic in the paving direction using the shoulder or lane in that direction as the detour. Traffic in the non-paving direction is relatively unaffected, although slowing will probably be required.
- Model 2. Highway cross section consists of two driving lanes (one each direction) with narrow shoulders. Paving operations will block one direction at a time, with traffic in the paving direction being diverted into the on-coming lane using an escort. Traffic in the non-paving direction will be required to stop when traffic is escorted from the opposite direction.
- Model 3. Highway cross section consists of two or more driving lanes in each direction. Paving operations will block one driving lane at a time, requiring traffic in the paving direction to channel down into fewer lanes. Traffic in the non-paving direction may be completely unaffected if the highway is a divided facility.
- Model 4. Highway cross section consists of two or more driving lanes in each direction. Directional traffic flow in the paving direction is completely blocked, with traffic diverted to at least one lane in the opposite direction. Traffic in the non-paving direction must be channeled down into fewer lanes to accommodate opposing traffic.
- Model 5. Highway cross section consists of two or more driving lanes in each direction. Directional traffic flow in the paving direction is completely blocked, with traffic diverted around

the overlay zone by special detour, alternate route, or combination of these. Traffic in the non-paving direction may be completely unaffected if the highway is a divided facility.

For low- to medium-volume highways, the Pavement Design Task Force (PDTF, 2009) recommended removing possible cost bias in accounting for these user costs by simply selecting detour model 3 and entering the posted approach speed for all traffic speed entries in the detour area.

Total Number of Lanes. This value includes all driving lanes in both directions. If a facility includes a continuous left turn lane, treat this lane as a shoulder (do not count it as a driving lane).

Number of Open Lanes, Overlay Direction. This value will depend upon the overlay model chosen above and the total number of lanes on the highway. In the case of Model 1, this number would be one (1). In the case of Model 2, this number would be zero (0).

Number of Open Lanes, Non-overlay Direction. This value will depend upon the overlay model chosen above and the total number of lanes on the highway. In the case of Model 2, this number would be one (1).

Distance Traffic Slowed in the Overlay Direction. This input calculates the time delay associated with the detour which is further reduced to a user cost. Input is in units of miles.

Distance Traffic Slowed in Non-overlay Direction. Same as above, except for traffic in the non-overlay direction.

Detour Distance, Overlay Zone. This field is used only if Detour Model 5 is selected. The distance in miles of the alternate route/special detour is input.

Traffic Inputs

In the Traffic Data section, average daily traffic (ADT) statistics, cumulative ESAL loading, and percent trucks in the ADT are parameters that must be obtained through the Traffic Analysis Section of the Transportation Planning and Programming Division (TPP) by requesting a "Traffic Analysis for Highway Design."

Use Form 2124, Request for Traffic Data[1], for design traffic requests (see , "Pavement Design Process," for guidance). This TPP report will also contain the Average of the Ten Heaviest Wheel Loads, Daily (ATHWLD) and percent tandems in the ATHWLD, parameters that are both required inputs to the Modified Triaxial Check (Chapter 2, Section 5--"Approved Pavement Design Methods").

1. Internal access only.

As noted by the Pavement Design Task Force (PDTF)(2009), districts should review the traffic analysis for highway design report to verify data reasonableness. For example, check reported ADTs and percent trucks in the the traffic stream against observations along the project corrider. Also, knowledge of the predominant types of trucks and commodities transported can influence the proportion of fully loaded trucks and the wheel/axle loading imparted to the pavement.

By scrutinizing the reported beginning ADT, percent trucks in the ADT, and backcalculatiing the average ESALs per truck from the TP&P data, a designer can roughly estimate the magnitude of the reported versus observed truck traffic and the damage caused by the average truck. The information gathered can be used as a basis for requesting a re-evaluaton of the forecasted traffic, if necessary.

For multi-lane highways, observation may determine the actual lane distribution of trucks. Determine whether the ATHWLD figure appears reasonable. Compare the reported load against a typical dual tire set on a fully loaded 18-wheeler with a static load of roughly 9,000 lbs.

Beginning ADT (vehicles/day)

This is normally a two direction volume parameter that is required to generate user delay costs during overlays at the end of a performance period and (along with the ending ADT) to determine the distribution of axle loading over time. The beginning and ending ADTs define the composite traffic growth rate over the analysis period. For special situations such as one-way frontage road and ramp analysis, or one-direction analysis in the case of unequal loading, this figure will be one-way volume. This value will be the ADT at the beginning of a 20-yr. analysis period which should correspond to the year the facility is opened to traffic after construction or structural overlay (design option 5) is placed.

Ending ADT (vehicles/day)

Same as above, except this will be the volume at the end of a 20-yr. analysis period. ADT is assumed to increase linearly over time.

20-Yr. 18-kip ESALs (One Direction)

This figure is entered in terms of decimal millions. If the analysis period is other than 20-yr., an internal traffic equation will adjust the cumulative ESALs to the correct value for the analysis period used. **The cumulative 20-yr. traffic MUST ALWAYS be entered in this field.** Standard or project specific adjustments for lane distribution factors

may be used when at least three lanes exist in the design direction (, "Pavement Design Process"); lane distribution factors can be applied to highways with two lanes in one direction when there is compelling data to warrant reduction of the 100% ESAL loading.

Average Approach Speed to the Overlay Zone

Figure is entered in miles/hour; typically the posted speed limit is used. This parameter forms part of the equation to determine user delay costs through the overlay zone.

Average Speed in the Overlay Direction

Figure is entered in miles/hour; the estimate will in part depend upon which detour model is used. This parameter is another part of the user delay cost equation. To avoid possible influence on the ordering of recommended thickness options in the FPS output, this value should be set equal to the "Average Approach Speed" to preclude computation of associated user costs in the overall estimate of project costs.

Average Speed, Non-overlay Direction

Same as above, except in the non-overlay direction. Again, this value should be set equal to the "Average Approach Speed" to preclude computation of the associated user costs in the overall estimate of project cost.

Percent of ADT per hour of Construction

Parameter is part of the user delay cost equation. An estimate of the percent of ADT that arrives each hour of overlay construction is needed. If no better information exists, use 5% for urban highways and 6% for rural.

Percent Trucks in the ADT

Parameter is part of the user delay cost equation. The higher the percentage of trucks, the higher the user delay costs will be during overlay operations. Use the percent figure provided by TPP traffic analysis.

Environment and Subgrade

The original intent of this module of the program was to compensate for regional 10-yr. average daily temperature differences with decreased performance in regions with cooler climates and allow the designer to account for accelerated deterioration in performance due to swelling soils.

Both of these program features have proven to be ineffective in controlling either thermal cracking or performance degradation caused by swelling soils. As a result, the default will be to neutralize any effect on structural thickness computations.

The designer can run designs using these features to observe the predicted difference in performance life. Use caution when changing design inputs to compensate for the decrease in the predicted performance life.

Mitigation of soil movement is best evaluated with Tex-124-E, Determining Potential Vertical Rise, rather than using FPS-19W. The details of approval requirements for soil movement mitigation is discussed in , "Materials Investigation and Selection Information."

District Temperature Constant

Use 31 as the default (corresponds to a Central Texas value). In FPS-19, this will require overwriting the value automatically generated that corresponds to the selected county where the project lies. This input is designed to address the increased susceptibility of hot mix asphaltic concrete (HMAC) to cracking in cold weather. Using the default values keyed to the project county will generate a steeper performance curve (shorter life) for a colder region that in a warmer region. Again, FPS-19 will not generate an effective design that will counter the effects of thermal cracking (based on thickness alone).

Swelling Probability

This parameter is the percentage of the project (expressed as a decimal) that is likely to experience roughness or serviceability degradation due to swelling soils (range 0.0 to 1.0). Use 0 (zero) as a default, unless checking degradation to expected performance life.

Potential Vertical Rise

This input is an estimate of how much the surface of the clay layer can rise if supplied with as much moisture as it can absorb. "Tex-124-E, Determining Potential Rise" may be used, but compensation can be made for any swelling that may have occurred over time in the existing roadbed.

Impact will be nullified if using 0 (zero) as directed in "Swelling Probability." See , "Materials Investigation and Selection Information" Section 2 — Geotechnical Investigation for Pavement

Structures for deliberate design measures to mitigate potential vertical rise, and guidelines on how and when it should be used.

Swelling Rate Constant

This is an indexed value used to evaluate how rapidly a swelling susceptible soil can swell. A nomograph is included in the help screen with variables on one scale for estimating how tight or fractured the soil is; and on the other scale the relative availability of moisture to cause swelling. A line joining these scales will traverse the swelling rate constant scale, which will be the value entered in this field. The value will range between 0.04 and 0.20. The effect of this field is automatically nullified when zero (0) has been entered for Potential Vertical Rise or for Swelling Probability.

Material Parameters

The material modulus will be very influential in layer thickness calculations; the combination of the unit cost and thickness will dictate the structure's initial construction cost. However, this simplistic approach can be disastrous if the big picture is overlooked.

Properly estimating in-place and proposed new material properties is key to deriving a good performing, yet economical structure. Materials selected must be compatible. Failure to properly consider existing materials, relative stiffnesses, moisture susceptibility, or the adjacent structure (in the case of widening) can lead to early failure and/or very high maintenance costs.

Layer (LYR) Column

The sequence number for each material layer is automatically placed for each pavement design type. The designer should not attempt to add layers to these pre-established sequences. The layer sequence number will appear on the output pages of the program for each design option next to the letter 'D' (depth) for each layer.

NOTE: A coding mistake exists for design type 5 (Structural Overlay); the subgrade is labeled "Layer 5" instead of "Layer 4."

Layer Code (CDE)

This is a one-letter alphabetic code that corresponds to the layer sequence number and appears at the top of the design output to show the material sequence. The designer should not attempt to reorder these characters from the default order.

Material Name

The designer may edit the name of the material to be used in the layer, if preferred.

Cost per Cubic Yard

All new materials proposed in a design must have a cost in terms of dollars/cubic yard. Even existing layers can have costs associated with them, such as reworking a flexible base or milling an existing HMAC layer prior to overlaying. Material costs should be tracked at the district level.

The Average Low Bid Unit Price found on the department's web site can be used as a basis for these material costs.

Modulus, E (ksi)

Each material layer used in the structure will have a modulus input that should characterize the average stiffness of that material. The construction process, inherent material variability (initially and over time), and effects of environment and traffic loading will typically introduce considerable variance about the average value.

Overestimating this material property can result in a structure with poor permanent deformation performance and may subject the surface to early fatigue, while underestimating can result in an uneconomical pavement.

Additionally, materials that have an average in situ modulus in one circumstance may have a different average modulus if placed in another environment. This is particularly true of unbound base materials' modulus, which can be significantly influenced by the confinement provided by the layer directly beneath, absence of paved shoulders, or by the amount of moisture infiltrating the structure if the materials are moisture susceptible.

In evaluating a design that consists of layers that were pre-existing (including the subgrade), the Falling Weight Deflectometer (FWD) is indispensable in determining what stiffnesses (through backcalculation) these layers can contribute to the new structure. Virgin material to be added will require knowledge by the designer (preferably through past use and subsequent evaluation), tempered by the specifics of the current project.

Recent studies indicate that "performance" hot mixes and thick composite HMAC structures, using any type of HMA, have a much stiffer in-place moduli values than conventional thinner surfaced HMAC structures. Laboratory and field testing continues on these mixes to establish stiffness-temperature curves and better define "design" stiffness values by type if necessary. All material modulus values are manually entered into their respective fields (values are not read from the MODULUS summary file).

Below is a partial listing of typical design moduli by material type for newly placed or modified in place materials using FPS-19W.

Table 5-3: Recommended Material Design Modulus Values

Material Type	2004 Specification	Design Modulus	Poisson's Ratio	Comment
Surface Treatment	Item 316, 318	250 ksi	0.35	Considered in the structural design only when placed on the surface (not as an underseal).
Limestone Rock Asphalt Pavement	Item 330	200–350 ksi	0.35	Material typically placed as asphalt stabilized base or surface for low volume roads.
Hot-mix Cold-laid ACP	Item 334	300–400 ksi	0.35	
Dense-graded Hot (Warm) Mix Asphalt	Item 340, 341	Combined HMA thickness: £ 8″ use 500 ksi >8.0″ use 650 ksi	0.35	
Permeable Friction Course	Item 342	300 ksi	0.35	Thinness of the lift and high air voids do not allow significant contribution to the overall structural capacity.
Performance Design Mixtures	Item 344	Combined HMA thickness: £6.0″ use 650 ksi 6″< T £ 8″ use 700 ksi >8.0″ use 850 ksi	0.35	Includes SuperPave and CMHB mixes. Rich Bottom Layer (RBL) use 500 ksi.
Stone-Matrix Asphalt	Item 346	Combined HMA thickness: £6.0″ use 650 ksi 6″<T £ 8″ use 700 ksi >8.0″ use 850 ksi	0.35	

Table 5-3: Recommended Material Design Modulus Values

Material Type	2004 Specification	Design Modulus	Poisson's Ratio	Comment
Asphalt Treatment (base)	Item 292	250–400 ksi	0.35	Use "Tex-126-E, Molding, Testing, and Evaluating Bituminous Black Base Materials" or alternatively Part III of "Tex-204-F, Mix Design for Large Stone Mixtures using the SuperPave Gyratory Compactor (SGC)" to establish optimum asphalt content.
Emulsified Asphalt Treatment (Base)	Item 314, various OTU special specs	50–100 ksi	0.35	Contact CST-M&P for assistance in establishing optimum emulsion concentration.
Flexible Base	Item 247	If historic data not available, modulus should be from 3–5 times the subgrade modulus or use FPS default. Typical range 40–70 ksi.	0.35	In general, a finer graded base will have lower moduli than one that is a coarser gradation. As angularity and soundness of particles decrease, modulus will decrease to the lower end of the scale. Limiting the minus 200 clay fraction will improve resistance to moisture damage.
Lime Stabilized Base	Item 260, 263	60–75 ksi	0.30–0.35	Use Tex-121-E, "Soil-Lime Testing" to establish optimum lime content. Long-term stiffness improvement will depend on concentration used and affinity of base material to undergo permanent chemical bonding.
Cement Stabilized Base	Item 275, 276	80–150 ksi	0.25–0.30	Use Tex-120-E, "Soil-Cement Testing" to establish optimum cement content. For Item 276, a **minimum** 7-day unconfined compressive strength of 300 psi is established for Class L stabilized base. TTI research indicates that higher strengths can lead to detrimental shrinkage cracking. Also, very stiff, stabilized bases are not modeled effectively in FPS-19W. Higher design moduli are not recommended.

Table 5-3: Recommended Material Design Modulus Values

Material Type	2004 Specification	Design Modulus	Poisson's Ratio	Comment
Fly Ash or Lime Fly Ash Stabilized Base	Item 265	60-75 ksi	0.30	Use "Tex-127-E, Lime Fly-Ash Compressive Strength Test Methods" to establish optimum fly ash or lime fly ash content.
Lime or Cement Stabilized Subgrade	Item 260, 275	30–45 ksi	0.30	Use Tex-121-E or Tex-120-E to establish optimum lime or cement content. Long-term stiffness improvement will depend on concentration used and affinity of subgrade material to undergo permanent chemical bonding. There are cases when a subgrade will be treated (2-3% lime) to provide a working platform for construction equipment and a platform for compactive effort of the overlying layers. **This layer should not be accounted for in the structural design.**
Emulsified Asphalt Treatment (Subgrade)	Item 314, various special specs	15–25 ksi	0.35	Contact CST-M&P for assistance in establishing optimum emulsion concentration.
Subgrade	(Existing)	Priority should be to use the project-specific back-calculated subgrade modulus. Defaults by county are available in the FPS design program. Typical range is 8-20 ksi.	0.35–0.45	Use of a back-calculated modulus is preferred. FPS-19W defaults to the average county subgrade modulus taken from a limited number of tests. For new highway construction on a new right of way, deflection testing on an adjacent highway, or intersecting highways can provide data for backcalculation. Alternatively, elastic modulus correlations to field or laboratory derived CBR or the program default may be used. Wetter or more highly plastic materials warrant higher Poisson ratios.

Minimum Depth

This is the minimum depth for a given layer that the designer will consider. This may be driven by nominal maximum aggregate size in the case of asphaltic concrete pavement (ACP) layers, minimum practical layer thickness for flexible bases, or district policy.

- For existing layers, the average existing layer thickness is input.
- If a layer is not used, zero is input.
- For subgrade, use the average depth to bedrock as determined by MODULUS or the default when the subgrade modulus was not backcalculated.

Maximum Depth

This is the maximum depth for a given layer that the designer will consider. This may be driven by the maximum practical compaction thickness for one or more lifts or by district policy.

- For existing layers, the average existing layer thickness is input (minimum equals maximum).
- If a layer is not used, zero is input.
- For subgrade, use the average depth to bedrock as determined by MODULUS or the default when the subgrade modulus was not backcalculated.

Salvage Percentage

The designer enters a percentage of the original cost of the material that may be recovered at the end of the analysis period. Guidelines are given in the built-in help screen for these fields for 10-, 20-, and 30-yr. analysis periods.

Poisson's Ratio

The designer enters a value typical for the material type used. Recommended values are given in Table 5-3. This input has very little sensitivity. Selection of a value within the ranges identified is sufficient.

Check (CHK) Column

Not used. (Outdated coding from an earlier version of program.)

Once all inputs are entered, the designer clicks the "Go!" button and the output screen appears. If there are no feasible solutions, a message box will appear that will allow the designer to re-run and modify necessary inputs.

When at least one design is produced, further evaluation using the design checks may begin. At a minimum, the Modified Texas Triaxial Design check must be run. Leeway is granted to the project engineer to override the outcome of this check if evidence of past performance can be cited.

Modified Texas Triaxial Check

This design check is accessed by using the following procedure

Table 5-4: Texas Triaxial Design Check

Step	Action
1	Below the FPS design output display, click on "Check Design." Once selected, a "pavement plot" window appears with several option buttons.
2	Click on "design check." A "FPS Design Check" window will appear.
3	Select "Texas Triaxial Design Check." The "Form 1" window appears.
4	Enter the Modified Texas Triaxial Check data.
5	The first two parameters (Average of the Ten Heaviest Wheel Loads Daily [ATHWLD] and percentage of tandem axles in the ATHWLD) are provided in the Transportation Planning and Programming Division (TPP) *Traffic Analysis for Highway Design* report.
6	The percent tandem axles in the AHTWLD value is used to trigger a multiplier of 1.3 times the ATHWLD for percent tandems at 50% or greater.
7	◆ The Pavement Design Task Force (PDTF, 2009) recommends a factor of 1.0 be used for all designs where traffic loading is below 5 M ESALs. ◆ The percent tandem axles must be entered as <50% for these designs. **or** ◆ For designs involving more than 5 M ESALs, use the appropriate percent tandem axles provided by TP&P.

Where heavy industry truck traffic, aggregate pits or concrete plants, etc. are common, special design considerations may be required.

Subgrade Triaxial Class Number

The Subgrade Triaxial Class number is derived through actual laboratory testing of soil samples from the project ROW (Tex-117-E, "Triaxial Compression for Disturbed Soils and Base Materials"), or through use of Soil Conservation Service maps and catalogued results from previous testing (Soil_Series.xls).

The modified cohesiometer value is the final input. The cohesiometer is a device that was once used to determine a relative index of sheer strength for highway materials as a way to assign greater weight to bound or stabilize materials used in the highway structure. By doing this, the calculated depth of "better granular material" needed to protect the subgrade (or other unbound layers) could be reduced by an amount obtained from a chart (use the Flexible Base

Design Chart followed by the Thickness Reduction Chart for Stabilized Layers) found in the "Tex-117-E" test procedure archived versions, August 2002 - December 2009.

These calculations are automatically performed in this design check routine.

Table 5-5: Application of Cohesiometer and Thickness Reduction for the Triaxial Design Check

Step	Action
1	When the "reference" button is activated, a table of cohesiometer values appears.
2	◆ Double-click on the material used in the proposed structure that has the highest cohesiometer value. ◆ The screen will revert back to the input screen with calculated values for Triaxial Thickness Required, Modified Triaxial Thickness, and the FPS Design Thickness derived earlier.

A "details" button will allow you to review the materials table from your selected FPS design.

The box at the bottom left of the Modified Triaxial input screen will indicate whether the FPS design will meet thickness requirements.

Section 4 — Pavement Detours and Pavement Widening

Pavement Detours

Detours are pavements onto which traffic is temporarily diverted until a permanent (new construction, reconstruction, etc.) structure is provided to carry the traffic over the time frame of a conventionally designed performance period.

Detours can generally be placed into two categories. The first category is the diversion of an existing road or roads that parallel or skirt the location of the permanent facility to allow traffic around the construction site. Based on the engineer's evaluation, the existing road(s):

- may be used as is if little detrimental effect caused by the additional traffic loading is anticipated
- may need structural upgrading prior to opening of the detour or
- may need rehabilitation once the detour is closed.

The second category entails the construction of a temporary facility, usually parallel to and in the general right of way of the permanent facility. This type of detour is usually removed when no longer needed or may become the base of the permanent structure.

When estimating traffic loading over a time frame, the detour in effect may be complicated, especially when multiple highway sections are involved. Traffic data for detours can be specifically requested from TPP. For typical detours, the traffic data requested for the parent project will suffice when used in conjunction with the following detour design procedures.

NOTE: The temporary nature of the detour is not a license for ignoring other basic considerations such as drainage, underlying support, and quality workmanship. These points should be emphasized during the pre-construction conference and actively monitored by inspectors in the field.

Structural Design of Detours

The falling weight deflectometer (FWD) should be used to evaluate the adequacy of any existing structure (e.g., shoulders or bypass routes) to carry detour traffic.

FPS-19W and the Modified Texas Triaxial Class (TTC) design procedure (as a standalone design option) are the primary analytical methods for detour design. As with all FPS designs, a check using the modified TTC check procedure is required; an allowable alternate

version of the standard procedure is described later in this section. If the detour incorporates existing roads/highways, traffic load estimates must include traffic from both sources. Districts may employ proven design strategies for detours and can develop catalog designs based on traffic levels and subgrade support.

When using FPS-19W, traffic loading must be entered as the 20-yr. cumulative ESALs. It is only the analysis period that is adjusted to reflect the expected duration of the detour. Set the analysis period and time to first overlay to one or two years. The version of the modified TTC check most commonly used assumes a design life of 20 yr. For detours expected to last less than 2 yrs., an alternate version of the modified TTC check may be considered – a version sometimes used for evaluating load-zoned highways where the design life is considered only 10 years.

In the alternate version, the allowable wheel load scale is essentially doubled, allowing for reduced cover over the standard procedure. The easiest way to perform this alternate version of the modified TTC check is to simply divide the ATHWLD reported from TPP by 2 on the Traffic Analysis for Highway Design report and enter this wheel load value into the appropriate field of the FPS-19W Modified Texas Triaxial Design check. Depending on the priority of maintaining uninterrupted traffic flow, the tandem axle multiplier can further reduce the overall thickness requirement when set to less than 50%. (NOTE: The tandem axle multiplier is employed when the reported percent tandem axles in the ATHWLD is 50% or greater.) If this alternative option is used, an additional mechanistic check **is highly recommended** to double check fatigue and rut life.

The current version of FPS-19W does not easily allow the M-E check for detour design because these checks are evaluated against the full 20-yr. traffic loading. One procedure the designer can use is estimate the cumulative ESALs over the detour design life by using TRAFFIC6 or a similar routine. Then compare this value against the predicted crack and rut life numbers that FPS-19W generates in the post-processing Mechanistic Design Check.

Material and Construction Considerations for Detour Structures

Once a structural design is generated by using any of the cited procedures, other considerations must be made to insure adequate performance over the expected life of the detour structure. Among these are:

- adequate pavement width to insure wheel loads do not encroach upon the pavement edge where lack of lateral support may result in shear failures at the edge

- properly prepared subgrade (including investigation into soil properties). If there are no time constraints for the construction of the detour, stabilize the subgrade for low to moderate strength with moisture susceptible subgrades. Alternately, a geogrid can be used as long as some separation (e.g., thin granular layer) from a proposed lift is planned. The stabilized subgrade will provide a stable construction platform and will help reduce the total thickness of the pavement structure for the detour.
- placing 4 in. of flexible base prior to placing any full depth hot mix structure if it is not feasible to stabilize the subgrade. The 4 in. of flexible base will help reduce the total thickness of the pavement structure and provide an improved platform for compacting the bottom HMA lift against failure. Improved density in the HMA will increase fatigue life and moisture resistance.
- adequate drainage by insuring proper cross slope, ditch lines, and prevention of ponding
- properly compacted pavement lifts including the HMAC surfacing, and
- sufficient HMAC density at mat joints.

Proof rolling should be conducted prior to applying surfacing to insure adequate sheer strength in the unbound layers. Consider the more stringent requirements in the applicable specifications to insure quality workmanship is used, especially where traffic volumes and level of loading are high.

When the detour structure is no longer needed, mill and recycle pavement layers used in the detour within the parent project to reduce costs.

Pavement Widening

Widening is conducted to increase the capacity of the highway or to improve safety aspects such as the inclusion of shoulders, turning bays, etc. Consideration should always be given to maintaining the original cross section for the widened portion. This serves two purposes:

- it maintains uniformity in the section which facilitates future evaluation and rehabilitation options for the section as a whole, and
- it maintains subsurface drainage patterns which are essential to preventing trapped moisture.

Exceptions to this philosophy are generally related to poor performance in the existing section or the desire to expedite construction of the widened section to minimize interference with traffic.

In addition to cross section considerations, widening will entail a full-depth joint that is `naturally` a weakened interface in the structure. Compaction against the vertical plane of the old structure will be more difficult to achieve than with full-width construction. Placement of this joint as far from the wheelpath as possible will improve performance. Also, applying a final HMAC overlay across the entire section with the overlay joint offset by 6-12 in. from the underlying vertical interface will improve the impermeability of the interface over the short term.

The underlying vertical interface at the widening will `usually` cause reflective cracking through to the surface. An aggressive crack sealing program will limit the amount of precipitation runoff from entering into the structure. Consideration should also be given to using geotextiles/stress absorbing membrane interlayer (SAMI) over the widening joint prior to applying the full-width overlay as a means to delay reflective cracking.

Recommendations for Similar Cross Section Widening Strategies

The objective will be to match as close as practical the same cross section and materials in the widened section as were used in the original section.

Unbound Granular Base Sections

Other considerations are:

- Treatment of the subgrade under the widened location. This may serve to reduce moisture fluctuations at the new pavement edge which in turn should reduce potential for longitudinal edge cracking. An alternative may be to use geogrids at the subgrade/base interface. Treatment should be accomplished below the level of the old flex base.
- When milling any HMAC to the outside of the existing lane, also remove an extra 12.0 in. to the inside of the lane edge (stripe). The object is to offset the HMAC joints above the widening interface.
- Selection of the new flexible base material should be based on laboratory evaluation of both new and existing materials to compare the moisture susceptibility of each. Preferably these should be about the same. A material that is more highly moisture susceptible may draw moisture from both the original section and from outside the structure. A material that is less moisture susceptible may send moisture into the original base, particularly during the original curing process.
- Apply level-up to match original HMAC surface elevation. Make repairs to original surface as necessary. Apply reflective crack retardant (geotextile, SAMI) if desired.
- Apply overlay across full section. Longitudinal mat joints should be placed at lane stripes when used.
- Insure ditch lines are sufficient to prevent hydraulic backflow into the pavement structure.

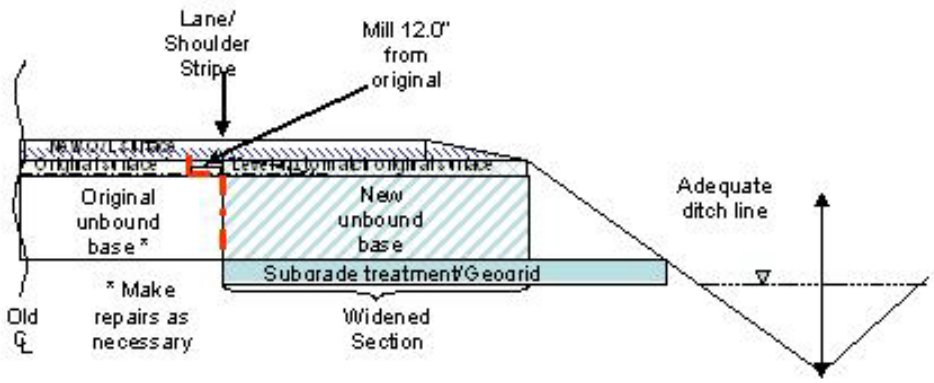

Figure 5-2. Unbound base sections.

Bound Base Sections

Other considerations will closely parallel those cited for the unbound base situation. Although an original bound base is cement-treated material, there are cases when it may be preferable to use full-depth HMAC for the widening to expedite construction. This strategy should not cause subsurface moisture flow problems ("bath tub" effect) when the cement treated base is not itself moisture susceptible. Laboratory evaluation of core samples will reveal the degree of moisture susceptibility of the existing base.

Recommendations for Dissimilar Cross Section Widening Strategies

This widening strategy is not encouraged because it often results in lateral subsurface drainage restrictions ("bath tub" effect). If it was possible to guarantee that no moisture would accumulate beneath the surface of the original structure, this type of widening would be easier to endorse.

Areas of the State that experience low rainfall and deep water tables may experience better performance with this type of widening strategy. If this type of widening is considered in wetter areas, consideration should be given to installing subsurface drainage at the widening interface, with laterals to the ditch line. Unfortunately, this remedy is generally counter to expediting construction. Sources of in-pavement moisture accumulation are shown below.

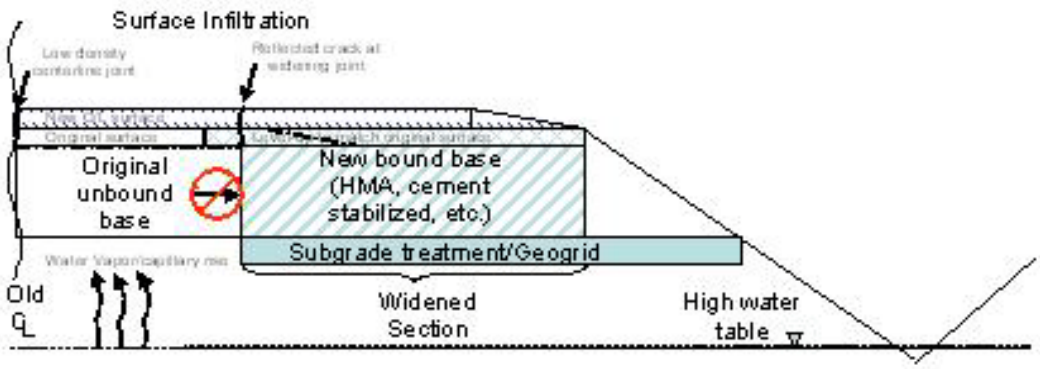

Figure 5-3. In-pavement moisture accumulation.

Section 5 — Perpetual Pavement Design and Mechanistic Design Guidelines

General

Structural Design of Pavements is an evolving process. Gradually, we have moved from purely empirical design relationships based largely on the results of observations at the AASHO Road Test to the current mechanistic-empirical process incorporated in FPS-19W where material stiffness is related to performance (loss of smoothness) through use of the surface curvature index (SCI, a deflection parameter).

Ultimately, the goal is to specify other material characteristics and their relationships (transfer functions) to the progression of specific forms of distress. As a result, multiple performance measures on any given structure subjected to a given regional environment and regional or site specific axle loads are available.

Limiting Criteria

While research continues in this area, there is one "special case," mechanistic structural evaluation. Experts have defined **limiting criteria** with some confidence. If these mechanistic benchmarks are not exceeded, then there is a very high likelihood that the pavement will not suffer traditional bottom-up fatiguing or full-depth (subgrade failure) rutting.

These limiting criteria find their full potential in the **perpetual pavement** design. Thinner structures are generally subjected to similar maximum axle loads, but offer insufficient stiffness or thickness to stay below the limiting criteria. As with other design criteria, these limiting criteria presuppose that quality materials are used and that proper construction procedures are followed. The **limiting criteria** reported by experts in the field such as Nunn and Monismith are:

- tensile strain at the bottom of the composite HMA layers: <70 μ-strain
- compressive strain at the top of the subgrade: <200 μ-strain.

To develop a design and determine whether these criteria are met is a two step process.

1. Either the designer must use a program such as FPS to roughly estimate an initial thickness design, then use an analysis program that calculates the primary responses to traffic loading at the critical locations or
2. the designer speculates at a desired structure and then evaluates it at the critical locations using an analysis program.

Designing a Perpetual Pavement Using FPS-19W

A summary of the steps to follow to design perpetual pavement using FPS-19W.

Table 5-6: Designing Perpetual Pavement Using FPS-19W

Step	Action
1	Pavement design Type 3 is recommended for this type of structure.
2	Select a 20- or 30-yr. analysis period.
3	Enter the 20-yr. cumulative ESALs in the 18-kip ESAL field, regardless of the analysis period length.
4	Use lane distribution reduction factors when three or more lanes are planned in one direction.
5	Because of limitations in the number of pavement layers allowed in FPS-19W, material inputs will require some judgment as to consolidating some layers and possibly disregarding those that contribute least (e.g., permeable friction course surface).
6	Select a minimum "time to first overlay."
7	Follow general guidelines for all other inputs. Select the **"Go!"** button to run the design.

Step 1-3: It is recommended that a pavement design Type 3 be used for this type of structure. Select a 20- or 30-yr. analysis period. The analysis period will not be critical, since meeting the limiting strain criteria specified above is the ulimate goal. As previously stated, you must enter the 20-yr. cumulative ESALs in the 18-kip ESAL field, regardless of the analysis period length.

Step 4 & 5: Use lane distribution reduction factors when three or more lanes are planned in one direction. Because of limitations in the number of pavement layers allowed in FPS-19W, material inputs will require some judgment as to consolidating some layers and possibly disregarding those that contribute least (e.g., permeable friction course surface).

Step 5 Example: As an example, the top 6.0 in. of HMA consists of one lift of SMA and one lift of 3/4 in. SuperPave (SP-B) where both mixes will use a PG 76-XX binder and the lower lifts of HMA will all use PG 64-XX binder. The top two lifts can be consolidated into one FPS-19 layer with a design modulus of 850 ksi, while the remaining HMA layers can be consolidated and designed using a modulus of 650 ksi. The third layer will be the prepared foundation (lime-stabilized subgrade, flexible base, etc.) with appropriate modulus.

Step 6: Select a minimum "time to first overlay." Fifteen years is recommended because this time frame will usually allow development of a structure of sufficient depth to meet the limiting strain criteria. Establishing a "time to first overlay" may not appear to be consistent with "perpetual" design. Actually, the performance module within FPS-19W is inconsistent with perpetual design. The suggested overlay will not be a structural requirement.

Step 7: What is consistent with perpetual design is the requirement to renew the surface at periodic intervals to mitigate the effects of surface wear and oxidation (something FPS cannot predict) and is more comparable to a maintenance activity. Follow general guidelines for all other inputs. Select the **"Go!"** button to run the design.

Depending on the thickness ranges selected, the resulting designs may indicate a requirement for an overlay prior to the end of the **analysis** period. (NOTE: Analysis period is not pertinent to perpetual design.) A mechanistic check using the mechanistic design check option should show "unlimited" fatigue and rutting life (>50 million ESALs).

Checking the Proposed Design for Compliance with Limiting Strain Criteria

FPS-19W has a built-in mechanistic analysis module (incorporates WESLEA®) that can be used to determine compliance with the recommended limiting strain criteria. Once you have run your FPS-19W design, select "check design" and then select the "stress analysis" button. Note, for full use of the features in this module, consult the *Flexible Pavement Design System (FPS) 19: User's Manual*. An example of the output is depicted in Figure 5-4 below:

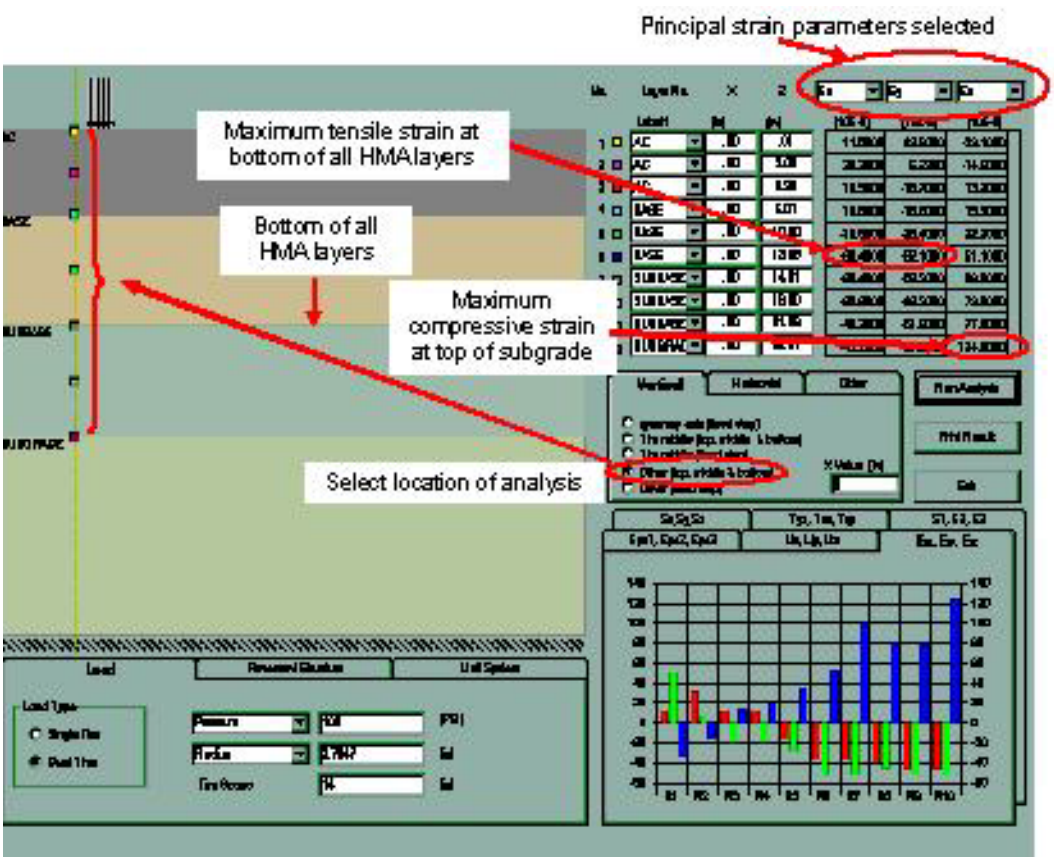

Figure 5-4. Analysis for limiting strain criteria using the FPS-19W Stress Analysis Module.

In this example, the proposed structure meets both of the limiting strain criteria; the maximum tensile strain at the bottom of all HMA layers is 46 μ-strain (< 70) and the maximum compressive strain at the top of the subgrade is 115 μ-strain (<200). While these calculated strains may show that this structure is somewhat conservative, some allowance should be made for the assumed material modulus (composite HMA lifts) and the expected variability inherent in subgrade support, material properties, and construction execution.

Strain computations are based on loading by a 9-kip dual wheel load (one half of an 18-kip standard axle). You are advised to check against strain levels derived from the ATHWLD loading by using computations outside the current FPS-19W program.

Alternatively, other analysis programs may be used to verify the limiting stain criteria (linear elastic, finite element, etc.). However, other analysis programs are not specifically supported within the department so knowledge in their use is the designer's responsibility.

Chapter 6 — Flexible Pavement Construction

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Section 1 — Introduction

Acknowledgement: Content of this chapter is made available from the University of Washington, Pavement Tools Consortium.

This chapter provides a general overview of the equipment and procedures involved in the construction flexible pavements. The equipment includes hot mix plant operations, trucks, placement equipment, and compaction equipment. Construction procedures such as surface preparation and compaction are also discussed.

This chapter will cover the following topics:

- pavement surface preparation
- plant operation
- mix transport
- mix placement
- compaction
- ride quality
- base and subgrade preparation.

Section 2 — Base and Subgrade Preparation

Introduction

Pavement performance can be largely attributed to the performance of its foundation, which is comprised of the subgrade and base layers. Base and subgrade layers must provide adequate and moisture resistent strength and modulus, in addition to durability and stability.

Frequently, in situ soils and local base materials do not meet project-specific requirements. Texas also has some of the most expansive soils in the country, which cause distresses in many pavements around the state.

Currently, a large portion of pavement construction consists of rehabilitating existing roads. These roads frequently contain subgrade or base material layers that are inadequate for current traffic loading demands. Shortages of high quality soil-aggregate sources are becoming more and more common statewide.

In order to achieve specified properties, subgrade, select fill, and base materials frequently require treatment with additives such as asphalt, cement, fly ash, and lime. Each of these materials must be properly designed to determine the most appropriate additive to achieve the desired improvement.

Additive selection criteria can be found in the "Guidelines for Modification and Stabilization of Soils and Base Materials for Use in Pavement Structures." This document also outlines the proper methodology of defining the goals for treatment and selecting, designing, and evaluating treated soils and base courses for pavement structures. In addition, this document provides some basic knowledge on the various treatment methods and the mechanism each treatment methods employs. Please contact the personnel listed in the document for further questions.

Section 3 — Pavement Surface Preparation

Surface Condition

The performance of a flexible pavement under traffic is directly related to the condition of the surface on which the pavement layers are placed. For a full-depth asphalt pavement, if the condition of the subgrade soil is poor, the ultimate durability of the roadway may be reduced. For hot mix asphalt (HMA) layers placed on top of a new, untreated granular base course, the base material should not be distorted by the trucks carrying the mix to the paver. For HMA placed as an overlay on top of an existing HMA layer, the surface should be free of major distresses, smooth and clean.

Prime Coat - Flexible Pavements

For flexible pavements, the graded subgrade or the top granular base layer may be prepared with a prime coat. A prime coat is a sprayed application of a cutback (MC-30 or MC-70) or emulsion asphalt applied to the surface of untreated subgrade or base layers. The prime coat serves several purposes:

- fills the surface voids and protect the base from weather
- stabilizes the fines and preserve the base material
- promotes bonding to the subsequent pavement layers.

The project plans specify the rate at which the prime coat is applied, and rates will vary from one project to another. Generally, however, medium curing cutback asphalt is applied at the range between 0.2 to 0.5 gal/yd^2, and emulsified asphalts are applied between 0.1 to 0.3 gal/yd^2. For emulsions it is recommended to mix the emulsion with the top 1 in. of the base.

Underseals

Existing Surface Preparation for Overlays includes an underseal which is a sprayed application of asphalt binder (emulsion or hot applied asphalt binder) immediately covered by a layer of one-sized aggregate. The underseal provides several benefits, such as waterproofing the surface, sealing small cracks and protecting the underneath surface from solar radiations. The application of an underseal should be strongly considered when placing a thin ACP layer on top of a flexible base. The *Seal Coat and Surface Treatment Manual* covers in detail the design and construction of seal coats.

Existing Surface Preparation for Overlays

Overlays make up a large portion of the roadway paving done today. The degree of surface preparation for an overlay is dependent on the condition and type of the existing pavement. Generally, the

existing pavement should be structurally sound, level, clean and capable of bonding to the overlay. To meet these prerequisites, the existing pavement is usually repaired, leveled (by milling, leveling or both), cleaned and then coated with a binding agent. This subsection covers:

- repair
- tack coats.

A tack coat material can be a PG binder or an emulsion layer applied between HMA pavement lifts to promote bonding. Adequate bonding between constructed lifts (especially between the existing road surface and an overlay) is critical for the constructed pavement structure to behave as a single unit and provide adequate strength. If adjacent layers do not bond to one another they essentially behave as multiple independent thin layers - none of which are designed to accommodate the anticipated traffic-imposed bending stresses. Inadequate bonding between layers can result in delamination (debonding) followed by longitudinal wheel path cracking, alligator cracking, potholes, and other distresses such as rutting that greatly reduce pavement life.

Application

Tack coats should be applied uniformly across the entire pavement surface and result in more than about 90% surface coverage. In order for this uniformity to be consistently achieved, all aspects of the application must be considered and carefully controlled. Specific aspects are:

- the condition of the pavement surface receiving the tack coat
- the application rate
- type of tack coat according to standard TxDOT specifications on Limestone Rock Asphalt Pavement, Item 330, and hot-mix items 334, 340, 341, 342, 344, and 346.

Condition of the Pavement Surface Receiving the Tack Coat

The pavement surface receiving the tack coat should be clean and dry to promote maximum bonding. Emulsified tack coat materials may be applied to cool and/or damp pavement; however, the length of time needed for the set to occur may increase (Flexible Pavements of Ohio, 2001). Since existing and milled pavements can be quite dirty and dusty, their surfaces should be cleaned off by sweeping or washing before any tack coat is placed, otherwise the tack coat material may bond to the dirt and dust rather than the adjacent pavement layers. This can result in excessive tracking of the tack coat material. Construction vehicles and equipment pick up the tack-dirt mixture on their tires and leave the existing roadway with little or no tack coat in the wheel paths (see Figure 6-1). Slippage cracking and delamination are distresses typically seen when cleanliness is lacking (Flexible Pavements of Ohio, 2001).

Figure 6-1. Tack coat tracking resulting in no tack coat in the wheel path.
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Application Rate

Tack coat application should result in a thin, uniform coating of tack coat material covering approximately 90% of the pavement surface (Flexible Pavements of Ohio, 2001). To achieve this result, application rate will vary based on the condition of the pavement receiving the tack coat. Too little tack coat can result in inadequate bonding between layers. Too much tack coat can create a lubricated slippage plane between layers, or can cause the tack coat material to be drawn into an overlay, negatively affecting mix properties and even creating a potential for bleeding in thin overlays (Flexible Pavements of Ohio, 2001). Table 6-1 shows recommended application rates from Flexible Pavements of Ohio (2001.)

Table 6-1: Recommended Emulsion Tack Coat Application Rates from Flexible Pavements of Ohio (2001)

Existing Pavement Condition	Application Rate in liters/m^2 (gallons/yd^2)	
	Residual	Undiluted
New HMA	0.14 - 0.18 (0.03 - 0.04)	0.23 - 0.32 (0.05 - 0.07)
Oxidized HMA	0.18 - 0.27 (0.04 - 0.06)	0.32 - 0.45 (0.07 - 0.10)
Milled HMA	0.27 - 0.36 (0.06 - 0.08)	0.45 - 0.59 (0.10 - 0.13)
Milled PCC	0.27 - 0.36 (0.06 - 0.08)	0.45 - 0.59 (0.10 - 0.13)
PCC	0.18 - 0.27 (0.04 - 0.06)	0.32 - 0.45 (0.07 - 0.10)

Residuals: The application rate of just the asphalt binder content of the emulsion.
Undiluted: The application rate of the undiluted emulsion

Basic application rate considerations are:

- **Roughness of the pavement surface receiving the tack coat**. Rough surfaces require more tack coat than smooth surfaces. For instance, milling produces a rough, grooved surface, which will increase the existing pavement's surface area when compared to an ungrooved surface. The surface area increase is dependent on the type, number, condition and spacing of cutting drum teeth but is typically in the range of 20 to 30%, which requires a corresponding increase in tack coat (20 to 30% more) when compared to an unmilled surface (TRB, 2000).
- **Distributor vehicle**. Several vehicle-related adjustments and settings are critical to achieving uniform tack coat placement. Essentially the nozzle patterns, spray bar height and distribution pressure must work together to produce uniform tack coat application. Generally, the best applications results from a "double lap" or "triple lap" coverage. "Double/triple lap" means that the nozzle spray patterns overlap one another such that every portion of the pavement receives spray from exactly two/three nozzles (see Figure 6-1; Figure 6-2; Figure 6-3; Figure 6-4; Figure 6-5; Figure 6-6; Figure 6-7; and Figure 6-8). Specific guidance follows:
  - Nozzle spray patterns should be identical to one another along a distributor spray bar. Differing coverages will result in streaks and gaps in the tack coat.
  - Spray bar height should remain constant. As tack coat is applied, the vehicle will become lighter causing the spray bar to rise. The tack coat application vehicle should be able to compensate for this. Excessively low spray bars result in streaks, while excessively high spray bars cause excessive nozzle overlap resulting in an excessive application rate.

- Pressure within the distributor must be capable of forcing the tack coat material out the spray nozzles at a constant rate. Inconsistent pressure will result in non-uniform application rates.
- Temperature within the distributor should be maintained between 24°C (75°F) and 54°C (130°F). Excessive heating may cause the emulsion to break while still in the distributor.

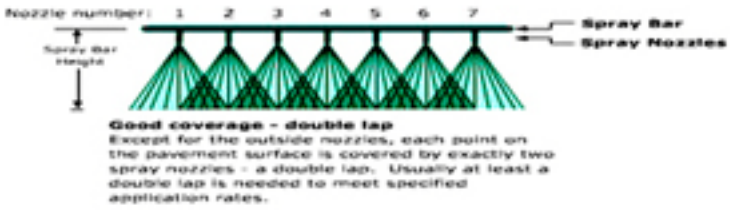

Figure 6-2. Tack coat application picture.
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Figure 6-3. Tack Coat Distributor Truck.
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Figure 6-4. Distributor Truck Spray Bar.
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Figure 6-5. Tack Coat Distributor Control Panel.
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Figure 6-6. Jet dryer used ahead of the tack coat distributor.
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Figure 6-7. Tack coat application.
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Figure 6-8. Tack coat application showing nozzle coverage area.
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Other Tack Coat Aspects

Timing

Generally, a tack coat should be allowed enough time to break and set (emulsion) before applying the next layer of hot mix asphalt (HMA).

Tracking

Tracking is the pick-up of tack coat material by vehicle tires. Tracking deposits tack coat material on adjacent pavement surfaces. Although this material is unsightly, it generally has little effect and

wears away quickly. In extreme cases, tracking may deposit enough tack coat material to distort pavement surfaces or hinder a driver's ability to navigate (Flexible Pavements of Ohio, 2001). Rubberized tack coats have an especially high propensity to stick to vehicle tires. Allowing tack coats to set (emulsions) before driving on them can substantially reduce tracking.

Traffic on Tack Coats

Generally, traffic should not be allowed on tack coats. When a tacked road surface is exposed to traffic, the potential exists for reduced skid resistance, especially during wet weather (Flexible Pavements of Ohio, 2001). When tack coat surfaces must be opened to traffic, they should be covered with sand to provide friction and prevent pick-up. A typical rate for applying a sand cover is 4 to 8 lb./yd^{2} (Flexible Pavements of Ohio, 2001).

Section 4 — Special Considerations for the Construction of Perpetual Pavements

Foundation

As with any pavement structure, quality begins at the foundation level. These structures must have a uniform, stiff, durable foundation that will adequately support the premium materials placed above. To establish the appropriate type and level of stabilization according to established test procedures, a geotechnical investigation of the underlying soils should be performed. Where stabilization of the subgrade is impractical, a high quality granualar base, cement-treated base, or other engineered foundation should be used.

Other Considerations

Once the placement of hot mix begins, it is important to consider the overall length of time and seasonal weather conditions that may occur prior to placement of the SMA lift to ensure surface water is not trapped in the structure through low density joints or segregation. Strict enforcement of density provisions in the applicable HMA specifications and considerations for use of positive drainage (edge drains) should be made. Where scheduling/traffic control plans prevent the SMA surface from being placed for extended periods of time, placement of a seal coat should be considered as an interim driving surface to minimize trapped moisture.

Construction considerations by layer are summarized in Generalized Perpetual Pavement Design Construction Considerations."

Section 5 — Plant Operations

HMA production is the first step in construction. The basic purpose of an HMA plant is to properly proportion, blend, and heat aggregate and asphalt to produce an HMA that meets the requirements of the job mix formula (JMF) (Roberts et al., 1996). There are two basic types of HMA plants commonly in use today:

- batch plant
- drum mix plant.

Batch plants produce HMA in individual batches while drum plants produce HMA in a continuous operation. Each type of plant can produce the same types of HMA and neither type of plant should impart any significant plant-specific HMA characteristics. The choice of a batch or drum mix plant depends upon business factors such as purchase price, operating costs, production requirements and the need for flexibility in local markets; both can produce quality HMA. This section gives a brief overview of batch and drum mix plants. More detailed information on plant operations can be found in:

Roberts, F.L., P.S. Kandhal, E.R. Brown, D.Y. Lee, and T.W. Kennedy. 1996. *Hot Mix Asphalt Materials, Mixture Design, and Construction.* National Asphalt Pavement Association Research and Education Foundation. Lanham, MD.

Transportation Research Board (TRB). 2000. *Hot-Mix Asphalt Paving Handbook 2000*. Transportation Research Board, National Research Council. Washington, D.C.

Batch Plants

Batch plants, which produce HMA in individual batches, are the older of the two types of HMA production facilities. HMA was originally made in batches; it was not until the 1970s that drum plants became a popular HMA production option. Currently, about 70% of all operational HMA plants in the U.S. are batch plants while about 95% of all newly manufactured plants in the U.S. are drum plants (Roberts, et al., 1996). This means that as older batch plants are retired they are more than likely to be replaced by new drum plants, which can provide greater mobility and production capacity. Typical batch quantities range from 1.5 to 5 tons of HMA. Figure 6-9 shows the basic components of a batch plant.

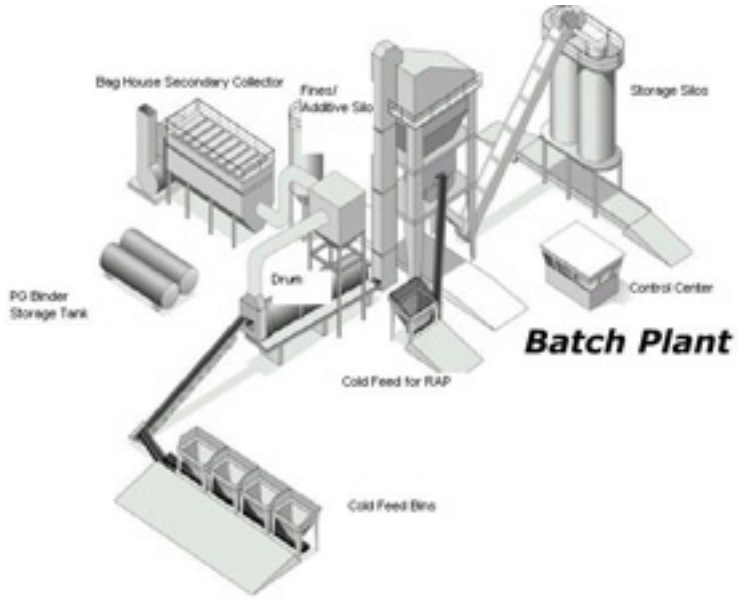

Figure 6-9. Batch Plant.
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Drum Plant

Drum plants, which produce HMA in a continuous manner, generally offer higher production rates than batch plants for comparable cost. Typical production rates for drum plants vary between about 100 tons/hr. up to over 900 tons/hr. depending upon drum design. Figure 6-10 shows the basic components of a drum plant and their function.

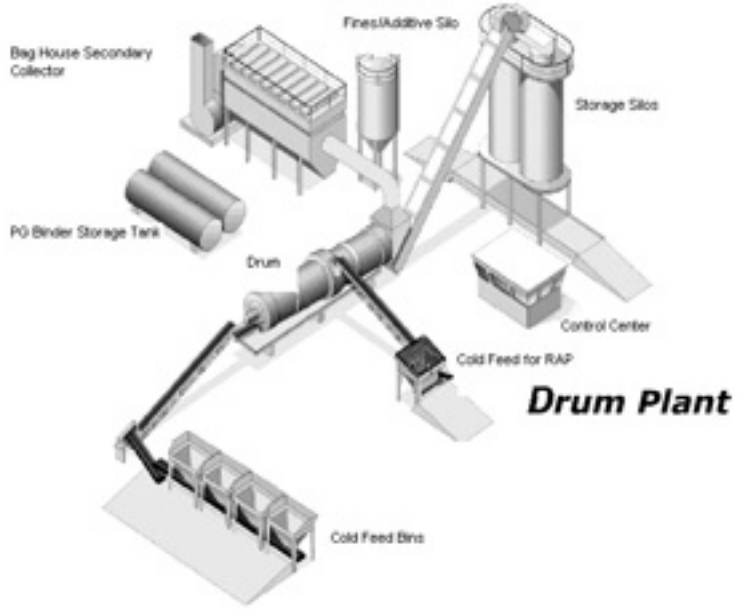

Figure 6-10. Drum Plant.
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Section 6 — Mix Transport

Introduction

Mix transport involves all actions and equipment required to convey HMA from a production facility to a paving site including truck loading, weighing and ticketing, hauling to the paving site, dumping of the mix into the paver or material transfer vehicle hopper, and truck return to the HMA production facility (Roberts et al., 1996). Ideally, the goal of mix transport should be to maintain mix characteristics between the production facility and the paving site. Transport practices can have a profound effect on mix temperature at the paving site, aggregate and/or temperature segregation of the mix and mat quality. This section will discuss the types of trucks used for mix transport and the various considerations involved with mix transport.

Truck Types

There are three basic truck types used for mix transport classified by their respective HMA discharge methods:

- end dump
- bottom dump (or belly dump)
- live bottom (or flo-boy).

End Dump Truck

End dump trucks unload their payload by raising the front end and letting the payload slide down the bottom of the bed and out the back through the tailgate (see Figure 6-11). End dump trucks are the most popular transport vehicle type because they are plentiful, maneuverable and versatile. Some general considerations associated with end dump trucks are:

1. **When the bed is raised, it should not contact the paver**. Bed contact with the paver may affect the screed tow point elevation, which can affect mat smoothness.
2. **The truck bed should be raised slightly before the tailgate is opened**. This allows the HMA to slide back against the tailgate, which will cause it to flood into the paver hopper when the tailgate is opened. HMA that trickles into the paver hopper is more susceptible to aggregate segregation.
3. **Truck-paver contact should be established by allowing the paver to move forward into a stationary truck**. This ensures that the truck does not bump the paver too hard and cause the paver to lurch to a sudden stop, which could cause a rough spot in the mat.
4. **Once the paver and truck are in contact, they should remain in contact.** This ensures that no HMA is accidentally spilled in front of the paver because of a gap between the truck and

paver. Usually the truck driver will apply the truck's brakes hard enough to offer some resistance to the paver but light enough so as not to cause the paver tracks to slip from excessive resistance. Most pavers can also be coupled to an unloading truck using truck hitches located on or near the push rollers.

Figure 6-11. End Dump Truck.
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Bottom Dump Truck

Bottom dump trucks (see Figure 6-12) unload their payload by opening gates on the bottom of the bed. Internal bed walls are sloped to direct the entire payload out through the opened gates. Discharge rates can be controlled by the degree of gate opening and the speed of the truck during discharge. The discharge is usually placed in an elongated pile, called a windrow (see Figure 6-13), in front of the paver by driving the truck forward during discharge.

A windrow elevator is used to pick up HMA from the windrow and feed it into the paver hopper. Windrow elevators do not have any method of regulating material flow, which makes it necessary to place the correct amount of HMA in the windrow to match the paving width and depth being placed without allowing the paver hopper to run out of mix or become overloaded.

Figure 6-12. Bottom dump truck in an urban setting.
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Figure 6-13. Windrow made by emptying a bottom dump truck.
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Live Bottom Dump Truck

Live bottom (or flo-boy) dump trucks (see Figure 6-14) have a conveyor system at the bottom of their bed to unload their payload. HMA is discharged out the back of the bed without raising the bed. Live bottom trucks are more expensive to use and maintain because of the conveyor system but they also can reduce segregation problems (because the HMA is moved in a large mass) and can eliminate potential truck bed – paver contact (because the bed is not raised during discharge).

Figure 6-14. Live Bottom Truck.
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Each truck type is capable of adequately delivering HMA from a production facility to a paving site. However, certain situations such as the ones listed in "Table 6-2: Truck Type Situations" below, may make one truck type advantageous over another.

Table 6-2: Truck Type Situations

Situation	Possible Truck Type	Reason
Paving on congested city streets	End Dump	Better maneuverability because it has no trailer and is smaller than a bottom dump or live bottom truck.
Paving using a mix highly vulnerable to segregation	Live Bottom	Live bottom trucks deliver the HMA by conveyor, which minimizes segregation.
Paving on rural highways	Bottom Dump	Usually has a larger capacity than end dump trucks (therefore fewer trucks are needed) but requires space and equipment for windrows.

Operational Considerations

There are several mix transport considerations, or best practices, that are essential to maintaining HMA characteristics between the production facility and the paving site. These considerations can generally be placed into four categories:

1. loading at the production facility
2. transport within the truck

3. unloading at the paving site
4. operation synchronization.

Loading at the Production Facility

Loading at the production facility involves transferring HMA from the storage silo or batcher (for batch plants) to the transport truck. There are two potential issues with this transfer:

1. **Truck bed cleanliness and lubrication**. Truck beds should be clean and lubricated to prevent the introduction of foreign substances into the HMA and to prevent the HMA from sticking to the truck bed. Non-petroleum based products should be used for lubrication such as lime water, soapy water or other suitable commercial products (Roberts et al., 1996). (The product should come from TxDOT's list of approved release agents.) Petroleum based products, such as diesel fuel, should not be used because of environmental issues and because they tend to break down the asphalt binder.
2. **Aggregate segregation**. HMA should be discharged into the truck bed so as to minimize segregation. Dropping HMA from the storage silo or batcher (for batch plants) in one large mass creates a single pile of HMA in the truck bed (see Figure 6-15). Large-sized aggregate tends to roll off this pile and collect around the base. Dropping HMA in several smaller masses (three is typical) at different points in the truck bed will largely prevent the collection of large aggregate in one area and thus minimize aggregate segregation.

Figure 6-15. Truck loading under a storage silo.
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Truck Transport

Truck transport affects HMA characteristics through cooling. HMA is usually loaded into a truck at a fairly uniform temperature between 250 to 350°F (see Figure 6-16). During transport, heat is transferred to the surrounding environment by convection and radiation and the HMA surface tem-

perature drops. This cooler HMA surface insulates the interior mass and thus transported HMA tends to develop a cool thin crust on the surface that surrounds a much hotter core (see Figure 6-17 and Figure 6-18). Things such as air temperature, rain, wind and length of haul can affect the characteristics and temperature of this crust. Several measures that can be taken to minimize HMA cooling during transport are:

1. **Minimize haul distance**. This can be accomplished by choosing an HMA production facility as close as possible to the paving site. Closer production facilities create shorter haul times and result in less HMA cooling during transport. Unfortunately, many paving locations may not be near any existing production facilities and economics may prohibit the use of a mobile production facility.
2. **Insulate truck beds**. This can decrease HMA heat loss during transport. Insulation as simple as a sheet of plywood has been used.
3. **Place a tarpaulin over the truck bed**. A tarp over the truck bed (see Figure 6-19) provides additional insulation, protects the HMA from rain and decreases heat loss. A study by the Quality Improvement Committee of the National Asphalt Pavement Association (NAPA) studied truck tarping and found that the HMA surface temperatures of tarped loads dropped more slowly than untarped loads but temperatures 100 mm (4 in.) below the surface between tarped and untarped loads were not significantly different (Minor, 1980).

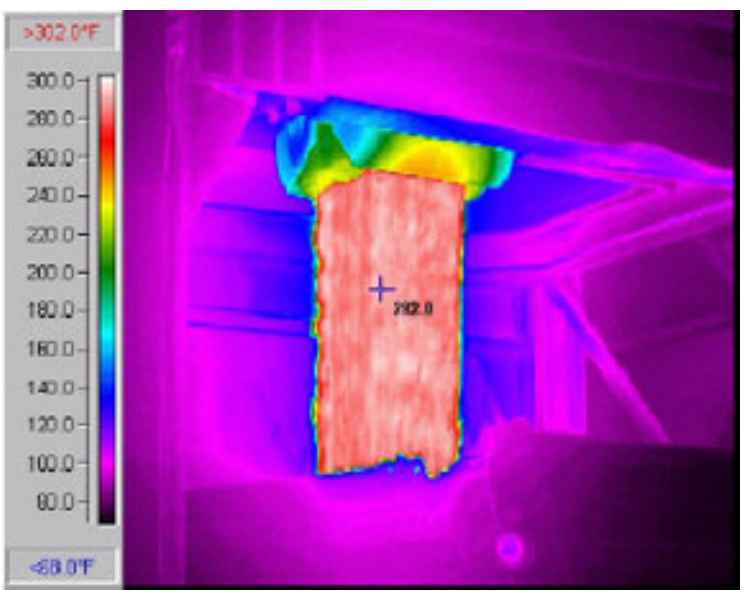

Figure 6-16. Infrared picture of an HMA storage silo loading a truck showing the hot uniform temperature of the mix.
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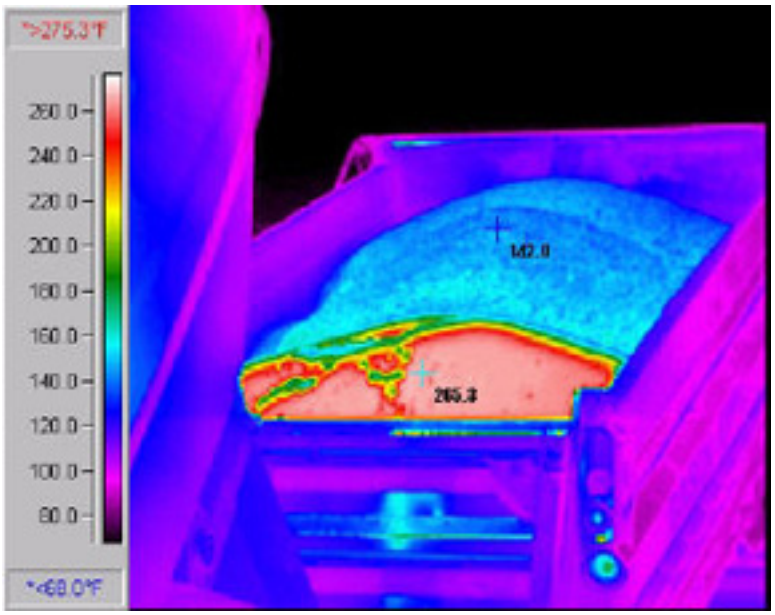

Figure 6-17. Infrared picture of a truck dumping HMA showing the cold surface layer crust (blue) and the hot inner mass (red).
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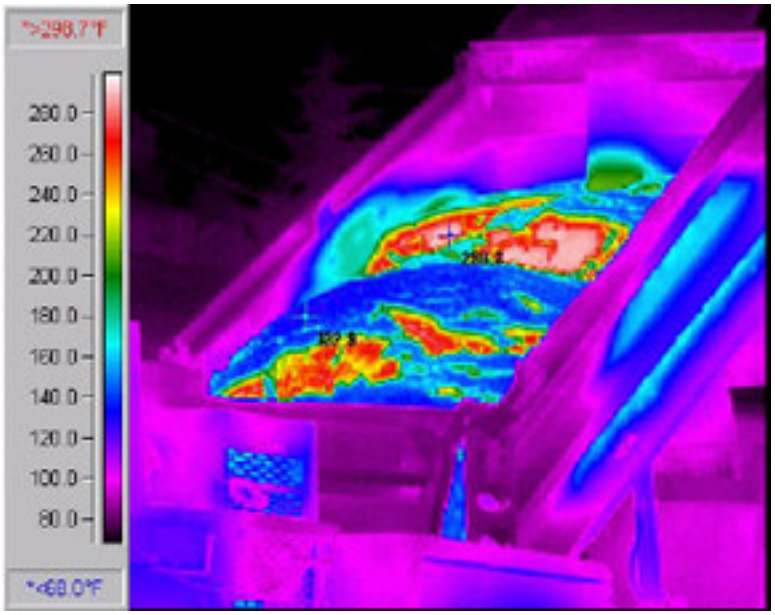

Figure 6-18. Infrared picture of a truck dumping HMA showing the cold surface layer crust (blue) and the hot inner mass (red).
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Figure 6-19. Driver covering his truck bed with a tarpaulin.
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In most cases, truck transport appears to cool only the surface of the transported HMA mass; however, this cool surface crust can have detrimental effects on overall mat quality if not properly dealt with. Actions such as reducing transport time, insulating truck beds or tarping trucks can decrease HMA surface cooling rate. Additionally, since the majority of the HMA mass is still at or near its original temperature at loading, mixing the crust and interior mass together at the paving site ("remixing") will produce a uniform mix near the original temperature at loading.

Unloading at the Paving Site

HMA unloading involves those procedures discussed in 'End Dump Truck' under "Truck Types" as well as a few other basic considerations such as:

- HMA should be unloaded quickly when it arrives at the paving site. This will minimize mix cooling before it is placed.
- Before HMA is loaded into the paver, the inspector and/or foreman should be certain it is the correct mix. Occasionally, paving jobs require several different mix designs (i.e., one for the leveling course and one for the wearing course) and these mixes should not be interchanged.

Operation Synchronization

Ideally, HMA production at the plant, truck transport and laydown at the paving machine should be synchronized to the same rate to minimize accumulation of excess HMA in any one of the three segments. Realistically, however, this synchronization can be quite difficult because of varying lay-

down rates, unpredictable truck travel times and variable plant production. Detailed information on operation synchronization can be found in:

National Asphalt Pavement Association (NAPA). 1996. *Balancing Production Rates in Hot Mix Asphalt Operations*, IS 120. National Asphalt Pavement Association. Lanham, MD.

Ideally, all operations are designed to meet optimal mat laydown rates. However, these rates can vary based on paving width and lift thickness. Also, complicated paving locations such as intersections or near manholes and utility vaults can temporarily increase or decrease the laydown rate.

Truck transport should be planned such that the HMA transport rate (expressed in tons/hr.) closely matches plant production rate and laydown rate. Some factors to consider are:

- number of trucks to be used
- truck type
- average truck hauling capacity
- production facility output rate
- availability and condition of storage silos at the production facility
- time to lubricate the truck bed before transport
- waiting time at the production facility
- loading, weighing and ticketing time at the production facility
- time to cover the load (when tarpaulins are used)
- distance between the production facility and the paving site
- average truck speed.

Traffic plays a large role in HMA delivery rates because it affects truck speed. Especially in congested urban areas, heavy and/or unpredictable traffic may substantially increase, or at least vary, truck travel time. As truck travel time increases, more trucks are needed to provide a given HMA delivery rate. Therefore, as traffic gets worse, trucking costs increase. Additionally, the unpredictability of traffic may result in either long paver idle times as it waits for the next truckload of HMA or large truck backups as several trucks all reach the paving site or production facility at the same time.

Finally, production facility output is typically controlled to match haul or laydown rate. However, this can result in suboptimal plant efficiency or HMA uniformity, which may increase plant exhaust output, shorten emission control device lifetimes, and affect contractual payment if payment is tied to HMA uniformity. It may often be more economical to run the production facility at maximum rate and store excess material in storage silos for discharge into trucks as they arrive. Storage silo insulation has progressed to a state where dense-graded HMA can be stored in them for up to a

week at a time without significantly affecting HMA characteristics. However, gap graded mixes such as SMA or OGFC should not be stored for more than 2 to 3 hr.

In sum, synchronization should be the goal but it is often difficult to achieve (based on varying laydown rates, haul time and traffic) and may result in plant inefficiency and HMA quality degradation. If a production facility has modern well-insulated, airtight storage silos and is producing a dense-graded HMA, it may be beneficial to run the plant at maximum production rate and store the mix until needed rather than try and match haul or laydown rate.

Summary

Mix transport can have a large impact on flexible pavement construction quality and efficiency. Mix characteristics such as laydown temperature, aggregate segregation and temperature differentials are largely determined by transport practices. In general, there are three types of HMA transport trucks: the end dump, bottom dump and live bottom dump (flo-boy). End dump trucks are most common, however, bottom dumps and live bottom dumps are well-suited for certain situations. Key considerations in mix transport are:

- truck bed cleanliness and lubrication
- proper mix loading techniques in order to prevent aggregate segregation
- haul distance and mix temperature
- timely mix unloading and unloading of the correct mix.

If properly managed, mix transport can successfully move HMA from the production facility to the paving site with little or no change in mix characteristics.

Section 7 — Placement

Introduction

Mix placement and compaction are the two most important elements in HMA pavement construction. Mix placement involves any equipment or procedures used to place the delivered HMA on the desired surface at the desired thickness. Mix placement can involve complicated asphalt paver operations or simple manual shoveling. This section provides a basic description of HMA placement operations. The *Hot Mix Asphalt Paving Handbook* (TRB, 2000) and the Asphalt Institute's *HMA Construction Manual* (2001) contain detailed information on asphalt paver components.

Placement Considerations

There are, of course, many considerations to take into account when placing HMA. Many are dependent upon local materials, weather, crew knowledge and training, and individual experience. This subsection presents a few of the basic considerations that apply in virtually all situations:

- **Lift thickness**. A "lift" refers to a layer of pavement as placed by the asphalt paver. In order to avoid mat tearing (which generally shows up as a series of longitudinal streaks) a good rule-of-thumb is that the depth of the compacted lift should be at least three times the nominal maximum aggregate size (TRB, 2000) Table 6-3 provides the minimum and maximum lift thickness for every mix type.

Table 6-3: TxDOT Min. Lift Thickness Requirement

Mix Type	Min. Lift Thickness (in.)	Max. Lift Thickness (in.)
Type A	3.00	6.0
Type B	2.50	5.0
Type C	2.00	4.0
Type D	1.50	3.0
Type F	1.25	2.5
SP-A	3.00	5.0
SP-B	2.25	4.0
SP-C	1.50	3.0
SP-D	1.25	2.0
CMHB-C	2.00	4.0
CMHB-F	1.50	3.0
SMA-C	2.25	4.0

Table 6-3: TxDOT Min. Lift Thickness Requirement

Mix Type	Min. Lift Thickness (in.)	Max. Lift Thickness (in.)
SMA-D	1.50	3.0
SMA-F	1.25	2.5
SMAR-C	2.00	4.0
SMAR-F	1.50	3.0

- **Longitudinal joints**. The interface between two adjacent and parallel HMA mats. Improperly constructed longitudinal joints can cause premature deterioration of multilane HMA pavements in the form of cracking and raveling.
- **Handwork**. HMA can be placed by hand in situations where the paver cannot place it adequately. This can often occur around utilities, around intersection corners and in other tight spaces. Hand-placing should be minimized because it is prone to aggregate segregation and results in a slightly rough surface texture. If hand placement is necessary, the following precautions should be taken (Asphalt Institute, 2001):
  - Place the HMA in a pile far enough away from the placement area that the whole pile must be moved. If the pile is located in the placement area, its appearance, density or aggregate distribution may be slightly different than the surrounding hand worked mat.
  - Carefully deposit the material with shovels and then spread with lutes. Do not broadcast (scoop and pitch) the HMA with shovels - this is likely to cause aggregate segregation.
  - All material should be thoroughly loosened and evenly distributed. Chunks of HMA that do not easily break apart should be removed and discarded.
  - Check the hand worked surface with a straightedge or template before rolling to ensure uniformity.
- **Performance Design Mixtures & Stone Matrix Asphalt (SMA)**. Open graded mixes behave differently than dense-graded mixes during placement and compaction. Experience and understanding of dense-graded mix placement should be augmented with specific training and precautions before attempting to place an SMA or a performance design mix for the first time. SMAs are generally stickier and more difficult to work with than dense-graded mixes because (1) they have more asphalt binder, (2) the asphalt binder is modified, and (3) the binder and filler combination creates viscous mastic. Also, it is not uncommon for large amounts of mastic (the combination of asphalt binder and mineral filler) to collect on paving equipment. If not carefully monitored, this mastic will release from the equipment into the mat leaving an over-asphalted area - commonly referred to as a "fat spot." These considerations only scratch the surface of SMA construction. A more thorough treatment can be found in:

  National Asphalt Pavement Association (NAPA). 1999. *Designing and Constructing SMA Mixtures - State-of-the-Practice*, Quality Improvement Series 122. National Asphalt Pavement Association. Lanham, MD.

- **Mat problems**. The asphalt paver, MTV, rollers, mix design and manufacturing introduce many variables into flexible pavement construction. A familiarity with common causes of the more typical mat problems can help improve construction quality. Some common mat problems are micro cracking, fat spots, joint problems, non-uniform texture, roller marks, shoving, surface waves, tearing (streaks) and transverse screed marks.

Asphalt Paver

In 1934, Barber-Greene introduced the Model 79 asphalt laydown machine, a self-propelled formless laydown machine with a floating screed (Tunnicliff, Beaty and Holt, 1974). Since then, the basic concept of the asphalt paver has remained relatively unchanged:

1. HMA is loaded in the front,
2. carried to the rear by a set of flight feeders (conveyor belts),
3. spread out by a set of augers,
4. then leveled and compacted by a screed.

This set of functions can be divided into two main systems:

- the tractor (or material feed system)
- the screed.

Tractor (Material Feed System)

The tractor contains the material feed system, which accepts the HMA at the front of the paver, moves it to the rear and spreads it out to the desired width in preparation for screed leveling and compaction. The basic tractor components are:

- **Push Roller and Truck Hitch**. The push roller is the portion of the paver that contacts the transport vehicle and the truck hitch holds the transport vehicle in contact with the paver (see Figure 6-20 and Figure 6-21). They are located on the front of the hopper.

Figure 6-20. Push Roller and Truck Hitch.
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Figure 6-21. Truck hitch engaged.
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- **Hopper.** The hopper is used as a temporary storage area for HMA delivered by the transport vehicle. Therefore, the paver can accept more material than is immediately needed and can use the volume in the hopper to compensate for fluctuating material demands created by such things as paving over irregular grades, utility access openings or irregular intersection shapes. Hopper sides (or "wings") can be tilted up (or "folded") to force material to the middle where it is carried to the rear by the conveyor system (see Figure 6-22). Hoppers can also be fit with inserts to allow them to carry more HMA (see Figure 6-23). These inserts are typically used in conjunction with a material transfer vehicle (MTV).

Figure 6-22. Hopper with wings folded up.
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Figure 6-23. Caterpillar AP-1055B with hopper insert.
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- **Conveyor.** The conveyor mechanism carries the HMA from the hopper, under the chassis and engine, and then to the augers (see Figure 6-24 and Figure 6-25). The amount of HMA carried back by the conveyors is regulated by either variable speed conveyors and augers or flow gates, which can be raised or lowered by the operator or, more often, by an automatic feed control system.

Figure 6-24. Conveyors.
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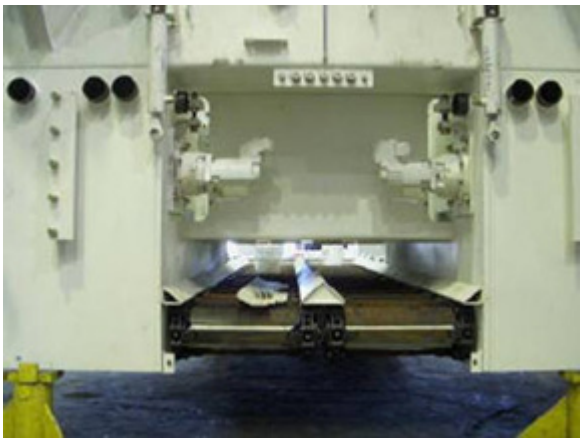

Figure 6-25. Conveyors (seen from the rear of a partially built paver).
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- **Auger**. The auger receives HMA from the conveyor and spreads it out evenly over the width to be paved (see Figure 6-26). There is one auger for each side of the paver and they can be operated independently. Some pavers allow the augers to be operated in reverse direction so that one can be operated forward and the other in reverse to send all the received HMA to one side of the paver. The auger gearbox can either be located in the middle (between the augers as shown in Figure 6-27) or on the outside edge of each auger. If an inadequate amount of HMA is distributed under a middle-located gearbox the result can be a thin longitudinal strip of mat aligned with the gearbox that exhibits lower densities from aggregate segregation and/or temperature differentials (see Figure 6-28 and Figure 6-29).

Figure 6-26. Augers distributing HMA.
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Figure 6-27. Paver Augers (note, gear box in between augers).
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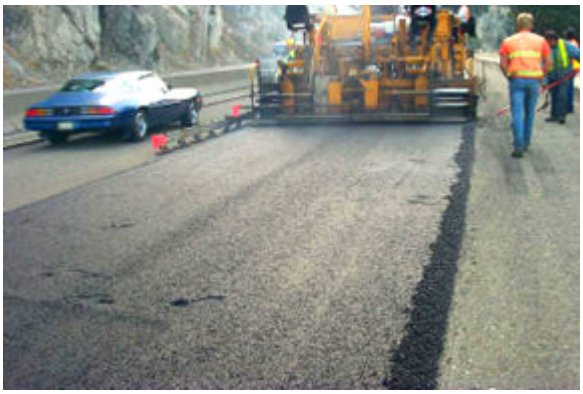

Figure 6-28. Gearbox Streak.
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Figure 6-29. Gearbox streak in a new mat.
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Operation of the tractor, and specifically the material feed system, can have significant effects on overall construction quality and thus long-term pavement performance. Although there are many detailed operational concerns, the two broad statements below encompass most of the detailed concerns:

1. **HMA must be delivered to maintain a relatively constant head of material in front of the screed**. This involves maintaining a minimum amount of HMA in the hopper, regulating HMA feed rate by controlling conveyor/auger speed and flow gate openings (if present), and maintaining a constant paving speed. As the next section will discuss, a fluctuating HMA head in front of the screed will affect the screed angle of attack and produce bumps and waves in the finished mat.
2. **The hopper should never be allowed to empty during paving**. This results in the leftover cold, large aggregate in the hopper sliding onto the conveyor in a concentrated mass and then being placed on the mat without mixing with any hot or fine aggregate. This can produce aggregate segregation or temperature differentials, which will cause isolated low mat densities.

If there are no transport vehicles immediately available to refill the hopper it is better to stop the paving machine than to continue operating and empty the hopper (TRB, 2000).

Screed

The most critical feature of the paver is the self-leveling screed unit, which determines the profile of the HMA being placed (Roberts et al., 1996). The screed takes the head of HMA from the material delivery system, strikes it off at the correct thickness and provides initial mat compaction. This section describes screed terminology, basic forces acting on the screed, screed factors affecting mat thickness and smoothness, automatic screed control, screed operation summary.

1. Screed Terminology. The following is a list of basic screed components and terms:

- **Screed plate**. The flat bottom portion of the screed assembly that flattens and compresses the HMA.
- **Screed angle** (angle of attack). The angle the screed makes with the ground surface.
- **Strike-off plate**. The vertical plate just above the leading edge of the screed used to strike off excess HMA and protect the screed's leading edge from excessive wear.
- **Screed arms**. Long beams that attach the screed to the tractor unit (see Figure 6-30).
- **Tow point**. Point at which the screed arm is attached to the tractor unit (see Figure 6-31).
- **Depth crank**. The manual control device used to set screed angle and ultimately, mat thickness (see Figure 6-30).
- **Screed heater**. Heaters used to preheat the screed to HMA temperature. HMA may stick to a cold screed and cause mat tearing. After the screed has been in contact with the HMA for a short while (usually about 10 min.) its temperature can be maintained by the HMA passing beneath it and the heater can be turned off. If the screed is removed from contact with HMA for an extended period of time, it may need to be preheated again before resuming paving.
- **Screed vibrator**. Device located within the screed used to increase the screed's compactive effort. Screed compaction depends upon screed weight, vibration frequency and vibration amplitude.
- **Screed extensions**. Fixed or adjustable additions to the screed to make it longer (see Figure 6-32 and Figure 6-33). Basic screed widths are between 2.4 m (8 ft.) and 3.0 m (10 ft.). However, often it is economical to use wider screeds or adjustable width screeds. Therefore, several manufacturers offer rigid extensions that can be attached to a basic screed or hydraulically extendable screeds that can be adjusted on the fly.

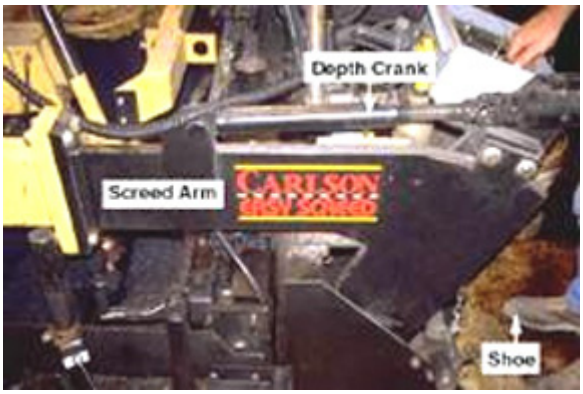

Figure 6-30. Screed close-up showing the screed arm and depth crank.
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Figure 6-31. Tow Point.
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Figure 6-32. Hydraulic Screed Extension.
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(This screed is extended too far [resulting in poor mix delivery and placement] and the tack coat is subpar.)

Figure 6-33. Screed Extension.
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2. Screed Forces. There are six basic forces (see Figure 6-34) acting on the screed that determine its position and angle (Roberts et al., 1996):

- **Towing force**. This is provided by the tractor and exerted at the tow point. Thus, towing force is controlled by paver speed.
- **Force from the HMA head resisting the towing force**. This is provided by the HMA in front of the screed and is controlled by the material feed rate and HMA characteristics.
- **Weight of the screed acting vertically downward**. This is obviously controlled by screed weight.
- **Resistive upward vertical force from the material being compacted under the screed**. This is also a function of HMA characteristics and screed weight.
- **Additional downward force applied by the screed's tamping bars or vibrators**. This is controlled by vibratory amplitude and frequency or tamping bar force.
- **Frictional force between the screed and the HMA under the screed**. This is controlled by HMA and screed characteristics.

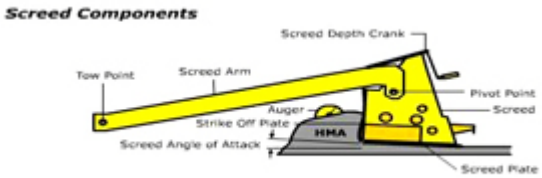

Figure 6-34. Screed components and forces.
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3. Factors Affecting Mat Thickness and Smoothness

- Since the screed is free floating it will slide across the HMA at an angle and height that will place these six forces in equilibrium. When any one of these forces is changed, the screed angle and elevation will change (which will change the mat thickness) to bring these forces back into equilibrium. Therefore, changing anything on the paver that affects these forces (such as paver speed, material feed rate or screed tow point) will affect mat thickness. Furthermore, since mat thickness needs to be closely controlled, pavers have controls to manually set screed angle rather than rely on a natural equilibrium to determine mat thickness.
- In typical paving operations the screed angle is adjusted to control mat thickness. In order to understand how a manually controlled screed angle affects mat thickness, a brief discussion of how the paver parameters of speed, material feed rate and tow point elevation affect screed angle, screed height and, therefore, mat thickness is provided.
  - **Speed**. Paver speed affects mat thickness by changing the screed angle. If a paver speeds up and all other forces on the screed remain constant, the screed angle decreases to restore equilibrium, which decreases mat thickness. Similarly, as paver speed decreases, screed angle increases, this increases mat thickness.
  - **Material Feed Rate**. The amount of HMA in front of the screed (the material "head") can also affect screed angle and thus mat thickness. If the material head increases (either due to an increase in material feed rate or a reduction in paver speed), screed angle will increase to restore equilibrium, which increases mat thickness. Similarly, if the material head decreases (either due to a decrease in material feed rate or an increase in paver speed), screed angle will decrease to restore equilibrium, which decreases mat thickness (TRB, 2000). Therefore, in order to maintain a constant mat thickness for a change in paver speed or material head in front of the screed, the natural equilibrium of forces on the screed cannot be relied upon and the screed angle must be manually adjusted using a thickness control screw or depth crank. Screed angle adjustments do not immediately change mat thickness but rather require a finite amount of time and tow distance to take effect. Figure 6-35 shows that it typically takes five tow lengths (the length between the tow point and the screed) after a desired level is input, for a screed to arrive at the new level. Because of this screed reaction time, a screed operator who constantly adjusts screed level to produce a desired mat thickness will actually produce an excessively wavy, unsmooth pavement.

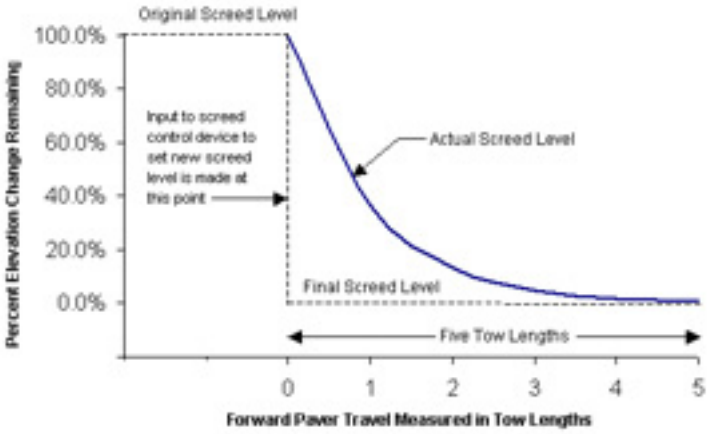

Figure 6-35. Screed reaction to a manual decrease in screed angle, (after TRB, 2000).
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- **Tow Point Elevation**. Finally, tow point elevation will affect screed angle and thus mat thickness. As a rule-of-thumb, a 25 mm (1 in.) movement in tow point elevation translates to about a 3 mm (0.125 in.) movement in the screed's leading edge. Without automatic screed control, tow point elevation will change as tractor elevation changes. Tractor elevation typically changes due to roughness in the surface over which it drives. As the tow point rises in elevation, the screed angle increases, resulting in a thicker mat. Similarly, as the tow point lowers in elevation, the screed angle decreases, resulting in a thinner mat. Locating the screed tow point near the middle of the tractor significantly reduces the transmission of small elevation changes in the front and rear of the tractor to the screed. Moreover, because the screed elevation responds slowly to changes in screed angle, the paver naturally places a thinner mat over high points in the existing surface and a thicker mat over low points in the existing surface (TRB, 2000). The interaction of paver speed, material feed rate and tow point elevation determine the screed position without the need for direct manual input. This is why screeds are sometimes referred to as "floating" screeds.

4. Automatic Screed Control. As discussed previously, the screed angle can be manipulated manually to control mat thickness. However, tow point elevation is not practical to manually control. Therefore, pavers usually operate using an automatic screed control, which controls tow point elevation using a reference other than the tractor body. Since these references assist in controlling HMA pavement grade, they are called “grade reference systems” and are listed below (Roberts et al., 1996):

- **Erected stringline**. This consists of stringline erected to specified elevations that are independent of existing ground elevation. Most often this is done using a survey crew and a detailed elevation/grade plan. Although the stringline method provides the correct elevation (to within surveying and erecting tolerances), stringlines are fragile and easily broken, knocked over or inadvertently misaligned. Lasers can be used to overcome the difficulties associated with stringlines because they do not require any fragile material near the pavement construction area.

Lasers can establish multiple elevation or grade planes even in dusty or high-electronic and light-noise areas and are therefore sometimes used to construct near-constant elevation airport runways. The laser method becomes quite complicated, however, when frequent pavement grade changes are required.

- **Mobile reference**. This consists of a reference system that travels with the paver such as a long beam or tube attached to the paver (called a "contact" device since it actually touches the road - see Figure 6-36) or an ultrasonic device (called a "non-contact" device since it relies on ultrasonic pulses and not physical contact to determine road elevation). The mobile reference system averages the effect of deviations in the existing pavement surface over a distance greater that the wheelbase of the tractor unit. Minimum ski length for a contact device is normally about 7.5 m (25 ft.) with a typical ski lengths being on the order of 12 to 18 m (40 to 60 ft.) (Asphalt Institute, 2001).
- **Joint matching shoe**. This usually consists of a small shoe or ski attached to the paver that slides on an existing surface (such as a curb) near the paver. Ultra sonic sensors accomplish the same task without touching the existing surface by using sound pulses to determine elevation. This type of grade control results in the paver duplicating the reference surface on which the shoe or ski is placed or ultra sonic sensor is aimed.

Figure 6-36. Automatic grade control using a mobile reference beam.
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In addition to grade control, the screed can also be set to control pavement slope and/or crown. A slope controller uses a slope sensor mounted on a transverse beam attached to the screed to determine screed slope, then adjusts screed slope to the desired amount. Generally, one side of the screed is set up to control grade and the opposite side is set up to control slope based on that grade. The usual practice is to run grade control on the side of the screed nearest the pavement centerline and run slope control on the screed side nearest the pavement edge because it is easier to match the centerline joint if grade control is used on that side of the paver (TRB, 2000.)

Screed crown (the elevation of the middle in relation to the edges) can also be controlled. Typically, screeds offer separate front and rear crown controls. If crown control is used, the front control is usually set to a slightly more severe crown than the rear control to allow for easier passage of HMA under the screed.

5. Screed Operation Summary

- The floating screeds used by today's pavers are acted upon by six basic forces, which when left undisturbed result in an equilibrium screed angle and elevation that determines mat thickness. Adjusting paver speed, material feed rate or tow point elevation will change these forces and result in a new equilibrium screed angle and elevation and eventually a new mat thickness. In order to achieve the most consistent thickness and smoothest possible surface, pavers attempt to maintain a constant speed, use automatic feed controls to maintain a consistent head of material in front of the paver, and use automatic screed control to maintain a consistent tow point. Although the screed angle can be adjusted manually to change mat thickness, excessive adjustments will result in a wavy, unsmooth mat. In addition to grade, screeds can also control mat slope and crown to provide almost complete control over mat elevation at any location.

Material Transfer Vehicles (MTVs)

MTVs are used to assist the paver in placing HMA. Most pavers are equipped to receive HMA directly from end dump or live bottom trucks; however in certain situations it can be necessary or advantageous to use an MTV. Paving using bottom dump trucks and windrows requires a windrow elevator MTV (see Figure 6-37 and Figure 6-38), while other MTVs are used to provide additional surge volume, which is advantageous because it allows the paver to operate continuously without stopping, minimizes truck waiting time at the paving site and may minimize aggregate segregation and temperature differentials. This subsection covers:

- Windrow elevators
- Surge volume and remixing MTVs.

Windrow Elevators

Windrow elevators are positioned directly in front of pavers and are designed to pick up HMA placed in a windrow and transfer it to the paver hopper. This allows for (1) windrows to be used and (2) virtually continuous paving without stopping. When using windrows and windrow elevators, the windrow laydown rate must match the paver laydown rate. If the amount of material in the windrow is too little or too much, the paver may become overloaded or may run dry and have to stop. To avoid this, windrow paving operations typically have some method (e.g., a loader) available to add or subtract material from the windrow. Some windrow paving operations establish a windrow laydown rate slightly less than the paver laydown rate then periodically add material to the windrow with an end dump truck. Other windrow paving operations leave periodic spaces in the windrow to avoid paver overloading.

Figure 6-37. Windrow paving.
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Figure 6-38. Windrow Elevator.
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Other MTVs are used to provide an additional surge volume for the paver (see Figure 6-39; Figure 6-40; Figure 6-41; Figure 6-42). This surge volume allows for continuous paver operations. With an MTV, the paver no longer has to stop while one truck leaves and the next truck backs up. Additionally, the MTV serves as a buffer between the paver and the haul trucks, which eliminates most truck bumping problems. Finally, most MTVs offer some sort of remixing capability that remixes the cool HMA crust formed during transport with the hot interior HMA to produce a more uniform mix entering the paver. This remixing can essentially eliminate aggregate segregation and temperature differentials. Some states have actually implemented specifications that require a remixing MTV for paving contracts where segregation and temperature differentials are of concern.

Surge Volume and Remixing MTVs

Surge volume / remixing MTVs are typically used in tandem with a paver hopper insert that increases the capacity of the paver hopper (see Figure 6-23). The insert is removable and sometimes contains remixing apparatus (such as a pugmill) near the bottom. At least one manufacturer

has developed a paver solely for use with an MTV. The Roadtec Stealth™ paver uses gravity feed and does not contain conveyors, hopper wings or push rollers, which reduces initial cost as well as maintenance costs (Roadtec, 2001).

Figure 6-39. Blaw Knox MC-30 MTV.
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Figure 6-40. Cedarapids MS-3 MTV.
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Figure 6-41. Roadtec Shuttle Buggy MTV.

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Figure 6-42. Roadtec shuttle buggy, front view, showing loading hopper for end dump and live bottom trucks.
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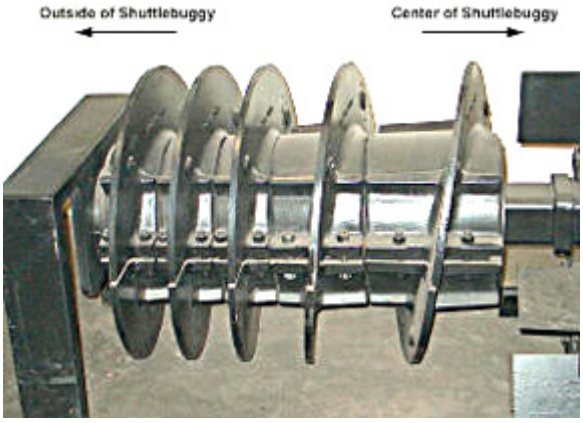

Figure 6-43. Roadtec Shuttle Buggy Patented Remixing Auger.
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Remixing thoroughness varies from one type of MTV to the next. One idea that seems to work well is the Roadtec Shuttle Buggy patented remixing auger (see Figure 6-43). The auger employs three different pitches that get progressively bigger towards the center of the MTV. This allows for additional material to enter the auger area each time the pitch is increased resulting in thorough remixing.

In summary, MTVs assist with the transfer of HMA from the transport trucks to the paver. There are two basic types of MTVs:

- windrow elevator MTVs, and
- surge volume/remix MTVs.

Windrow elevator MTVs are used to pick up HMA from a windrow and place it into the paver hopper while surge volume/remix MTVs provide an additional material surge volume that allows for continuous paving and/or a remix capability that can reduce aggregate/temperature segregation. MTV use costs money and will increase the per ton cost of HMA paving but can help provide superior mat quality. However, MTVs should not be used as a substitute for good production and laydown practices, which are fundamental to good mat quality.

Section 8 — Compaction

Introduction

The top three factors in HMA pavement construction are "compaction, compaction, compaction." Compaction is the process by which the volume of air in an HMA mixture is reduced by using external forces to reorient the constituent aggregate particles into a more closely spaced arrangement. This reduction of air volume in a mixture produces a corresponding increase in HMA unit weight, or density (Roberts et al., 1996). Numerous researchers have stated that compaction is the greatest determining factor in dense graded pavement performance (Scherocman and Martenson, 1984; Scherocman, 1984; Geller, 1984; Brown, 1984; Bell et. al., 1984; Hughes, 1984; Hughes, 1989). Inadequate compaction results in a pavement with decreased stiffness, reduced fatigue life, accelerated aging/decreased durability, rutting, raveling, and moisture damage (Hughes, 1984; Hughes, 1989).

Compaction Measurement and Reporting

Compaction reduces the volume of air in HMA. Therefore, the characteristic of concern is the volume of air within the compacted pavement. This volume is typically quantified as a percentage of air voids by volume and expressed as "percent air voids." Percent air voids is calculated by comparing a test specimen's bulk density with its theoretical maximum density (TMD) and assuming the difference is due to air. Once TMD is known, portable devices can be used to measure HMA density in-place. The terms "percent air voids" and "density" are often used interchangeably. This is not wrong, since density is used to calculate percent air voids, the fundamental parameter of concern is always percent air voids.

Percent air voids is typically calculated by using AASHTO T 269, ASTM D 3203 or an equivalent procedure (AASHTO, 2000). These procedures all use laboratory-determined bulk specific gravity and theoretical maximum specific gravity in the following equation:

$$Percent\ Air\ Voids = 100\left(\frac{G_r - G_a}{G_r}\right)$$

Where:

- G_r = theoretical maximum specific gravity of the particular HMA in question
- G_a = bulk specific gravity of the HMA in question.

These procedures require a small pavement core (usually 100 - 150 mm [4 - 6 in.] in diameter), which is extracted from the compacted HMA (see Figure 6-44 and Figure 6-45). This type of air voids testing is generally considered the most accurate but is also the most time consuming and expensive.

Figure 6-44. Core extraction.
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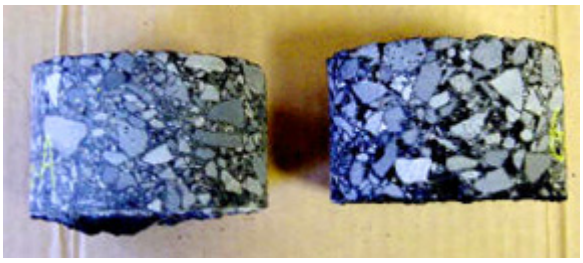

Figure 6-45. Two cores – the core on the right has significantly higher air void.
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Since core extraction is time consuming and expensive, air voids are often measured indirectly using a portable density-measuring device such as a nuclear density gauge (see Figure 6-46) or electrical density gauge (see Figure 6-47).

Figure 6-46. Nuclear Density Gauge.
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Figure 6-47. Electrical Density Gauge.
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Each contracting agency usually specifies the compaction measurement methods and equipment to be used on contracts under their jurisdiction. Most agencies stipulate some sort of extracted core density testing and usually allow testing by nuclear gauge. Electric density gauges are relatively new on the market (in the last five years). Accurate calibration of these devices is essential for their proper use.

Although percent air voids is the HMA characteristic of interest, measurements are usually reported as a measured density in relation to a reference density. This is done by reporting density as:

1. percent of TMD (sometimes called Rice density),
2. percent of a laboratory density, or
3. percent of a control strip density (a control strip is a short pavement strip that is compacted to the desired value under close scrutiny then used as the compaction standard for a particular job).

In sum, percent air voids is the critical HMA characteristic with which compaction is concerned. It can be measured using pavement cores or portable gauges (nuclear or electric) ; measurement specifications vary from one contracting agency to the next. Percent air voids is usually reported as a density in one of three forms:

1. percent TMD,
2. percent of laboratory density, or
3. percent of control strip density. Regardless of the measurement device or reporting method, the key characteristic is percent air voids.

Compaction Importance

The volume of air in an HMA pavement is important because it has a profound effect on long-term pavement performance. An approximate rule-of-thumb is for every 1% increase in air voids (above 6 to 7%), about 10% of the pavement life may be lost (Linden et al., 1989). Keep in mind that this rule-of-thumb was developed using limited project data. This rule-of-thumb applies to air voids above 6 to 7% and should be used with extreme caution. According to Roberts et al. (1996), there is considerable evidence that dense-graded mixes should not exceed 8% nor fall below 3% air voids during their service life. This is because high air void content (above 8%) or low air void content (below 3%) can cause the following pavement distresses (this list applies to dense-graded HMA and not open-graded HMA or SMA):

1. **Decreased stiffness and strength**. Kennedy et al. (1984) concluded that tensile strength, static and resilient moduli, and stability are reduced at high air void content.
2. **Reduced Fatigue Life**. Several researchers have reported the relationship between increased air voids and reduced fatigue life (Pell and Taylor, 1969; Epps and Monismith, 1969; Linden et. al., 1989). Finn et al. (1973) concluded "...fatigue properties can be reduced by 30 to 40% for each 1% increase in air void content." Another study concluded that a reduction in air voids from 8% to 3% could more than double pavement fatigue life (Scherocman, 1984a).
3. **Accelerated Aging/Decreased Durability**. In his Highway Research Board paper, McLeod (1967) concluded that "compacting a well-designed paving mixture to low air voids retards the rate of hardening of the asphalt binder, and results in longer pavement life, lower pavement maintenance, and better all-around pavement performance."
4. **Raveling**. Kandhal and Koehler (1984) found that raveling becomes a significant problem above 8% air voids and becomes a severe problem above 15% air voids.
5. **Rutting**. The amount of rutting which occurs in an asphalt pavement is inversely proportional to the air void content (Scherocman, 1984a). Rutting can be caused by two different mechanisms: vertical consolidation and lateral distortion. Vertical consolidation results from continued pavement compaction (reduction of air voids) by traffic after construction. Lateral distortion – shoving of the pavement material sideways and a humping-up of the asphalt concrete mixture outside the wheelpaths – is usually due to a mix design problem. Both types of rutting can occur more quickly if the HMA air void content is too low (Scherocman, 1984a).
6. **Moisture Damage**. Air voids in insufficiently compacted HMA are high and tend to be interconnected with each other. Numerous and interconnected air voids allow for easy water entry (Kandhal and Koehler, 1984; Cooley et al., 2002) which increases the likelihood of significant moisture damage. The relationship between permeability, nominal maximum aggregate size and lift thickness is quite important and can change significantly as these parameters change.

Air voids that are either too great or too low can cause a significant reduction in pavement life. For dense-graded HMA, air voids between 3 and 8 percent generally produce the best compromise of pavement strength, fatigue life, durability, raveling, rutting and moisture damage susceptibility.

Factors Affecting Compaction

HMA compaction is influenced by a myriad of factors: 1) environment, 2)

mix and structural design, 3) construction (see "Table 6-4: Factors Affecting Compaction").

Table 6-4: Factors Affecting Compaction

Environmental Factors	Mix Property Factors	Construction Factors
Temperature	**Aggregate**	**Rollers**
◆ Ground temperature ◆ Air temperature ◆ Wind speed ◆ Solar flux	◆ Gradation ◆ Size ◆ Shape ◆ Fractured faces ◆ Volume	◆ Type ◆ Number ◆ Speed and timing ◆ Number of passes ◆ Lift thickness
	Asphalt Binder	**Other**
	◆ Chemical properties ◆ Physical properties ◆ Amount	◆ HMA production temperature ◆ Haul distance ◆ Haul time ◆ Foundation suppor

Environmental factors are determined by when and where paving occurs. Paving operations may have some float time, which allows a limited choice of "when," but paving location is determined by road location so there is essentially no choice of "where." Mix and structural design factors are determined before construction and although they should account for construction practices and the anticipated environment, they often must compromise ease of construction and compaction to achieve design objectives. Obviously, construction factors are the most controllable and adaptable of all the factors affecting compaction. Although some factors like haul distance/time, HMA production temperature, lift thickness and type/number of rollers may be somewhat predetermined, other factors associated with roller timing, speed, pattern and number of passes can be manipulated as necessary to produce an adequately compacted mat. This subsection discusses:

- temperature (the environmental factor)
- mix property factors

NOTE: "Compaction Equipment" and "Roller Variables" discuss construction factors.

Temperature

HMA temperature has a direct effect on the viscosity of the asphalt cement binder and thus compaction. As HMA temperature decreases, its asphalt cement binder becomes more viscous and resistant to deformation, which results in a smaller reduction in air voids for a given compactive effort. As the mix cools, the asphalt binder eventually becomes stiff enough to effectively prevent

any further reduction in air voids regardless of the applied compactive effort. The temperature at which this occurs, commonly referred to as **cessation temperature**, is a function of the mix property factors in the "Table 6-4: Factors Affecting Compaction". In some literature it is reported to be about 79°C (175°F) for dense-graded HMA (Scherocman, 1984b; Hughes, 1989). Below cessation temperature rollers can still be operated on the mat to improve smoothness and surface texture but further compaction will generally not occur. Conversely, if the binder is too fluid and the aggregate structure is weak (e.g., at high temperatures), roller loads will simply displace, or "shove" the mat rather than compact it. In general, the combination of asphalt cement binder and aggregate needs to be viscous enough to allow compaction but stiff enough to prevent excessive shoving.

Mat temperature then, is crucial to both the actual amount of air void reduction for a given compactive effort, and the overall time available for compaction. If the initial temperature and cool-down rate are known, the temperature of the mat at any time after laydown can be calculated. Based on this calculation rolling equipment and patterns can be employed to:

1. **Take maximum advantage of available roller compactive effort**. Rollers can be used where the mat is most receptive to compaction and avoided where the mat is susceptible to excessive shoving.
2. **Ensure the mat is compacted to the desired air void content before cessation temperature is reached.** This can be done by calculating the time it takes the mat to cool from initial temperature to cessation temperature. All compaction must be accomplished within this "time available for compaction."

The major factors affecting time available for compaction are (Roberts et al., 1996):

- **Initial mat temperature**. Higher initial mat temperatures require more time to cool down to cessation temperature, thus increasing the time available for compaction. However, overheating the HMA will damage the asphalt binder and cause emissions.
- **Mat or lift thickness**. Thicker lifts have a smaller surface-to-volume ratio and thus lose heat more slowly, which increases the time available for compaction.
- **Temperature of the surface on which the mat is placed**. Hotter surfaces will remove heat from the mat at a slower rate, increasing the time available for compaction.
- **Ambient temperature**. Hotter air temperatures will remove heat from the mat at a slower rate, increasing the time available for compaction.
- **Wind speed**. Lower wind speeds will decrease mat heat loss by convection, which will increase the time available for compaction.

Jordan and Thomas (1976) pointed out additional factors affecting mat cool-down rate that include mat density, pavement layer thermal conductivity, specific heat, convection coefficient, incident solar radiation and coefficients of emission and absorption of solar radiation for the pavement surface.

HMA temperature affects its binder viscosity, which affects compaction in two ways:

1. the colder and more viscous the binder, the less actual amount of air void reduction for a given compactive effort, and
2. HMA can only be compacted until it reaches cessation temperature, therefore initial HMA temperature and mat cool-down rate establish a fundamental compaction parameter for the overall time available for compaction.

Many factors influence HMA temperature and cool-down rate including initial mat temperature, mat thickness, temperature of the surface on which the mat is placed, ambient temperature and wind speed.

Mix Properties

Mix aggregate and binder properties can also affect compaction. They do so by affecting the:

- ease with which aggregate will rearrange under roller loads, and
- viscosity of the binder at any given temperature.

Gradation affects the way aggregate interlocks and thus the ease with which aggregate can be rearranged under roller loads. In general, aggregate effects on compaction can be broken down by aggregate size (TRB, 2000):

1. **Coarse aggregate**. Surface texture, particle shape and the number of fractured faces can affect compaction. Rough surface textures, cubical or block shaped aggregate (as opposed to round aggregate) and highly angular particles (high percentage of fractured faces) will all increase the required compactive effort to achieve a specific density.
2. **Midsize fine aggregate (between the 0.60 and 0.30-mm [No. 30 and No. 50] sieves)**. High amounts of midsize fine, rounded aggregate (natural sand) cause a mix to displace laterally or shove under roller loads. This occurs because the excess midsize fine, rounded aggregate results in a mix with insufficient voids in the mineral aggregate (VMA). This gives only a small void volume available for the asphalt cement to fill. Therefore, if the binder content is just a bit high it completely fills the voids and the excess serves to (1) resist compaction by forcing the aggregate apart and (2) lubricate the aggregate making it easy for the mix to laterally displace.
3. **Fines or dust (aggregate passing the 0.075-mm [No. 200] sieve)**. Generally, a mix with a high fines content will be more difficult to compact than a mix with a low fines content.

The asphalt binder grade affects compaction through its viscosity. A binder that is higher in viscosity will generally result in a mix that is more resistant to compaction. Additionally, the more a binder hardens (or ages) during production, the more resistant the mix is to compaction.

Asphalt binder content also affects compaction. Asphalt binder lubricates the aggregate during compaction and, therefore, mixes with low asphalt content are generally difficult to compact

because of inadequate lubrication, whereas mixes with high asphalt content will compact easily but may shove under roller loads (TRB, 2000).

Sometimes, a combination of mix design factors produces what is known as a tender mix. Tender mixes are internally unstable mixes that tend to displace laterally and shove rather than compact under roller loads.

Compaction Equipment

There are three basic pieces of equipment available for HMA compaction:

1. the paver screed,
2. the steel wheeled roller and
3. the pneumatic tire roller.

Each piece of equipment compacts the HMA by two principal means:

1. **By applying its weight to the HMA surface and compressing the material underneath the ground contact area**. Since this compression will be greater for longer periods of contact, lower equipment speeds will produce more compression. Obviously, higher equipment weight will also increase compression.
2. **By creating a shear stress between the compressed material underneath the ground contact area and the adjacent uncompressed material**. When combined with equipment speed, this produces a shear rate. Lowering equipment speed can decrease the shear rate, which increases the shearing stress. Higher shearing stresses are more capable of rearranging aggregate into more dense configurations.

These two means of densifying HMA are often referred to collectively as “compactive effort." This section discusses the paver screed, the steel wheeled roller (both static and vibratory) and the pneumatic tire roller as they apply to HMA compaction. 'Compaction Sequence' discusses how each one of these pieces of compaction equipment work together in a typical construction scenario. This subsection covers:

- the paver screed
- steel wheel rollers (including vibratory rollers)
- pneumatic tire rollers.

Paver Screed

The paver screed has previously been discussed in 'Screed.' Of additional note here is that approximately 75 to 85% of the theoretical maximum density of the HMA will be obtained when the mix passes out from under the screed (TRB, 2000).

Steel Wheel Rollers

Steel wheel rollers are self-propelled compaction devices that use steel drums to compress the underlying HMA. They can have one, two or even three drums, although tandem (2 drum) rollers are most often used. The drums can be either static or vibratory and usually range from 86 to 215 cm (35 to 85 in.) in width and 50 to 150 cm (20 to 60 in.) in diameter. Roller weight is typically between 0.9 and 18 tonnes (1 and 20 tons) (see Figure 6-48 and Figure 6-49).

Figure 6-48. Small Static Steel Wheel Roller (1.32 tonnes [1.45 tons], 86 cm [34-inch] wide drum).
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Figure 6-49. Large Vibratory Steel Wheel Roller (17 tonnes [18.7 tons], 213 cm [84-inch] wide drum).
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In addition to their own weight, some steel wheel rollers can be ballasted with either sand or water to increase their weight and thus, compactive effort. Although this ballasting is a fairly simple process (see Figure 6-50), it is usually done before rolling operations start and rarely during rolling operations. Since asphalt cement binder sticks to steel wheels, most steel wheel rollers spray water on the drums to prevent HMA from sticking, and are equipped with a transverse bar on each drum to wipe off HMA. Note, however, that this water will cool the HMA and can reduce the time available for compaction.

Figure 6-50. Filling up with water.
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Vibratory Steel Wheel Roller

Some steel wheel rollers are equipped with vibratory drums. Drum vibration adds a dynamic load to the static roller weight to create a greater total compactive effort. Drum vibration also reduces friction and aggregate interlock during compaction, which allows aggregate particles to move into final positions that produce greater friction and interlock than could be achieved without vibration. Roller drum vibration is produced using a rotating eccentric weight located in the vibrating drum (or drums) and the force it creates is proportional to the eccentric moment of the rotating weight and the speed of rotation (TRB, 2000). Operators can turn the vibrations on or off and can also control amplitude (eccentric moment) and frequency (speed of rotation). Vibration frequency and amplitude have a direct effect on the dynamic force (and thus the compactive force) as shown in Table 6-5:

Table 6-5: Vibratory Steel Wheel Roller Parameters (after TRB, 2000)

Parameter	Typical Values	Effect on Dynamic Force
Frequency	1,600 to 3,600 vibrations per minute	Frequency $\propto$ (Dynamic Force)^2
Amplitude	0.25 to 1.02 mm (0.01 to 0.04 in.)	Amplitude $\propto$ Dynamic Force

The ideal vibratory frequency and amplitude settings are a compromise based on desired mat smoothness, HMA characteristics and lift thickness. Low vibration frequencies combined with high roller speeds will increase the distance between surface impacts and create a rippled, unsmooth surface. In general, higher frequencies and lower roller speeds are preferred because they decrease the distance between surface impacts, which:

- increases the compactive effort (more impacts per unit of length), and
- provides a smoother mat.

The recommended impact spacing is 3 - 4 impacts/m (10 - 12 impacts/ft.). The following table shows basic guidance for vibratory settings.

Table 6-6: Typical Vibratory Settings (from TRB, 2000)

HMA/Mat Characteristics	Frequency	Amplitude
Thin Lifts (< about 30mm [1.25 in.])	Operate in static mode. Under vibratory mode, as the pavement increases in density the drums may begin to bounce, which may cause the HMA to shove and become less dense. Also, some of the aggregates may be crushed.	
Lifts between 30 mm and 65 mm (1.25 and 2.5 in.)	High frequency	Low amplitude
Lifts beyond 65 mm (2.5 in.)	High frequency	Higher amplitude
Stiff (more viscous) HMA	High frequency	Higher amplitude

As a general rule-of-thumb, a combination of speed and frequency that results in 3 - 3.5 impacts/m (10 - 12 impacts/ft.) is good. At 3,000 vibrations/min. that gives a speed of 4.5 - 5.5 km/hr. (2.8 - 3.4 mph).

When density is difficult to quickly achieve with a vibratory steel wheel roller, the tendency may be to increase vibratory amplitude to increase compactive effort. However, high amplitude is only advisable on stiff mixes or very thick lifts that can support the increased amplitude without fracturing the constituent aggregate particles. For typical mix types and lift thicknesses a better solution is usually to maintain low amplitude vibrations and increase the number of roller passes at low amplitude.

Vibratory steel wheel rollers offer potential compaction advantages over static steel wheel rollers, but they also require the operator to control more compaction variables (amplitude, frequency and vibratory mode use) and there are certain situations they must be used with caution (e.g., over shallow underground utilities, in residential areas, thin overlays).

In general, steel wheel rollers provide the smoothest mat finish of all compaction equipment. When operated in the vibratory mode, they also provide substantial compactive effort.

Pneumatic Tire Rollers

The pneumatic tire roller is a self-propelled compaction device that uses pneumatic tires to compact the underlying HMA. Pneumatic tire rollers employ a set of smooth (no tread) tires on each axle; typically four on one axle and five on the other. The tires on the front axle are aligned with the gaps between tires on the rear axle to give complete and uniform compaction coverage over the width of the roller. Compactive effort is controlled by varying tire pressure, which is typically set between 400 kPa (60 psi) and 800 kPa (120 psi) (TRB, 2000).

Asphalt binder tends to stick to cold pneumatic tires but not to hot pneumatic tires. A release agent (like water) can be used to minimize this sticking, however if asphalt binder pickup (the asphalt

binder sticking to the tires) is not permanently damaging the mat, it is better to run the roller on the hot mat and let the tires heat up to near mat temperature. Tires near mat temperature will not pick up an appreciable amount of asphalt binder. Insulating the tire area with rubber matting or plywood helps maintain the tires near mat temperature while rolling (see Figure 6-51).

Figure 6-51. Pneumatic Tire Roller (notice rubber matting insulation around tire area as well as tire marks left in the new mat in front of the roller).
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In addition to a static compressive force, pneumatic tire rollers also develop a kneading action between the tires that tends to realign aggregate within the HMA. This results in both advantages and disadvantages when compared to steel wheel rollers:

- Advantages (Brown, 1984)
  - They provide a more uniform degree of compaction than steel wheel rollers.
  - They provide a tighter, denser surface thus decreasing permeability of the layer.
  - They provide increased density that many times cannot be obtained with steel wheeled rollers.
  - They compact the mixture without causing checking (hairline surface cracks) and they help to remove any checking that is caused with steel wheeled rollers.
- Disadvantages
  - The individual tire arrangement may cause deformations in the mat that are difficult or impossible to remove with further rolling. Thus, they should not be used for finish rolling.
  - If the HMA binder contains a rubber modifier, HMA pickup (mix sticking to the tires) may be so severe as to warrant discontinuing use of the roller.

In summary, pneumatic tire rollers offer a slightly different type of compaction than steel wheel rollers. The arrangement of multiple tires on both axles serves to both compress and knead the mat, which may or may not be advantageous over steel wheel rollers.

Roller Variables

There are several variables associated with rollers that can be adjusted from job to job. These variables are:

- sequence and number of rollers
- roller speed
- number of roller passes over a given area of the mat
- location at which each roller works
- pattern that each roller uses.

Not all these variables are infinitely adjustable, but by adjusting a combination of them a rolling plan can be developed that will optimize mat compaction.

Compaction Sequence

HMA compaction is typically accomplished by a sequential train of compaction equipment (see Figure 6-52). This allows each piece of equipment to be used only in its most advantageous situation resulting in a higher quality mat (both in density and in smoothness) than could be produced with just a single method of compaction.

Figure 6-52. Breakdown and Intermediate Rollers.
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A typical compaction train consists of the following (in order of use):

1. **Screed**. The screed is the first device used to compact the mat and may be operated in the vibratory mode.
2. **Breakdown Roller**. The breakdown roller is the first roller behind the screed and therefore, generally affects the most density gain of any roller in the sequence. Breakdown rollers can be of any type but are most often vibratory steel wheel and sometimes pneumatic tire.
3. **Intermediate Roller**. The intermediate roller is used behind the breakdown roller if additional compaction is needed. Pneumatic tire rollers are sometimes used as intermediate rollers because they provide a different type of compaction (kneading action) than a breakdown steel wheel vibratory roller. This can help further compact the mat or, at the very least, rearrange the aggregate within the mat to make it receptive to further compaction.
4. **Finish Roller**. The finish roller is last in the sequence and is used to provide a smooth mat surface. Although the finish roller does apply compactive effort, by the time it comes in contact with the mat, the mat may have cooled below cessation temperature. Static steel wheel rollers

are almost always used as finishing rollers because they can produce the smoothest surface of any roller type.

5. **Traffic**. After the rollers have compacted the mat to the desired density and produced the desired smoothness, the new pavement is opened to traffic. Traffic loading will provide further compaction in the wheel paths of a finished mat. For instance, a mat compacted to 8% air voids and then opened to heavy traffic (e.g., an interstate freeway) may further compact to about 3 to 5% air voids in the wheel paths over time.

Each position in the roller train (breakdown, intermediate and finish) may be performed by one roller or several rollers in parallel. For instance, a large paving project may use two vibratory steel wheel rollers for breakdown rolling, one pneumatic tire roller for intermediate rolling and two static steel wheel rollers for finish rolling. The determination of the best rolling sequence and the number of rollers is generally made on a case by case basis and depends upon the desired final air voids, available rollers and their operating parameters, rolling patterns, mix properties, and environmental conditions.

Roller Speed

Rollers are slow; the fastest, operating speeds may reach about 11 km/h (7 mph). In order to provide complete and uniform mat compaction, rollers should be operated at a slow, constant speed. Operating at high speeds will reduce compactive effort while varying roller speed can cause non-uniform compaction. The following table shows typical roller speeds.

Table 6-7: Typical Roller Speed Ranges (from TRB, 2000)

Type of Roller	Breakdown	Intermediate	Finish
Static Steel Wheel	3.2 – 5.6 km/h (2.0 – 3.5 mph)	4.0 – 6.5 km/h (2.5 – 4.0 mph)	4.8 – 8.0 km/h (3.0 – 5.0 mph)
Pneumatic	3.2 – 5.6 km/h (2.0 – 3.5 mph)	4.0 – 6.5 km/h (2.5 – 4.0 mph)	6.4 – 11.2 km/h (4.0 – 7.0 mph)
Vibratory Steel Wheel	3.2 – 4.8 km/h (2.0 – 3.0 mph)	4.0 – 5.6 km/h (2.5 – 3.5 mph)	not used

As mentioned previously, roller compactive effort comes in two forms:

1. material compression under the ground contact area, and
2. shear stress between the compressed area and adjacent uncompressed areas.

Operating at lower speeds allows the roller to remain in contact with a particular mat location longer than it would at higher speeds. This results in more compression per roller pass and, therefore, increases compactive effort. Speed also affects the magnitude of shear stress developed. Lower speeds result in the shearing force between compressed and uncompressed areas being applied for a longer period of time for a particular area (giving a lower shear rate), which results in a higher

shear stress. The higher the shear stress, the better able it is to rearrange aggregate into a denser configuration. Therefore, as roller speed decreases, shear stress increases and compactive effort increases.

Because speed affects compactive effort, varying roller speed will vary compactive effort resulting in uneven compaction. Varying roller speed typically occurs when operators are not closely monitoring their speed or when they speed up to roll an area more quickly so that they can keep pace with the paver. If the mat is being laid down at a faster rate than it can be rolled, the solution should not be to speed up the rollers but rather should involve one of the following options (TRB, 2000):

1. **Slow down the paver.** This may involve adjusting production and material delivery rate as well.
2. **Use more rollers**. Adding rollers can increase the number of roller passes in a given time without reducing the compactive effort per pass.
3. **Use larger, wider rollers**. Wider rollers allow greater coverage per pass.

Finally, rollers should not be stopped on a fresh mat because they can cause large indentations that are difficult, if not impossible, to remove.

Roller speed directly affects compactive effort. The best compactive effort and most uniform densities are achieved by slow, consistent roller speeds. If rollers cannot keep up with the pace of the paving operation, they should not be operated at higher speeds because this reduces compactive effort. Rather, the paving operation should be slowed or more/larger rollers should be used.

Number of Roller Passes

Generally, it takes more than one roller pass over a particular area to achieve satisfactory compaction. A roller pass over a specific mat area is defined as one complete trip over the area in question by the entire roller. This means that if the roller uses two steel drums, both drums must travel over the area in question to make "one pass." In general, earlier passes over hotter HMA will increase density (decrease air voids) more than later passes over cooler HMA (see Figure 6-53)

Reference and Conditions Information for Figure 6-53

Supporting Data:
Graph and data taken from Chadbourn et al. (1998).

- Dense graded HMA
- 64-mm (2.5-in.) lift thickness
- 16 km/h (10 mph) wind speed
- 19.4°C air temperature
- Existing surface is milled HMA
- 22.7°C surface temperature
- 50% cloud cover.

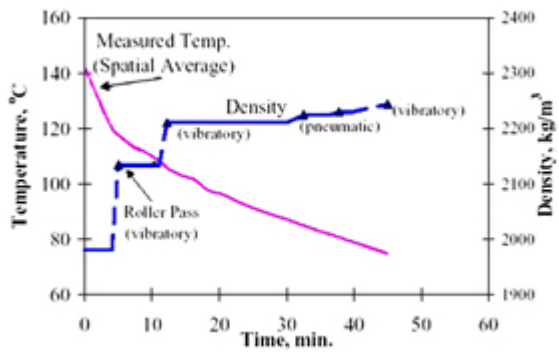

Figure 6-53. Density and Measured Mat Temperature vs. Time (note the increase in density for each roller pass) (Chadbourn et al., 1998).
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- Test Strip
  - Contractors will often (and are sometimes required to) construct a "test strip" to help determine the necessary number of passes.
  - A test strip is a small section of mat laid out at the beginning of a project with the purpose of determining the best roller type, sequence, number of passes and rolling pattern to use.

Rolling Location

Determining where the different rollers in the train should physically be is actually a question of mat temperature and roller characteristics and not one of physical distance. 'Compaction Sequence' under "Roller Variables" described the roller sequence and its reasoning while this section describes some more general rules-of-thumb.

In general, the greatest compaction per roller pass can be achieved directly behind the paver because the mat is the hottest and least viscous in that position. Therefore, the breakdown roller(s) should operate as close to the paver as possible to achieve the most compaction per roller pass. Likewise, the intermediate roller(s) and finish roller(s) should be placed on the mat at a safe distance from the roller in front of them and begin rolling as soon as possible. Sometimes, when a tender mix is placed, these general rules do not apply.

Roller Pattern

The roller pattern combines roller sequence, speed, number of passes and location to provide complete coverage of the entire mat in such a manner that results in:

- uniform compaction to a specified level of air voids,
- acceptable surface smoothness and
- complete compaction before cessation temperature is reached.

Uniform compaction depends on getting the same number of roller passes over each area of the mat. This means that a pattern must be developed that covers the entire mat with an equal number of roller passes from each type of roller. For example, if two vibratory steel wheel rollers are operating as the breakdown rollers, they must work together so each portion of the mat receives the same number of passes. Since they are the same type of roller, it is not necessary for each roller to cover the entire mat. If two different rollers, such as, a vibratory steel wheel roller and a pneumatic tire roller are performing breakdown rolling, each roller should cover the entire mat an equal number of times, otherwise compaction may be non-uniform. Although roller patterns can vary widely, some general rules-of-thumb are:

- Overlap between two successive passes should be at least 15 cm (6 in.) (Roberts et al., 1996; Ingersoll-Rand, 2001). This ensures that small steering inaccuracies do not leave gaps between successive passes.
- The roller should be turned slightly to the side when reversing directions or stopping. Rollers tend to create a slight bow hump when moving and will leave this hump in place when reversing directions or stopping. Often, it is difficult to flatten out this hump on subsequent passes if it is perpendicular to the direction of roller travel. By turning the roller slightly before changing direction or stopping, the resulting hump will be diagonal to the direction of roller travel and easier to flatten out with subsequent passes. However, hard steering should be avoided because it can tear or shove the mat.
- Roller passes should end at different points to prevent developing a hump (caused by the direction change) that spans the entire transverse length of the mat.
- Where there is an unconfined edge on the mat, the first roller pass should stay about 0.15 - 0.30 m (0.5 - 1 ft.) away from the mat edge. The small resultant strip of uncompacted mat helps confine the rest of the mat and minimize lateral displacement near mat edges. This strip should then be compacted on the next roller pass (Ingersoll-Rand, 2001).
- Do not roll over a designed crown in the road. Rolling over a crown will flatten it out.
- When compacting a longitudinal joint, the first roller pass should be entirely on the hot mat about 0.15 - 0.30 m (0.5 - 1 ft.) away from the joint. On subsequent passes, the roller should travel mostly on the newly constructed mat and only overlap the older mat by about 0.15 m (0.5 ft.) (Roberts et al., 1996; Ingersoll-Rand, 2001).
- Joints should be compacted with the roller operating parallel to the joint. Although transverse joints cannot always be compacted this way, perpendicular rolling does not compact the hot/new side as well.
- For steel wheeled rollers, operate the powered wheel on the paver side. This will minimize humps that can be caused by the drive wheel.

The above thumb-rules are just general guidance; other methods may work. However, without a clear roller pattern, the center of a lane typically receives more roller passes than the outsides. This is of particular concern because most wheel loads occur nearer the edges of any particular lane in

the wheel paths. In summary, any method that achieves uniform coverage, acceptable density and acceptable smoothness without damaging the mat can be considered a good method.

Summary

Although compaction looks like a simple job, it is far from it. Variables such as sequence, speed, number of passes, location, pattern and mat temperature make it quite complex. All these variables have a profound effect on air voids and thus pavement performance. Simply put, good compaction is essential to quality pavement.

Section 9 — Ride Quality

Ride Quality Parameter (IRI)

IRI is the International Roughness Index and measures pavement smoothness. The lower the calculated IRI, the smoother the pavement will ride. The higher the IRI, the rougher the pavement will ride. The units of IRI are usually in./mile, m/km, or mm/m. For more detailed information on IRI, refer to *The Little Book of Profiling* (Sept 1998) by Michael W. Sayers and Steven M. Karamihas.

Equipment for Measuring Ride Quality

The inertial profiler is a device that uses laser sensors to measure the profile of the pavement surface. There are generally three types of inertial profilers:

1. one that can travel at highway speeds, a high speed profiler,
2. a light weight profiler that can travel at a minimum speed of 12 mph, and
3. a portable profiler.

Figure 6-54 and Figure 6-55 are pictures of a high speed profiler and a light weight profiler.

Figure 6-54. High Speed Inertial Profiler.
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Figure 6-55. Light Weight Profiler.
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Pay Schedule

The pay schedule is used to determine the level of bonus or penalty for each 0.1-mi. section on the project. Pay schedules 1 and 2 have the same bonus schedule. Under Item 585, pay schedule 3 has a lower bonus schedule than pay schedules 1 and 2. Penalty schedule is most severe for schedule 1 less severe for schedule 2 and there is no penalty for schedule 3. The Construction Division has a written guideline that should be used to determine the most appropriate pay schedule. The pay schedule should be selected based on the existing IRI, class of the road, posted speed, the number of smoothness opportunities, and other mitigating factors. Refer to the Guidance Document on Item 585 Ride Quality for Pavement Surfaces for more detail.

Smoothness Opportunity

A smoothness opportunity is defined as a chance for the contractor to improve ride quality. Examples:

1. placing an overlay on an existing pavement surface is considered one smoothness opportunity,
2. placing a level up and overlay is considered two smoothness opportunities,
3. hot in-place recycling with a new overlay is considered two smoothness opportunities, and
4. grading for a base course is considered one smoothness opportunity.

Spot level-ups, milling operations, or seal coats are not considered a smoothness opportunity. Mill and fill operations that require matching the existing pavement are not considered a smoothness opportunity.

Analysis of Ride Data

The ride data is analyzed using TxDOT's Ride Quality computer program. The analysis includes a description of the project:

District, county, highway, beginning reference marker, end reference marker, CSJ, lane designation, IRI values every 0.1 mile, pay, locations of bumps or dips and total pay adjustment.

Chapter 7 — Flexible Pavement Rehabilitation

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Section 10 — Alternate Pavement Rehabilitation Options

Section 1 — Overview

Developing a rehabilitation design generally requires extensive investigation into the condition of the existing pavement structure, performance history, and laboratory testing of materials to establish suitability of existing and proposed materials for use in the rehabilitation design. The field investigation will require a deflection survey, drainage survey, and perhaps additional non-destructive testing (NDT) surveys such as ground penetrating radar (GPR), dynamic cone penetrometer (DCP), and seismic. Examination of multi-year Pavement Management Information System (PMIS) distress and ride data will show performance related issues. Once these preliminary surveys are conducted, locations for material sampling can be established. In addition, for projects where full-depth reclamation is being considered, samples of the structure should be taken at intervals not to exceed 0.5-mi. These samples will be evaluated in the lab to verify field survey conclusions and establish basic properties necessary to quantify moisture susceptibility, stabilizer compatibility, blending requirements, etc.

The preferred rehabilitation strategy should consider:

- cost-effectiveness
- repair of the specific problems of the existing pavement
- prevention of future problems
- meeting all existing constraints of the project.

Section 2 — In-place Surface Recycling

Hot In-place Recycling (HIPR)

This rehabilitation technique is meant to address asphalt concrete surface distress and texture issues only, therefore the underlying base layers must offer adequate support.

The HIPR process involves recycling the existing asphalt surface layer by heating, scarifying, and adding a recycling agent. There are three basic HIPR processes:

1. Heater-scarification - heating, scarifying, rejuvenating, leveling, reprofiling, and compacting.
2. Repaving - heating, scarifying, rejuvenating, leveling, laying a new hot mix layer, reprofiling, and compacting.
3. Remixing - heating, scarifying, rejuvenating, mixing (adding a new hot mix), leveling, reprofiling, and compacting.

A specialized train is used to heat the surface of the pavement to 300 - 350°F which allows the top 1 to 2 in. of material to be scarified, rejuvenated, remixed and replaced in a multi-step on-site process. Minor cross slope and aggregate gradation corrections can be made. Virgin hot mix can also be added to correct the recycled asphalt pavement (RAP) shortcomings as in the case of the remixing process. Coring of the existing asphaltic concrete pavement (ACP) surface is necessary to determine the material properties of the existing asphalt pavement. This will allow for evaluation of any necessary adjustments to aggregate gradation to develop the required voids in mineral aggregate (VMA) and selection of the appropriate asphalt concrete (AC) binder. It is recommended that the vendor/contractor be contacted with regard to the mix design.

Pavements with delaminations, especially saturated delaminations, in the top 2 in. should not be considered for HIPR projects. Also, the state of practice does not recommend pavements that have been rutted, heavily patched, or chip-sealed as good candidate projects. Typically, hot in-place recycling operations are conducted when the ambient air temperature is 50ºF and rising, but 2004 specifications cite a minimum surface temperature for hot mix asphalt (HMA) placement of 60ºF. An additional overlay may be placed over the recycled surface if additional structural strength is needed as in the case of the repaving process. An equivalent alternative strategy is to mill 1 in. of the existing top layer and place back 2 in. Up to 30% RAP may be allowed in the new 2-in. overlay.

Figure 7-1. Hot In-place Recycling Train.

Cold In-place Recycling (Bituminous Layers Only)

As with the hot in-place process, this rehabilitation technique is meant to address distress within the bituminous portion of the pavement structure, but can reach as deep as 4 to 6 in. Therefore, the base must also be sound, with repairs made to locations that have failed or show potential for failure. The process also involves a specialized train with a cold milling machine, crushing/screen unit, and mixing unit that is capable of reclaiming the old asphalt, crushing (screening and sizing) the RAP, and mixing the RAP with virgin aggregate (if necessary) and emulsion. Coring of the existing ACP surface is necessary to determine the material properties of the existing asphalt pavement to properly design proportions of virgin aggregate, emulsion and rejuvenator, if necessary. Cores are also inspected for the presence of variations in the pavement layers, delaminations and whether voids are saturated. The industry does not recommend pavements that have been rutted, heavily patched, or chip-sealed as good candidate projects. Generally, a seal coat or additional overlay will be required after adequate curing since the cold re-processed mix has higher void ratios and is more difficult to compact than hot mix.

Section 3 — Geosynthetics

Geosynthetics in Hot Mix Asphalt (HMA) Applications

Geosynthetic products, defined herein as fabrics, grids, composites, or membranes, have been used by TxDOT since the mid-1980s. The primary purpose of incorporating the use of geosynthetics in the pavement design process is to reduce reflective cracking in HMA overlays and to resist moisture intrusion into the underlying pavement structure. Geosynthetics can be part of an overall rehabilitation strategy that will, as a minimum, include the placement of a new wearing/surface course of hot mix asphaltic concrete (HMAC). TxDOT investigated these products in 2001 as part of research project 0-1777. One of the products from this research was the publication of *Geosynthetic Guidelines.* This document discusses the advantages of using geosynthetics in HMA applications, guidance on the selection of materials, cost considerations, pavement design, as well as construction considerations. One concern that the geosynthetic users should keep in mind is future rehabilitations as any anticipated milling of HMAC layers must avoid RAP contamination and possible fouling of milling equipment.

Geogrids

The main function of a geogrid in an HMA application is to retard the occurrence of reflective cracking. In evaluating the appropriateness of use, cracking in the existing structure should be limited to cases in which the crack faulting does not fluctuate significantly with traffic loading and crack width does not fluctuate significantly with temperature differentials. The pavement should be structurally sound with existing cracks limited to less than 3/8 in. width. Hence, low to moderate levels of alligator cracking, or random cracking may benefit from application of grids in HMA, whereas widely spaced thermal cracking or underlying rocking/faulted Portland cement concrete (PCC) slabs will probably not benefit. It is necessary to repair localized, highly distressed/weak areas and apply a level-up course of HMAC. Where rutting exceeding 1/2 in. exists, milling prior to applying the level-up should be considered. A minimum 2.0-in. surfacing course is recommended. Installation of this type of product has proven to be problematic and will result in premature failure (fatiguing) of the surfacing overlay where a lack of bonding (surface to grid to level-up) occurs. It is highly recommended that the manufacturer's installation procedures be strictly followed and that a manufacturer's representative be present during the planning and construction process.

Fabrics, Composites, and Membranes

These products provide a moisture barrier in addition to varying degrees of resistance to reflective cracking. Application guidelines are similar to those recommended above for the geogrid. The impermeable qualities of these products can be a double-edged sword in that they prevent trapped moisture within the structure from transpiring out. This may result in debonding of HMA layers and/or stripping of HMA layers below the product, especially if the lower mixes are moisture susceptible. Also, if the surfacing overlay is permeable and surface moisture can not readily escape the

section laterally (mill and inlay technique is especially prone), stripping of the surface mix may also occur. It is incumbent upon users of these products to insure laboratory testing is performed to determine HMAC stripping susceptibility of existing mixes (highway cores) and the proposed level-up and overlay mixes.

Geosynthetics in Pavement Bases (non-HMA Applications)

Geosynthetics are placed in pavement bases to perform one or more of the following functions:

- reinforcement
- separation
- filtration.

Base reinforcement results from the addition of a geogrid or composite at the bottom or within a base course to increase the structural or load-carrying capacity of a pavement system by the transfer of load to the geosynthetic material. The primary mechanism associated with this application is lateral restraint or confinement of aggregates in the base. Where very weak subgrades exist, geosynthetics can increase the bearing capacity by forcing the potential bearing capacity failure surface to develop along alternate, higher strength surfaces. Geogrids may also be considered for use in locations where chemical stabilization of the subgrade is not desirable due to possible reaction with sulfates in the subgrade, or not practical because of expedited construction concerns, particularly in urban settings. There have been assertions that the resultant increase in restraint or confinement should allow for design of thinner structures using these products versus structural designs which do not, however their benefits may only be noticeable over the long term and there appears to be an absence of long-term controlled monitoring. For purposes of geosynthetic reinforcement, the Construction Division, Materials & Pavements Section (CST-M&P) recommends that its application be viewed as an "insurance policy" rather than a "modulus- multiplier" or structure-reducing product.

Geosynthetics used for separation have classically been applied to prevent subgrade soil from migrating into the unbound base (or subbase), or to prevent aggregates from an unbound base (or subbase) from migrating into the subgrade. A small amount of fines introduced into the granular base can significantly reduce the internal friction angle and render the flex base weaker. Potential for these circumstances increases where wet, soft subgrades exist. Typically, a geocomposite will be used for this application, placed at the subgrade/unbound base interface. Geotextile separators act to maintain permeability of the base materials over the life of the section, and they allow the use of more open-graded, free-draining base and subbase materials. Another form of separation is being increasingly explored in Texas where there is a high potential for reflective cracking originating in the subgrade or chemically-bound base. A grid or composite is used to dissipate stresses induced by the opening crack. Longitudinal edge cracking is particularly an issue in areas where moderate to high plasticity index (PI) soils are exposed to prolonged cycles of wetting and drying. Geogrids will typically be employed at the subgrade/bound base interface, or if a flex base is placed

above a bound base (e.g., full-depth reclamation/recycling [FDR] projects), the grid may be placed at this location. Grids should be a minimum of 10-ft. wide to reduce the potential for longitudinal cracking due to edge drying.

The function of filtration is to allow for in-pavement moisture transfer, but restrict movement of soil particles; hence composites or fabrics that are placed for the classical purpose of separation will usually incorporate this function as well.

Section 4 — Flexible Base Overlay and Flexible Base Thickening

Flexible Base Overlay

The flexible base overlay has been used as an intermediate layer of a new structure, placed directly on top of an old structure. The primary purpose, in addition to adding additional structure, has been to resist the propagation of reflective cracks from the old structure. The existing structure still offers good/uniform support. A suitable surface designed for the expected traffic loading must also be planned. As such, this type of rehabilitation can increase the overall highway profile considerably. The material proposed for the new flexible base should be evaluated for moisture susceptibility and durability; trapping moisture in the flexible base overlay between the old highway surface and the new surface must be avoided.

Flexible Base Thickening

Flexible base thickening can be used to improve the structural capacity of low to moderate volume highways. This technique can be used where there are low levels of existing structural damage, the existing base is uniform and the subgrade offers good (>15 ksi) support. The new base material should be of equal or higher quality than the existing base; where this is not the case, it is essential that the new material be thoroughly blended with the existing base. When the materials are blended, cutting into the subgrade must be avoided. This subject was explored as a part of in research project 0-4182. Field performances were evaluated in TTI report 4182-1, "Field Performance and Design Recommendations for Full Depth Recycling in Texas."

Section 5 — Full Depth Reclamation/Recycling (FDR)

This rehabilitation procedure entails pulverizing the old pavement structure, blending in a stabilizing agent, compacting, adding additional material, and resurfacing. This procedure is meant to address deep structural problems ranging to depths as great as 12 in. in the existing structure. This is the practical limit of recycler cut-in and subsequent compaction of the recycled layer. Deeper problems must be addressed by temporary storage such as windrowing removed material in the right of way (ROW) or otherwise moving existing material off site. Efficiency is improved by using a specialized train to reprocess the existing structure. New base and surfacing is then applied to provide appropriate additional structure, ride quality, skid, weatherproofing, etc. It is imperative that representative samples be taken from the layers to be recycled, preferably in similar gradations as would be produced by the recycling process, in order to evaluate stabilizer type and content in the lab. This may require the use of a small milling machine or a larger auger drill working in one or more representative locations. Additional aggregates may be necessary to improve the gradation if the resulting blend is poor and/or if the depth of desired stabilization is greater than the depth of existing materials above the subgrade. Alternatively, additional passes of the pulverizer can be considered to achieve the desired gradation. Cutting into the existing subgrade should be avoided as this may introduce excessive fines into the blend that may be detrimental to the moisture susceptibility of the recycled layer.

"The Wirtgen™ Cold Recycling Manual" offers a sequence of steps for developing a mix design suitable for the recycled layer and is summarized here:

1. Select stabilizer based on cost, availability (sufficient quantity, quality), suitability with respect to type and quality of material to be recycled, and desired (design) properties of the recycled mix. Stabilizer suitability must address the plasticity and quantity of the fines in addition to the overall gradation. The environment also must be considered; preliminary research has shown that fly ash and emulsified asphalt do not work well in high moisture environments. These sensitivities to moisture may be mitigated when used in combination with other additives (lime-fly ash, or cement with emulsified asphalt). Also, where freezing conditions are expected, chemical stabilized materials should be in place well before (one month) the onset of freezing weather.
2. Prepare samples (test methods Tex-120, Tex-121, Tex-126, Tex-127) using varying percentages of stabilizer, bring samples to optimum fluid content (water plus any additional liquid such as emulsified AC) for compaction.
3. Compact sample using standardized compaction procedures.
4. Cure samples.
5. Subject samples to unconfined compressive strength (UCS) and moisture susceptibility testing. Optimum stabilizer content for chemical stabilizers should target the sample UCS at 300 psi. Moisture susceptibility evaluation should limit dielectric to <10 after a 10-day capillary rise and an 85% retained UCS.

Additional guidelines can be found in TTI report 4182-1, Field Performance and Design Recommendations for Full Depth Recycling in Texas.

Section 6 — HMA Overlays

Structural Overlays

For flexible pavements, structural hot mix overlay thicknesses are designed using FPS-19 design, option 5. Currently, the only department-approved rational method for design of structural HMAC overlays on rigid pavements is by using the appropriate overlay option in DARWin™ (AASHTO 93). In considering the actual overlay thickness, guidelines established for lift thickness based on the type of mix / nominal maximum aggregate size must be followed. In addition, the type of mix selected should complement the overall structure in terms of resilience, durability, permeability, texture, etc.

A reasonable investigation of the condition of the existing substructure and hot mix on the project must be made to insure the desired performance of the structural overlay. In addition to deflection measurements, ground penetrating radar (GPR), when combined with selective coring, is a rapid method of determining the depth and extent of delamination or stripping problems. Where rutting-susceptible mixes exist in the old structure, if these mixes are within 5 in.of the newly overlaid surface, the chance of renewed rutting originating in the old mix will still exist (zone of high shear and compression). Evaluation of road cores for rutting and stripping susceptibility using Tex-242-E (Hamburg) should be used if in doubt. Even if there is no evidence of current stripping, it is advisable to evaluate the existing material for stripping susceptibility; experience has shown that stripping problems often start only after the new overlay is placed. There are also certain surface materials that should not be overlaid, including plant-mix seal or permeable friction courses (open-graded friction courses). Where poor substructure is located, full-depth repair should be accomplished prior to the overlay. In the case of Portland cement concrete (PCC) pavements, a determination must be made into the uniformity of the underlying support. The rolling dynamic deflectometer (RDD) has been a useful tool in identifying areas of low support. Slabs must be prevented from moving by stabilizing the material beneath them. This involves drilling holes in an unstable PCC slab or section and injecting an asphaltic or cementitious material to fill any underlying voids. Typically, this method is only an option for isolated instances of instability. It does not work well as a general roadway treatment. Application of a stress absorbing membrane interlayer (SAMI) may be useful in retarding reflective cracking when overlaying jointed concrete pavements. SAMI is a fine mix that is designed for cracking resistance using the overlay tester or flexural beam fatigue. The mix is typically placed 1 in. thick and should be covered with an adequate overlay thickness to provide adequate resistance to rutting (3.0 in. minimum is recommended).

Other reasons for removing a portion of the existing HMA surface include leveling because of rutting, reducing crack width caused by spalling, and eliminating raveling.

As a minimum, a higher rate of tack coat application will be needed on a milled surface prior to overlaying. For planning purposes, a seal coat may be applied to the surface of the milled structure,

especially if there are visible or latent cracks. The designer should also consider other measures to thwart reflective cracking through the new overlay. Geotextiles have been used successfully for this purpose, but require increased vigilance on the part of the contractor to insure manufacturer's guidelines are strictly followed. Mix design, selecting a mix that incorporates increased resilience, low permeability, and overall mat thickness are also important considerations.

Non-structural Overlays

These overlays are designed using a combination of experience and guidelines established herein. Generally, this type of overlay is used to improve ride, texture, cross-slope drainage, and weather-proofing, and is often categorized as "preventive maintenance." Often, the process is combined with milling of a like amount of existing surface, essentially keeping the same profile while improving functional characteristics (mill and fill).

Condition surveys (including deflection testing and GPR) should be undertaken to insure that a non-structural overlay is appropriate. This would also include evaluating the existing HMAC or PCC slabs for stability (and HMA stripping potential) if necessary. As with overlays designed for structural purposes, corrective action to the substructure should be accomplished prior to the overlay. The condition survey should also reveal whether minor milling, leveling, and undersealing (or grout injection beneath PCC slabs) are necessary.

The type of mix selected should complement the overall structure in terms of durability, permeability, and texture. The designer should consider appropriate lift thickness based on the desired mix type and nominal maximum aggregate size. Where permeable friction courses (PFC) are applied, it is imperative that the underlying structure is water-tight, hence these overlays are almost always applied with an underseal. Where active cracks exist (especially jointed PCC structures), thin overlays will rarely perform well without the use of geotextiles and careful consideration of the resilient properties of the mix. Some success has been realized by saw-cutting and sealing thin overlays over active joints on jointed PCC pavements.

Two **thin** overlay options are available by special specification. The ultrathin bonded wearing course (SS 3001) and thin bonded PFC (SS 3000) both use a warm polymer-modified asphalt emulsion membrane followed immediately by the application of a hot plant mixed paving mixture. The ultrathin bonded wearing course is placed in thicknesses from 1/2 - 3/4 in. The thin bonded PFC is placed in thicknesses of 3/4 to 2 in., so some minor improvement in ride is possible. The PFC mixture will allow for the rapid removal of surface water, improving splash/spray characteristics; the open void structure will also reduce tire noise. These treatments should be considered on higher volume highways where average speeds exceed 45 mph, and where chip pickup and road noise from the alternate surface treatment are more objectionable to the traveling public. The application of the warm polymer-modified asphalt emulsion membrane is designed to seal the existing surfaces where minor cracking (< 1/4 in.) is the most severe distress, and is seen as a potential remedy for pavements that have leaky joints and/or segregation problems. Existing rutting or more severe cracking must be addressed separately before using these options.

There are several construction-related concerns in placing these non-structural HMA overlays, including:

- Thin lifts require less HMA per foot of road length than thick lifts. This can result in high paver speeds (in excess of 70 ft./min.). Compaction may not be able to keep pace with these high speeds.
- Thin lifts will cool quicker than thick lifts. This can result in little time available for compaction before the thin overlay reaches cessation temperature (sometimes as little as 3 to 5 min.). Therefore, laydown and roller variables should be set to account for this (e.g., slower laydown machine speed, enough rollers and an adequate roller pattern to compact the material before it reaches cessation temperature).
- Thin lift construction produces greater screed wear. If the lift depth is less than about twice the maximum aggregate size, the HMA may tear under the paver screed. Very thin lifts (less than 25 mm [1 in.]) can be damaged by the screed dragging large articles.
- Thin lifts are more sensitive to vibratory rolling. Incorrectly chosen amplitude, frequency or roller speed can result in aggregate degradation (i.e. breaking) and damage of the bond between the overlay and the existing pavement.
- Density control is difficult. Thin lifts provide fewer options for aggregate particles to rearrange under compaction. Thus, mat densities will tend to be less uniform than those associated with a thicker lift. This should be recognized if pay is in any way tied to mat density.

In general, compaction is more difficult and more variable on thin lifts.

Section 7 — Surface Treatments

For purposes of discussion in this guide these procedures will include treatments under items 315 (fog seal), 316 (surface treatments), 318 (hot asphalt-rubber surface treatments), and 350 (micro-surfacing). Surface treatments are applied to restore texture and weatherproofing (including protection from oxidation), but do not contribute to improvement in ride or increased structural capacity. They have become an essential part of the preventive maintenance program in Texas, particularly for lower volume highways. A fog seal is a light application of a diluted slow-setting asphalt emulsion to the surface of an aged (oxidized) pavement surface. Microsurfacing is a blend of emulsified asphalt, water, well-graded fine aggregate (top size < 1/2 in.) and mineral filler. Condition surveys as described in Chapter 4, "Pavement Evaluation," should be conducted to assess whether localized repair should be conducted prior to application of the surface treatment.

Surface treatments perform best when the underlying structure is still in good shape (minimal cracking, raveling, bleeding). In addition to a lane or road-width treatment, microsurfacing has been used to fill minor rutting. Relative performance of several traditional surface treatments (asphalt rubber chip seal, polymer modified emulsion chip seal, latex modified AC chip seal, unmodified chip seal, and microsurfacing) were evaluated in the TTI study 4040, *Supplemental Maintenance Effectiveness Research Program (SMERP)* (TTI-4040-3). In general, chip seals are more effective in preventing reappearance of cracking than microsurfacing, with the asphalt rubber chip seal performing best overall. Microsurfacing proved to be more effective than chip seals in preventing the reappearance of bleeding. In fact, if applied incorrectly, (AC application rates too high, especially in the wheelpaths) chip seals were shown to actually increase the amount of bleeding over time. While a fog seal was also applied to test sections in this study, its effectiveness was limited essentially to anti-oxidation and anti-raveling roles. Fog seals are suitable for low-volume roads which can be closed to traffic for the 4 to 6 hrs. it takes for the slow-setting asphalt emulsion to break and set. An excessive application rate may result in a thin asphalt layer on top of the original hot mix asphalt (HMA) pavement. This layer can be very smooth and cause a loss of skid resistance. Sand should be kept in reserve to blot up areas of excess application.

As with standard HMA overlays, the existing bituminous layers should be evaluated in the lab for stripping potential prior to applying the surface treatment. A surface treatment will generally seal off the vertical escape of moisture migrating upward out of a pavement which can set up accelerated stripping in the existing HMA layer beneath the seal.

For more information on surface treatments and seal coats, refer to the *Seal Coat and Surface Treatment Manual*.

Underseals

Underseals may be applied to the surface of an existing pavement prior to the placement of a new surface lift of HMA for two primary reasons. The first is to seal the old HMA layer, which has

small to moderate amounts of cracking or is otherwise permeable because of poor compaction. The other reason is to improve the adhesion or bond of the new surface lift of HMA to the existing structure. Items 316 and 318 are the governing specifications. Cautions cited above regarding the stripping susceptibility of the existing HMA layers apply with application of underseals as well.

Section 8 — Concrete Overlays

Applying a concrete overlay on a HMA surfaced pavement may be a viable rehabilitation strategy under certain circumstances. Where existing distress in an HMA-surfaced structure is confined to the HMA itself (mix rutting, shoving, washboarding, cracking), but otherwise the existing substructure is sound, a concrete overlay can offer a durable replacement surface. These circumstances may present themselves at intersections or along open sections of highway. The process of applying a concrete overlay on a HMA-surfaced pavement is sometimes referred to as "whitetopping," a term used by the American Concrete Pavement Association (ACPA). The ACPA further divides whitetopping into three sub-categories:

- conventional whitetopping
- concrete inlay
- ultrathin whitetopping.

Conventional whitetopping will use a slab at least 4.0 in. thick placed on top of the existing HMA surfaced pavement.

For 4.0 in. to 7.0 in. thick slabs, refer to Chapter 8, Section 8, "Thin Concrete Pavement Overlay (Thin Whitetopping)."

For 8.0 in. and thicker slabs, use the same design procedures as new pavement and refer to Chapter 8, "Rigid Pavement Design."

Concrete inlays may be placed on thicker HMA pavements that have been partially milled out.

Ultrathin whitetopping will use slabs from 2.0 to 4.0 in. thick that are placed on an HMA surface that has been milled or otherwise prepared to enhance the bond. Concrete overlays thinner than 4.0 in. are not currently allowed under department guidelines. This may restrict the use of concrete overlays at certain curb and gutter intersections where vertical profile may not allow direct placement on top of the existing HMA structure, and milling to the appropriate depth may leave insufficient support.

Design of whitetopping slabs that are thinner than what would be derived using conventional rigid pavement design procedures (AASHTO 93, using department input guidelines) must be approved by the Director of CST-M&P. Typically, the thinner the slab, the more critical the uniformity and strength of the underlying structure will be. Thinner slabs may contain fiber reinforcement in place of conventional steel rebar. For jointed PCC overlays that are less than 8.0 in. thick, reduced joint spacing over conventional design is recommended. The ACPA recommends a joint spacing of 12 to 15 times the slab thickness to reduce curling stresses and bending stresses due to loads (adjusted to coincide with lane width/striping). Green cut sawing should be used as soon as possible before the internal stresses of cement hydration begin.

Section 9 — Reducing Rigid Pavements

Acknowledgement: Content of this section is made available from the University of Washington, Pavement Tools Consortium.

Various procedures can be used to reduce poor performing or functionally inadequate rigid (Portland cement concrete [PCC]) pavements into a subbase or base for a bituminous surfaced structure. The object is to reduce the size of "free-moving" PCC slabs to minimize differential movement at existing cracks, joints and punchouts. This will in turn reduce the occurrence and severity of reflective cracking through the newly applied bituminous surface. Three processes are generally recognized for reducing rigid pavements: break and seat, crack and seat, and rubblizing. These procedures may not be well suited for locations where a thin, old slab rests directly on saturated subgrade (non free-draining) since the slab will often tend to rock under the impact and may not reduce down to desirable sized particles. If free moisture exists in the structure, placing an edge drain system prior to reducing the PCC slabs may be effective in improving the stability of the slab enough to make these reduction methods possible.

Break and Seat

This process may be used to reduce jointed reinforced concrete pavement (JRCP) to pieces on the order of about 1-ft.^{2} to 2-ft.^{2} by repeatedly dropping a large weight. The objective is to retain some slab action but reinforcing steel **must** be ruptured or the bond broken. Typically, a drop hammer or "guillotine" type device is used. Broken PCC is then seated into the foundation using several passes of a 35 to 50 ton pneumatic roller.

Figure 7-2. Drop hammer breaking slab for break and seat operations.
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Figure 7-3. Drop hammer "breaking" slab for "break and seat."
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Crack and Seat

This process may be used to reduce jointed plain concrete pavement (JPCP) to pieces 1 to 3 ft^2 in size. Equipment similar to that used for break and seat is used to reduce the slab. Some slab action is retained. Broken PCC is then seated into the foundation using several passes of a 35 to 50 ton pneumatic roller.

Rubblizing

This process will completely fracture any type of PCC slab into pieces smaller than 1 ft^{2} and is very effective in breaking the bond of any reinforcing steel to the slab fragments. The slab is reduced to a high strength granular base that is not susceptible to causing reflective cracking in the HMA overlay. The fractured layer is typically compacted using a 10 ton vibratory roller. This is the preferred process for reducing JRCP and CRCP pavements. Rubblizing is typically done with one of the following two pieces of equipment:

Resonant Pavement Breaker

This equipment strikes the rigid pavement at low amplitude with a small plate at the resonant frequency of the slab (usually about 44 Hz) causing the slab to break apart. Usually it takes about 14 to 18 passes for a resonant pavement breaker to rubblize an entire 3.6 m (12 ft.) lane (NCAT, 2001). Daily production rate is estimated to be about 6,000 yd^{2}/day.

Figure 7-4. Resonant Pavement Breaker.
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Figure 7-5. Rubblized pavement texture following resonant pavement breaking.
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Multi-head Breaker (MHB)

This equipment uses a series of independently controlled high amplitude drop hammers to smash the slab. Typically, there are between 12 and 16 hammers, each weighing between 1,000 - 1,500 lbs. Hammers can be dropped from variable heights 1 - 5 ft. to create impact energies between 2,000 - 12,000 ft.-lbs. Hammers cycle at a rate of 30 - 35 impacts per minute. MHBs can rubblize an entire lane up to 13 ft. in a single pass (Antigo Construction, 2001.)

Figure 7-6. Multi-head Pavement Rubblizer.
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Section 10 — Alternate Pavement Rehabilitation Options

Alternate pavement rehabilitation options are used to allow more competition on pavement rehabilitation projects when using specialty processes.

This section will provide information on alternate pavement rehabilitation options to:

- hot in-place recycling (HIR)
- ultrathin bonded hot mix wearing course (UBHMWC)
- thin bonded permeable friction course (TBPFC)
- reflective crack relief interlayer (RCRI)

Alternate Options to Hot In-Place Recycling

The purpose of hot in-place recycling (HIR) is to correct asphalt pavement surface distress such as surface cracking, roughness, or lack of skid resistance through recycling the existing pavement. HIR can be effective in correcting surface defects, as long as the defects are not caused by structural inadequacy of the pavement.

The Cutler HIR process involves recycling the existing asphalt surface layer by heating, scarifying and adding a recycling agent. The recycled material is placed on the roadway surface, a thin overlay of new hot mix is placed over the recycled layer and the entire pavement layer is compacted using conventional compaction equipment.

Alternate options are mill and overlay or surface recycling process also referred to as the Dustrol process. Mill and overlay involves milling one inch of the existing top layer and replacing it with two inches of new overlay. A high percentage of RAP should be allowed in the new two-inch overlay.

The Dustrol process is an alternate to the Cutler process. The Dustrol process is a surface recycling process. The Dustrol process involves heating and scarifying the top layer of asphaltic pavement (typically the top one inch), mixing a recycling agent. A new overlay is placed on top of the new recycled surface using standard paving operations.

Alternate Options to Ultrathin Bonded Hot Mix Wearing Course (UBHMWC)

The ultrathin bonded hot mix wearing course (UBHMWC) is used as a preventative maintenance treatment to restore skid resistance and improve ride quality.

The UBHMWC is an open-graded mix placed on polymer modified rapid setting emulsion tack coat. The mix is placed before the emulsion breaks using a specialty paver. Note: The paver must meet the requirements of the Special Specifications 3142.

The UBHMWC may also be used when there are restrictions on overlay thickness due to clearance requirements.

The alternate option to this process is placing an underseal (seal coat) plus a 3/4-inch thick PFC with two separate pieces of equipment.

Alternate Options to Thin Bonded PFC (TBPFC)

Placement of a thin bonded PFC (TBPFC) surface is intended to provide increased safety and better ride quality on a high speed roadway.

The conventional use of TBPFC involves a surface course placed on a sealed pavement surface. TBPFC is the PFC mix placed on a polymer-modified rapid setting emulsion tack coat before the emulsion breaks using a specialty paver.

The alternate option to this process is placing an underseal (seal coat) plus a PFC, with two separate pieces of equipment.

Alternate Options to Reflection Crack Relief Interlayer (RCRI)

The purpose of the reflection crack relief interlayer (RCRI) is to retard reflection cracks on asphalt concrete overlays located on top of jointed concrete pavements.

The RCRI is a fine mix that is designed for cracking resistance using the flexural beam fatigue. The mix is typically placed one-inch thick and should be covered with an adequate overlay thickness to provide adequate resistance to rutting.

An alternate option is the crack attenuating mix (CAM). The cracking resistance of the CAM will be evaluated using the overlay tester.

Chapter 8 — Rigid Pavement Design

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Section 1 — Overview

Rigid Pavement Types

Different pavement types use different types of joints and reinforcement to control the forces acting on the concrete pavement. These forces include curing shrinkage of the concrete, environment changes and traffic loads. Forces in the concrete pavement will cause cracks to form without traffic.

The designer can select the location where the joints will be placed and, consequently, where the cracks will form. Joints may be thought of as "controlled cracks" that will reduce the stresses the concrete will experience during its life and greatly increase the life of the concrete pavement. Through the use of reinforcement, the location and spacing of cracks can also be controlled.

Two types of concrete pavements commonly used in Texas are contnuously reinforced concreted pavement (CRCP) and jointed concrete pavement (JCP).

Continuously Reinforced Concrete Pavement (CRCP)

CRCP contains both longitudinal and transverse steel. CRCP does not contain transverse joints except at construction joints.

The function of the longitudinal steel is not to strengthen the concrete slab, but to control concrete volume changes due to temperature and moisture variations and to keep transverse cracks tightly closed. The function of the transverse steel is to keep longitudinal joints and cracks closed. If the steel serves its proper function and keeps cracks from widening, aggregate interlock is preserved and concrete stresses in the concrete slab due to traffic loading are reduced.

The CRCP thickness design is detailed in Section 2, "Approved Design Method," and Section 3, "Rigid Pavement Design Process." Steel reinforcement and other design details are governed by the CRCP standards that can be obtained from TxDOT's internet web site at the following address: http://www.dot.state.tx.us/insdtdot/ orgchart/cmd/cserve/standard/rdwylse.htm. Some districts have district-wide CRCP standards.

Figure 8-1. Continuously Reinforced Concrete Pavement

Jointed Concrete Pavement (JCP)

JCP has transverse joints spaced at regular intervals. The transverse joints are used to control temperature induced contraction and expansion in the concrete. Smooth dowel bars are used at the transverse joints for load transfer. The transverse joints are spaced at 15 ft.

Longitudinal joints are used to control random longitudinal cracking. Longitudinal joints are tied together with tie bars.

The thickness design of JCP is same as CRCP and is detailed in Section 2, "Approved Design Method," and Section 3, "Rigid Pavement Design Process." Other JCP details are governed by the concrete pavement contraction design (CPCD) standards that can be obtained from TxDOT's internet web site at the following address: http://www.dot.state.tx.us/insdtdot/ orgchart/cmd/cserve/ standard/rdwylse.htm.

Selection of Rigid Pavement Type

CRCP provides excellent long-term performance requiring very low maintenance. The Department policy is to utilize CRCP for new or reconstructed rigid pavements in Texas. There are situations where jointed pavement may be desirable. If jointed pavement is to be utilized, its use must meet one of the following criteria:

- for roadways controlled and maintained by another government entity
- for aesthetic reasons where pedestrian traffic might interpret the randomly spaced cracks of CRCP with premature failure
- for parking areas or roadways with crosswalks, adjacent parking, or sidewalks
- for railroad crossings, approaches to structures or to widen existing jointed pavement

- for intersections and approaches in flexible pavement roadways that are associated with vehicle braking and acceleration which could cause shoving and rutting of an asphalt pavement
- for other situations approved by the Administration, after submittal of request and justification to the Materials & Pavements Section of the Construction Division (CST-M&P).

Rigid pavement types other than CRCP and JCP may be appropriate for a wide variety of situations. Refer to Chapter 7, "Concrete Overlays" and Chapter 10, "Bonded Concrete Overlay" and "Unbonded Concrete Overlay" for a description of other rigid pavement applications.

Performance Period

For rigid pavements, the initial pavement structure shall be designed and analyzed for a performance period of 30 yr. A performance period other than 30 yr. may be utilized with justifications. For example, an existing pavement to be widened will be completely reconstructed within 15 yr. In this case, a selection of a 15-yr. performance period is more reasonable and justifiable for the widening.

Section 2 — Approved Design Method

AASHTO Rigid Pavement Design Procedure

The 1993 AASHTO *Guide for Design of Pavement Structures* is the only approved design method for rigid pavements for TxDOT. This design procedure is available in automated form. Automated procedures include the AASHTO DARWin® program and the TxDOT's TSLAB86 program.

These programs are available to TxDOT personnel through the district pavement engineer. Consultants may obtain the TSLAB86 program from the district pavement engineer or the Materials & Pavements Section of the Construction Division (CST-M&P).

The AASHTO Guide also contains design procedures for rehabilitation of rigid pavements, including asphalt concrete overlays or Portland cement concrete (PCC) overlays of existing rigid pavements. Contact the district pavement engineer or CST-M&P for further assistance.

Section 3 — Rigid Pavement Design Process

The following figure summarizes the rigid pavement design process.

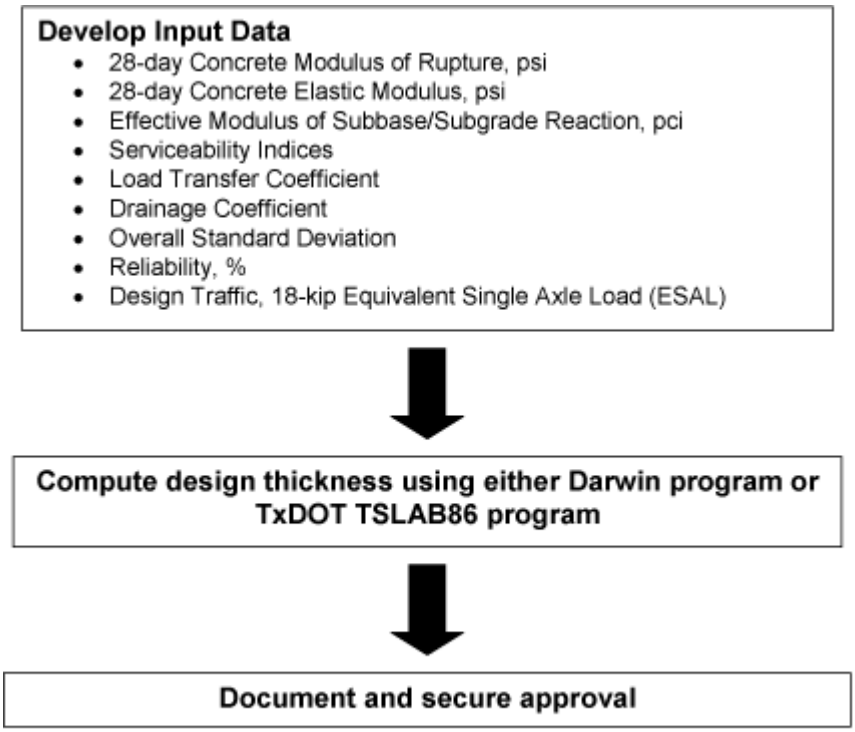

Figure 8-2. Rigid Pavement Design Process.

Section 4 — Recommended Input Design Values

Input Values

The following input variables are needed for the AASHTO rigid pavement design procedure:

- 28-day Concrete Modulus of Rupture, psi
- 28-day Concrete Elastic Modulus, psi
- Effective Modulus of Subgrade Reaction, pci
- Serviceability Indices
- Load Transfer Coefficient
- Drainage Coefficient
- Overall Standard Deviation
- Reliability, %
- Design Traffic, 18-kip Equivalent Single Axle Load (ESAL).

28-day Concrete Modulus of Rupture, M_r

The M_r of concrete is a measure of the flexural strength of the concrete as determined by breaking concrete beam test specimens. A M_r of 620 psi at 28 days should be used with the current statewide specification for concrete pavement design. If the engineer selects an alternate value for M_r, it must be documented with an explanation.

28-day Concrete Elastic Modulus

Elastic modulus of concrete is an indication of concrete stiffness. It varies depending on the coarse aggregate type used in the concrete. Although the value selected for pavement design could be different from the actual values, the elastic modulus does not have a significant effect on the computed slab thickness. A modulus of 5,000,000 psi should be used for pavement design. The use of a different value must be documented with an explanation.

Effective Modulus of Subgrade Reaction: k-value

The AASHTO guide allows pavement designers to consider the structural benefits of all layers under the concrete slab. The guide also allows designers to consider the effect of loss of support of the underlying material due to erosion or deterioration. The slab support is characterized by the modulus of subgrade reaction, otherwise known as k-value.

Field performance evaluations of concrete pavement have revealed that the soundness of the base is essential to the long-term performance of concrete pavement. If the base underneath the concrete slab does not provide good support, long-term pavement performance will be severely compromised regardless of the concrete slab thickness.

TxDOT-Required Base Layer Combinations

TxDOT recognized this and requires one of the following base layer combinations for concrete slab support:

- 4 in. of asphaltic concrete pavement (ACP) or asphalt stabilized base (ASB) or
- a minimum 1 in. asphalt concrete bond breaker over 6 in. of a cement stabilized base.

A k-value of 300 psi/in. should be used in the rigid pavement design procedure with one of the stabilized base layer combinations listed above.

The designer may use a higher k-value, up to 800 psi/in., with the requirement of field verification tests during the construction. Conduct verification tests in accordance with "Method 1: On-site Static Load Test" found in Tex 125-E, "Determining Modulus of Subgrade Reaction (k-value)."

Use a 30-in. diameter steel plate and test on top of the base for every 0.1 mile of roadbed, for each 20 ft. of plan dimensional steel plate width or a portion of that. The k-values obtained from verification tests may be adjusted by the loss of support factor as directed by the Engineer. These k-values should meet the design k-value at all times during construction.

TxDOT requires these stabilized bases because they do not erode over time under truck traffic loading. TxDOT's general philosophy is to prevent water intrusion and pumping of underlying materials by using bases that are dense graded, non-erosive, and stabilized. Bases that are properly designed and constructed using TxDOT specifications and test methods should provide adequate long-term support.

Where long-term moisture susceptibility of ACP is a concern, using a plan note to increase the target laboratory density (and thus total asphalt content) may be beneficial. To ensure long-term strength and stability of cement stabilized layers, sufficient cement must be used in the mixture.

Selecting Appropriate Strength

Item 276, Cement Treatment (Plant-Mixed) currently designates three classes of cement-treated flexible base, based on 7-day unconfined compressive strength. Classes L and M are intended for

use with flexible pavements. Class N, which has a minimum strength as shown on the plans, is intended for use with rigid pavements.

There are several approaches to selecting an appropriate strength (and, indirectly, cement content as a result):

- successful long-term experience with similar materials
- laboratory testing using 100% of the retained strength of a conditioned specimen to determine if the design cement content and strength are acceptable
- laboratory testing using the tube suction test to determine if the design cement content and strength are acceptable.

Use of Bond Breaker

A bond breaker should always be used between concrete pavement and cement stabilized base. There have been several instances across Texas where excessive cracking and premature failures occurred when a concrete slab was placed directly on cement stabilized base. These problems occur because concrete slabs tend to bond directly to cement stabilized bases. This increases the chances for cracks in the base to reflect through the overlying slab. This also increases tensile stresses in the concrete slab due to temperature and moisture changes, resulting in higher chances of additional cracking.

TxDOT recommends a minimum of 1 in. asphalt concrete stress-relieving layer be used between cement stabilized base and the concrete slab. A polyethylene sheet is not recommended for use as a bond breaker, due to construction problems evident from experience.

The subgrade is usually stabilized or treated with lime or cement to facilitate construction as well as to provide additional support to the pavement structure. Large volume changes in the subgrade resulting from moisture variations or other causes can cause the deterioration of concrete pavement. These volume changes in the subgrade should be minimized by appropriate means. Contact CST-M&P for further assistance, if needed.

Subgrade/Base Widths. The subgrade/base should be designed 2 ft. wider than the concrete slab on each side to accommodate slipform pavement equipment.

If the engineer elects to use a "drainable base," then coordination with CST-M&P personnel is required.

Serviceability Indices

For concrete pavement design, the difference between the initial and terminal serviceability is an important factor. An initial serviceability value of 4.5 and a terminal serviceability value of 2.5 are

to be used in the procedure, which results in a difference of 2.0. Different values, if used, must be documented and justified.

Load Transfer Coefficient

The load transfer coefficient is used to incorporate the effect of dowels, reinforcing steel, tied shoulders, and tied curb and gutter on reducing the stress in the concrete slab due to traffic loading. The coefficients recommended in the AASHTO Guide were based on findings from the AASHO Road Test.

Table 8.1 lists the load transfer coefficient values to be used for rigid pavement design:

Table 8-1: Load Transfer Coefficients

CRCP or Load Transfer Devices at Transverse Joints	Tied PCC Shoulders, Curb and Gutter, or Greater Than Two Lanes in One Direction	
	Yes	**No**
Yes	2.6 for CRCP 2.9 for JCP	3.2
No	3.7	4.2

There is substantial evidence that tied Portland cement concrete (PCC) shoulders improve PCC pavement performance significantly. Therefore, it is strongly recommended that tied PCC shoulders be provided, if possible. In case it is not feasible to provide tied PCC shoulders, the use of a minimum 2-ft. wider outside lane should be considered.

PCC shoulders should be tied to the main lane pavement by tie bars or by the main lane's transverse steel. The joint between the concrete shoulder and the concrete main lane pavement should be a warping or hinge joint, not a construction or expansion joint. The PCC shoulder should have the same thickness and the same base layers as the main lane pavement. This will allow traffic to be routed on the shoulder during future maintenance and construction while reducing the chances of structurally damaging the shoulder. It will also facilitate the construction sequence in most cases.

Tied or monolithic curb and gutter help in reducing edge stresses and serve as a barrier that discourages traffic from riding too close to the edge of the pavement structure. Although tied curb and gutter sections usually contain tie bars, the tie bars are too small in either size or number to transfer load stresses effectively by themselves. Construction joints usually exist between tied curb and gutter and the concrete pavement. This means no aggregate interlock exists. Since the number of edge stresses in the pavement are much less for monolithic curb sections than for tied curb and gutter sections, the use of monolithic curb is recommended when practical.

Drainage Coefficient

The drainage coefficient characterizes the quality of drainage of the subbase layers under the concrete slab. Good draining pavement structures do not give water the chance to saturate the base and subgrade; thus, pumping is not as likely to occur.

The AASHTO Guide provides a table of drainage coefficients based on the anticipated exposure of the pavement structure to moisture and on the quality of drainage of the base layers. Higher drainage coefficients represent better drainage. The most credit is given to permeable bases with edge drains.

TxDOT has not had much experience with positive drainage systems. As stated earlier, TxDOT's philosophy on this issue is to prevent water intrusion and pumping by using bases that are dense graded, non-erosive, and stabilized. TxDOT has had good performance with such bases and it is believed that the bases recommended earlier in this section provide performance equivalent to a fair level of drainage.

Currently, drainage coefficients in Texas for non-erosive stabilized bases are based on the anticipated exposure of the pavement structure to water. "Table 8-2: Drainage Coefficients" shows the recommended drainage coefficients for Texas. The coefficients are selected based on the annual rainfall in the project area.

Table 8-2: Drainage Coefficients

Annual Rainfall (in.)	Drainage Coefficients
58 – 50	0.91 - 0.95
48 – 40	0.96 - 1.00
38 – 30	1.01 - 1.05
28 – 20	1.06 - 1.10
18 – 8	1.11 - 1.16
NOTE: Higher drainage coefficients decrease the pavement thickness in the AASHTO procedure.	

Overall Standard Deviation

Overall standard deviation accounts for both chance variation in the traffic prediction and normal variation in pavement performance prediction for a given traffic. The AASHTO Guide recommends values in the range of 0.30 to 0.40, with 0.35 being the overall standard deviation from the AASHO Road Test. Higher values represent more variability; thus, the pavement thickness increases with higher overall standard deviations. A value on the high end of the range is considered reasonable for Texas since it is believed that the inputs recommended for Texas are less accurate than the inputs determined at the AASHO Road Test. A value of 0.39 is to be used for rigid pavement design.

Reliability, %

The reliability value represents a "safety factor" with higher reliabilities representing pavement structures with less chance of failure. The AASHTO Guide recommends values ranging from 50% to 99.9%, depending on the functional classification and the location (urban vs. rural) of the roadway. Based on TxDOT's experience, a reliability of 95% should be used for rigid pavement with more than 5 million design ESALs; a reliability of 90% should be used for rigid pavement with 5 million or less design ESALs. If the engineer decides to use a different value, then it must be documented and justified.

Design Traffic 18-kip ESAL

The AASHTO Guide requires a prediction of the number of 18-kip ESALs that the pavement will experience over its design life.

The traffic projections for a highway project (in terms of ADT and one-way total 18-kip ESALs) are obtained from the traffic analysis report provided by the Transportation Planning and Programming (TPP) Division. This report is requested during the design phase of a project and, upon receipt, should be evaluated for reasonableness.

The predicted 18-kip ESALs is multiplied by a lane distribution factor (LDF). This factor represents the percentage of the total one-way 18-kip ESALs that could be expected in the design lane. The design lane is the lane that will carry the most traffic. Usually, it is assumed that the outer lane of a highway with two lanes in each direction carries the most traffic. For a three-lane facility, the middle lane is assumed to carry the most traffic. Traffic distribution in urban areas is somewhat more complex due to merging and exiting operations, but the same assumptions could apply.

Table 8-3: Lane Distribution Factor

Total Number of Lanes in Both Directions	LDF
≤ 4	1.0
6	0.7
≥ 8*	0.6
*Unless field observations show otherwise	

The lane distribution factor (LDF) decreases with an increase in the number of lanes of a facility. Highways with two lanes in each direction would have a higher LDF than highways with three or more lanes in each direction. This is because traffic tends to spread out over the available lanes. If there is a legal constraint requiring the trucks to use a particular lane, or avoid using certain lanes, then a review should be made of the appropriate lane distribution factor.

The traffic analysis report also lists a directional distribution of traffic. This value indicates the percent distribution of the design hourly volume in each direction of a highway facility. This value is used for capacity analysis and applies to all vehicles in the design hourly volume.

TPP assumes the directional distribution of heavy vehicles on any project is evenly split in both directions, unless specifically stated otherwise. Typically, the directional distribution shown on the traffic projections from TPP applies only to the 30th highest hourly volume. Therefore, **the directional distribution factor listed in the TPP report should not be used to modify the design 18-kip ESALs**.

Local conditions may cause the directional distribution of heavy vehicles to be unequal. For example, a location near a major quarry adjacent to a highway with otherwise modest levels of truck traffic. If the designer is aware of local conditions that may result in unequal distributions of heavy trucks, TPP should be informed of this condition when requesting traffic projections and the reported 18-kip ESALS for pavement design should be adjusted. The reasons and methods used to make the adjustment should be documented.

Section 5 — Determining Concrete Pavement Thickness

The computed concrete slab thickness usually contains fractions of inches, round the thickness up to the nearest full or half inch. For example, use slab thickness of 11.5 in. for computed thickness of 11.4 in. and use slab thickness of 10 in. for computed thickness 10.24 in.

- Use 6 in. for slab thickness if the computed thickness is less than 6 in.
- Use 13 in. for slab thickness if the computed thickness is greater than 13 in.

The minimum concrete pavement thickness is reduced from 8 in. to 6 in. Allowing the design to account for an increased effective modulus of subgrade reaction by utilizing thickened base/subbase layers will help ensure the structural adequacy for the pavement system.

The maximum concrete pavement thickness is reduced from 15 in. to 13 in. Districts should submit designs thicker than 13 in. to the District Engineer for approval along with their justification for doing so.

Section 6 — Concrete Pavement Design Standards

The following TxDOT pavement-related standards should be included in the pavement design report where applicable.

- CRCP (1)-09, Continuously Reinforced Concrete Pavement, One-Layer Steel Bar Placement - applies to CRCP that is from 6 to 13 in. thick.
- CRCP (2)-09, Continuously Reinforced Concrete Pavement, Two-Layer Steel Bar Placement – applies to CRCP that is from 14 to 15 in. thick.
- FDR(CP)-05, Full-Depth Repair of Concrete Pavement – standard for repairing existing concrete pavement.
- CPCD-94, Concrete Pavement Details, Contraction Design (CPCD) – standard for plain jointed concrete pavement and covers pavement thickness from 8 to 15 in.
- TA (CP)-99, Terminal Anchorage for Concrete Pavement - shows a terminal anchorage system usually constructed near bridge ends to restrain the movement at the joint between the bridge approach slab and the end of the pavement. Restraining the slab movement will hopefully reduce the chance of damaging adjacent structures. Such anchors are mainly used for CRCP, but they can also be used for CPCD as well.
- JS-94, Concrete Paving Details, Joint Seals - specifies joint sealing requirements for concrete pavement.
- TWT-04, Thin Whitetopping Details – standard for 4 to 7 in. thick bonded concrete overlay of an asphaltic surface.
- BAS-A, Bridge Approach Slab, Asphaltic Concrete Pavement.
- BAS-C, Bridge Approach Slab, Concrete Pavement.

These standards are available from TxDOT's internet web site at http://www.dot.state.tx.us/insdt-dot/orgchart/ cmd/cserve/standard/rdwylse.htm. For more detailed information about these standards, contact the district pavement engineer or the Materials & Pavements Section of the Construction Division (CST-M&P).

Section 7 — Bonded and Unbonded Concrete Overlays

Bonded concrete overlay (BCO) consists of a 2- to 8-inch thick concrete layer placed on top of the existing concrete pavement with operations conducted to ensure full bond between new and old concrete layers. BCO is one of the most cost effective ways of enhancing structural capacity of under-designed pavements. It increases structural capacity of the pavement system by reducing deflections.

TxDOT maintains many miles of thin PCC pavement. Although PCC pavement has served more traffic than it was originally designed for, most sections are in reasonably good condition. BCO can be a cost-effective method of rehabilitation to extend PCC pavement service life even further. The use of BCO on PCC pavement is based on the fundamental design assumption in BCO; that is, the old and new concrete layers behave as a monolithic layer. Providing full bond is of the utmost importance. During construction, specific steps are taken to enhance and ensure the full bond between old and new concrete. This will be discussed further in , "Rigid Pavement Rehabilitation."

Bonded concrete overlays are not recommended for construction over jointed concrete pavements. CRCP-bonded concrete overlays have been constructed and have performed successfully in several districts but have not been used widely throughout the state. For any district considering bonded concrete overlay that has no experience in this design, obtain training presentations materials from CST-M&P, Rigid Pavement & Concrete Materials Branch is recommended.

The 1993 AASHTO Guide for Design of Pavment Structures is recommended for "Unbonded Concrete Overlay" thickness design. The designer can use the automated procedure AASHTO DARWin® program and select the "Overlay Design" Module and "Bonded PCC Overlay of PCC Pavement."

Overlay Thickness Design

Dol = Df - Deff

Where:

Dol = required slab thickness of overlay, in.

Df = slab thickness to carry future traffic, in.

Deff = thickness of existing slab, in.

The slab thickness Df to carry future traffic is determined for the design traffic, as if it were built as a new pavement on the prepared base.

The design engineer will determine the design life and request the future traffic from TP&P. The modulus of subgrade reaction of 300 pci is recommended for new pavement structures with stabilized bases. However, some existing pavements might not have stabilized bases, and the k-value should be evaluated using FWD or DCP. Other input values will follow the same recommendations as listed at Section 2 — Approved Design Method.

Determination of Effective Slab Thickness by Condition Survey Method

Deff = Fic * Fdur * Ffat * D

Where:

Fic = joints and cracks adjustment factor

Fdur = durability adjustment factor

Ffat = fatigue adjustment factor

D = thickness of existing slab, in.

For BCO design, the condition of existing pavement is one of the most important factors. If the pavement condtion is deteriorated in the form of punchouts and deteriorated cracks, BCO may not be a good alternative.

The items that should be surveyed are:

1. the number of punchouts per mile
2. the number of deteriorated transverse cracks/joints per mile
3. the number of existing and new repairs prior to the overlay per mile

4. presence of D-cracking or ASR cracking and
5. evidence of pumping of fines or water

Punchouts are the only structural distresses in CRCP; the number of punchouts per mile is a good indication of the structural condition of the existing pavement. If there are more than 10 punchouts per mile, then the pavement is in poor structural condition and may not be a good candidate for BCO.

The number of deteriorated transverse cracks per mile is the next item to be surveyed. Even though some transvers cracks may appear to be deteriorated, quite often then are in good condition. In Texas, it is rare to observe deteriorated transverse cracks, except for spalled cracks. Most spalled cracks are not necessarily structurally deficient. Based on the research findings, it is recommended that only transverse cracks much wider than normal, all along the transverse crack throughout the lane width should be counted as "deteriorated transverse cracks."

Number of patches per mile should be recorded. Durability related problems should be noted and their severity recorded. Overall, it has been quite rare to observe durability related problems in CRCP in Texas. However, if D-cracking or ASR-related cracks are observed, their severity should be evaluated and recorded. In Texas' old CRCP, base was not stabilized and pumping has been observed. Evidence of pumping should be recorded.

Joints and Cracks Adjustment Factor, Fjc

The Joints and Cracks Adjustment Factor, Fjc, accounts for the extra loss in the present serviceability index (PSI) caused by deteriorated reflection cracks in the overlay. Deteriorated reflection cracks develop due to unrepaired deteriorated joints, cracks, and other discontinuities in the existing slab prior to the overlay.

A deteriorated joint or slab will rapidly reflect through an overlay and contribute to loss of serviceability. Therefore, full-depth repair is recommended on all deteriorated reflection cracks and any other major discontinuities in the existing pavement prior to overlay. The target Fjc is 1.00.

If it is not possible to repair all the deteriorated areas, use the total number of unrepaired deteriorated joints, cracks, punchouts, and other discontinuities per mile in the design lane to determine the Fjc from Figure 8-3.

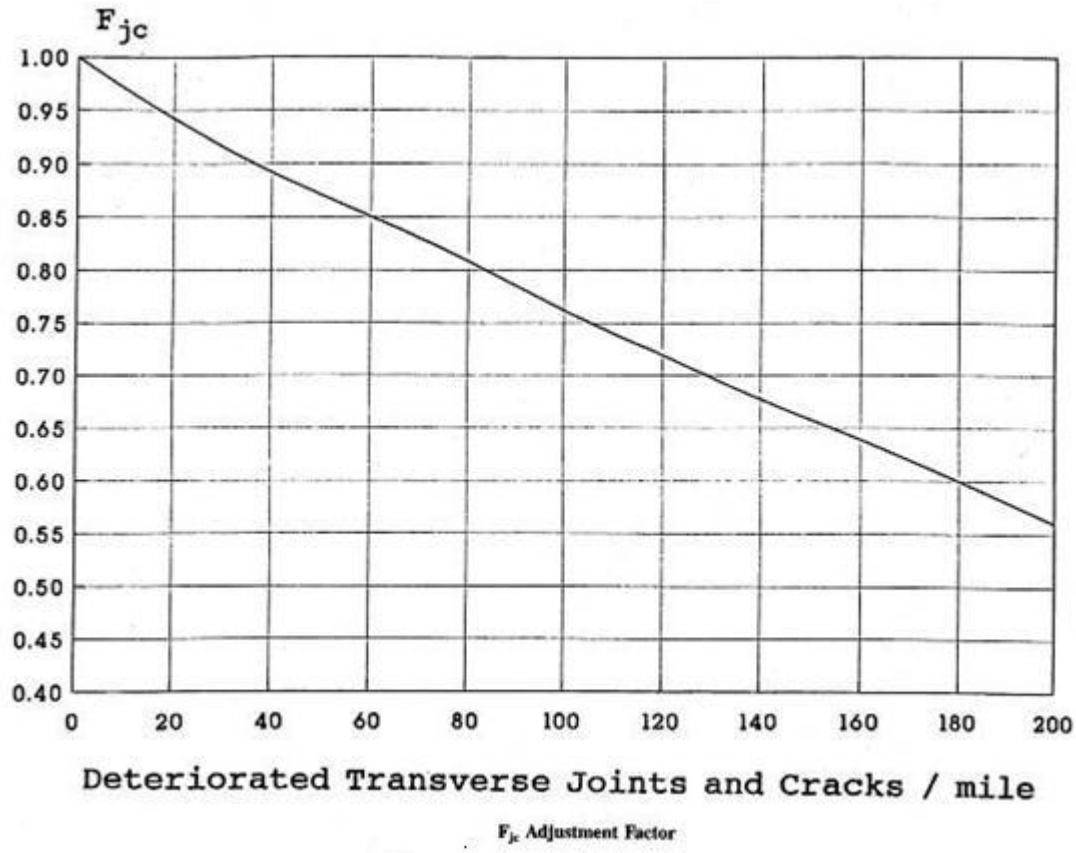

Figure 8-3. Joints and Cracks Adjustment Factor for Bonded Overlays, F_{jc}

Source: AASHTO Guide for Design of Pavement Structures (1993)

Durability Adjustment Factor, Fdur

The Durability Adjustment Factor, Fdur, adjusts for an extra loss in PSI of the overlay when the existing slab has durability problems, such as D-cracking or reactive aggregate distress. Fdur is determined using historical records and condition survey data.

Durability Adjustment Factor, F_{dur}	Historical Records and Condition Survey Data
1.00	No evidence or history of PCC durability problems

Durability Adjustment Factor, F_{dur}	Historical Records and Condition Survey Data
0.96—0.99	Pavement is known to have PCC durability problems, but there is no visible spalling.
0.88—0.95	Cracking and spalling exist (in these conditions, a bonded PCC overlay is not recommended)

Fatigue Damage Adjustment Factor, Ffat

The Fatigue Damage Adjustment Factor, Ffat, adjusts for past fatigue damage in the slab. It is determined by observing the extent of punchouts (CRCP) that may be caused primarily by repeated loading.

Fatigue Damage Adjustment Factor, F_{fat}	Historical Records and Condition Survey Data
0.97—1.00	Few transverse cracks/punchouts exist (none caused by D-cracking or reactive aggregate distress), < 4 punchouts per mile
0.94—0.96	A significant number of transverse cracks/punchouts exist (none caused by D-cracking or reactive aggregate distress), 4 to 12 punchouts per mile
0.90—0.93	A large number of transverse cracks/punchouts exist (none caused by D-cracking or reactive aggregate distress), >12 punchouts per mile BCO is not recommended

Steel Design

Steel should be placed at a depth that provides a minimum concrete cover of 3 in. When BCO thickness is 3 in. or less, reinforcement in the form of longitudinal steel is not needed. In fact, fibers have been successfully used.

The design engineer will determine the steel bar size and numbers of longitudinal bars, tiebars, and transverse bars. The performance of CRCP depends on the amount of longitudinal reinforcement, and BCO with 4 in. or thicker without longitudinal reinforcement will effectively reduce the amount of longitudinal steel in the combined slab (existing plus BCO layers), which could increase steel stresses and make transverse crack widths larger. See the

recommended steel percentage and vertical location in the table below.

BCO Thickness	Longitudinal Steel (%)	Vertical Location	Fibers
<=3 in.	No	N/A	Yes
<= 5 in.	0.6%	Bottom of BCO	No
> 5 in.	0.6%	Middle of BCO	No

Unbonded Concrete Overlays

Unbonded concrete overlay consists of a concrete layer (5 in. or greater) on top of an existing concrete with a "separation inter-layer" to separate new overlay and existing concrete. An unbonded overlay is a feasible rehabilitation alternative for PCC pavement for practically all conditions. These types of rehabilitation methods are most cost-effective when the existing pavement is badly deteriorated due to the reduced amount of repairs made to the existing pavement.

Unbonded CRCP concrete overlays may be used over CRCP, jointed concrete pavement (JCP), or jointed reinforced concrete pavement (JRCP). Unbonded CRCP overlay uses the same design procedure as new CRCP pavements. This use of unbonded CRCP overlay can be credited for contributing to the structural capacity of the existing concrete pavement and results in a thinner concrete pavement design than required for CRCP constructed on a new location.

The 1993 AASHTO Guide for Design of Pavement Structures is recommended for unbonded concrete overlay thickness design. The designer can use the AASHTO DARWin® program and select the "Overlay Design" Module and "Unbonded PCC Overlay of PCC Pavement."

Overlay Thickness Design

Dol = (Df2 - Deff2)1/2

Where:

Dol - required slab thickness of overlay, in.

Df = slab thickness to carry future traffic, in.

Deff = effective thickness of existing slab, in.

Determination of Effective Slab Thickness by Condition Survey Method

Deff = Fjcu * D

Where:

D = existing slab thickness, in. (Use 10 in. when existing D >10 in.)

Fjcu = joints and cracks adjustment factor

Fjcu adjusts for PSI loss due to unrepaired joints, cracks, and existing expansion joints, exceptionally wide joints (>1 in.), or AC full-depth patches.

Use the total number of unrepaired deteriorated joints, cracks, punchouts, and other discontinuities per mile in the design lane to determine the Fjcu from Figure 8-4.

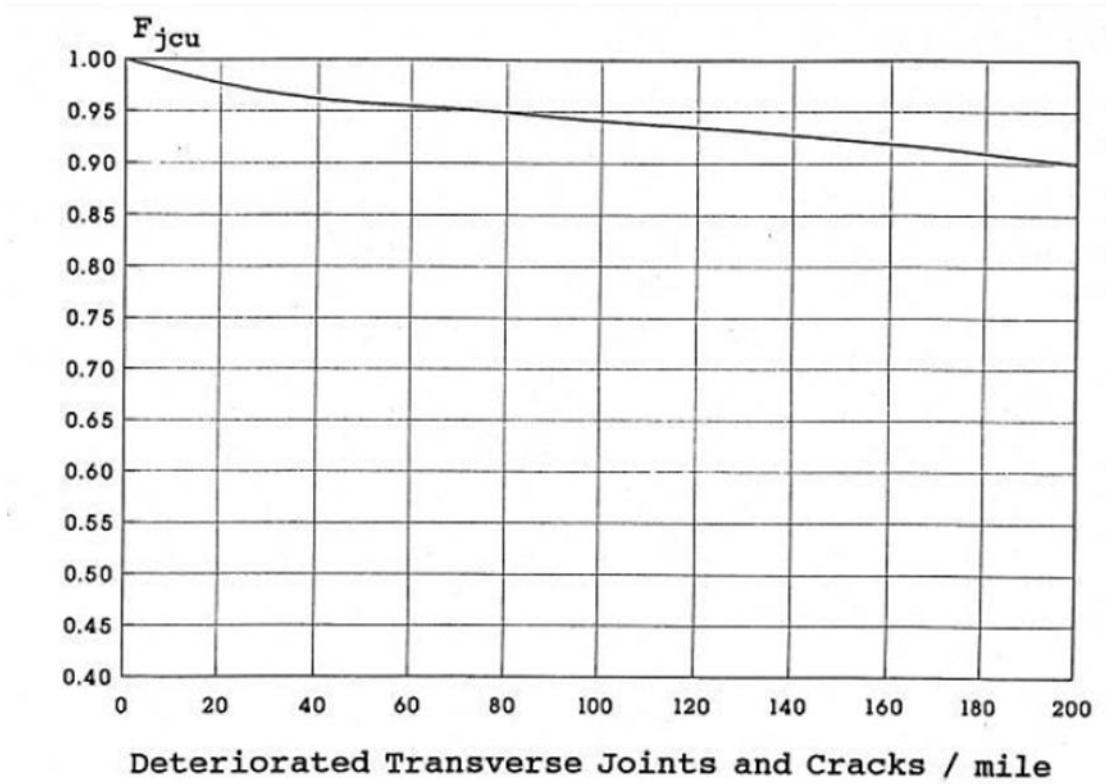

Figure 8-4. Joints and Cracks Adjustment Factor for Unbonded Overlays, F_{jcu}

Source: AASHTO Guide for Design of Pavement Structures (1993)

Steel Design

The steel placements are the same as new CRCP when unbonded overlay is 6 in. or thicker. When unbonded overlay is less than 6 in., use longitudinal reinforcement at about 0.6% of concrete cross-sectional area. The design engineer will determine the steel bar size and quantities for longitudinal bars, tiebars, and transverse bars and consult with the Rigid Pavement & Concrete Materials Branch of CST-M&P.

More information about bonded and unbonded concrete overlays is detailed in "Bonded Concrete Overlay" and "Unbonded Concrete Overlay" of , "Rigid Pavement Rehabilitation."

Section 8 — Thin Concrete Pavement Overlay (Thin Whitetopping)

Thin whitetopping (TWT) is a 4 to 7 in. thick concrete overlay bonded to an existing asphalt concrete pavement (ACP) to create a composite section. TWT is constructed normally at intersections where rutting and shoving in asphalt pavement continue to cause problems. TWT may also be used at access or exit ramps to interstate highways, entire sections of urban roadways, low-volume rural roads, bus lanes, and parking area. TWT provides better serviceability, longer service life, lower life-cycle cost, and improved safety.

This rehabilitation technique purposely seeks to bond the concrete overlay to the existing asphalt. The composite action significantly reduces the load-induced stresses in the concrete overlay. Therefore, the concrete overlay can be significantly thinner for the same loading as compared to a whitetopping section with no bond to the underlying asphalt. TWT will significantly reduces time and delays accompanying the frequent maintenance of an asphalt surface.

Figure 8-5. Whitetopping.

TxDOT has developed the *Preliminary Guidelines for Thin Whitetopping (TWT)* (June 2004). It can be used as a general guide for construction and proper TWT thickness design. More research will be conducted to improve the design procedure for optimal design life, thickness and size of the concrete panels through Life Cycle Cost analysis.

Preliminary Guidelines for Thin Whitetopping (TWT)

Preliminary Guidelines provide the recommendations for TWT design thickness, joints spacings, ACP support layer, and better bond.

- Design Life of 5 to 10 yr. is recommended for TWT.
- Normally used at intersections where rutting and shoving in asphaltic pavement continue to cause problems.

- Designs for < 4 in. TWT were not included because the initial cost difference between 4 in. slabs and those with less than 4 in. is negligible.
- Designs for > 7 in. TWT were not included because our standard concrete pavement designs should be used and load transfer devices included.
- Contraction joints spacing are set at 6 ft. to prevent edge loading and reduce the costs for saw cutting. All sawed TWT panels shall be square except as necessary in pavement width transitions.
- Use engineering judgment to allow the contractor to meet plan requirements when using the ride specification at intersections or curb and gutter sections.
- Recommend ≥ 4 in. asphalt concrete pavement (ACP) support layer (including ASB and level-up).
- Uniform support will improve performance.
- Recommend milling the existing ACP to provide a better bond and remove rutting ≥ 1/2 in.
- Opinions differ on the need for fibers to control shrinkage cracking.
- Saw cuts must be made as soon as possible, without delay.
- Saw cut depths must be adjusted accordingly in thickened sections.
- At bridge approaches, an approach slab is recommended with an expansion joint as shown in existing standards.
- TxDOT has yet to establish when expansion joints are needed for wide pavements.
- The use of steel fibers is not recommended at locations where deicing salts may be used.
- Apply governing Special Specification, Thin Whitetopping (Concrete Overlay).
- Use Thin Whitetopping Details, TWT-04 Standard Sheet.
- Use Table 8-4 as a general guide for determining proper TWT thickness. More research will be conducted to improve the design procedure.

Table 8-4: Thin Whitetopping Thickness (in.) Design

	Design Life (yr.)					
Trucks per Day per Lane	**5**	**6**	**7**	**8**	**9**	**10**
≤ 200	4	4	4	4	4	4
250	4	4	4	4	5	5
300	4	4	4	5	5	5
350	4	4	5	5	5	5
400	4	5	5	5	5	5

Table 8-4: Thin Whitetopping Thickness (in.) Design

Trucks per Day per Lane	Design Life (yr.)					
	5	**6**	**7**	**8**	**9**	**10**
450	5	5	5	5	5	6
500	5	5	5	5	6	6
600	5	5	5	6	6	6
700	5	5	6	6	7	7
800	5	6	6	7	7	7
900	6	6	6	7	7	n/a
1,000	6	6	7	7	n/a	n/a

The traffic data (in terms of ADT, Percent Directional Distribution, and Percent Trucks) are obtained from the traffic analysis report provided by the Transportation Planning and Programming (TPP) Division.

$$\textit{Trucks per day per lane} = ADT \times PDD \times PT \times LDF$$

Where:

- ADT = average daily traffic for the first year of design period
- PDD = percent directional distribution of traffic
- PT = percent trucks
- LDF = lane distribution factor, use 1.0 for 4 lanes or less, 0.7 for 6 lanes, and 0.6 for 8 lanes or more. The lane number is for both directions.

Example:

- Design Life = 10 years, ADT = 13,500, PDD = 63%, PT = 3.0%, and 4 lanes roadway, LDF =1.0
- Trucks per Day per Lane = 13,500 × 0.63 × 0.03 ×1.0 = 255
- Read TWT Catalog, for 255 trucks per day per lane and 10-yr. design, the required TWT thickness is 5 in.

Chapter 9 — Rigid Pavement Construction

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Section 1 — Overview

The long-term performance of rigid pavement depends not only on proper pavement design and materials selection, but on good construction practices as well. Poor construction practices have resulted in premature failures of rigid pavement. The construction of a rigid pavement is a fairly complex process. It involves many processes including proper preparation of the subgrade and sub-base, placing reinforcing bars or dowels, choice and handling of aggregates and other materials, development of concrete mix design, production and transport of the concrete, and placing, finishing, curing and joint sawing the concrete. This chapter describes the construction of rigid pavement system. For subgrade construction, base construction, and surface preparation refer to Chapter 6, "Flexible Pavement Construction."

Section 2 — Concrete Mix Design

The concrete mix design is performed to ensure that the concrete mix formulation meets or exceeds the specification requirements. The mix design is used to establish the proper proportioning of components (hydraulic cement, aggregates, water, pozzolans, and admixtures) in the mixture to achieve the specified properties. Significant properties are strength, air content, slump, and sometimes the coefficient of thermal expansion (COTE). The mix design may be developed for the current project or may have been previously developed. TxDOT developed a Concrete Mix Design Guidance document that provides useful information. Concrete mix design can be facilitated by using the Mix Design spreadsheet developed by TxDOT. The mix design must be formally approved by an engineer.

In addition to the required mix design strength, the job control strength needs to be established. The job control strength is used to verify that the concrete being used on the project will perform similarly to the concrete used to develop the mix design. The default is to use the 7-day mix strength as the job control strength. This 7-day strength testing may be altered with the approval of the engineer. The contractor may want to develop job control strengths at an earlier age, such as four days, to permit the job control specimen to also be used to open the pavement to traffic at an earlier age. Reduction of curing time for the job control specimens may reduce the reliability of the test in ensuring that the specified strengths will be reached. Testing at four days should still provide a reliable estimate of the long term strength.

The standard job control testing at seven days was established many years ago and ensured that the strength test of the job control specimens occurred on the same day of the week as the concrete paving. The result is that there would be no testing on Saturday or Sunday, unless the paving work was performed on those days. This was also a management tool to eliminate the need for laboratory technicians to report to work on Sunday to perform a single strength test.

It is recommended that air-entrainment requirements be maintained for concrete pavement at all locations around the state, regardless of the number of freeze-thaw cycles in that area. The benefits provided by entrained air to slip-formed pavement in workability and water demand are substantial. A good air-entrained system also benefits the long-term durability of hardened concrete.

For concrete pavements, the target air content will be between 2.5 and 5.5%, with values as high as 7% acceptable as long as strength requirements are met. Since Item 360 allows occasional and minor (less than 10%) strengths below job-control targets, an entrained-air range of 2.5 to 7% should not be a production challenge, nor should it require excessive cement to ensure minimum strengths are consistently obtained.

Many urban highway construction projects have severe traffic control and congestion issues. To expedite construction and minimize travel delays for the public, contract restrictions on dates and times that travel lanes can and can not be closed to travel are imposed. Project contracts may also

impose large bonuses and disincentives for time of completion. Pavement using Class P concrete may be opened in as little as two days to contractors' vehicles and as little as three days to all traffic, if opening strength is achieved. After curing is complete and when earlier age job control testing is permitted or required, and the tested strength is greater than the required opening strength, the pavement may be opened to traffic.

Class HES (high early strength) concrete may be used in small areas and leave-outs. Class HES has additional strength requirements beyond Class P concrete to ensure that the high early strengths for opening to traffic are realized.

The maturity method, "Tex-426-A, Estimating Concrete Strength by the Maturity Method," may be used to open the pavement to traffic at an earlier age than seven days with either Class P or Class HES concrete. It is still necessary to complete the specified curing. A maturity curve may be developed during the mix design process to establish the relationship between the concrete maturity and concrete strength. This may be used to identify what the maturity of the concrete should be when the opening strength has been reached. This maturity value may be used to estimate the in situ strength and open pavement to traffic that has completed the required curing. A maturity test should not be used in lieu of job control strength testing to determine the conformance of the mix to the mix design.

Section 3 — Concrete Plant Operation

A batch plant is the concrete mix plant where the aggregates, cementitious materials, chemical admixtures, and water are metered and combined together to produce hydraulic cement concrete. The batch plant can be a commercial concrete batch plant or a temporary batch plant brought in and assembled by the contractor near the paving site. On large concrete paving projects, most contractors choose to mix their concrete using their own temporary batch plant. This is because commercial batch plants are usually too small to produce the large amount of concrete needed for a concrete paving project.

In Item 421, five types of mixers are allowed for TxDOT projects:

- central-mixed,
- shrink-mixed,
- truck-mixed,
- volumetric mixer-mixed, or
- hand-mixed.

Volumetric mixing and hand-mixed concrete are rarely used for concrete pavement construction and are not covered in this document. The other three mixing operations are described below.

Central-mixed Plants

Central-mixed concrete is batched and completely mixed in a stationary mixer at the plant site . A central-mixed concrete plant completely mixes concrete before discharging it into haul vehicles. Central-mix plants are sometimes referred to as wet batch plants. Figure 9-1 shows a diagram of the central-mixing plants. About 20% of the concrete plants in the US use a central mixer. Principal advantages of the central-mixing plants include the following:

- higher production capability
- better concrete quality control and consistency, and
- reduced wear on the truck mixer drums.

A batch of concrete is the amount of material mixed at one time. Sequencing is the order that materials are batched. A typical batch of concrete includes:

- water
- admixtures
- coarse aggregate
- fine aggregate

- cementitious materials (cement, fly ash, ground granulated blast furnace slag, ultra fine fly ash).

Figure 9-1. Central-mixing Plant.

In this central-mixing plant, aggregates (coarse and fine) are fed by conveyer belts in the left side. Cementitious materials, water, and chemical admixtures are supplied from the right side. Once all the ingredients are put into the mixer, they are mixed thoroughly and dumped into the trucks.

Accurate proportioning or batching of these materials per approved mix design is essential to producing concrete with satisfactory properties. The first step to achieve proper proportioning is to have all the weighing and measuring equipment properly calibrated.

Concrete materials are batched in three groups. One group is the aggregate group, and another is the cementitious materials group. These materials are either weighed individually or cumulatively. Water and admixtures comprise the last group. In most current central-mixing plants, the batching is done by computerized control system as shown in Figure 9-2.

Figure 9-2. Most batch plants are computerized.

Consistency of the concrete is greatly affected by the amount of water in the concrete. The accurate estimate of moisture condition of the coarse and fine aggregate and proper "consideration" is critical to the production of consistent concrete. In normal operations, moisture contents of fine

aggregates are evaluated using moisture probes as shown in Figure 9-3. Normally, the moisture content of the sand affects the concrete more than the coarse aggregate. The coarse aggregate typically has low absorption values.

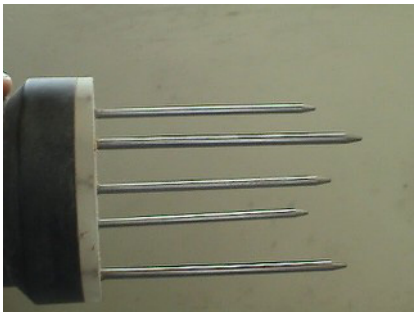

Figure 9-3. Moisture probe for aggregates.

The amp meter in a central mixed plant provides an indication of concrete mix consistency (see Figure 9-4). The amp meter is sometimes called the slump meter. Care must be taken when using this meter to control the mix consistency, since the reading in the amp meter is an indirect indicator of the consistency, which is derived from the correlation between consistency and current needed to rotate the mixer.

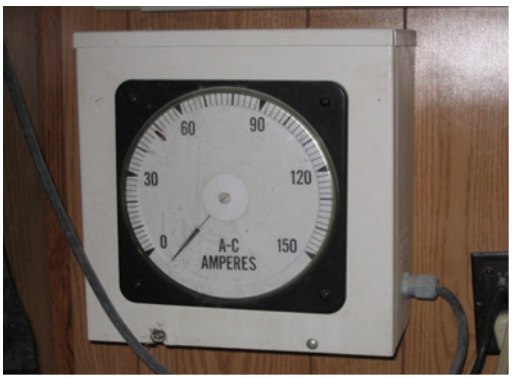

Figure 9-4. Amp meter for workability.

The first step in the sequence of concrete production is charging the mixer. Charging the mixer consists of transferring all of the weighed or measured materials from weigh hoppers and silos into the central-mixer. Aggregates are ribbon loaded on conveyer belts. Initial blending takes place on the feed belt. This initial blending enhances mixer performance. Specific charging sequences vary.

Materials are blended in approximate proportions as they enter the mixer. One batch of concrete is being mixed while another is being batched. These operations occur simultaneously.

It is important that admixtures are not blended, since blending before discharging to the mixer can create undesirable interactions. Shown in Figure 9-5 are three orange hoses that feed admixtures. The big blue tube is the water supply line, which is diverted to both sides. Therefore, admixtures are not blended directly with each other; rather they are blended with water first.

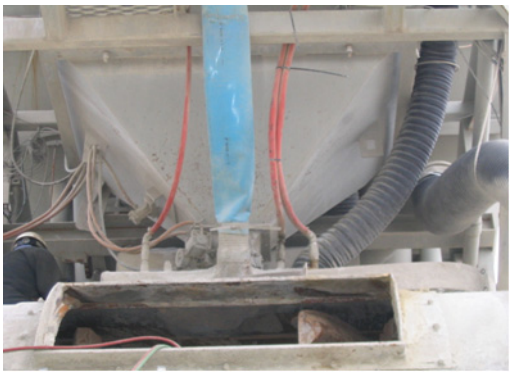

Figure 9-5. Water and Admixture Supply System.

The blades or paddles and the mixing action of a central-mix drum are quite different than a truck mixer, where there is little folding action compared with that in a stationary mixer. See Figure 9-6 for an illustration of the inside of a central-mix drum.

Figure 9-6. Paddles are shown inside a central mix drum.

Although a large variety of central plant mixers have been used over the years, the large tilting drum mixer is most popular. An example of a tilting drum mixer is shown in Figure 9-7.

Figure 9-7. Tilting Drum Mixer.

Mixing time begins after all materials have been added to the mixer. Item 421 requires a mixing time as a function of the quantity of the concrete to be produced – 1 min. for 1 cu. yd. and 15 sec.

for each additional cu. yd. of rated capacity of the mixer. If mixer performance, tested according to "Tex-472-A, Uniformity of Concrete," shows uniform mix can be achieved with shorter mixing time, this mixing time requirement can be waived.

Central-mixed concrete is transported in non-agitating trucks, truck mixers, or agitating trucks. In most of TxDOT's projects, contractors prefer non-agitating (dump) trucks for the efficiency. Figure 9-8 illustrates the charging of concrete into a dump truck.

Figure 9-8. Discharging concrete into a dump truck.

When non-agitating trucks are used to haul the concrete, the haul distance should not be far. Transporting concrete in dump trucks in a long distance can cause segregation and non-uniform consistency due to bleeding and setting.

After dumping the concrete, the truck bed needs to be washed out to the point where no clumps of concrete remain in the corners of the bed (Figure 9-9). If concrete is allowed to build up in the corners of the bed, the clump becomes big and hard, and will break loose from the bed when the truck is dumping a load of concrete.

Figure 9-9. Rinsing truck between deliveries.

Shrink-mixed Concrete

Shrink-mixed concrete is used to increase the truck's load capacity and retain the advantages of transit-mixed concrete. In shrink-mixed concrete, concrete is partially mixed at the plant to reduce or shrink the volume of the mixture and mixing is completed in transit or at the jobsite. This type of concrete is rarely used for TxDOT projects.

Truck-mixed Concrete

In truck-mixed concrete, all of the ingredients are charged directly in the truck mixer. In truck-mixed concrete or "dry batch" operation, as it is sometimes called, the batches are measured by the plant operator and charged from the weigh hoppers directly into the truck mixers for mixing. The plant operator is responsible for accurately batching the concrete in the proper and pre-determined sequence.

Sequences for charging truck-mixers are more critical than those for central mixers. Figure 9-10 illustrates this process. The first ingredients into the drum are usually a portion of the water and a portion of the coarse aggregate. The water is shut off and aggregates and cementitious materials are ribboned together until all of the cementitious material is in the drum. A final portion of water is added with the last of the aggregates to clean and wash any cementitious material clinging to the hoppers, rear fins, and chutes.

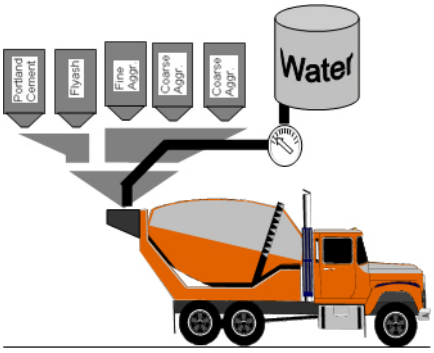

Figure 9-10. Truck-mixed or dry batching of concrete.

One of the problems when the proper batching sequence is not followed is so-called "head packs." A head pack occurs when sand or sand and cementitious material packs in the head of the drum and remains lodged in the head without being mixed into the concrete. Typically, head packs will be about 12 to 24 in. thick. Head packs are difficult to detect because they break up after approximately half of the concrete has been discharged. Head packs are principally responsible for sand streaks, which occur in the final three-quarters of the discharge.

Once the batching is complete, generally the truck mixer driver completes the mixing in the yard. This allows the driver to examine the mix and determine if it is acceptable prior to leaving the patch batch plant site. When the mix is correct at the plant, it will most likely be correct when it arrives at the job site. Item 421 requires the mixing of the concrete from 70 to 100 revolutions at the mixing

speed. It also allows additional mixing at the job site at the mixing speed as long as concrete is discharged before the drum has revolved a total of 300 revolutions.

The truck mixer is equipped with a revolution counter and slump meter as shown in Figure 9-11. The driver is capable of estimating the slump to within 1/2 in. This meter is used for estimating only. The slump is still measured per the project specifications.

Figure 9-11. Slump meter mounted in the mixing truck.

It is necessary to rinse off the rear fins of the mixer between loads and wash and discharge the entire mixer only at the end of the day. Figure 9-12 illustrates the washing of the mixer drum.

Figure 9-12. Washing the mixer drum.

Concrete mixed in a truck mixer is transported to the jobsite at agitating speed. Between each load of concrete, no clumps of concrete should remain on the paddles or fins. Figure 9-13 shows examples of concrete clumps. If concrete is allowed to build up, it will eventually break loose from the drum when a load of concrete is discharged.

Figure 9-13. Large clumps of concrete.

Section 4 — Delivery of Concrete

Concrete to be used in concrete pavement may be delivered to the paving operations in several ways without segregation. The low slump of the paving concrete makes it possible for the concrete to be delivered in non-agitated dump trucks, concrete mixing trucks and agitator trucks.

Concrete mixing trucks may be used for mixing or agitation. Typically, a concrete mixing truck will have two capacity ratings, as a mixer and as an agitator. The agitator rating is usually significantly larger. For example, a mixer truck may be rated for 6 cu. yd. as a mixer and for 8 cu. yd. as an agitator. This means that if the concrete is centrally mixed and then loaded into the mixer truck, a larger load can be transported to the project. Otherwise, the smaller mixer rated capacity must be used.

True agitator trucks are sometimes used on TxDOT projects. Agitator trucks can be used to deliver agitated concrete to the project, but never to mix concrete. Typically, these trucks will resemble a mixer truck, except that the mixer drum is open on the top. Instead of a drum turning, a series of paddles will rotate within the agitator drum. These paddles can be seen as they rotate above the mix. Use of agitator trucks is more common when higher slump concretes are used that may be more susceptible to segregation during deliver.

The concrete delivery time needs to be carefully monitored. This is the time from when water is first added to the dry ingredients of the mix until the truck is ready to discharge the concrete on the project. This is particularly critical in hot weather when the concrete may start to set prior to delivery. It should be regularly monitored using the time stamp on the batch ticket from the plant.

Concrete mixer trucks typically have a water tank mounted on the truck. This water should never be added to the concrete batch unless approved by the engineer. Occasionally, the use of an absorptive aggregate, such as recycled aggregate will cause a drop in the slump of the mix, as water is withdrawn from the mortar into the aggregate. Even then, the water should be measured carefully using a certified gauge to ensure that the added water does not cause the concrete to exceed the water content of the approved mix design. Water should never be added to “re-temper” concrete that has begun to set. This can occur in hot weather or when delivery times are extended. It is also more pronounced when higher cement contents are used. The water tank should be full when the truck leaves the plant and full when it reaches the project.

Wash water from the concrete trucks should never be disposed of in other than an environmentally sound method, as approved by the Texas Commission on Environmental Quality (TCEQ). The preferred method is to capture the wash water at the concrete plant and recycle the wash water as concrete mix water into subsequent concrete batches.

Section 5 — Steel Placement

In this section, only the placement of longitudinal and transverse rebars is described. The placement of other rebars, such as tie bars and dowel bars, is described in Section 7, "Joints."

Reinforcing Steel

The longitudinal reinforcing steel keeps the naturally occurring transverse cracks in continuously reinforced concrete pavement (CRCP) tight, thereby providing high level of load transfer across cracks. When transverse cracks are kept tight, aggregate interlock also transmits the shear forces across the crack, resulting in reduced wheel load stress and fatigue damage in concrete.

Transverse reinforcing steel is used to provide support for longitudinal steel. It also keeps longitudinal cracks tight if they occur.

ASTM A966 Type R bars may only be used as straight bars and only in concrete pavements. This type of bar is permitted to encourage the use of recycled steel in TxDOT concrete paving projects. It may not meet the "pin" test requirements of ASTM and may not have sufficient strength if bent. If this type of reinforcing steel is used in concrete pavement, care must be taken that these bars are not diverted to other uses on the project.

Only Grade 60 deformed steel bars that meet the requirements of the Item 440 are used. Grade 70/75 steel bars have been used in some TxDOT projects and no pavement performance improvements have been noted. On the other hand, when reduced amount of longitudinal steel was used with Grade 70/75 steel, transverse cracks were wider than those in sections with Grade 60 steel. Cracks with larger widths reduce load transfer across cracks, resulting in larger wheel load stress and poor long-term CRCP performance.

The reinforcing steel should be placed at the locations shown on the plans. As the pavement thickness increases, the amount of steel is increased. For thinner pavements, all the steel is fit into one mat or layer of steel placed at the mid-height of the pavement.

For 14- and 15-in. thick concrete pavements, two layers of steel are necessary. Enough room must be provided between adjacent bars to allow the plastic concrete that is placed on top of the steel mat to pass completely through the steel mat to the lower portion of the concrete placement.

Storing Reinforcing Steel

The steel must be stored above the surface of the ground upon platforms, skids, or other supports and shall be protected from damage and deterioration. This prevents excessive rusting that would occur if sitting directly on the ground. It also prevents mud and dirt from collecting on the steel.

When placed in the work, reinforcement shall be free from dirt, paint, grease, oil, or other foreign materials. Reinforcement shall be free from defects such as cracks and laminations.

Splicing Longitudinal Steel

In CRCP, maintaining steel continuity in the longitudinal direction is important in ensuring good performance of the pavement. The length of the reinforcing steel bars is 60 ft. The continuity of the longitudinal steel is achieved by overlapping individual steel bars. Extensive testing shows that as long as the overlapping is more than 33 times bar diameter, stresses in one steel is effectively transferred to the next steel via surrounding concrete. For example, No. 6 bars would need a 25-in. splice for effective stress transfer. Figure 9-14 shows an example of spliced or lapped reinforcing steel.

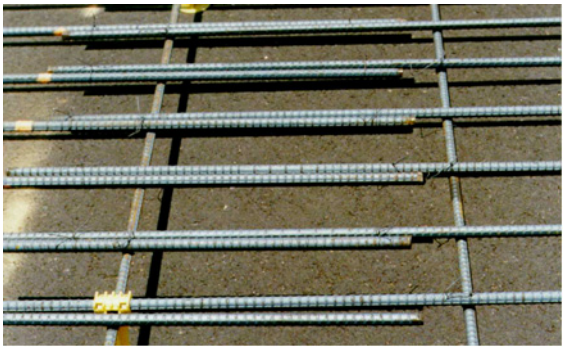

Figure 9-14. Spliced or Lapped Reinforcing Steel.

Splice Locations

If all the splices occur at the same longitudinal location, transverse cracks that occur at the location could cause steel bonding failure, resulting in wide cracks and performance problems. To make sure that all the splices do not occur at the same transverse location, Item 360 requires staggering splices to avoid having more than 1/3 of the splices within a 2-ft. longitudinal length of each lane of the pavement.

Following are drawings (Figure 9-15, Figure 9-16, Figure 9-17, and Figure 9-18) that show acceptable and non-acceptable splice locations or patterns.

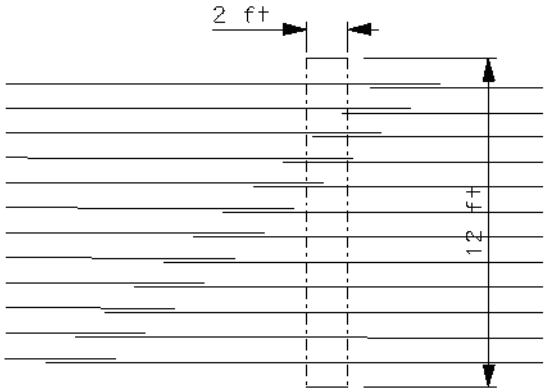

Figure 9-15. Acceptable Splicing Pattern- 33% (4 of the 12 bars) within the box are spliced.

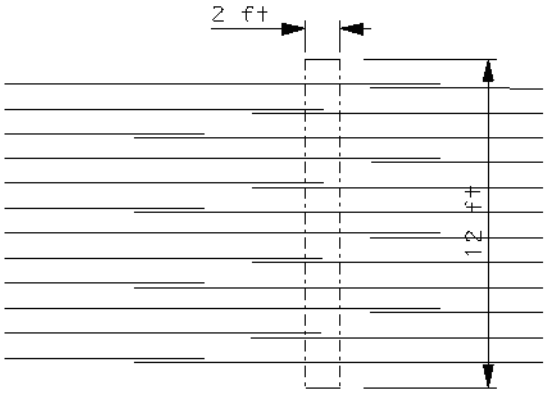

Figure 9-16. Acceptable Splicing Pattern.

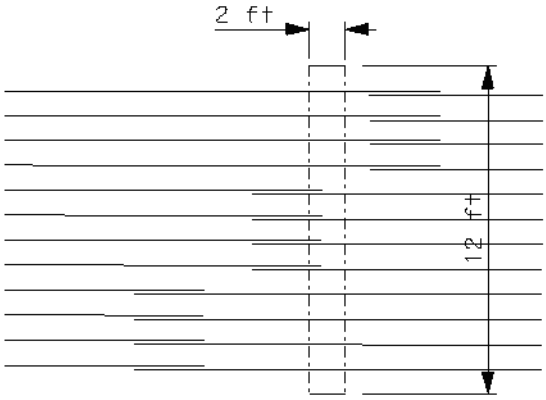

Figure 9-17. Acceptable Splicing Pattern.

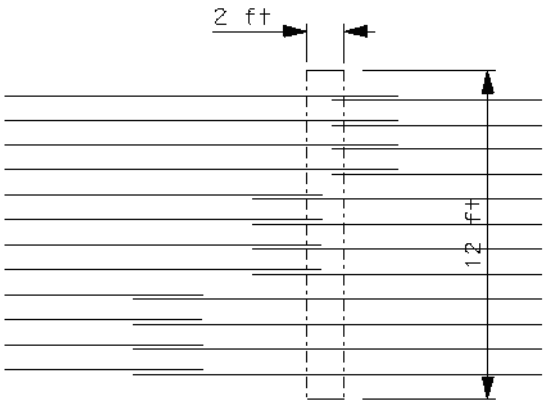

Figure 9-18. Non-acceptable Splicing Pattern – 66% of bars in the box are being spliced.

Holding the Reinforcing Steel in Place

Contractors like to keep the steel mat from moving excessively prior to paving due to the paving operation itself or because of temperature changes. Figure 9-19 shows a piece of reinforcing steel being used as a pin. This pin can be left, or can be removed before the concrete pour.

Figure 9-19. Pinning the mat.

Steel depth has an effect on CRCP performance. Design Standards CRCP (1)-03 and CRCP (2)-03 requires the tolerance of longitudinal steel at +/- 1 in. horizontally and +/- 0.5 in. vertically. Figure 9-20 illustrates the checking of the depth of steel.

Figure 9-20. Checking depth of steel.

Section 6 — Paving Operations

Two types of paving operations are used for the construction of rigid pavement. One is fixed-form paving, and the other is slip-form paving. Both methods have some common operations as further delineated here. First, both methods require accurate survey controls that are used to establish the proper alignment and grade of the concrete pavement. Second, both methods require proper curing of the concrete in order to facilitate producing a durable long lasting pavement. Third, any required joint sawing must be accomplished in a timely manner to prevent random cracking of the pavement.

Fixed-form Paving

Although not used very often in Texas, paving machines that ride on forms are still in use. Paving machines that use steel wheels to ride on paving forms are very heavy machines, almost comparable to a slip-form paving machine without the drive tracks. To prevent sagging under the weight of this machine, the forms need to be uniformly supported on a very firm subbase.

Smaller paving machines, such as the Clary, have three long steel roller tubes that extend across forms in both directions. The rollers propel and screed the concrete to the level of the forms. This type of paving machine is still seen on small and irregular placements, such as ramps and turnarounds. It is limited to smaller productivity and narrower placements.

Another type of paving machine that has actually been used in recent years on main lane urban freeways is the Bidwell, a machine that is more commonly seen on bridge deck construction. This machine rides on forms or steel pipes. It is characterized by a truss extending across the forms with a suspended longitudinal screed roller that moves transversely across the pavement. An auger mounted on the front end of the roller screed spreads the concrete and the roller screeds smooth the surface.

Neither the multiple roller type paver or the bridge type machine enjoy a reputation for producing a very smooth riding surface and are unlikely to be seen on a pavement surface where a ride specification with bonus/penalty is in effect. Productivity is also relatively low for both types of machines.

Fixed-form paving is a form of concrete pavement construction where fixed forms are used to hold the concrete in place at the proper grade and alignment during construction. This type of paving is different from slip-form paving, which utilizes the forms of the slip-form paving machine to form or mold the concrete in place at the proper grade and alignment, in lieu of fixed forms, during paving. In general, fixed-form paving is not as productive as slip-form paving because of the difference in the efficiency of the placing, spreading, and finishing operations with form riding equipment. In addition, it takes time and effort to set and remove forms before and after paving. However, fixed-form paving is more applicable than slip-form paving in certain situations such as ramps, blockouts, small paving areas, or where slip-form paving is not feasible or not economical. Most of the

main lane concrete paving in TxDOT projects is done by slip-form paving. Usually, only minor portions of the concrete paving are done by fixed-form paving.

To facilitate the placing, spreading, consolidation, and finishing operations, concrete used for fixed-form paving usually has more workability or higher slump than the concrete used with slip-form paving.

Forms

A key element in fixed-form paving for constructing a smooth concrete surface is the form. Typical forms have the following characteristics:

1. Most of the forms are made with metal with the minimum thickness of at least 0.2 in. and 10 ft. long.
2. The height of the forms determines the slab thickness.
3. The bases of the forms are wide and flat. These wide and flat bases provide stability of the forms in place.
4. Flange braces extend outward on the base not less than 2/3 the depth of the form.

The final grade and smoothness of the concrete surface is determined, to a large extent, by how securely and how closely the forms are set to the final grade and alignment lines. One of the most important requirements of the forms is to provide stability while concrete is being placed. Unstable forms will cause irregularities in the finished concrete pavement. The top and the face of the forms should be as flat as possible. A straightedge or stringline can be used to check for variance. Forms must be free from detrimental kinks, bends, or warps that could affect ride quality or alignment. The forms should allow for tightly locking the ends of adjacent form sections. The ends of the forms should be flush when they are in position. All forms must be cleaned and oiled before use.

Setting Forms

A survey line is established to facilitate setting forms at proper grade and alignment. Form setting is a critical construction operation since the final grade and smoothness of the pavement surface depends, to a large extent, on how accurately the forms are set to line and grade and how well and uniformly the forms are supported by a firm foundation.

The finished smoothness of the pavement depends on the care with which the forms are set and maintained because the finishing equipment rides on the forms. Proper alignment and elevation of the forms will contribute to a smooth pavement. It is important to provide a firm and level foundation under all forms. Figure 9-21, Figure 9-22, Figure 9-23, and Figure 9-24 illustrate the process of form setting.

Once the forms have been set, they are checked for overall alignment and tolerance before any paving takes place. If any form section is out of line, it needs to be corrected immediately. Joints between forms must be tight and smooth. Specification Item 360 requires that the contractor provide metal side forms of sufficient cross-section, strength, and rigidity to support the paving equipment and resist the impact and vibration of the operation without visible springing or settlement.

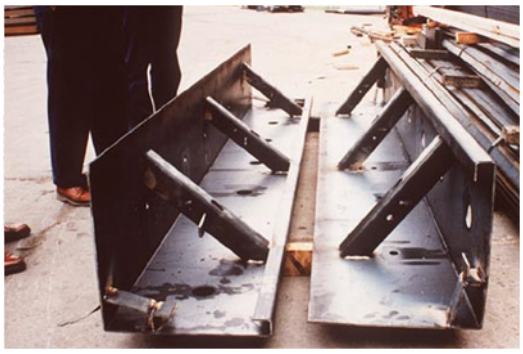

Figure 9-21. Forms ready for setting.

Figure 9-22. Using jackhammer to set staking pin.

Figure 9-23. Staking Pin.

Figure 9-24. Side forms used on a curve.

The preferred depth of the form shall be equal to the required edge thickness of the pavement. Forms with depth greater or less than the required edge thickness of the pavement can be used if the difference between the form depth and the design pavement depth is not greater than 2 in., and:

1. forms of a depth greater than the pavement edge thickness may be used if the supporting material is planed to construct a form trench.
2. forms of a depth less than the pavement edge thickness shall be brought to the required edge thickness by securely attaching metal strips or wood shims of approved section to the full width and length of the base of the form, as shown in Figure 9-25. Use grout to fill the gap with wood shims to establish grade.

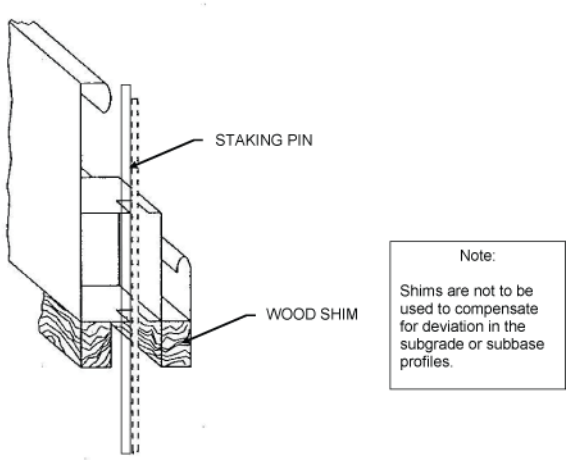

Figure 9-25. Side Form.

Checking Forms

Unlike a slip-form paving operation, where the thickness of concrete pavement can be adjusted for "knots" or rises in the subbase to maintain proper thickness, the thickness of concrete pavement is

established by the height of the side forms. That is, side forms that are 8 in. tall will produce a concrete pavement that is at least 8 in. tall only if the subbase between the two forms does not have an area that is above the level of the bottom of the forms. To check that there are no bumps in the subbase, a "scratch" template may be used.

The scratch template is typically a lightweight truss that is supported on the side forms by wheels and has long metal tines spaced about a foot apart that extend down from the truss for the thickness of the concrete pavement. The scratch template is moved along the side forms. If the tines on the scratch template come in contact with the subbase and "scratch" the surface, that would indicate an area with a rise in the subbase and where the concrete pavement would have insufficient thickness. If there is a high spot in the subbase, then that area needs to be milled down, or more commonly, the side forms need to be shimmed up to achieve proper concrete pavement thickness. Subbase planers were used in the past with side form paving to excavate excess subbase material. These planers were suitable for use on non-stabilized subbase materials. Any milling of stabilized subbase should be approved by the engineer to ensure that no damage to the subbase or deficient subbase thickness result.

Paving Operations

As with any paving method, it is important that the concrete be discharged, consolidated, and finished to provide optimum ride quality and long-term performance. Equipment used for placing and finishing concrete in fixed form paving varies substantially from project to project, and detailed descriptions of the placing and finishing operations are not provided in this document.

One of the key elements in fixed-form paving is to maintain a consistent and uniform head of concrete in front of the strike-off screed. The strike-off screed used in fixed-form paving is lighter than the one used in slip-form paving, and too much variation in the head of concrete will result in reduced smoothness of the concrete surface. A head of concrete that does not run over forms or the screed works best. Also, steady machine progress improves pavement smoothness and concrete should be delivered to ensure steady machine progress.

Finishing operations such as floating, burlap/carpet drag, tining, and curing operations are similar to those in slip-form paving, and they are described under "Slip-form Paving".

Figure 9-26 shows how a vibrating screed is pulled forward with a winch at each end of the screed connected to a cable hooked ahead of the screed. A manually operated finishing screed shall be a strike template and a tamping template or a vibratory screed with adequate length to cover the width of the slab.

Figure 9-26. Vibrating Screed.

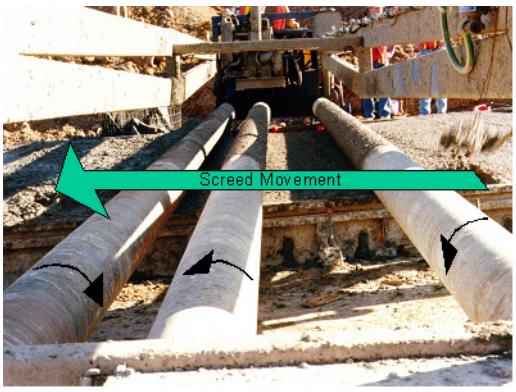

Figure 9-27. A Clary or Roller Screed.

Figure 9-28. Knock down the ridge of mortar formed by the Clary screed.

When the concrete is setting up on the screed and the screed needs to be cleaned, have the contractor move the screed ahead of the concrete so that cleaning water does not fall on the plastic concrete. Figure 9-29 shows the cleaning of the screed between haul trucks.

Figure 9-29. Cleaning screed between haul trucks.

For finishing, texturing, and curing operations, please refer to "Slip-form Paving" below.

Removing Forms

Forms can be removed as early as practical without damaging the concrete. In most instances, forms can be removed within 6 to 8 hr. after concrete placement. The forms shall be carefully removed in such a manner that minimal or no damage will be done to the edge of the pavement. All damage resulting from this operation and any honeycombed areas shall be repaired with a mortar mix within 24 hr. after form removal. Immediately after removing the side forms from the concrete pavement, apply membrane cure to all concrete surfaces not previously treated. Forms should be cleaned right after removal. If not, they become difficult to clean. Figure 9-30 shows removed side forms.

Figure 9-30. Removed side forms.

Slip-form Paving

Slip-form paving is the most widely used paving method in modern concrete paving construction. The slip-form paver consolidates screeds and initially finishes the concrete in one continuous operation without the need of forms. Modern slip-form paving equipment is of the extrusion-type process. The extrusion process can be simply defined as forcing, pressing, or pushing a material

through a die or mold to create the desired shape. Slip-form paving is very efficient and can provide smooth concrete pavement. Because there are no forms, the plastic concrete must be able to hold the pavement edge. When slip-forming, the desired slump of the concrete is 1-1/2 in.

Alignment and Grade

The "stringline" is actually a slender wire rope. It is usually tensioned fairly high to reduce sags in the wire between the supports. A sensor on the paving machine will follow the stringline and any sags will show up in the final ride surface as waves that can produce an unpleasant ride quality.

A stringline is established to control the slip-form paving equipment at the proper grade and alignment. A surveying crew establishes a stake line every 25 or 50 ft. along, but offset from the edge of the pavement to be placed. Wood stakes used for this line are about 1-1/2 to 2 in. square, commonly called hubs, and are driven into the ground. The alignment is then established with a tack in the top of each stake. Grade is also established by elevation grading of each stake. For hard, dense subgrade such as an asphalt bond breaker, nails driven into the subgrade are used instead of wooden stakes with tacks. Using the line and grade from this survey line, a stringline is established for the slip-form paving machine to follow utilizing the machine's electronic grade and alignment controls. Usually another graded stake line is established along, but offset from, the other edge of the pavement to control the grade of that side of the paving machine; however, alignment is only controlled from one stringline.

Overview of Slip-form Paver

Slip-form paving is accomplished by the use of several self-propelled machines in a line which is commonly known as a paving train.

The first machine in line is a concrete placing machine and, depending on the manufacturer, this machine is sometimes referred to as a placer/spreader. This machine receives the mixed concrete from the delivery vehicles and places and spreads the concrete in front of the second machine which is the slip-form paver. Sometimes this first machine is eliminated if the concrete can be deposited directly in front of the slip-form paver from the concrete delivery units or if a concrete placing attachment is installed in the front of the slip-form paver.

The slip-form paver spreads the concrete uniformly across the paving area with an auger, consolidates the concrete with spud vibrators, and strikes off the top of the concrete to a suitable elevation to feed into the mold that shapes the pavement into the proper geometric configuration. Depending on the manufacturer, some slip-form pavers also utilize what is known as a tamping bar. The tamping bar slightly tamps large aggregates into the top of the concrete slab to prevent the paver's mold from snagging the aggregate and causing a tear in the top of the slab.

The third machine in the train is a tube float. This machine smoothes and seals the top of the pavement by dragging diagonally mounted aluminum tubes forward and back along the top of the

pavement. This machine is sometimes eliminated by attaching what is known as an auto float to the back of the slip-form paver. The auto float automatically travels across the top of the pavement while simultaneously oscillating in a forward and back motion to smooth and seal the top of the pavement.

The last machine in the train is a combination tine/cure machine. This machine installs the tining in the pavement top with a metal comb that is automatically dragged across the top of the pavement. This machine is then used to spray the curing compound on the pavement. Sometimes a second curing machine is required if the tine/curing machine can't perform the curing operation in a timely manner. Either the tube float or the tine/cure machine is also used to install any required texture such as a burlap or carpet drag texture after all finishing is completed and prior to any tining texture required.

Slip-form pavers contain various combinations of all or some of the following components: auger spreader, spud vibrators, oscillating screeds, clary screed, tamping bars, and pan floats. A slip form paver is shown in Figure 9-31.

Slip-form pavers are equipped to spread the concrete uniformly and strike off the concrete to the required section, using a power driven device, either a reciprocating blade, a screw conveyor (auger), or a belt conveyor, without loss of traction. A slip-form paver with two augers is shown in Figure 9-32.

Figure 9-31. Slip-form Paver.

Figure 9-32. Slip-form paver with two augers (idle).

A machine with two tracks is steered by varying the speed of the tracks from one side to the other side. The bigger machines with four tracks, two tracks on each side, are stirred by pivoting each track, much like the front wheels on a car. The tracks can either ride on the base or on a previously placed pavement. Figure 9-33 shows a paver machine on tracks.

Figure 9-33. Paver machine on tracks.

The base should extend out, past the paver's tracks, to give adequate support for the machine. This is illustrated in Figure 9-34. The track path should be level and clean to allow for a smooth concrete surface. Do not force the grade control sensor to makeup for trash, such as spilled concrete or tie bars, in the track path.

Figure 9-34. Track Path.

Vertical Alignment Control

Usually, slip-form pavers have an electronic sensor system or equivalent to provide grade control. Figure 9-35 and Figure 9-36 show examples of such sensors. Sensors come in both electronic and hydraulic models. The electronic sensors work better.

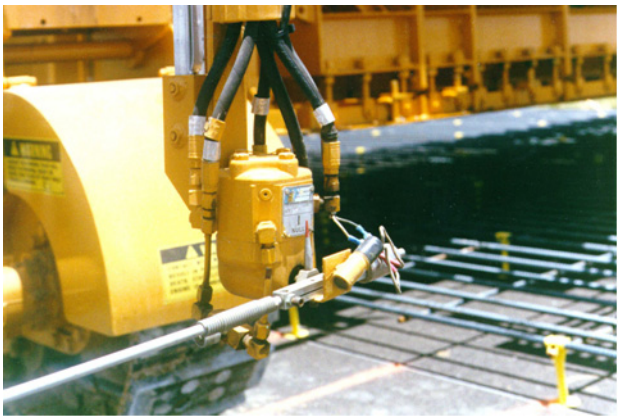

Figure 9-35. Electronic Sensor.

Figure 9-36. Electronic Sensor.

The grade control sensors, one on each side of the paver, have wands that ride on the bottom of a guide wire that adjusts the height of the entire machine by raising or lowering the vertical hydraulic cylinders on each side of the machine. Figure 9-37 shows vertical and horizontal sensors. The guide wire is supported and tensioned to prevent any measurable sag. Figure 9-38 shows the side view of an idle paver. Notice the two hydraulic cylinders on the side of the paver (right side of Figure 9-38). Figure 9-39 shows the guide wire controls.

Figure 9-37. Horizontal and Vertical Sensors.

Figure 9-38. Side view of an idle paver.

Figure 9-39. Guide Wire Controls.

Figure 9-40. Another view of guide wire controls.

Horizontal Alignment

The alignment or steering of the paver can be either sensor controlled or operator steered.

Paver's Forward Speed

The speed the paver moves forward is controlled by the operator. The speed should be as uniform as possible, but should vary with the rate of concrete delivery so that complete stops are held to a minimum. The paver speed is up to 20 ft. per minute.

If the paver stops moving forward, the vibrators must be turned off within five seconds. Continued running of the vibrators will result in segregation of the concrete, forcing water and fines to the surface.

Augers

Rotating the augers moves the concrete sideways to the entire lane being paved. The augers are controlled by the operator and can be turned either clockwise or counterclockwise as needed. Figure 9-41 shows a close-up of an auger.

The augers on the spreader of a paving machine help distribute the concrete over the width of the pavement placement ensuring that a uniform head of concrete is maintained across the pavement width and extending well above the vibrators. This overburden over the vibrators helps direct the energy from the vibrators downward into the concrete pavement and helps ensure proper consolidation of the concrete. Figure 9-42 shows the spreading of the concrete.

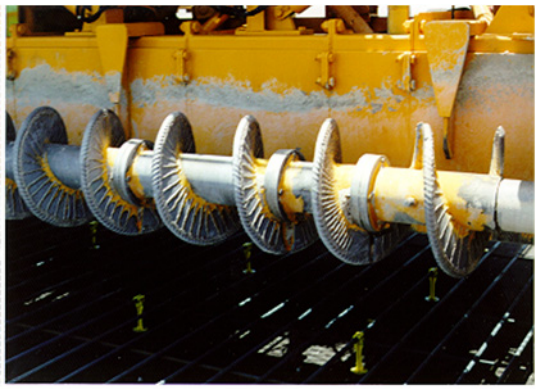

Figure 9-41. Close-up of an auger.

Figure 9-42. Auger and strike-off spreading the concrete.

Vibrators

The frequency in air of the immersion vibrator units shall be not less than 8,000 cycles per minute. Figure 9-43 shows immersion of vibrators. Handheld vibrators are driven by an electric motor.

Vibrators on a slip-form paver are hydraulically driven. Figure 9-44 shows the underside of a slip-form. Going from right to left observe the:

1. augers
2. strike-off screed
3. vibrators
4. finishing screed
5. pan float.

Vibrators need to be mounted high enough that they will not snag any of the reinforcing steel. An isolated streak in the concrete pavement behind the paving machine may indicate that there is a non-working vibrator.

Figure 9-43. Immersion Vibrators.

Figure 9-44. Underside of slip form paver.

Figure 9-45 shows the normally recommended vibration position. The paver is moving to the left. A concrete head or surcharge should be maintained over the vibrators during placing operations. The vibrators must be turned off whenever the forward motion of the paver is stopped. Figure 9-46 shows the vibrated zones.

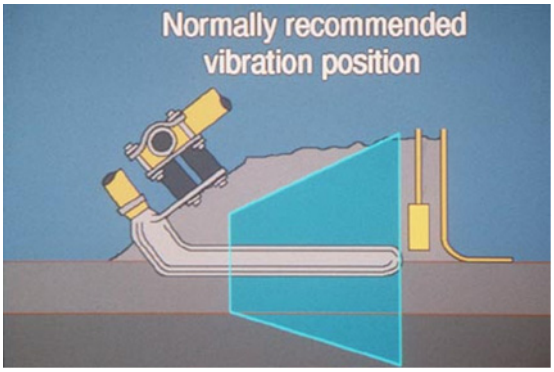

Figure 9-45. Normally recommended vibration position.

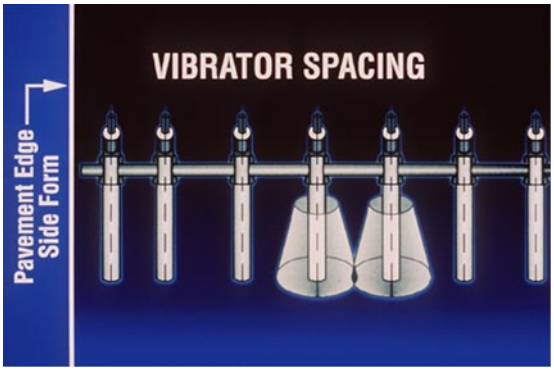

Figure 9-46. Drawings showing the vibrated zone.

No vibrator streaks or trails should be apparent behind the paver. Figure 9-47 shows unacceptable vibrator streaks. Here, the vibrators are not vibrating enough. Because of this, the vibrators are plowing a trail instead of vibrating the concrete.

Figure 9-47. Unacceptable – vibrator streaks.

Pan Float

After screeding, the concrete surface is finished with a pan float that further smoothes and consolidates the concrete. Pan floats are solid plates anywhere from 18 to 60 in. wide and slightly narrower than the pavement width. Figure 9-48 shows pan floats on an idle paver.

Figure 9-48. Looking up at a pan float on an idle paver.

Inserting One-piece Tie Bars into the Edge

The tie bars are inserted manually or mechanically in the concrete while it is still in the slip-form paver. Larger finishing machines have the capacity to mechanically insert the tie bars evenly with little disturbance to the plastic concrete. Insertion by hand often causes the edge of the plastic concrete to drop down and requires constant remedial work. Figure 9-49 shows a surface drop down due to hand insertion.

Figure 9-49. Surface drop down due to hand insertion.

Check tie bar dimensions and locations often. Care must be taken to prevent any movement of the tie bars after placement. Figure 9-50 shows an example of a wood side form at a transverse construction joint. The tie bars at this side form location were not inserted into the plastic concrete. A hole will be drilled and filled with epoxy grout for each tie bar not originally placed.

Figure 9-50. Wood side form at a transverse construction joint.

Finishing Operations

Sealing the Surface of the Concrete

To close all surface openings and produce a uniformly smooth and flat surface, the plastic concrete behind the screed is either floated or straight edged, or both. A float may be used only when using a finishing machine. Figure 9-51 shows an example of tube floats. Tube floats are pulled back and forth in the longitudinal direction. Notice that each tube float is suspended and pulled by chains. The tube floats are set at 60 degree angles to the pavement's centerline. The leading end of the tube float may be on either the inside or outside of the pavement. The tube floats must extend across the entire width of the pavement being placed. For wide concrete placements, more than one tube is used.

Figure 9-51. Tube Floats.

Tube float machines must have spray nozzles to provide a fine, light fog mist to the concrete surface just ahead of the tubes.

A longitudinal float, also called auto-float, slides back and forth in the transverse direction. An example of a longitudinal float is shown in Figure 9-52. Notice the surface marks left by the float.

Another type of float is the drag float, an example of which is shown in Figure 9-53. Because a drag float is heavier than a tube float or a longitudinal float, the drag float applies greater surface pressure, making it more effective at smoothing out a knot or bump in the plastic concrete. A drag float needs to be more effective because it only makes a single pass across the plastic concrete.

Figure 9-52. Longitudinal Float.

Figure 9-53. "V" shaped drag float pulled by paver.

When using a drag float, be careful that the float does not slowly slide to the downhill side when paving a crowned or superelevated section.

When the float can not reach to the edges, the edges are floated with hand trowels. Such a process is shown in Figure 9-54.

Figure 9-54. Hand Trowel.

Care must be exercised while floating to avoid distorting the surface. A bull float (Figure 9-55) can produce a depression if excessively used in one spot. Longer bull floats will produce better results. Bull floating should be limited to small areas and should not be used to float the entire surface. Avoid using a bull float, if possible.

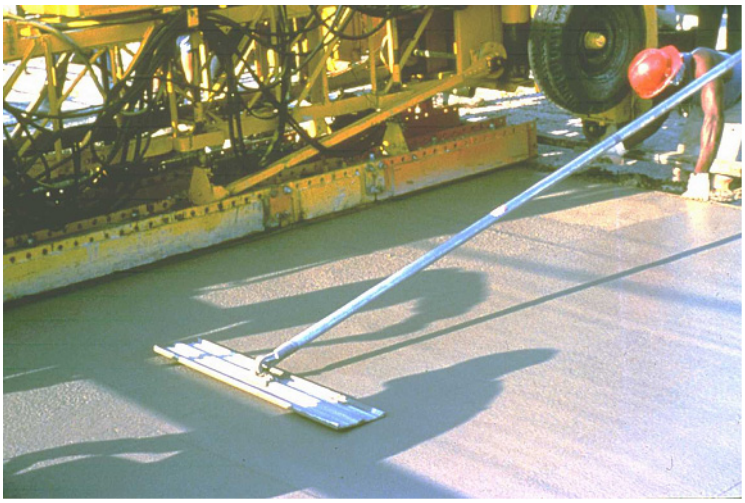

Figure 9-55. Bull Float.

Bleed Water or Free Surface Water

Any finishing should be held to a minimum during the period of greatest bleeding since working the surface in the presence of excess water leaches out a portion of the cement and produces low-strength surface mortar. In the 2004 Specifications, the maximum allowable water cement ratio has been reduced to 0.45 from 0.5. It is expected that the reduced water cement ratio will result in less bleed water. Also, the use of supplementary cementious materials such as fly ash and ground granulated blast furnace slag reduces bleeding water.

Under calm wind and high relative humidity conditions, the concrete pavement should provide enough bleed water that little or no water needs to be added to float and finish the concrete surface.

Under very dry conditions or when there is a strong breeze, the bleed water may evaporate rapidly before floating can be performed. A fine mist of water may be used to reduce the loss from evaporation, but never to form slurry. The fine mist should come from a misting nozzle mounted on a pipe frame that is not pointed directly at the pavement surface. Water should never be applied from a nozzle or thumb over a garden type hose.

If liquid mortar slurry is running over the edges of the pavement, no additional water should be applied. If possible, the water content of the paving mix should be reduced to control this free water. Other states have reported contractors developing mortar slurry on the surface to help improve the ride quality and receive a higher payment. Although this can help improve initial ride quality, it will result in a weak surface concrete prone to early age deterioration. This practice should be strongly discouraged.

No finishing operation should be conducted when free water is on the surface of the concrete (see Figure 9-56). Finishing with water on the surface will cause the cement at the surface of the concrete to become diluted or even washed away as shown here.Wait until the water evaporates before continuing with the finishing.

Today, the major source of surface water is the external supply of water as shown in Figure 9-57. This practice is detrimental to the durability of the pavement surface.

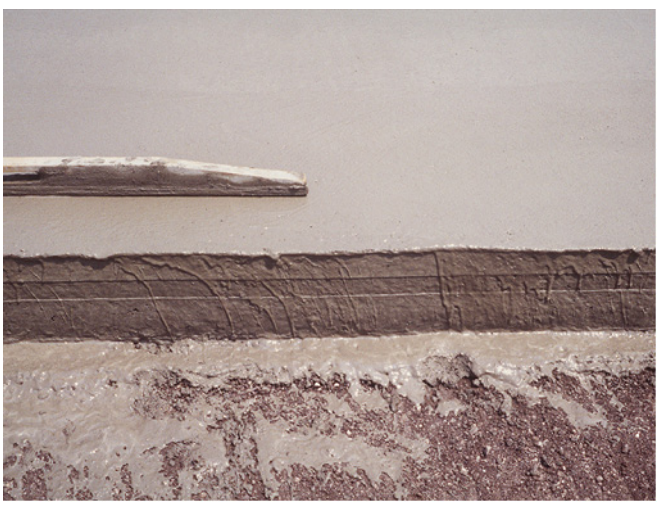

Figure 9-56. Unacceptable – finishing with free surface water.

Figure 9-57. Unacceptable – water being added.

Finishing with water on the surface makes the finishing easy to perform, so finishers tend to add water. This practice dilutes and washes away the surface mortar, resulting in poor performance of concrete pavement in terms of skid resistance, scaling, durability, and others. This practice should be severely discouraged.

Straightedge Finishing

A straightedge is a 10-ft. or 15-ft. long, steel or magnesium, square tube, which contacts the concrete surface. An illustration of straightedge finishing is shown in Figure 9-58, Figure 9-59, Figure 9-60, and Figure 9-61. Straightedging flattens the concrete surface and in the case where floats were not previously used, can close the surface tears produced by a screed. A straightedge is better at flattening surface bumps and seams than a float, but straightedging takes longer than floating because straightedging cannot be mechanized. The straightedges should be parallel to the centerline of the pavement.

Figure 9-58. Straightedge Finishing.

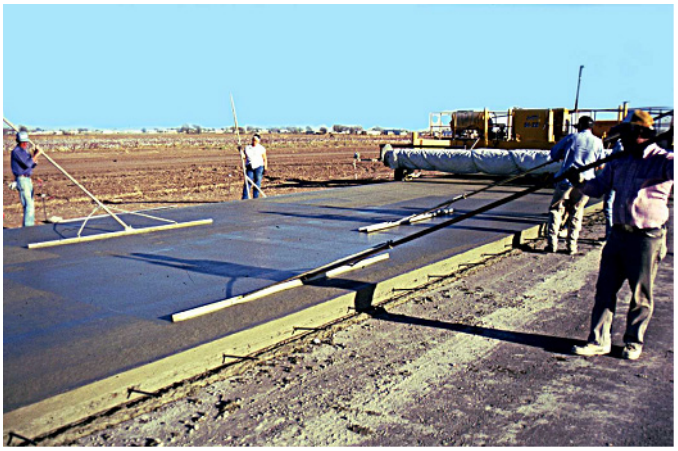

Figure 9-59. Pulling the straightedge – keeping the handle high.

Figure 9-60. Pushing the straightedge – keeping the handle low.

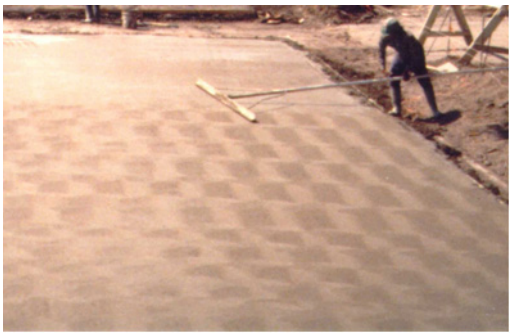

Figure 9-61. Straightedging behind a longitudinal float.

Notice in Figure 9-61 that the straightedged pavement no longer shows the irregularities left by the longitudinal float. Once the finishing operation is completed, measures should be taken to keep the surface from drying before the curing compound is applied. It may take a while for the concrete to set up sufficiently after float finishing before the tine texturing can be performed. This delay may increase if the concrete temperature is cool or a high dose of a retardant admixture is included in the concrete mix. The evaporation retardant is applied in a fine mist to uniformly cover the pavement surface. An important property of the evaporation retardant, in addition to reducing evaporation, is that it is specifically formulated to dissolve harmlessly into the concrete when further finishing of the surface occurs.

The evaporation retardant (see "DMS-4650, Hydraulic Cement Concrete Curing and Evaporation Retardants") should be applied according to the manufacturer's recommendation when finishing operations cease and maintained until the final texturing occurs. This may be before or after the carpet drag is performed.

Historically, TxDOT has required the contractor to have a small quantity of evaporation retardant (typically in a small garden type sprayer) on the project to be used if there was a temporary breakdown of the paving machine or a brief disruption in the delivery of concrete. In these circumstances, the contractor would apply the evaporation retardant to the mass of concrete in front of the paving machine to reduce evaporation and keep a crust from forming on the concrete until paving could resume.

Texturing Operations

Surface texture is required to provide skid resistance and to prevent hydroplaning. Two types of texturing are provided:

- carpet drag (Astroturf), and
- tining.

Item 360 requires both types of texturing to be provided. However, special provision allows carpet drag only on the roadways with design speeds of 45 mph or less as long as the average texture depth is not less than 0.04 in. Figure 9-62 shows an upside down Astroturf carpet pulled longitudi-

nally on the plastic concrete. This gives the concrete surface a rough texture similar to a broom finish (Texture Requirements for Pavement Surfaces with Design Speeds of 45 Miles per Hour or Less).

Figure 9-62. Astroturf carpet drag.

The Astroturf shall be a single piece of carpet long enough to reach across the width of the plastic concrete and capable of contacting 4 longitudinal ft. of the concrete at a time. The engineer may allow changes in the length and width of the Astroturf to accommodate specific applications.

The Astroturf shall be molded polyethylene pile face carpet with a blade length of 5/8 to 1 in. and a minimum weight of 70 oz./yd.^{2}. The backing shall be a strong, durable material not subject to rot, and shall be adequately bonded to the facing to withstand dragging on plastic concrete.

To keep the mortar from hardening on the Astroturf, the Astroturf should first be wetted before using and it should be cleaned with water often enough to produce a uniform finish on the concrete. Streaking of the surface behind the Astroturf is caused by hardened grout on the Astroturf. The contractor should stop the drag as soon as the streaking is observed and remove any dried grout from the Astroturf.

Figure 9-63 shows the mounting of an Astroturf. The Astroturf shall be mounted on a moveable support system capable of varying the area of carpet in contact with the pavement and can be pulled either mechanically or manually.

Figure 9-64 illustrates unacceptable Astroturf contact with the concrete. Here, a corner of the Astroturf wants to roll up. To keep the Astroturf in contact with the concrete, the trailing end of the Astroturf may be weighted with metal pipe or a 2 × 4.

Figure 9-63. Mounting of Astroturf carpet.

Figure 9-64. Unacceptable Astroturf carpet contact.

Keep an eye on the drag - it can indicate some things about the concrete. If it starts to catch and pull aggregates to the surface instead of leaving a uniform texture, there are two things that could be wrong:

1. The concrete might not be properly vibrated. Check the vibrators to ensure that all concrete is being vibrated.
2. There may not be enough sand in the mix to allow grout to be worked to the surface.

Tining

The primary objective of tining is to provide resistance to hydroplaning. Figure 9-65 shows a tined surface. The metal tines should make **transverse** grooves in the plastic concrete which are 3/16 in. deep, 1/12 in. wide and spaced an average of 1 in. part. The tining spacing has a lot to do with the noise level. The minimum groove depth is 1/8 in.

Figure 9-65. Tined Surface.

To ensure that the grooves are straight, the tines should be pulled by a mechanized device. The mechanical device is designed to tine up to a 24 ft. wide concrete pour. Figure 9-66 shows a mechanical tining device.

Figure 9-66. Mechanical Tining Device.

Curing the Concrete Pavement

Curing is the process by which hydraulic cement concrete matures and develops hardened properties over time as a result of the continued hydration of the cement in the presence of sufficient water and heat. In concrete pavement, the surface area is relatively large compared with the volume. Moisture can be evaporated from this large surface area unless measures are taken to keep the moisture in the concrete. With loss of moisture, the surface concrete will not develop enough strength and could result in scaling, spalling, and other distresses. It is important that proper curing is provided. The most widely used curing method is the use of curing compound.

Curing Compound

Curing compound should meet the requirements of Type 2 membrane curing compound in "DMS-4650, Hydraulic Cement Concrete Curing and Evaporation Retardants." Curing compound is similar to paint and must be thoroughly agitated just before using. The curing compound should not be

thinned or diluted. The curing compound should be re-agitated during its use if it becomes non-uniform.

Two separate coats of curing compound are to be applied to all exposed concrete surfaces. Applying the membrane cure in two coats has several advantages over a single coat:

- it results in a thicker film that better resists evaporation,
- it reduces rundown of the curing compound into the tine grooves that might occur in a single heavy coat, and
- it helps ensure a more even coating of the concrete surface if one or more of the spray nozzles were partially obstructed.

The first and second coats should be applied within 10 min. and 30 min. after completing texturing operations, respectively. The individual application rate should be not more than 180 ft^2/gal.

Where the coatings show discontinuities, pinholes, or other defects, or if rain damages the newly applied curing compound, an additional coat of the compound shall be applied.

Once dried, the curing compound will bead water. If the water does not bead but instead wets the surface of the concrete as determined by darkening the concrete, or by visibly soaking into the surface, an additional coat or coats of the curing compound should be applied immediately.

Should the membrane be damaged from any cause before the expiration of 72 hr., the damaged portions should be repaired immediately with additional compound.

A mechanized spraying unit (Figure 9-67) is essential when more than one lane of pavement is placed. Note that the above machine is applying curing compound to the vertical edge of the concrete as well.

Figure 9-67. Mechanized Spraying Unit.

Figure 9-68 shows another mechanized unit for spraying curing compound. Note the uniform and complete coverage of the curing compound, even on the side of the concrete.

Figure 9-68. Mechanized Spraying Unit.

Figure 9-69 illustrates unacceptable application of curing compounds. In this figure, some areas are whiter than others, indicating that curing compound has not been applied uniformly.

Figure 9-69. Unacceptable – non-uniform application of curing compounds.

Plastic shrinkage cracks occurred in the pavement section shown in Figure 9-70 within 5 hr. of concrete placement, which will cause durability problems.

Figure 9-70. Plastic Shrinkage Cracks.

Figure 9-71 shows that the tie bars have been covered with a plastic sleeve to prevent curing compound from dropping on the tie bars. Any curing compound that ends up on the tie bars or on the "U's" for the curbs will need to be removed with a steel brush or by sand blasting.

Figure 9-71. Preventing curing compounds from dropping on tie bars.

Section 7 — Joints

Concrete undergoes volume changes due to temperature and moisture changes. If these volume changes are not controlled properly, excessive stresses will develop, resulting in uncontrolled cracks. These cracks can be sources of distress and need to be controlled. Joints can be considered as intentional cracks and are provided where the cracking is most likely. These joints relieve stresses, thus preventing uncontrolled cracks. Provisions are made at the joints to provide wheel load transfer.

In CRCP, however, the concrete volume changes are controlled by random cracks, which are tightly held by longitudinal reinforcing steel. Figure 9-72 shows an example of random cracks in CRCP. These random cracks do not cause distress.

Figure 9-72. Random cracks in CRCP.

There are four types of joints:

1. transverse contraction joint
2. warping joint
3. construction joint
4. expansion joint.

Contraction joint is to relieve tensile stresses resulting from temperature drops and moisture variations in concrete. An example is shown in Figure 9-73. This type of joint is used only in plain jointed pavement (JCP or also called, in Texas, concrete pavement contraction design [CPCD]) at the transverse joint. In TxDOT, transverse contraction joints are provided every 15 ft. TxDOT's design standards for JCP can be found in ftp://ftp.dot.state.tx.us/pub/txdotinfo/cmd/ cserve/standard/roadway/cpcd94.pdf.

Figure 9-73. Transverse Contraction Joint.

Problems can occur when contraction joints are sawed too late or too shallow. Attempts to saw-cut too early might cause raveling of aggregates, while too late saw-cutting might result in uncontrolled transverse cracking. There are a number of factors that determine the optimum time of saw-cutting. They include, but are not limited to, setting characteristics of concrete, ambient and concrete temperatures, and thermal coefficient of concrete, to name a few. Even though it is possible to theoretically determine the optimum time for saw-cutting, it is almost impossible to do at the job site due to so many variables involved. The best strategy is to saw-cut as early as possible without severe raveling. Item 360 allows some minor raveling due to early saw-cut.

Dowel bars are provided at the contraction joints to provide load transfer, which reduces deflections and stresses due to wheel load applications. To allow the slabs to move freely in the longitudinal direction due to temperature and moisture variations while providing efficient load transfer, dowel bars are lubricated at least one-half of their length.

The details requirements of dowels, such as dowel bar size, length, and spacing, are shown in the Standards CPCD-94. Figure 9-74 shows a picture of a dowel bar assembly.

Figure 9-74. Dowel Bars.

Dowel bars are typically placed in heavy welded wire baskets that hold the bars in proper spacing, depth, and alignment. Full lane width baskets are typically used, although baskets that are one-half of the lane width have also been used. These wire baskets are shop fabricated. Typically, one end of the dowel bars is tack welded to the wire basket. The wire basket is nailed to the subbase to prevent the basket from being pushed along the subbase once submerged under concrete and under the paving machine.

Alternatively, dowel bars may be mechanically placed by the paving machine. When this method is used, it is important to ensure that the bars are properly aligned and positioned. When this equipment is first used, a few bars should be carefully exposed using a small trowel to verify alignment and positioning. This is done from a work bridge behind the paving machine.

Whether wire baskets or mechanical inserters are used, it is extremely important the dowel bars be parallel for the contraction joint to function properly.

Concrete saws are self-propelled and rotate their blades in a cut-down direction, unlike a handheld circular saw which rotates its blade in a cut-up direction (see Figure 9-75). Most concrete saws use diamond-tipped saw blades which require water to cool and lubricate the blade.

Figure 9-75. Riding Saw.

Some saws are riding saws, like a riding lawnmower, and others are self-propelled walk behinds (Figure 9-76). Some newer saw models do not require the use of water and can actually saw the concrete earlier than the concrete saws shown here.

Figure 9-76. Walk-behind Saw.

Figure 9-77 shows that the initial saw-cut was being made from the top of the figure to the bottom. For some reason, the initial saw-cutting was delayed. When the saw was halfway across the pavement, the concrete cracked ahead of the saw.

Even the crack is sealed and allowed to stay, the concrete between the crack and the saw-cut will eventually spall out. To prevent fruther uncontrolled cracking, every second or third transverse joint should be sawed first then the intermediate joints. This is called skip-sawing.

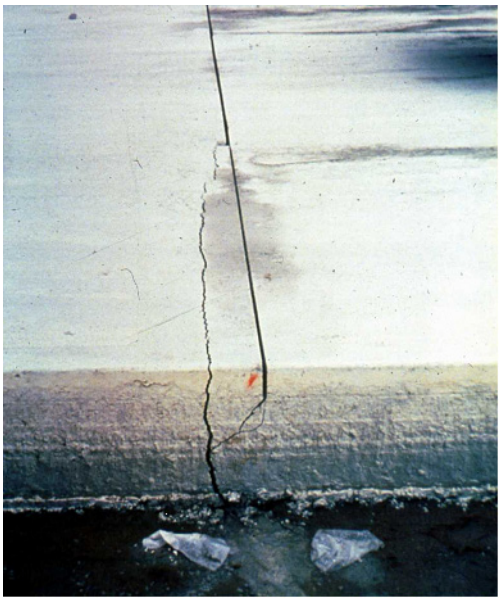

Figure 9-77. Uncontrolled crack developed while initial sawing. The left side of the slab needs to be removed and replaced.

Depth of the joints also plays a role in ensuring the proper working of the joints. Shallower cuts than specified will increase the chances of cracks somewhere else. Currently, TxDOT requires a minimum of 1/3 of slab thickness when siliceous coarse aggregate is used, and 1/4 of slab thickness when limestone coarse aggregate is used. The reason for this difference is that concrete with siliceous river gravel coarse aggregate has higher coefficient of thermal expansion (CTE) and modulus of elasticity than concrete with limestone coarse aggregates. For given temperature variations, concrete with higher thermal coefficient and modulus of elasticity result in higher stresses compared with concrete with lower values of CTE and modulus. Deeper saw-cut is needed to relieve these higher stresses. Item 360 states that "Saw joints to the depth shown on the plans as soon as sawing can be accomplished without damage to the pavement regardless of time of day or weather conditions." Once the saw-cut is completed, it is cleaned and sealant materials are applied. TxDOT standards for sealing joints are found in ftp://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/standard/roadway/js94.pdf.

Figure 9-78 shows a pavement that has 13-in. slab thickness with siliceous river gravel coarse aggegate, which will require a saw-cut depth of 4.3 in. The actual saw-cut depth is about 2.5 in. As a result, a crack was not induced.

Figure 9-78. Crack was not induced due to shallower saw-cut depth than required.

In normal concrete pavement construction, the width of the concrete placement varies. It can be as wide as 50 ft. If the width of the slab is more than 15 ft., the chances of cracking, in longitudinal direction, increase. TxDOT standards require that when the concrete placement width is more than 15 ft., warping joint be provided in the longitudinal direction. Otherwise, longitudinal cracks occur and there are two primary causes for these uncontrolled cracks: late saw-cutting or shallow saw cut depth (as in contraction joints).

Transverse construction joints are needed when the paving operation is stopped in the middle of a lane. A stoppage can occur because of the end of the workday, or because of a mechanical breakdown.

When the placing of concrete is stopped, a bulkhead (header) of sufficient cross sectional area to prevent deflection, accurately notched to receive the load transmission devices and shaped accurately to the cross section of the pavement needs to be provided (Figure 9-79). At transverse construction joints, additonal longitudinal reinforcing steel is required (Figure 9-80).

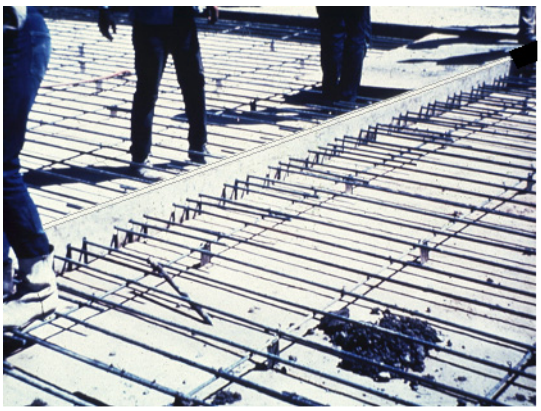

Figure 9-79. Bulkhead (Header).

Figure 9-80. Additional steel is required at header.

When two adjoining lanes are placed with separate concrete placements, the joint between the two placements is called a longitudinal construction joint. Across longitudinal construction joints are short pieces of reinforcing steel called tie bars. The tie bars keep the two adjoining slabs from pulling away from each other and keep the surface across the joints flat. In Item 360, either single piece tie bars or multiple piece tie bars are allowed at the longitudinal construction joints. The detailed requirements for tie bars, such as length, size, and spacing are shown in the Standards CPCD-94.

One piece tie bars may be acceptable when they don't interfere with traffic or construction conditions.

Multiple piece tie bars are threaded in the middle to allow the two halves of the tie bar to be separated and then reconnected only prior to the casting of the second pavement. The tie bars shown in standard plan sheets CPCR(1)-03 and CPCR(2)-03 for continuously reinforced concrete pavement specify multiple piece tie bars. Unless shown on the plans, the spacing for multiple piece tie bars shall be equal to or less than that of the transverse bars.

Multiple piece tie bars shall develop a tensile strength over their entire length equal to 1-1/4 times the yield strength of the tie bars shown on the plans. Each end of multiple piece tie bars shall consist of deformed reinforcement of at least the size shown on the plans.

In the past, bent tie bars were commonly used to keep the tie bars from interfering with traffic or construction. This practice is no longer allowed, since there were problems such as lane separations.

Notice in Figure 9-81 that the spacing of the transverse reinforcing steel and the spacing of the tie bars match.

If the tie bars are inserted exactly at the transverse bar location, there is a higher chance of hitting the transverse bars. It would be a better practice to insert tie bars a few inches away from the transverse bar locations.

In Figure 9-82 half the tie bar length was embedded into the concrete. The other half sticks out, ready to be cast in the adjacent concrete placement. These tie bars were manually inserted into the plastic concrete during the paving operation.

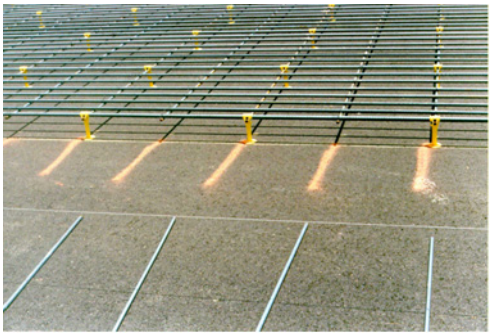

Figure 9-81. Before paving. Tie bars (laying on base in lower part of picture) will be placed at the paintmarks during paving.

Figure 9-82. After paving.

Tie bars are also used in plain jointed concrete pavement. When these tie bars are used to connect two lanes, or a lane and shoulder, the tie bar may be mechanically inserted by the paving machine into the fresh concrete ahead of the float or it may be held in position by support devices. Unlike CRCP, there is not an available mat of steel reinforcement that can be used to position and secure the tie bars. Each tie bar will need to be separately supported and held in position by stakes or pins driven into the subbase. These supports need to be sturdy enough to hold the tie bar in position when concrete is placed and consolidated over it.

Expansion joints are installed to provide space for concrete to expand without causing excessive compression in concrete, which could result in blowups or structural damages in bridges. These joints are explained in “Terminal Anchors” described below.

Terminal Anchors

Concrete experiences volume changes due to temperature and moisture variations. When the temperature goes down, concrete contracts and transverse contraction joints in CPCD and transverse cracks in CRCP open up and could allow incompressible foreign material to get into joints or

cracks. This process can continue and more material could accumulate. When the temperature goes up in the summer, concrete expands and the incompressible material in the joints or cracks does not provide the space that once existed to allow for concrete volume changes. The result is the expansion of the total length of the concrete pavement, called "pavement growth." Normally, concrete pavement is abutted to the bridge structures via approach slabs and if this "pavement growth" is not contained, the approach slabs will be pushed and serious damage could occur to the bridge structures. To prevent this problem, some form of system is installed at the bridge and pavement interface. Currently, there are three systems in use at TxDOT:

- expansion joint system
- wide-flange system
- anchor lug system.

The basic premise of the expansion joint system is that the expansion joint width will be able to absorb any concrete pavement expansion without transmitting the compression forces to the bridge structure. A typical design standard for expansion joint system can be found at ftp://ftp.dot.state.tx.us/pub/txdot-info/ftw/specinfo/cptdfw.pdf.

Wide flange system is similar to the expansion system, except that the expansion joints exist under the wide flange and are not seen from the pavement surface. One advantage is that, since the joint is not exposed to the pavement surface, joint maintenance is minimized. Typical design standard for wide-flange system can be found at ftp://ftp.dot.state.tx.us/pub/txdot-info/cst/pdm/stdb3_all.pdf.

Anchor lug system tries to contain the concrete movement at the interface between bridge and pavement by providing five anchor lugs. The design standard for this system can be found at ftp://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/standard/roadway/tacp99.pdf. In all three systems, expansion joints are provided.

At this point, it is not known which of the three systems is the most efficient in minimizing damage to the bridge structures. There will be a research project that will compare the effectiveness of these three systems. It is expected that, based on the findings from the research study, TxDOT will develop one system for statewide use.

Chapter 10 — Rigid Pavement Rehabilitation

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Section 1 — Overview

Rigid pavement, if designed and built properly, provides excellent long-term performance with little maintenance required. However, for various reasons, distresses take place and repair/rehabilitation becomes necessary. Compared with repair/rehabilitation of flexible pavement, rigid pavement repair/rehabilitation costs more and if not done properly, similar distresses will develop and additional repair/rehabilitation will be required. Developing optimum repair/rehabilitation strategies requires the understanding of the causes and mechanisms of the distress. The number of distress types in rigid pavement is limited and the causes are fairly well understood.

In this chapter, pavement distress types that require specific repair/rehabilitation strategies are described along with repair procedures. Not much research has been done on the effectiveness of various repair/rehabilitation strategies. TxDOT recognized this and recently funded several research projects on maintenance-related issues in rigid pavement. As the findings become available, this chapter will be updated.

Section 2 — Full-Depth Repair

In this chapter, Portland cement concrete (PCC) pavement distress types that require full depth repair are discussed, followed by the repair procedures.

Pavement Distress Types that Require Full Depth Repair (FDR)

In concrete pavement contraction design (CPCD), the following distresses require FDR:

- transverse cracks
- shattered slabs and corner breaks.

Transverse cracks occur due to temperature/moisture variations and/or wheel load stress that extend through the depth of a slab (but not plastic shrinkage cracks which occur when the rate of evaporation from the surface exceeds the rate at which the bleed water is available – the depth of plastic shrinkage cracks is limited to about 1 to 2 in. from the surface).

Transverse cracks in CPCD that extend through the depth of a slab are caused by inadequate slab lengths and deficient slab thickness (design issues), concrete with high coefficient of thermal expansion or modulus (materials issues), or non-uniform or insufficient base support (construction issues).

Shattered slabs and corner breaks result from insufficient slab thickness (design issue) and base support (design/construction issue).

In continuously reinforced concrete pavement (CRCP), the following distresses require FDR:

- punchouts
- deep spalling.

Punchouts in CRCP are caused by excessive wheel loading applications and insufficient structural capacity of the CRCP, such as deficient slab thickness (design issue) or subbase support (design/construction issue). It is manifest by block(s) of concrete connected by transverse and longitudinal cracks which are depressed and those block(s) are depressed. Normally, longitudinal steel at the transverse cracks of the punchouts eventually ruptures. Punchouts are the most serious distress type in CRCP. Better design and construction practices by TxDOT over the years significantly reduced the frequency of punchouts. Figure 10-1 shows typical punchouts. Note the asphalt patch was applied to restore the surface elevation, implying that the concrete block was pushed into the subbase.

Figure 10-1. Typical punchouts in CRCP.

Spalling is another distress type in CRCP. Spalling is the breaking, chipping, or fraying of concrete at the cracks. There are several causes for spalling. In Texas, spalling is more prevalent when certain types of coarse aggregates are used. The depth of spalling varies widely, from less than half an inch to as deep as half the slab thickness. Shallow spalling causes functional rather than structural problems in PCC pavement. However, deep spalling cause substantial structural damage to the pavement and requires FDR. Figure 10-2 shows deep spalling.

Unlike punchouts, it's not easy to distinguish deep versus shallow spalling. The most efficient way to identify deep spalling is by coring or non-destructive testing such as ground penetrating radar (GPR) or portable seismic pavement analyzer (PSPA).

Figure 10-2. Deep spalling.

Full Depth Repair (FDR) Procedures

Once it has been identified that FDR is required, the procedures below need to be followed:

1. Identify the repair limits
2. Saw-cut the perimeters
3. Remove the concrete slab
4. If needed, repair damaged subbase
5. Drill holes for longitudinal and transverse tie bars
6. Provide longitudinal and transverse steel continuity
7. Place concrete.

Each step is explained in more detail.

1. Identify the repair limits.

Item 361 requires the repair areas to be at least 6 ft. long and at least half a full lane width, unless otherwise shown on the plans.

It is important to properly identify the limits of the FDR needed. Repair area needs to include all areas that have developed voids under the concrete pavement. This area typically extends beyond the boundary of the failed areas. In TxDOT, it is a normal practice to determine the limits of the repair by evaluating the extent of the distress by visual observations only. Sometimes, this method does not include all the damaged area and results in pavement failures later on as shown in Figure 10-3. It illustrates that the repair limits should have been extended further to the left side.

Figure 10-3. Full depth repair of punchouts in continuously reinforced concrete pavement (CRCP).

For the FDR of extensive transverse cracking and shattered slabs in concrete pavement contraction design (CPCD), it is a normal practice to remove and replace the whole slab to ease sawing and

removal operations. Use good engineering judgment to determine the repair limits for corner breaks.

For the punchouts repair limit determination, evidence such as the extent of pumping or the use of falling weight deflectometer (FWD) testing provides useful evaluation tools. For deep spalling, coring or non-destructive testing is the most efficient evaluation method. It is recommended that the limits of FDR are somewhat extended beyond the limits determined by the evaluations.

2. Saw-cut the perimeters.

Once the FDR limits are established, saw-cut the concrete using diamond-bladed saws through the full depth of the concrete slab. Saw-cutting with diamond-bladed saws will result in a smooth cut surface with little damage to the surrounding concrete. This operation will cut all the existing reinforcing bars along the perimeter. Carbide-tooth wheel saws can cause damage to the surrounding concrete and shouldn't be used for saw-cutting for FDR.

During the summer, saw-cuts should be done in the morning while the concrete temperature is relatively low. When the temperature is high, the concrete is in compression and the diamond blade saw may bind.

Even though Item 361 allows up to seven days between saw-cutting and concrete placement, the saw-cut concrete blocks should be removed and subsequent repair operations should follow as quickly as possible. When this is not feasible, adjust saw-cut operations so that the subsequent repair operations can immediately follow. Since there is no load transfer in CRCP between saw-cut concrete blocks and the surrounding CRCP, any wheel loading applications will result in higher deflections in both saw-cut boundaries and the concrete block. These large deflections could cause damage not only in the subbase but also in the concrete, which will increase the potential for pavement distress for the full-depth repaired pavement.

3. Remove the concrete slab.

Once the repair limits are saw-cut, the concrete is removed in two ways. One is the lift-out of the slab, and the other is the breakup of the slab. At TxDOT, most of the slab removals are done using lift-out method, as it is faster and less labor intensive than the breakup method. While the breakup method could cause damage to the surrounding concrete, a properly conducted lift-out method will not damage the subbase and surrounding concrete.

To lift the slab, it is necessary to drill holes and insert pins as shown in Figure 10-4.

Figure 10-4. Slab ready for lift.

Once the lift pin arrangements are complete, cranes or front-end loaders lift the slabs vertically. The lifting should be done as vertically as possible with minimum sway, since any deviation from this can damage the surrounding concrete.

4. If needed, repair the subbase.

After the slab removal, the subbase condition must be evaluated to determine whether subbase repair work is needed. If distresses are extensive in the subbase, the subbase should be repaired with an approved subbase material to the original top of the subbase grade. TxDOT districts typically use the same subbase material type in the project for repair.

For punchouts, shatter slabs, or corner breaks, chances are that there will be some evidence of distress in the subbase. If the distress is not extensive, it is usually sufficient to remove the loose materials and compact the remaining subbase. However, if the distress is extensive, the subbase should be repaired with an approved base material to the original top of the subbase grade. Also, if the distress in the subbase extends beyond the perimeter of the FDR, a decision must be made whether further saw-cutting and additional concrete removal is needed. If the distressed subbase beyond the perimeter of the FDR is not repaired, the same type of pavement distress will take place.

For deep spalling, chances are that the subbases will be in good condition and no repairs will be needed except removal of loose materials and compacting the remaining subbase. However, if there is extensive distress in the subbase, the subbase needs to be repaired with an approved base material to the original top of the subbase grade.

5. Drill holes for longitudinal and transverse tie bars.

In concrete pavement contraction design (CPCD), dowels need to be provided in all transverse perimeters. In transverse patch joints, a minimum of four dowels are recommended for each wheel path. Holes are drilled in the existing concrete for dowels.

In continuously reinforced concrete pavement (CRCP), providing continuity of the reinforcing steel in the longitudinal direction is essential to maintaining good performance of CRCP. To provide tie bars in the existing concrete, holes are drilled in the concrete in the longitudinal direction. The length of the drill extended into the remaining concrete is important as the continuity of longitudinal steel is critical to the performance of CRCP. The spacing and depth of the holes are specified in the design standard "Full Depth Repair of Concrete Pavement: FDR(CP)-05." For transverse tie bars, the bar spacing should be the same as that for tie bars or transverse steel in the adjacent lane. Figure 10-5 shows the drilling operation using a single drill. Sometimes, a multiple drill system, called a gang drill, is used for higher efficiency.

Figure 10-5. Drilling holes at longitudinal joint.

The design standard FDR(CP)-05 requires a minimum length of the drilled hole of 12 in. and Item 361 requires the contractor to demonstrate the bond strength is sufficient. Bond strength testing in the field rarely happens. Whether this length requirement is adequate or not should be evaluated, since failures have been observed due to pull-out of the embedded rebars. Failure of bars that have been drilled and epoxied to achieve proper strength may be more related to complete filling of drill holes with epoxy than the length embedded.

6. Provide longitudinal and transverse steel continuity.

If tie bars installed at the repair perimeters are not functioning well, the potential problem of lane separation at the longitudinal joint or distresses at the transverse repair perimeters will increase. For detailed steel designs for FDR, refer to TxDOT standard, FDR(CP)-05. For FDR of CRCP, this step of providing longitudinal continuity of tie bars is of utmost importance. To achieve the optimum bond between concrete and tie bars inside drilled holes, all dust inside the holes should be completely removed. The most commonly used method to achieve this is to apply compressed air to the holes. During this operation, it is important that the air is not contaminated with grease or oil, which will weaken the bond between the concrete and steel bars. Otherwise, the condensation of oil

vapor inside the drilled holes will act as a bond breaker. To check whether there is oil vapor, compressed air is shot into white paper for a few seconds and the paper is checked for any oil residue.

Once the drilled holes are cleaned, they are completely filled with epoxy materials. The epoxy materials used in this operation have a low viscosity and it's not easy to fill the holes completely. Item 361 requires the use of grout retention disks to keep the epoxy in the holes. However, this requirement is not always met in the field. More attention needs to be paid to this requirement.

Design standard FDR(CP)-05 indirectly requires a minimum lap length of 24 in. for #6 bars. This requirement comes from the fact that, to prevent pull-out failure, the steel embedded length should be a minimum of 33 times the diameter of the reinforcing steel. This will work as long as there are no cracks near the transverse perimeter of the repair area. If cracks develop near the transverse perimeter, this requirement will not be met. FDR(CP)-05 should be revised to increase the lap length requirement.

Figure 10-6 shows the full-depth repaired slab that was removed for additional repair due to distresses. The problem was that continuity of the longitudinal steel was not provided. The bars in the red circle show tie bars inserted into the drilled holes during the previous repair. The bar in the blue circle is the existing longitudinal steel. It is noted that tie bars are not fully bonded to the concrete. Epoxy is not fully filled in the holes either. If the pullout testing was done, it would have failed. The inability of the tie bars to provide continuity of the longitudinal steel caused wide joints on one side of the transverse perimeters in the repair area, which resulted in premature failure and another repair was required.

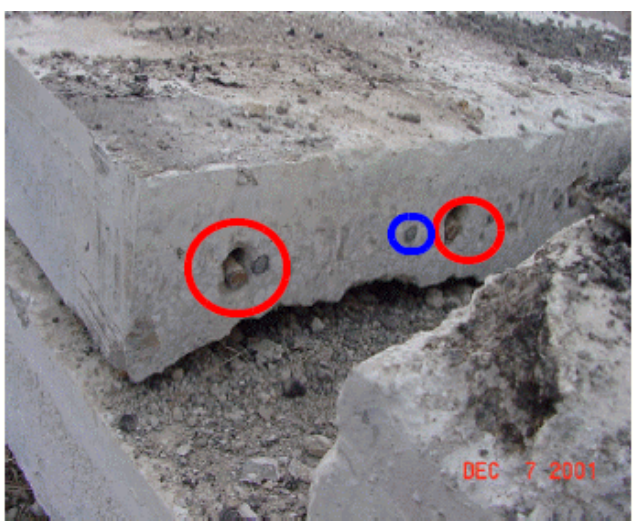

Figure 10-6. Previously repaired slab.

7. Place concrete.

Once the steel placement is complete, concrete is poured and finished. Since the concrete placing is done manually in FDR, the consolidation of concrete is achieved by hand vibration. In order to pro-

vide a good bond between the steel and concrete, a high quality consolidation operation is required. Surface texturing comparable to the surrounding concrete should be provided.

Normally, the full-depth repaired pavement is required to be opened as soon as possible. Prior to 2004 Specifications, Class K concrete was used for FDR. However, there were no statewide uniform strength requirements for Class K concrete. For 2004 Standard Specification Item 421 created a new class of concrete, high early strength (HES) concrete, when high early strength is required. The maximum water-cement ratio is set at 0.45 and strength requirements are specified in Item 360. Item 361 requires the use of class HES concrete if the pavement has to be opened within 72 hr. after concrete placement. Since high early strength is required for early opening to traffic, it's normal practice to use high cement content as well as Type III cement, which is for high early strength. That practice could be beneficial if the repairs are done in winter when the ambient temperature condition is not conducive to achieving high early strength. However, if the repairs are done in the summer when the ambient temperature is high, the practice could result in high heat of hydration and premature pavement distress due to thermal cracking problems.

Another important item in concrete finishing is curing. Since the concrete used in FDR will often have requirements of high early strength as described above, higher cement content than normal Class P concrete is used. Also, Type III cement is sometimes used. These practices result in high heat of hydration as well as larger drying shrinkage of concrete. High heat of hydration creates large temperature gradient through the slab depth, especially near the surface. Also, high concrete temperature encourages the evaporation of the water from the surface. If the proper curing is not provided, the combination of large temperature gradients as well as high evaporation will cause cracking problems as shown in Figure 10-7. Proper curing is essential.

Figure 10-7. Map cracking in FDR slab.

For FDR project, the use of maturity in accordance with Tex-426-A, Estimating Concrete Strength by the Maturity Method" rather than conventional strength testing, is strongly encouraged. It is because the accurate estimate of the in situ strength can be best achieved by the maturity method.

To use the maturity method, contact the Rigid Pavements and Concrete Materials Branch of the Construction Division.

Section 3 — Partial Depth Repair

Pavement Distresses that Require Partial Depth Repair (PDR)

In TxDOT, the pavement distress types requiring partial depth repair (PDR) are limited to shallow spalling problems. For this document, shallow spalling is defined as the spalling with depths less than 4 in. It appears that concrete material properties and construction practices have a significant effect on the occurrence of spalling. Also, horizontal delaminations are frequently observed at spalled areas. Figure 10-8 shows typical spalling in CRCP.

Figure 10-8. Spalling in CRCP.

Spalling causes ride as well as noise problems for the traveling public. Research studies are underway to develop design/construction methods to prevent or minimize spalling. Spalling occurs more often when certain coarse aggregate types are used. Spalling can occur as early as within two years after construction. In some projects, it took more than 10 yr. before spalling occurred. Also, spalling appears to progress with time. Initially, spalling is confined to small areas, and then it extends to larger areas. To repair spalling problems, TxDOT has tried a number of different materials for PDR with varying results. Currently, there is a research study to evaluate the PDR procedures including patching materials. It is expected that the study findings will improve the effectiveness of PDR and this section will be revised as findings become available.

Partial Depth Repair (PDR) Procedures

The following procedures need to be followed for PDR.

1. Identify the repair limits.
2. Remove deteriorated concrete.
3. Clean the repair surfaces.

4. Place the repair material.
5. Finish the surface.

Each step is explained in more detail.

1. Identify the repair limits.

When it is determined that PDR is required, it is important to properly evaluate the extent of the spalling and determine the limits of the repair. When spalling occurs, the damage is often extended beyond the visible spalled area. Since most of the spalls that require PDR appear to be caused by shallow delaminations, the extent of delaminations should be identified. One of the most efficient ways of evaluating the extent of delamination is the use of a sounding test. A hammer or steel rebar can be used for sounding testing. A hammer or rebar is dropped near the spalls. If there is no delamination, the sound will be solid. On the other hand, a dull or hollow sound indicates a high probability of delamination. Figure 10-9 shows coring in the spalled area that has been repaired.

Figure 10-9. Coring from spall repaired area.

The repair material applied (left side of the picture) has greenish color. A core was taken several inches away from the limits of the spall repair and several delaminations at various depths were observed. Note the top portion of the core came out disintegrated; this is an example of a PDR where the limits were not properly identified. If a sounding test had been conducted, areas with delamination would be identified and the area could be included in the repair. To assure removal of all delaminated concrete, it is good practice to extend the limits of the repair boundaries several inches beyond the limits determined by the sounding tests. As mentioned earlier, a research project is currently underway to identify the best method to identify delaminated areas. So far, researchers have evaluated ground penetrating radar (GPR) and portable seismic pavement analyzer (PSPA) for their potential to detect delaminations. Once the research is complete and more information becomes available, this section will be updated to provide information on which non-destructive testing (NDT) devices are the best at determining delaminated areas.

2. Remove deteriorated concrete

After the repair limits are determined, the delaminated concrete should be removed. A typical method for removing spalled concrete is chipping. A shallow vertical saw-cut, approximately the depth of the spall, made around the perimeter of the spalled area can be used to prevent a tapering of the repair around the perimeter. Tapering to a shallow depth can contribute to subsequent spalling. Sawing and chipping is sometimes used for large repair areas. As described in the previous section, it is important to remove all the delaminated concrete. Chipping is done with light pneumatic tools.

3. Clean the repair surfaces.

For PDR to succeed, good bonding between the exposed concrete surface and repair material is essential. Without good bonding, the repair material will be separated from the concrete due to environmental and wheel loading. When delaminated concrete is removed, it is important to expose a fresh concrete surface. Fresh concrete surfaces should have rough textures and be cleaned with water to remove any dust, which will improve bonding of the patching material.

4. Place repair materials.

As described above, a number of different repair materials have been used for PDR. There are different opinions as to which repair materials work best. However, the general characteristics of good repair materials should include:

- good bond strength,
- less volume change potential due to temperature and moisture variations, and
- strength and modulus of elasticity comparable to those of the existing concrete.

As mentioned before, a research study is underway to evaluate repair materials for spalling and it is expected that the findings will provide more complete information on the best repair materials.

When applying the repair materials, it is desirable to screed from the center of the patch out to the patch boundaries. This construction process will improve the bond of the repair materials to the concrete.

5. Finish the surface.

If cementitious materials are used for PDR, good curing is essential. Because the areas and depth of PDR are relatively small, this makes the surface to volume ratio of the repair material applied higher than that of normal concrete pavement. This high surface to volume ratio makes the repair material subject to larger volume changes due to drying shrinkage if moisture is not kept by good curing. The loss of moisture due to poor curing also results in evaporation heat loss.

The optimum time for the application of curing compounds is when the bleeding water no longer comes to the surface. However, repair materials for PDR have a low water-cement ratio; there may be no visible bleeding water . In that case, curing operations should begin as soon as possible.

Section 4 — Bonded Concrete Overlay

This chapter describes bonded concrete overlays (BCO) on continuously reinforced concrete pavement (CRCP), not on concrete pavement contraction design (CPCD). BCO is not a good option for the rehabilitation of CPCD.

In the past, concrete pavements were designed and constructed with insufficient thicknesses for today's traffic demand. This insufficient thickness often resulted in pavement distresses such as punchouts for CRCP and mid-slab cracking or joint faulting in CPCD. If the Portland cement concrete (PCC) pavement is structurally sound (in other words, if the slab support is in good condition) except for the deficient thickness, BCO can provide cost-effective rehabilitation strategies to extend the pavement life. In bonded concrete overlays, new concrete layer is applied to the surface of the existing PCC pavement. This increases the total thickness of the concrete slab, thereby reducing the wheel load stresses and extending the pavement life. There are BCO projects in Texas that have provided an additional 20 yr. of service. At the same time, there are BCO projects that did not perform well. The difference between good and poorly performing BCOs is the bond strength between new and old concretes.

The critical requirement for the success of BCO is a good bond between a new and old concrete layers. If a good bond is provided, the new slab consisting of old and new concrete layers will behave monolithically and increased slab thickness. The increased slab thickness will reduce the wheel load stress at the bottom of the slab substantially, prolonging the pavement life. On the other hand, if a sufficient bond is not provided, the wheel load stress level in the new concrete layer will be high and the pavement performance will be compromised.

Bonded Concrete Overlay (BCO) Procedures

The design and construction of bonded concrete overlay (BCO) involves the following procedures.

1. Evaluate whether the project is a good candidate for BCO.
2. Develop adequate slab thickness and steel designs.
3. Repair distresses in the existing pavement.
4. Prepare surface of existing pavement for overlay.
5. If needed, place steel.
6. Place concrete and provide optimum curing.

Each step is explained in more detail.

1. Evaluate whether the project is a good candidate for BCO.

This task involves the evaluation of pavement conditions to determine whether a BCO is the most appropriate rehabilitation solution. At this time, TxDOT does not have guidelines on Portland cement concrete (PCC) pavement rehabilitation type selections. However, there is a research study underway to develop guidelines on optimum rehabilitation type selections for PCC pavement. The guidelines to be developed will help TxDOT engineers evaluate the existing PCC pavement to see whether BCO is a good candidate.

In general, BCO could be a good candidate for PCC pavements that are currently structurally sound, but are structurally insufficient to carry future projected truck traffic. A BCO can be used to increase the structural capacity of these pavements to enable them to carry traffic for many more years. It is because in order for BCO to work, severe distresses need to be repaired prior to BCO placement, and the required repairs are often too expensive for BCO to be economically feasible. In that case, other alternatives, such as an unbonded concrete overlay should be considered.

A feasible alternative is the one that addresses the cause of the problem motivating the rehabilitation; therefore, the pavement condition must be investigated before making these decisions. The reason for the rehabilitation may be the structural or the functional condition of the pavement. Structural condition refers to whether or not the pavement is fit to support current and future traffic loads over the desired design period. The functional condition encompasses pavement characteristics related to the way the user experiences the safety and comfort of the road such as skid resistance, roughness, appearance and hydroplaning.

The evaluation of the structural condition involves studying the distress patterns of the pavement, which will provide information on the impact of past traffic loadings. This is assessed by means of a visual condition survey. Historical information on patching, slab replacement and other repairs are other valuable sources for structural condition assessments. Finally, destructive and non-destructive testing (NDT) methods are helpful in determining the structural integrity of the pavement. Among the NDT procedures, the most common is a deflection testing using falling weight deflectometer (FWD). Destructive testing implies the extraction of samples from the pavement for their laboratory evaluation. The evaluation of the functional condition requires the measurement of roughness and skid resistance and an assessment of the present serviceability.

2. Develop adequate slab thickness and steel designs.

In the design phase, the necessary thickness of the overlay is determined. For this, properties of the pavement and parameters, determined by the overlay purpose, are analyzed. Various design procedures are available to determine the design thickness for the overlay. Most of the procedures are based on the principle that the overlay is being placed for the purpose of structural improvement. The required thickness of the overlay is a function of the structural capacity of the existing pavement under current conditions and the structural capacity necessary to fulfill future traffic demands. In order to assess the structural capacity of the existing pavement, a comprehensive evaluation of its condition should be conducted, which includes visual surveys, field and laboratory tests. If the

overlay is being placed only to remedy functional failures, normally a thinner overlay would suffice. However, 2 in. is the minimum practical constructible thickness for an overlay.

For the steel design, when the overlay thickness is more than 40% of the existing CRCP, longitudinal steel should be provided for the overlay. If the steel is not provided:

- the longitudinal steel in the existing CRCP will be in much higher stress, diminishing its ability to restrain concrete volume changes
- the distance between the overlaid concrete surface and the existing steel will be increased and the ability of the existing steel to control the concrete volume changes at the surface will be diminished, resulting in more concrete volume changes and larger crack widths at the surface.

The amount of steel needed should be sufficient to control the overlaid concrete volume changes. The guidelines to be developed in the current research study are expected to address steel design.

3. Prepare surface of existing pavement for overlay.

As described above, the critical factor for the good performance of BCO is the bond between existing concrete and overlaid concrete. An insufficient bond will result in the compromise of the pavement performance. If a good bond is provided, the overlaid concrete will be in most part in compression when subjected to wheel loading. In other words, the concrete strength of the overlaid concrete should not be a great concern as long as good bond is provided and the durability of overlaid concrete is not compromised. One of the requirements for good bond is to provide adequate surface texture of the existing concrete pavement.

Surface preparation encompasses the operations conducted on the existing substrate to roughen its texture in such a way that enables the new concrete layer (BCO) to become bonded to it as if both strata were a single structure.

One of the most critical facets of a BCO construction is surface preparation, because it is highly accountable for the bonding of the overlay to the existing concrete. The bond of a BCO usually determines the success or failure of the rehabilitation. The bond at the interface between the BCO and the existing concrete is subject to considerable stresses from concrete volume changes and loading. The goal of surface preparation is to provide a rough surface that favors the bonding of the BCO to the existing pavement by facilitating the bonding mechanisms. One important aspect of surface preparation is that, once the surface is roughened, the section should not be opened to traffic until the overlay is completed and cured.

There are several surface preparation methods to achieve a roughened substrate. The most common are:

- shotblasting
- sandblasting
- cold milling.

For shotblasting, a spinning drum equipped with compressed air blasts tiny steel balls (shot), which impact the surface at an angle to scarify it. A vacuum collects both the shot and the dust. The shot is separated from the dust by magnetic action for continuous reuse. Regulating the blaster speed allows control of the level of scarification, wherein lower speeds yield a higher level of scarification. Shotblasting removes the matrix surrounding the coarse aggregates in a uniform way, but keeps the aggregate intact. It is a clean procedure that minimizes dust and air pollution.

Sandblasting is similar to shotblasting, but instead of shot, sand particles are used. However, unlike shotblasting, sandblasting generates airborne dust, and sand may remain on the surface after it is scarified, making it necessary to air blast the surface to remove debris prior to paving. The sandblasting surface finishing is not as uniform as that executed by shotblasting operations.

Cold milling removes the top of the substrate to a specified depth by the chipping action of rotating mandrels with sharp tips mounted in a machine like the Rotomill as shown in Figure 10-10. As a result of its action, the surface texture after cold milling is rougher and more angular than after sandblasting or shotblasting. Cold milling is the most widespread method for large areas of concrete surface preparation requiring deep scarification. However, it generates a high amount of dust and contamination, which must be removed prior to overlaying.

Figure 10-10. Teeth of the Rotomill.

Cold milling, while being an efficient way to remove the grout matrix, has the drawback of fracturing the exposed aggregate because the procedure relies on breaking the surface. The microfractures of the exposed substrate may be detrimental to its structural integrity. Shotblasting is a more economical and efficient alternative.

The scarification depth and texture should be specified for each project, depending on economical considerations as well as the materials properties, for both the existing pavement and the new overlay. For instance, if the substrate grout paste is relatively soft and the coarse aggregate is especially hard, a light shotblasting will be sufficiently strong to remove the paste to reach the specified depth, while the aggregate will remain intact, resulting in a good surface texture. Normally, the depth of surface removal is about 1/4 in. deep into the coarse aggregate. It can also be specified in terms of some standardized texture test method, such as "Tex-436-A, Measuring Texture Depth by the Sand Patch Method." Typical texture readings from this test are between 0.050 in. and 0.095 in.

All the major distresses present in the existing pavement should be repaired prior to the overlay placement. The main guideline to follow when performing this work is to assess whether the distress is likely to affect the performance of the overlay within a few years. If that is the case, the distress has to be repaired before the BCO is built.

Concrete repair must replace the damaged concrete and reinforcement, restore structural function, and protect itself and adjacent concrete and underlying subbase from aggressive environmental elements. Areas of localized breakup are more susceptible to these elements; therefore, the repair should address any possible attack from extraneous factors.

Deep spalling, delaminations, punchouts, and deteriorated patches must be repaired. Asphalt concrete (AC) patches should be removed and replaced with PCC patches, because AC prevents bonding of the PCC layers. Also, no AC should be used for any of the pre-overlay repairs. Most distresses require FDRs, which is described in Section 2, "Full-Depth Repair." This will preserve the load transfer capability of the slab, for which the bars must be properly tied to the reinforcing steel in the existing concrete. Partial-depth repairs, in which the distressed concrete is removed by a combination of sawing and chipping, or by cold milling, are suitable only when the deterioration is limited to the surface of the concrete. Working longitudinal cracks may be repaired by stitching, as described in Section 6, "Stitching."

It is common practice to remove and replace the deteriorated area when the presence of structural distresses is extensive. When the distress is caused by a localized foundation weakness, it is convenient to remove and replace the weak material and to stabilize the foundation before replacing distressed slabs. When voids are detected under existing slabs, grout should be injected to stabilize the pavement.

Surface cleaning refers to the removal of dust and debris after the surface preparation is complete and prior to the placement of the BCO, to ensure that no foreign elements interfere with the achievement of bonding between both concrete layers.

After the surface preparation operations are finalized, and the reinforcing steel is in place, the last cleaning of the surface is done just before concrete placement by airblasting as shown in Figure 10-11.

Figure 10-11. Airblasting to clean the concrete surface.

Air blasting is not capable of removing paint stripes, tire marks or grout matrix. Therefore, it should be used only as a supplementary cleaning procedure to remove loose material and debris from the surface after milling, shotblasting, or sandblasting. Airblasting is to be used just before overlaying to thoroughly remove debris from milling or shotblasting operations. It is important not to leave a large time lag between the final surface cleaning and paving in order to prevent the contaminants from resettling.

If the pavement has been overlaid with AC layers, these layers should be milled prior to BCO placing and prior to surface preparation and repair of distresses. Remnants of AC will hinder the bonding of both PCC layers and are likely to trigger delaminations because AC works as a bond-breaking layer between PCC layers. Complete milling of these layers will ensure that all surface contaminants such as oil, carbonates and acids are removed.

The most efficient method is by means of shotblasting equipment; however, a milling machine, such as the Rotomill, could also be used.

4. If needed, place steel.

Steel should be placed at a depth that provides a minimum concrete cover of 3 in. If the overlaid thickness layer is not large enough, reinforcement steel bars can be placed directly over the surface of the existing pavement as shown in Figure 10-11, rather than at mid-depth of the overlay. For overlaid thickness that is not large enough, it may not be feasible to use a slip-form paving machine to place steel at the mid-depth of the overlay due to the use of vibrators.

Placing steel directly on top of the surface of the existing pavement has advantages and disadvantages. Advantages include: saving construction time and costs, since it does not require chairs. Another advantage: the steel will restrain concrete volume changes at the interface most effectively, which will prevent or retard debonding. The only disadvantage is the reduction of the interface area between the new and old concrete. Taken together, for overlaid thickness up to about 5 in., placing steel on top of the existing concrete appears to be a better construction practice. A research study currently underway will address this issue. Guidelines will include recommendations.

5. Place concrete and provide optimum curing.

The concrete and especially aggregates of the bonded concrete overlay (BCO) should be compatible with those of the existing pavement. The basic premise for material compatibility is to use aggregates for the BCO concrete that produce moduli and thermal coefficients lower than those of the materials in the existing slab, which will result in lower stresses at the interface, regardless of the season of placement.

Differences in moduli between layers have a significant influence on the thermally induced stresses. The main factor affecting the modulus of concrete is coarse aggregate type. High-modulus aggregate will result in high-modulus concrete.

The type of aggregate used in the mix has a significant impact on the concrete thermal expansion. Normal concrete has a thermal coefficient range from four to six millionths per degree Fahrenheit. Large differences in thermal expansion coefficients between existing and new concrete result in increased stresses at the interface, which will impact the bonding. Therefore, it is recommended that the coarse aggregate in the BCO should have a thermal coefficient that is as low as possible.

The maximum aggregate size of the BCO concrete should be one-third of the overlay thickness. This will ensure a uniform distribution of the concrete constituents when placing the BCO. If the aggregate is larger than one-third of the BCO thickness, segregation of the oversized aggregates is likely to occur, especially in areas where it cannot mix properly (e.g., under reinforcement bars).

As for the reinforcement steel, the amount should be determined from careful analysis. Currently, TxDOT does not have design standards or guidelines on this.

Type I cement is most commonly used for general construction where no special properties are needed. If a faster-than-normal strength gain is necessary, Type III cement can be used. Since current Item 360 requires that Type III cement be only used for class HES concrete, and class HES concrete has specific curing requirements, special attention should be paid when Type III cement is used. For a BCO, the use of Type I cement is recommended, as it produces less heat from hydration than Type III cement and, therefore, reduces the development of thermal stresses. Customary concrete placement procedures for new pavement apply for placement of BCOs.

Special attention should be given to adverse environmental conditions during paving. Hot, dry climates pose the most problematic setting for BCO placement because these conditions favor the loss of moisture from fresh concrete. Excessive water evaporation from the concrete can cause volume changes large enough to cause debonding problems at the interface between old and new concrete layers. A combination of high wind velocity and high air temperature, low relative humidity and high concrete temperature is the most harmful for paving conditions; these conditions favor high water evaporation.

Curing is a key component for the preservation of satisfactory moisture content and temperature in the concrete during its early stages so that desired properties may develop. For BCO, curing is crit-

ical since the surface to volume ratio of the BCO layer is greater than normal paving concrete. Moisture loss and resulting drying shrinkage are approximately proportional to the surface to volume ratio. Curing can be accomplished by a variety of methods, which include the use of membrane curing and wet mat curing.

The duration of construction is critical mostly in urban areas or highways with heavy traffic. A BCO inherently represents a quick construction process because it requires only a limited number of operations. An expedited BCO takes this concept further; by utilizing special materials, the road can be opened to traffic in a minimal time after placement. An expedited BCO can be opened within 6 to 24 hr. after placement. To make this possible, normally the BCO is constructed with a high-early-strength PCC mix, which is attained by using Type III cement, as opposed to the normal cement (Type I). In general, if Type III cement is not utilized, the mix is supplemented by the use of super plasticizers.

The required concrete strength is equal to the strength required for Class P concrete, unless it is an expedited BCO. As long as there is good bond, the tensile stress in BCO will not cause debonding problems.

Section 5 — Unbonded Concrete Overlay

The Portland cement concrete (PCC) overlay pavement system consisting of a concrete layer over existing PCC pavement with an interlayer between them to break the bond is called unbonded concrete overlay (UBCO).

UBCO has been used successfully in many parts of the country. While the existing PCC pavement should be in good condition for bonded concrete overlay (BCO) to work, UBCO can be successfully used where the existing pavement is in a poor condition. This is a significant advantage of UBCO over BCO or other rehabilitation methods. Also, not much preparation work is needed, except for shattered slabs in concrete pavement contraction design (CPCD) or punchouts in continuously reinforced concrete pavement (CRCP).

For BCO, proper bond was the most critical factor determining its performance. For UBCO, it's the thickness and quality of the interlayer. It's been shown that bituminous mixtures provide the best materials for interlayer. Part of the reason is that bituminous mixtures have lower modulus, thus reducing curling and warping stresses in the overlaid slab. Also, the bituminous layer provides some protection for distresses in the existing pavement from affecting the overlaid concrete.

In TxDOT, the layer right underneath the new CRCP slabs is always a bituminous layer, whether the subbase is cement treated or asphalt stabilized. The 1-in. bituminous layer over cement stabilized subbase is called "bond breaker." Its primary contribution to the performance of CRCP is to reduce warping and curling stresses, thus prolonging the life of CRCP. In UBCO, the existing PCC pavement, whether it is CPCD or CRCP, takes the role of a stabilized subbase like cement treated base (CTB). In this sense, from a structural standpoint, UBCO is similar to TxDOT's new CRCP system with CTB and bond breaker.

UBCO Procedures

Even though unbonded concrete overlay (UBCO) is a rehabilitation method, the design and construction of UBCO is very similar to that of new construction. The UBCO procedures are as follows:

1. Evaluate whether the project is a good candidate for UBCO.
2. Develop adequate slab and interlayer thickness and steel designs.
3. Repair distresses in the existing pavement.
4. Place concrete and provide optimum curing.

1. Evaluate whether the project is a good candidate for UBCO.

This process is identical for the BCO procedures. If the analysis shows that BCO is not a good rehabilitation candidate, UBCO could be a good alternative. TxDOT is currently sponsoring a

research study on BCO and guidelines will be developed to help TxDOT engineers determine the optimum PCC overlay options. The guidelines will provide information on whether the project is a good candidate for UBCO as well.

Since UBCO will increase the thickness of the total pavement system, if vertical clearance at numerous locations is an issue, UBCO might not be a good candidate.

2. Develop adequate slab and interlayer thickness and steel designs.

Once UBCO is determined to be a good candidate, the slab and interlayer thicknesses are determined considering future traffic loading and the conditions of the existing PCC pavement. There are several methods available for this analysis. The most widely used method is the method included in the AASHTO 1993 Design Guide. The thickness designs of PCC slab and interlayer are similar to the new pavement design. The major difference is the assessment of the support provided by existing concrete pavement. Therefore, the thickness of the overlaid PCC slab is sensitive to design traffic, but not as sensitive to the conditions of the existing pavement.

Steel design for UBCO should be in accordance with TxDOT design standards CRCP (1)-03.

3. Repair distresses in the existing pavement.

If the UBCO is placed over CPCD, shatter slabs must be removed and replaced. If it is placed over CRCP, any punchouts should be repaired with full depth repair (FDR). Even though spallings, whether they are partial depth or full depth, do not require repairs, some districts repair them before UBCO.

4. Place concrete and provide optimum curing.

Once all the distresses requiring repairs are completed, the interlayer is placed. The rest of the construction is very similar to the new PCC pavement which should be constructed according to Item 360 and TxDOT design standards.

Section 6 — Stitching

Stitching is the operation done at cracks to maintain aggregate interlock and provide added reinforcement to minimize the relative movement of concrete slabs at the cracks. It is also used at the longitudinal joints to keep the slabs from separating.

Pavement Distresses that Require Stitching

In concrete pavement contraction design (CPCD), cracks are not intended and could cause performance problems. Examples include transverse and longitudinal cracks.

From a pavement distress standpoint, transverse cracks are more serious than longitudinal ones, because wheel load is applied on both sides of the crack in sequence. Under heavy truck traffic, there is a differential movement at the crack and it deteriorates rather quickly. The normal practice to repair slabs with transverse cracks is to remove and replace. Stitching transverse cracks in CPCD will cause stress buildup from the volume changes due to temperature variations somewhere else and result in cracks or spalls. Stitching should never be used at transverse cracks in CPCD. On the other hand, longitudinal cracks in CPCD do not necessarily deteriorate at the same rate as transverse cracks and do not need to be removed and replaced. For some longitudinal cracks, simple sealing is all that's required. For some other longitudinal cracks, stitching might be a good repair option. It depends on the location of the longitudinal cracks.

In CRCP, the only time stitching is utilized is where lanes are separating at the longitudinal joints. In CRCP, transverse and longitudinal reinforcements are provided and cracks are held tight and do not cause pavement distress, unless longitudinal cracks extend through the full slab depth. In that case, punchouts will result and FDR is the most appropriate repair option.

Tie bars at construction joints (for both CPCD and CRCP) and transverse steel at warping joints (for CRCP) are used to keep the lanes from separating. However, for various reasons, lanes are separating at longitudinal joints, primarily longitudinal construction joints. In some TxDOT projects, tie bars were not straightened after the slip-form paving machine passed. In other projects, corrosion of the tie bars was noted. These construction errors and steel deterioration resulted in lane separation. Lane separation as much as over 2 in. has been observed. The negative effect of lane separation is that, in addition to the safety hazard to motorcycles, load transfer to the next lane is not achieved, and the slab will be subject to edge loading, rather than interior loading conditions. Edge loading condition causes higher wheel load stresses than interior loading conditions do, and will reduce the fatigue life of concrete, reducing the pavement life due to longitudinal and/or corner cracks in CPCD and punchouts in CRCP. TxDOT is sponsoring a research study to investigate what causes lane separations and the best repair methods. The findings will enhance TxDOT's ability to rehabilitate PCC pavement with lane separation problems more effectively.

Stitching has been rarely used by TxDOT. However, lane separations have taken place at the longitudinal joints and lately stitching has been used in several TxDOT projects.

Types of Stitching

There are three types of stitching used; cross-stitching, slot-stitching, and U-bar stitching. Cross-stitching is the most widely used method.

In cross-stitching, holes are drilled at an angle so that they intersect the longitudinal cracks or joints at about mid-depth of the slab. Dusts are removed by compressed air and epoxy is injected into the holes. Tie bars are inserted and excess epoxy is removed.

In slot-stitching, slots with lengths no smaller than 25 in. are cut approximately perpendicular to the longitudinal joints or cracks using a slot cutting machine or walk-behind saw. Slots are prepared by removing the concrete and cleaning the slot. Deformed bars are placed and backfill material is applied, finished, and cured. In slot-stitching, the concrete slabs are held together by the shear stress of deformed bars. It is important to provide high strength backfill material and good consolidation around the bars and concrete surface.

In U-bar stitching, slots are cut using a slot cutting machine and concrete is broken and removed by pneumatic hammer. Figure 10-12 shows the slots and U-bars (Special Specification 3408). In this method, anchoring action by the U-bars provides most of the restraining force. Proper backfilling around the ends of the U-bars is important.

Figure 10-12. Slots and U-bars.

Among these three methods, there is not enough field performance information to judge their performances. A research study will be conducted on the effectiveness of these methods and this section will be updated as findings become available.

Section 7 — Dowel Bar Retrofit

This rehabilitation technique is applicable to only jointed concrete pavement (JCP or concrete pavement contraction design [CPCD]), not continuously reinforced concrete pavement (CRCP). Load transfer is defined as the ability of a joint or a crack to transfer wheel load from one slab to the next. In CPCD, the majority of load transfer is achieved by dowels. Aggregate interlock is supposed to provide some load transfer; however, the effectiveness of aggregate interlock diminishes with time as concrete contracts due to drying shrinkage. Also, the contribution of aggregate interlock to the load transfer is substantially diminished during winter when the temperature is low and joint opening becomes large. Good load transfer at the joints improves the performance of jointed concrete pavement, and dowels have been used to provide good load transfer. However, there are jointed concrete pavement sections that do not have dowels. The absence of dowels along with deficient subbase or subgrade support could result in faulting and cracking problems at joint areas. Figure 10-13 shows faulting in CPCD.

Figure 10-13. Severe faulting at the transverse joint.

This pavement shown in Figure 10-13 does not have dowel bars and the subbase support was not adequate. Faulting increases the stresses in concrete due to increased dynamic wheel loading, which further deteriorates the pavement. One of the most efficient ways to restore load transfer at the joints is dowel bar retrofit (DBR). In DBR, slots are cut, concrete removed, dowel bars inserted, backfill material filled, the surface finished, cured, and most often diamond ground. Many states have used DBR to restore old CPCD where dowels were not used and have had success. In TxDOT, DBR is a relatively new procedure and currently, TxDOT does not have guidelines for using DBR. However, TxDOT developed special specifications and design standards, even though they are not statewide (Special Specification 3012).

DBR Procedures

The following describes the steps needed to complete the DBR.

1. Identify the need for DBR.
2. Cut the slots.
3. Prepare the slots.
4. Place dowels.
5. Backfill the slots.

1. Identify the need for DBR.

As described above, currently TxDOT does not have established faulting or load transfer efficiency criteria for DBR. However, ACPA and FHWA identified conditions that will benefit from DBR. Those conditions are:

1. load transfer efficiency (LTE) of 60% or less
2. faulting greater than 0.1 in.
3. differential deflection of 10 mils or more.

Since the load transfer efficiency and deflections depend on the temperature conditions of concrete, the above first and third criteria need to have reference temperature. Also, the determination of LTE and deflections require the use of FWD, while the magnitude of faulting requires a simple ruler. Even though Georgia Fault Meter is much more sophisticated than a simple ruler and has been used in other states, a simple ruler will do the job for the determination of the need for DBR. Currently, TxDOT does not have a Georgia Fault Meter. If the faulting is due to deficient base support, DBR alone should not be used to restore LTE. In this case, subbase repair should be done along with DBR.

For young pavement, where the pavement condition is satisfactory but dowels were not used, DBR can be used to strengthen the pavement's structural conditions. This is more of a preventive maintenance type operation.

2. Cut the slots.

Once the need for DBR is established, the next step is to cut the slots using diamond saw slot cutters. It is customary to provide three dowel bars in each wheel path. The diamond saw slot cutter shown in Figure 10-14 cuts multiple slots in a single pass. These cuts form the edges of the slots. When cutting slots, it is important that the slots are aligned parallel to centerlines. The slots need to be sufficiently wide to permit the largest aggregates in the concrete mix or proprietary mix to flow around the bars and consolidate properly.

Figure 10-14. Diamond Saw Slot Cutter.

Figure 10-15 shows the cuts made for slots. Slots are usually 12 in. apart center to center. Note that the cuts are clean and no spalling is observed on the edges of the slots.

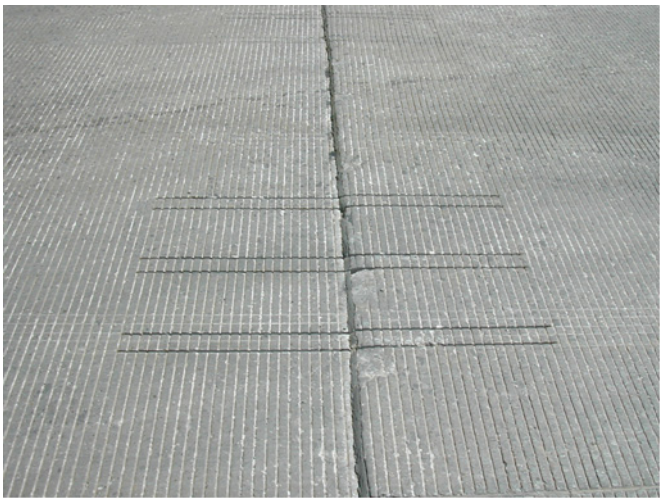

Figure 10-15. Cuts made for slots.

3. Prepare the slots.

Slot preparation consists of:

1. removing the concrete fins,
2. flattening the bottom,
3. cleaning the slots, and
4. caulking the joint.

Small handheld jackhammers are used to remove the fins as shown in Figure 10-16. Once the fins are removed, the bottom of the slots is flattened using a small brush hammerhead. The flat slot bottom allows the dowels to sit level and be properly aligned. Normally, the slots are cleaned with sand blasting first as shown in Figure 10-17. Figure 10-18 shows the flat bottom after being water cleaned. Maintaining clean slots is important since the bond between backfilling material and slots is critical to providing good load transfer. Joint caulking is necessary to prevent patch material from entering the joint.

Figure 10-16. Removing concrete fins.

Figure 10-17. Sandblasting the slots.

Figure 10-18. Flat bottom after being water cleaned.

4. Place dowels.

Dowels used for DBR are similar to the ones used for new CPCD. Dowels are lubricated with some type of bond breaker. Figure 10-19 shows the dowel bar assembly used for DBR with a joint reformer, endcaps, and chairs. Joint reformer and endcaps allow movement for the slab to expand into without bearing on the patch or bar. Chairs are used to support dowels in the base of the slots and allow backfill material to surround the dowels, and should fit snugly in the slot to keep dowels properly aligned.

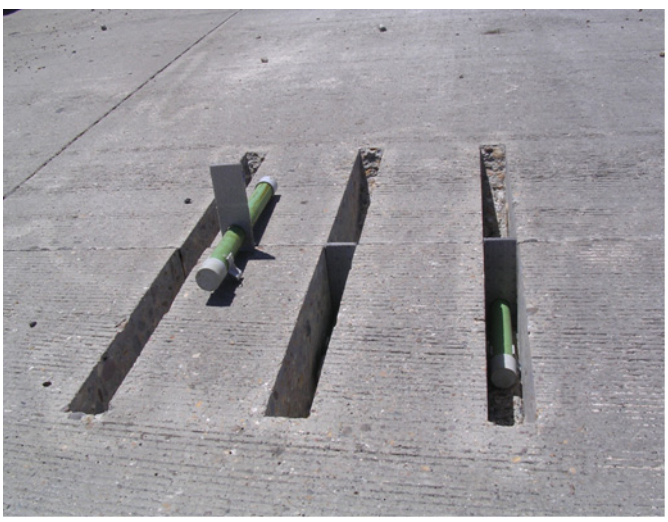

Figure 10-19. Dowel bar in place for backfill material application.

5. Backfill the slots.

Once the dowels are placed, backfill materials are applied. Backfill materials should have similar thermal properties to the concrete, provide strong bond to the existing concrete, be fast setting, have little shrinkage, and develop enough strength to allow traffic in a short time. Both high early

strength concrete and proprietary mixes have been used successfully. High early strength concrete usually contains Type III cement, accelerators, and aluminum power. Accelerators and aluminum powder improve set times and reduce shrinkage. Aggregates in the mix should be small enough to allow the concrete to flow around the bar and consolidate properly. Consolidation of backfill material is done with a small spud vibrator. Care should be taken not to hit the dowels with the vibrator, since touching dowels with a vibrator will knock it out of alignment.

Once the backfill material is applied, the surface is finished flush with the surrounding surface. Curing compound should be applied as soon as practical. Sawing is done over reformer and the entire project is diamond ground, with resealing the joints as a final step. Figure 10-20 shows the section shown in Figure 10-13 after DBR, slab jacking, and diamond grinding (DG).

Figure 10-20. After DBR, slab jacking and diamond grinding of the section shown in Figure 10-13.

Section 8 — Joint Repair

In PCC pavements, joints are provided to accommodate concrete volume changes due to temperature and moisture variations. In concrete pavement contraction design (CPCD), both transverse and longitudinal joints are used. Only longitudinal joints are used in continuously reinforced concrete pavement (CRCP) except for expansion joints at the bridge approaches and transverse construction joints. These joints relieve stresses in concrete, thereby preventing or minimizing the potential for uncontrolled cracks. The concrete movement at the joints could be substantial, and joint sealants are provided to seal and protect the joints from the infiltration of water and incompressible foreign materials.

Joints are one of the weakest areas in PCC pavement. There is a discontinuity in concrete at saw-cut joints and wheel load is not 100% transferred from one slab to the next, which results in higher wheel load stress. When incompressible materials get into the joints, expansion of concrete slabs due to high temperature will squeeze the incompressible materials and high localized stresses will develop in the concrete. Localized high concrete stresses can cause spalling in the joints. Also, if a good joint seal is not maintained, water can get into the joint and cause rust problems in dowel bars as shown in Figure 10-21.

Figure 10-21. Rusted dowel bar.

At the transverse joints in CPCD, dowel bars are used to improve load transfer efficiency (LTE). The higher the LTE the lower the wheel load stress and the better the pavement performance. For dowel bars to perform as they are intended, their alignment should be parallel to the direction of concrete movement, both horizontally and vertically. If dowel bars are misaligned, high stress concentration and cracking will result in concrete. Figure 10-22 illustrates a misaligned dowel bar.

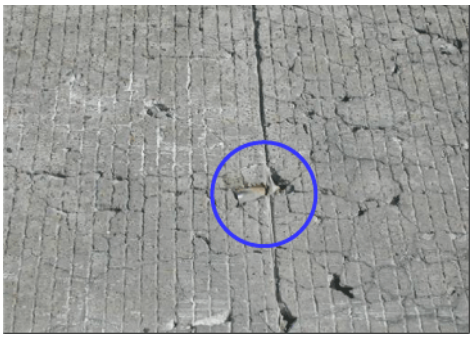

Figure 10-22. Misaligned dowel bar.

Pavement Distresses that Require Joint Repairs

In CPCD, any cracking, breaking, or spalling of slab edges on either side of a transverse joints need to be repaired. The repair can be full depth repair (FDR) or partial depth repair (PDR), depending on the extent of the breaking or spalling. If the distress extends through the depth of the slab or hinders the ability of dowel bars to transfer load, FDR is required.

In CRCP, failures in transverse construction joints require joint repairs. Additional longitudinal steel is used at the transverse construction joints to accommodate large concrete stresses, which requires special attention for concrete consolidation under the longitudinal steel. If proper consolidation is not provided at the transverse joints, delamination and distress can result as shown in Figure 10-23. Any repair of the transverse construction joints requires FDR.

Figure 10-23. Construction joint failure.

As for the boundaries of joint repairs in CPCD, if the patch boundary falls within 6 ft. of an existing undoweled transverse joint that does not require repair, extend the patch to the transverse joint. If the boundary falls on an existing doweled transverse joint, and the other side of the joint does not require repair, extend the patch beyond the transverse joint by about 1 ft. to remove the existing dowels.

Load Transfer Devices

During the joint repair in CPCD, dowels need to be provided at transverse joints and tie bars at longitudinal joints. For CRCP joint repair, tie bars need to be provided as described in Section 2, "Full-Depth Repair."

Section 9 — Diamond Grinding

Diamond grinding (DG) removes a thin layer at the surface of hardened PCC pavement using closely spaced diamond blades. DG used to be an expensive operation; however, new, high production equipment as well as improved synthetic diamonds for saw blades made DG a more cost competitive option for PCC pavement rehabilitation. DG hasn't been extensively used in TxDOT as a pavement rehabilitation option. Rather, it has been used primarily for removing bumps in the newly placed concrete pavement, especially at the transverse construction joints. The level surface is achieved by running the blade assembly at a predetermined level across the pavement surface, which produces saw-cut grooves. The uncut concrete between each saw-cut breaks off more or less at a constant level above the saw-cut grooves, leaving a level surface with longitudinal texture as shown in Figure 10-24.

Figure 10-24. Close-up view of diamond ground surface.

The advantages of DG as a PCC pavement rehabilitation option include:

- improves pavement smoothness
- costs substantially less than an overlay
- enhances surface friction and safety of an old concrete pavement surface
- significantly reduces noise generated by tire-pavement interaction
- does not affect fatigue life or material durability of the pavement
- can be accomplished during off-peak hours with short lane closures and without encroaching into the adjacent lanes
- eliminates the need for taper, which is required with overlay alternatives, at highway entrances, exits, and at side streets

- does not affect overhead clearances underneath bridges or hydraulic capacities of curbs and gutters on municipal streets.

As can be seen above, DG can be a useful tool primarily to correct functional distress (noise, roughness, and skid resistance) in PCC pavement. DG has been used extensively in other state departments of transportation. For example, Georgia DOT uses DG as a routine rehabilitation option. For the rehabilitation of CPCD where dowel bars were not used, they do not use dowel bar retrofit; rather, they diamond grind their CPCD every few years when the faulting at the joints exceeds a threshold value. Their diamond ground surface rides smooth and quite.

Pavement Distresses that Require Diamond Grinding (DG)

As stated above, DG can be a useful tool to correct functional distresses caused by pavement surface defects such as, noise, roughness and skid resistance. Currently, TxDOT is using 1-in. spacing transverse tining when noise has become an issue. Research data shows that DG can reduce noise level significantly. Pavement roughness increases dynamic loading and resulting wheel loading stresses, thus reducing pavement lift. Pavement roughness can be effectively corrected by DG. DG increases macro-texture of the surface, thus improving drainage and skid resistance, and reducing the potential for hydroplaning.

Other Issues with DG

The depth of DG is normally between 0.1 and 0.25 in. The increase in wheel load stress due to the reduced slab thickness by DG can be more than compensated by the reduction in dynamic loading. Also, the effect of reduction in thickness for thicker slabs is not as much as that for thinner slabs. TxDOT has been building thicker slabs in anticipation of increased wheel load applications. DG will have a negligible effect on structural capacity of rigid pavement.

Deeper diamond grinding will expose the coarse aggregate in the concrete pavement. Softer coarse aggregates that are exposed may polish and result in lower skid numbers.

The latest cost information of DG in one of TxDOT's projects shows the unit price of about $5/yd^2. As more DG is used and the quantity increases, the price is expected to go down. TxDOT developed special specifications for diamond grinding, even though they are not statewide (Special Specification 3014).

Section 10 — Thin ACP Overlays

The service life of an older existing CRCP pavement can often be extended many years by the addition of a thin Asphaltic Concrete Overlay (ACP), approximately 2 in. thick. This treatment can be applied to pavements that are beginning to show deterioration, getting rough, experiencing an increasing number of punchouts, or a have experienced a loss of skid numbers.

There are two principal reasons for adding a thin ACP overlay to an existing PCC pavement; either to greatly retard structural deterioration and formation of punchouts, or to improve skid resistance. Both types of treatments have been very successful. There are several theories as to the reason for the great success of these thin overlays on pavements experiencing punchouts, the principal reasons usually include that a new smooth surface reduces dynamic loads from trucks driving over rough pavement, and that the new ACP helps keep water from penetrating to the subbase. All the reasons may be true, but it is essential to keep surface water from infiltrating on a CRCP that has experienced significant punchouts. Little or no credit is given to the added structural capacity from the overlay.

As a CRCP ages, it may start developing punchouts, which require a full depth repair. The ACP overlay will not treat any area where a punchout has started to form. It is essential to perform a full depth repair of any likely punchout prior to overlaying. If punchouts have developed, it is likely that surface water (rain) infiltrating to the subbase has greatly contributed to the formation of the punchouts. For the overlay to be successful, it needs to treat the underlying factors to the distress. In this case, the overlay needs to keep any surface water from getting to the base. This requires either a seal coat or a dense graded hot mix with low permeability. Asphalt rubber seals have had good success. A higher permeability hot mix, such as a permeable friction course (PFC), may be used as a surfacing if there is something under it that will prevent water infiltration.

Thin ACP overlays can dramatically improve the skid resistance of CRCP pavements that are still in very good structural condition, with few or no punchouts. The PFC hot mix is an excellent choice for improving skid resistance and reducing the potential for hydroplaning. If the CRCP is older and has not experienced water related distress, an impermeable layer may not be necessary. Once the overlay has been placed, it is more difficult to install an impermeable pavement layer. A PFC by itself may retard the water from draining from the surface and retain a significant amount of water within the PFC matrix. This could lead to a significant increase in water infiltration. This could lead to a softening of an unstabilized subbase and the beginning of punchouts.

ACP overlays have also been used on older jointed concrete pavements. These overlays are frequently much thicker, often with 6 in. or more of ACP added to structurally deficient JCP that has experienced pervasive transverse contraction joint failures with faulting. The loss of load transfer at the transverse joints results in independent movement of the JCP slabs which causes a crack to form in the ACP that reflects through to the surface. ACP spalls from either side of the joint. If the joint in the JCP is open wide enough, two cracks will reflect through, one from each side of the

joint. Large pieces of ACP can break off and leave an opening above the joint. The success of those overlays has been marginal to poor. A "rule of thumb" has been cited as to the success – each inch of ACP adds a year of life. This makes ACP overlays of structurally failing JCP an expensive treatment for a relatively short life. One exception to the performance of thin ACP overlays on JCP occurs when the existing pavement remains in excellent structural condition, particularly the joints, and the only distress is a loss of skid resistance.

Rubblization of the existing concrete pavement combined with a thick ACP overlay has also been a successful rehabilitation technique. However, once the existing PCC pavement has been rubblized, it no longer behaves like a rigid pavement and can no longer be considered a rigid pavement. Rehabilitation designs using this technique should use a flexible pavement approach.

Section 11 — Retro-fitting Concrete Shoulders

Most of the rigid pavements constructed before the mid 1980s did not include a tied concrete shoulder. Until 1986, the *AASHTO Guide for the Design of Pavement Structures* did not consider or give credit for the inclusion of a tied concrete shoulder. Rarely were tied concrete shoulders included. It is possible to infer what the performance may have been by examining the urban freeways where frequent entrance and exit ramps encouraged the heavy volumes of truck traffic to use an interior lane where the lane edges were supported by another PCC travel lane. The performance of these interior travel lanes with heavy volumes of truck traffic, have greatly exceeded expectations. The reasons for the tremendous success of truck travel lanes supported at the edge by concrete pavement could be attributed to a combination of structural support from the tied concrete pavement reducing stresses in the travel lane, or from the tied concrete pavement helping to keep water from infiltrating into the subbase.

Chapter 11 — Forensics

Contents:

Section 1 — Overview

Section 2 — Objectives

Section 3 — Pavement Forensics Team's Composition and Specialization

Section 4 — Requesting Pavement Forensic Team Assistance

Section 5 — Pavement Forensics Team Investigation Procedures

Section 6 — Obtaining Copies of Pavement Forensics Studies

Section 1 — Overview

Despite advancements in pavement technology in past decades, premature failures and chronic pavement distresses continue to occur. Although improvements have been made to construction specifications, equipment, and construction processes, poor quality construction can occur due to a number of complex and sometimes competing variables such as:

- reduced inspection staffing
- employee turn over
- fluctuations in levels of experience inspectors and project managers
- incompatibilities between new admixtures and construction materials
- implementation of new technologies and construction methods
- environmental constraints and recycled materials
- other issues unforeseen during design and construction phases.

To prevent, and to reduce the probability of premature pavement failures and poor long term pavement performance, the root causes of these problems have to be identified. It is a challenging task to determine the causes of pavement failures. In conducting forensic studies, a thorough review and analysis of existing quality construction records and tests, nondestructive testing like ground penetrating radar (GPR) and the falling weight deflectometers (FWD) are essential to identify problematic areas and probable causes. Field testing such as dynamic cone penetration (DCP), portable FWD, dirt seismic pavement analyzer (DSPA), coring, trenching, and laboratory testing are also conducted to validate/confirm the initial hypothesis. From time to time, the outcomes from forensic studies have been used to validate/change/modify the existing design plan, and to resolve the disputes involving construction claims. In general, the forensic study should be completed 3 months after receiving the request.

For routine technical assistance, consult previous chapters. These types of requests for technical assistance are informal and are best made as early as possible during construction so as to allow maximum opportunity to correct problems, prevent premature pavement failures, and to allow time for appropriate contract administration.

Requests for Forensics Review and Reports (FR&R): After a project has been finally inspected and accepted by TxDOT, and the contractor has been released from contractual requirements, a FR may be requested by the districts/others. Premature pavement failures 0 – 5 years after the time of TxDOT acceptance of the project would likely be good candidates for FR&R.

Section 2 — Objectives

A Pavement Forensics Team has been established to provide technical assistance to the districts and divisions to determine the causes of, and recommend solutions to, premature pavement failures or chronic pavement distresses. The Pavement Forensic Team's involvement in pavement projects is strictly by request. The team will act as a consultant to the districts and divisions. Each FR&R will contain findings and recommendations. The findings section of the FR&R will identify elements causing premature failure. The recommendations section of the FR&R will identify recommendations for corrections necessary to rehabilitate the prematurely failing pavement segments and to improve processes and procedures to insure repeated failures are avoided and eliminated.

The Pavement Forensics Team's objectives are to:

- enhance the district/division's problem solving capabilities
- identify, document, and communicate causes of premature pavement failures so that they may be prevented in the future
- provide remedial solutions for premature pavement failures or chronic pavement distresses.

The team represents a step forward in the implementation of pavement management concepts and practices within the department.

Section 3 — Pavement Forensics Team's Composition and Specialization

The team is composed of members from the Construction Division (CST) and universities (Texas A&M, University of Texas at Austin and University of Texas at El Paso). Members of the team from CST Materials and Pavements Section (CST-M&P) and universities are specialized in any one (or more) of the following areas:

- Pavement Design
- Pavement Structural Analysis
- Pavement Evaluation
- Flexible Pavements
- Rigid Pavements
- Asphalt Cement
- Soils and Aggregates
- Construction Practices.

The director of CST-M&P will serve as the Pavement Forensics Team Coordinator. The director is responsible for:

- assigning project leaders to specific projects
- reviewing progress on individual projects and ensuring timely completion.

Section 4 — Requesting Pavement Forensic Team Assistance

The following table lists the steps for requesting the assistance of a forensics team.

Table 11-1: Requesting Pavement Forensics Team Assistance

Step	Responsible Party	Required Action
1	Requesting district/division, FHWA	◆ Assign district/division coordinator to serve on team. ◆ Request assistance by memorandum from the district engineer or division director to the director of CST. Include the following information in the memorandum and complete Form 2192, the Forensics Investigation Request form: ● District/Division Coordinator for Pavement Forensics Team to contact ● Pavement history (such as date of initial construction, overlays, and widening) ● Pavement structure and materials information used in design and any modifications made during construction ● Traffic information ● Description of the predominant distress or failure modes ● Construction records relevant to the problem (including, acceptance testing, construction inspection findings, weather conditions, etc.) ● Soil and geologic information.
2	Director of CST-M&P	Assign project leader.
3	Pavement Forensic Team Project Leader	Contact district/division coordinator within one week of the date of receipt of request to schedule an initial meeting and preliminary on-site investigation.
4	Pavement Forensic Team Project Leader	Assemble a forensic team that consists of TxDOT and University experts, and FHWA as necessary.

Section 5 — Pavement Forensics Team Investigation Procedures

The following table describes the investigation procedures for the forensics team.

Table 11-2: Pavement Forensics Team Investigation Procedures

Step	Responsible Party	Required Action
1	Pavement Forensics Team and District/Division Coordinator	Conduct preliminary project meeting and perform on-site investigation.
2	Pavement Forensics Team and District/Division Coordinator	Formulate action plan for detailed investigation based on information included in the Forensics Investigation Request form, background information, the preliminary meeting and on-site investigation. Detailed condition survey may include: ◆ Surface distress condition ◆ Ground Penetrating Radar ◆ Spectral Analysis of Surface Waves ◆ Falling Weight Deflectometer (FWD) ◆ Core Rig ◆ Materials sampling and testing ◆ Other testing as required.
3	Pavement Forensics Team and requesting District or Division	◆ Perform review and analysis of project quality construction inspection and sampling and acceptance testing. ◆ Perform materials sampling and laboratory testing to the extent possible and as recommended by the Forensics Team. ◆ Provide necessary traffic control. ◆ Provide equipment and personnel to acquire material samples. ◆ Provide equipment and personnel for repair of cored or trenched sections.
4	Pavement Forensics Team and requesting District or Division	Analyze data and identify most likely causes of problem
5	Forensics Team Leader's Supervisor	Reviews findings/recommendations made by Forensics Team Leader.

Table 11-2: Pavement Forensics Team Investigation Procedures

Step	Responsible Party	Required Action
6	Pavement Forensics Team and requesting District or Division	Present findings/recommendations to the district engineer through a technical report and a presentation. The Forensics Review and Reports (FR&R) will consist of the following and those elements outlined in the STANDARD FR&R: ◆ detailed report for district circulation. Copy to FHWA (Federal-aid Full Oversight projects). ◆ generic report (non-specific location) for statewide distribution ◆ reports should contain at least the following items: ● project history and background ● project findings including a summary of construction quality inspection and sampling and testing acceptance ● traffic history and projections ● description of pavement structure, soil conditions, material types, pavement condition, and distress types ● summary of evaluation/testing strategies employed, and the determined failure mechanism ● project recommendations prioritizing corrective strategies and necessary changes to process and procedures.
7	Requesting District or Division	◆ Monitor project subsequent to corrective measures to assess effectiveness ◆ Provide periodic feedback to Pavement Forensics Team regarding the effectiveness of the repair strategy. This feedback should be in the form of a technical memorandum and should be completed and sent to the CST-M&P in 2 years.

If you have any questions or need additional information on the Pavement Forensics Team, contact the director of CST-M&P.

Section 6 — Obtaining Copies of Pavement Forensics Studies

As indicated under item 6 of Table 11.2, a FR&R will be developed for each forensic investigation. To obtain copies of past forensic studies, please contact CST-M&P.

Pavement FR&R should not be sent to individuals or agencies outside TxDOT (not including university personnel involved in the study and FHWA) without first obtaining an open records request.

Chapter 12 — Load Zoning and Super Heavy Load Analysis

Contents:

Section 1 — Overview for Load Zoning

Section 2 — Changing Load Zones on Roads

Section 3 — Emergency Load Zones on Roads

Section 4 — Changing Load Zones on County Roads and Bridges

Section 5 — Super Heavy Load Analysis Background

Section 6 — Super Heavy Load Evaluation Process

Section 7 — Damage from Super Heavy Load Moves

Section 8 — Damage Claim Procedure

Section 1 — Overview for Load Zoning

Background

There are more than 16,600 mi. of load-restricted highways in Texas. These facilities were generally constructed prior to the late 1950s, and were designed for lighter wheel loads and axle configurations than are currently allowed by law. In addition, environmental effects have weakened some structures.

The Construction Division, Materials & Pavements Section (CST-M&P) manages a database for load-restricted highways. The load-restricted highways can be accessed through the following link http://www.dot.state.tx.us/gis/loadzone/viewer.htm.

The Bridge Division manages a database for load-restricted bridge structures. Refer to http://www.txdot.gov/business/motor_carrier/roadways/restricted_bridge.htm for the load restricted bridge application that includes both on and off-system load restricted bridges.

Minute Orders

In an effort to protect facilities that were originally designed for lighter wheel loads from accelerated deterioration, a provision exists in the state law (Texas Transportation Code, §621.102). The law authorizes the Transportation Commission to set load limits. The Commission must accomplish all actions to revise, post, or remove postings from restricted facilities by issuing a legal document called a Minute Order. CST-M&P is responsible for preparing and submitting, to the Commission, proposed Minute Orders involving load zoning. Minute Orders are approved by the Commission on a monthly basis.

What is in This Chapter?

This chapter discusses the following:

- Changing Load Zones on Roads
- Emergency Load Zones on Roads
- Changing Load Zones on County Roads and Bridges.

Section 2 — Changing Load Zones on Roads

Adding

New restrictions may be required for a number of reasons. Highways undergo periodic evaluations and, during the course of these evaluations, highways may be discovered to be structurally deficient. If the deficiency is severe enough to cause potential for accident or injury, then an emergency posting of restrictions under Minute Order 51070 or 84587 may be required. These Minute Orders authorize temporary load posting up to 60 or 120 days, respectively. In addition, if repairs or upgrades are not anticipated for more than 120 days, restrictions must be authorized and promulgated by a subsequent "permanent" Minute Order. See 'Emergency Load Zones on Roads' for more information on emergency postings.

Refer to the step-by-step instructions in "Table 12-1: Changing Load Zones on Roads" for adding a load zone.

Removing

Removing a load zone may be requested if rehabilitation or reconstruction has been performed or the load-zoned can carry the traffic until the next scheduled rehabilitation without premature failure. If the highway was upgraded by using an approved design process that accounts for future projected traffic, then no further analysis is necessary and recommendation for removal of restrictions by Minute Order should be made. However, if an upgrade was accomplished by means of a maintenance effort (or 2R program), then the district must perform a deflection survey of the upgraded highway using the load zone setup on the falling weight deflectometer (FWD).

Refer to the step-by-step instructions in "Table 12-1: Changing Load Zones on Roads" for removing a load zone.

Changing

The following table lists the steps, responsible party, and required action for changing load zones on roads:

Table 12-1: Changing Load Zones on Roads

Step	Responsible Party	Required Action
1	District	◆ completes the "Recommended Change in Road Load Zoning" Form 1084R ◆ attaches available photos of existing pavement, pavement deflection and design data, sub grade soil lab test reports, or other pertinent information such as a copy of the pavement report ◆ collects pavement structural data using the load zone setup in the falling weight deflectometer (FWD) if a pavement design report does not exist ◆ submits to Construction Division, Materials and Pavements Section (CST-M&P).
2	CST-M&P	analyzes data, determines allowable wheel load capacity, and makes recommendation to district.
3	District	makes decision on load requirements based on recommendation from CST-M&P.
4	CST-M&P	◆ prepares Minute Order for Commission approval ◆ notifies, by memorandum, affected district upon Commission approval of load zone revision. A copy of approved Minute Order will accompany district notification.
5	MCD	◆ updates permit maps ◆ notifies District Permit Coordinator upon completion of changes ◆ distributes the change in the load zone status to MCD Permit Officers and to Remote Permit System (RPS) customers.
6	District	◆ erects or removes signs consistent with the proper load limits once approved Commission Minute Order has been received.

Section 3 — Emergency Load Zones on Roads

Setting Emergency Load Zones on Roads

The district shall notify the Construction Division, Materials and Pavements Section (CST-M&P), by telephone, e-mail, or fax, that an emergency load restriction is required. The following table lists the steps, responsible party, and required action for establishing emergency load zones on roads:

Table 12-2: Emergency Load Zones on Roads

Step	Responsible Party	Required Action
1	District	◆ provides a map showing location of road and alternate routes for legally loaded vehicles to use ◆ recommends gross load or axle load limits ◆ documents deficiencies justifying the placement of emergency load limits ◆ submits a request for a 120-day extension of the emergency load restriction if the load restriction is needed for more than 120 days ◆ requests a permanent load restriction by memorandum and submittal of the "Recommended Change in Road Load Zoning" form prior to the expiration of the emergency load restriction, if necessary.
2	CST-M&P	◆ evaluate information received from district ◆ prepares letter for the director of CST authorizing the emergency load restriction for 120 days (director can also authorize 120-day extension of emergency load zone restriction, if necessary) ◆ verbally notifies district and the Motor Carrier Division (MCD) of approval.
3	MCD	◆ update permit maps ◆ notifies District Permit Coordinator upon completion of changes ◆ distributes the change in the load zone status to MCD Permit Officers and to Remote Permit System (RPS) customers.
4	District	◆ erects signs indicating the emergency load limits once approval from the director of CST has been received.

Section 4 — Changing Load Zones on County Roads and Bridges

Law Ruling

Texas Transportation Code (§621.301) requires counties to obtain TxDOT concurrence for proposed county road and bridge load limits. Counties will petition TxDOT for concurrence with a load limit by submitting a request to the district engineer. The county's request must include an engineer's evaluation of the proposed change along with supporting documentation.

The procedure outlined in this section will help expedite review of county requests that involve emergency or other temporary situations that pose a risk to the public or potential damage to a road or bridge. The department is not responsible for monitoring county compliance with this amendment of the Transportation Code. Also, a change in a county road limit does not require an approved Commission Minute Order.

Coordination between County and District

Each district will be responsible for communicating the requirements of the statute and providing information to the counties. The following table lists the steps that the district should take when a county submits a proposed change in load limits to the district engineer.

Table 12-3: Changing Load Zones on County Roads and Bridges

Step	Required Action
1	Determine if TxDOT concurrence is necessary. ◆ A county must always obtain TxDOT concurrence for a proposed change in: ● road load limits, or for a proposed road load limit on a county road built on a new location, if the proposed load limit is less than the legal load limit permissible under the Texas Transportation Code. ● bridge load limits not supported by TxDOT inspections.
2	Check the completeness of the request. ◆ If TxDOT concurrence is required for the proposed change, the request must include the information and supporting documentation listed in "Table 12-4: Required Information and Supporting Documentation".
3	Evaluate the request. ◆ If the district does not have the resources or expertise to perform the review, contact CST for roads and Bridge Division for bridges to assist in evaluating the proposed change.
4	Provide written concurrence to the county. ◆ If the submitted documentation and calculations accord with accepted engineering principles and practice, the district engineer should provide written concurrence to the county within 30 calendar days of receiving the complete request, including all supporting documentation. ◆ If the district engineer does not respond within 30 calendar days, the county may consider the proposed limit to have TxDOT concurrence.

Table 12-3: Changing Load Zones on County Roads and Bridges

Step	Required Action
5	Provide the district pavement engineer with copies of concurrence correspondence. ◆ If concurrence is given for changes in: ● road load limits, provide the district pavement engineer with a copy of the county request, supporting documentation, and district concurrence letter for record purposes. ● bridge load limits, provide the district bridge inspection office with the new load ratings and a copy of the documentation for updating the database.
6	Provide written notification withdrawing concurrence. ◆ The district engineer may review a changed load limit and withdraw concurrence at any time by providing written notification to the county.
7	Review request for appeal. ◆ The county may appeal a decision of the district engineer by submitting a written request along with the required documentation to the executive director of TxDOT. ◆ The executive director will review the request to determine if department concurrence will be granted. ◆ The executive director's decision is final.

Required Information and Supporting Documentation

The following table lists the required information and documentation that must be included in the request for changing load limits on county roads and bridges.

Table 12-4: Required Information and Supporting Documentation

Roads	Bridges
◆ Name, phone number, email address for a county contact person. ◆ Route name/number, location, and length (in feet) of the section to be load zoned. ◆ Reason for the proposed change. ◆ The current and proposed limits. The Gross Vehicle Weight (GVW) and allowable axle weights should be specified. ◆ Evaluation of traffic loads based on a recommended 10-year interval. ◆ Description of pavement typical sections and section limits, including layer thickness, material types, lane and shoulder widths, and curb and gutter locations, if applicable. ◆ Pavement visual distress condition evaluation, including an engineering description of the pavement distress type and extent. ◆ Date of the last rehabilitation or reconstruction performed on the road. ◆ Engineering Analysis - Supporting documentation that has been sealed by a professional engineer and includes a visual evaluation of the pavement, a full-depth structural analysis, and calculations supporting the proposed load limit. ◆ For a surfaced county road comparable to a state FM road or higher type pavement, an assessment of the pavement and layers is required within the project limits based on one of the following methods: ● Manual field tests using the dynamic cone penetrometer or similar device; ● Laboratory test results; ● Deflection analysis based on the falling weight deflectometer or similar device; or ● Other test methods based on accepted engineering practice. ◆ For an unsurfaced county road or a county road constructed to a lower standard than is typical for the state maintained system: an assessment of the pavement and subgrade layers is required based on engineering judgment including a visual inspection.	◆ Name, phone number, email address for a county contact person. ◆ Route name/number, location, feature crossed, and structure number (if known). ◆ Reason for the proposed change. ◆ Supporting documentation that has been sealed by a professional engineer and includes a structural evaluation report documenting the condition of the bridge and calculations supporting the proposed limits. Calculations should include the inventory and operating ratings for the bridge as defined by the AASHTO *Manual for Condition Evaluation of Bridges*, Chapter 6.

For an optional method to determine the maximum allowable wheel load, when only the depth of cover and subgrade strength are known, the Pavement & Materials Systems Branch of CST-M&P has provided the following engineering analysis method based on the Texas Triaxial Classification Procedure (Determining Allowable Pavement Load Limit using the Texas Triaxial Classification Procedure).

Section 5 — Super Heavy Load Analysis Background

Super heavy loads are defined as loads that exceed 254,300-lb. gross vehicle weight or exceed the maximum permissible weight on any axle or axle group, or exceed 200,000 lb. with less than 95 ft. of axle spacing. However, these loads may approach 2,000,000 lb. Due to the magnitude of these loads; there is a risk of causing severe pavement damage in one pass. All loads of this magnitude must be permitted through the Motor Carrier Division (MCD). Empirical information obtained during field studies of super heavy load moves provided the basis for the 500,000 lb. GVW / 6,000 lb. trailer tire load criteria that was established by the Construction Division, Materials and Pavements Section (CST-M&P). If a super heavy load exceeds 500,000 lb. regardless of trailer tire load; or if any super heavy load has a trailer tire load exceeding 6,000 lb. regardless of GVW, the CST-M&P performs an analysis of the super heavy load route. MCD will notify the CST-M&P of all proposed super heavy load moves when gross vehicle weight (GVW) is over 500,000 lb. or the trailer tire load exceeding 6,000 lb. Notification includes proposed movement dates, routes, and load configuration diagrams. CST-M&P will provide recommendation on proposed route to MCD when the GVW exceeds 500,000 lb. or a trailer tire load exceeding 6,000 lb.

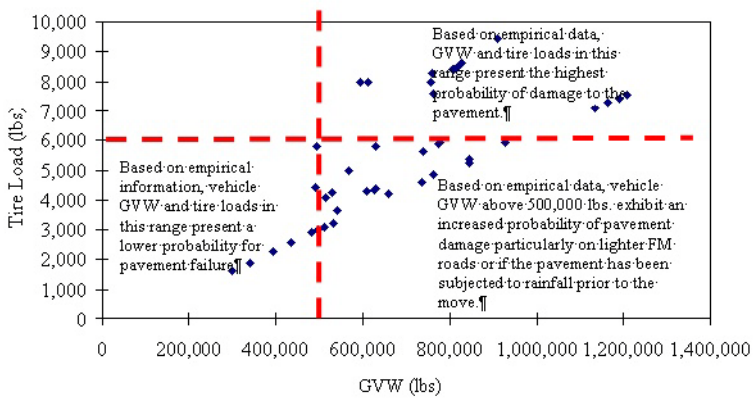

Figure 12-1. The general relationship in gross vehicle weight (GVW) and increased trailer tire load based on loading diagrams. Super heavy loads with GVW above 500,000 lb. and/or trailer tire load above 6,000 lb. are analyzed by CST-M&P.

The GVW load limit criteria was established based on the occurrence of observed pavement distress and the super heavy load GVW and tire load; as well as practical limitations regarding the number of analyses that can be performed by the Pavements and Material Systems Branch. Emphasis has been placed on analyzing super heavy loads with a greater potential for pavement damage. As depicted in Figure 12-1, the general observed trend is that as gross vehicle weight of the super heavy load increases, the load per tire increases. It is uncommon to have greater than a 6,000 lb. trailer tire load when the GVW is less than 500,000 lb.

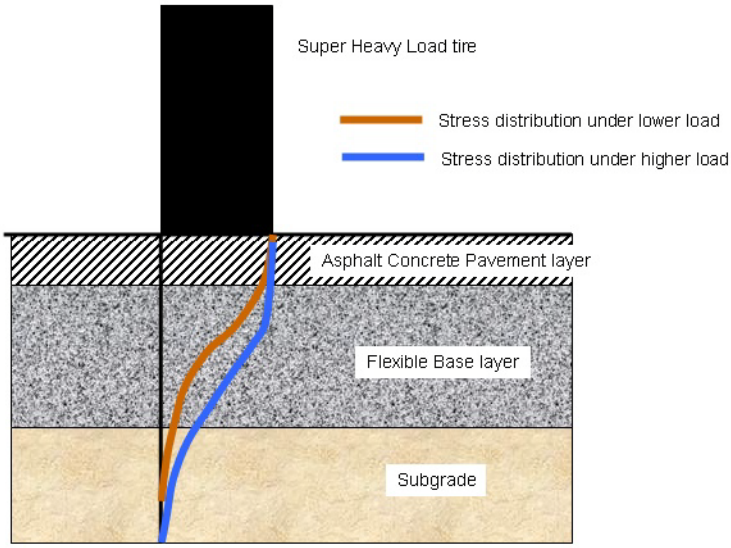

Figure 12-2. This figure shows the stress distribution within a flexible pavement structure for a low and a high tire load.

Flexible pavements, which constitute 93% of the Texas state highway system, typically are constructed with weaker materials at lower levels within the pavement structure. If these weaker materials are subjected to high stresses due to high tire loads, there is a greater chance for permanent damage to the pavement such as rutting or cracking. As can be seen in Figure 12-2, as tire load increases the stress within the pavement structure increases throughout the structure including the lower levels. For this reason, higher tire loads (established to be 6,000 lb.) are evaluated for super heavy loads regardless of the gross vehicle weight.

As mentioned previously, the trailer tire load limit of 6,000 lb. was established based on empirical information related to observed distress which occurred during super heavy load moves. In addition, approximately 6,000 lb. represents the highest manufacturer's single tire load limit that has been observed to date for standard truck tires used on super heavy loads. Although a super heavy load with a standard single truck tire loaded to its capacity has not yet been observed, this arrangement could pose a high potential for damage especially on thinner pavements. Also, as loads increase above 6,000 lb. super single tires may be employed by the carrier since they can carry maximum loads exceeding 9,000 lb. Super single tires have been shown to cause more damage to a pavement than standard dual truck tires based on tests conducted by the Federal Highway Administration (FHWA) and the California Department of Transportation (CALTRANS).

Super heavy loads can damage (by peeling up) pavement surfaces that have been overlaid or chip-sealed for less than 5 weeks. It is the district's decision to place the seal coat restriction on their pavements. The process allows movement over fresh seal coats, regardless of time since placement, if the district feels there will not be significant damage done to the new surface. If the district feels that the surface may sustain damage, the district may restrict movement, as long as the district can offer an acceptable reroute. If no reroute is available, then the original route will be

used and TxDOT personnel will need to be present to document any damage, should it occur. Motor Carrier Division (MCD) will advise the carrier of fresh seal coat locations, along a given route, so the carrier can make arrangements to travel over these sections early in the morning. If this is not possible, use of a water truck or some other method to reduce the possibility of seal coat damage may be considered. Upon receiving an email from MCD or CST-M&P, the District Pavement Engineer will have 72 hr. to alert MCD or CST-M&P, if a reroute is needed. MCD and CST-M&P maintain a contact list (a task force made up of at least 3 people per district) provided by districts on who should be notified. If after 72 hr. no reroute requests have been sent to MCD or CST-M&P, the route will be considered approved and the permit application request will continue to be processed. The exact date and time of the super heavy load is difficult to know due to numerous factors. District personal may take the proactive role to contact the carrier to obtain the dates of movement. For super heavy loads with a GVW over 500,000 lb., MCD requires the carrier to notify district personnel 24 to 48 hr. prior to the move when the Pavements and Material Systems Branch of CST-M&P has determined it is necessary on a case by case basis. Districts need to provide district contact information with at least three personnel to MCD and CST-M&P to keep the list updated.

It's the responsibility of the carrier (permit holder) to check their total load and wheel load. The axle information can be found on the axle diagram MCD sends to the Permit Coordinator along with a copy of the permit. If required, an engineer or maintenance personnel can check the axle configuration to ensure that the load matches the permit. Only the Department of Public Safety (DPS) or local law enforcement can verify the tire load since TxDOT has no enforcement authority. MCD should be the point of contact that notifies DPS when a load needs to be weighed.

Section 6 — Super Heavy Load Evaluation Process

CST-M&P will perform analyses for GVW over 800,000 lb. or GVW over 500,000 lb. and a trailer tire load exceeding 6,000 lb. with input data supplied by the affected districts and the Pavement Management Information System (PMIS). The pertinent information includes pavement typical section, visual distress evaluation, and FWD deflections. CST-M&P may conduct additional testing as needed. Figure 12-3 shows the flow chart of the super heavy load evaluation process.

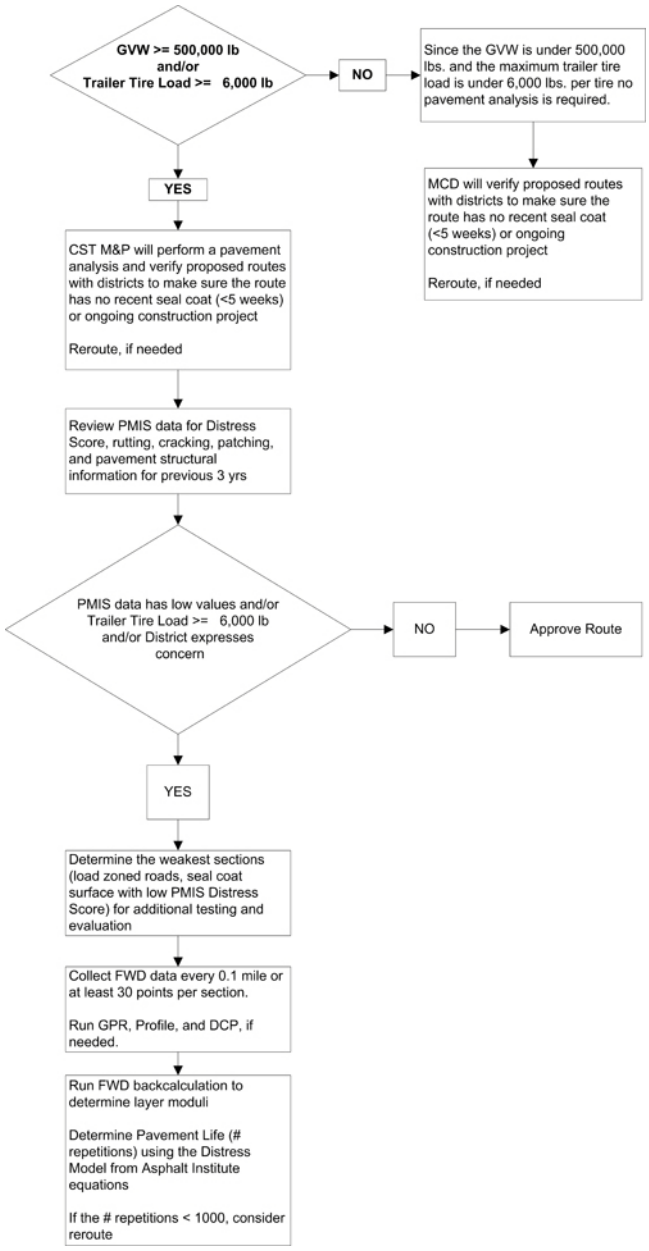

Figure 12-3. Flow chart showing the super heavy load evaluation process.

Section 7 — Damage from Super Heavy Load Moves

The five typical damages related to super heavy load moves are given as follows:

1. Shearing of the pavement surface during turning movements (Figure 12-4). The figure shows the need to consider road geometry in the super heavy load route evaluation. This damage can be prevented by using trailers with steerable axles. If the mover does not have this equipment, then the engineer can ask that the mover protect the surface at the turn by placing plywood sheets on the surface or laminated mats.
2. Rutting or cracking due to overload of the pavement structure. Recent rainfall or poor drainage conditions resulting in weak or wet subgrade or base materials are often associated with structural damage of this type (Figure 12-5). The figure also shows the need to consider road geometry. The road is narrow, and the outside tires are tracking on the unsurfaced shoulder. Moving of super heavy loads should be avoided on such roads. If there is no other route available, the engineer can have the mover mat the width of road where this trailer is going to track.
3. Peeling of fresh seal coats or asphalt concrete pavement overlays (Figure 12-6 and Figure 12-7). Moves should be avoided on fresh seal coats. If there is no other alternative, the mover can protect the surface by placing plywood sheets.
4. Bleeding of seal coats or asphalt concrete surfaces (Figure 12-8).
5. Lateral shear failure at the pavement edge (Figure 12-9 and Figure 12-10).

Figure 12-4. Shearing of the pavement surface during turning movements.

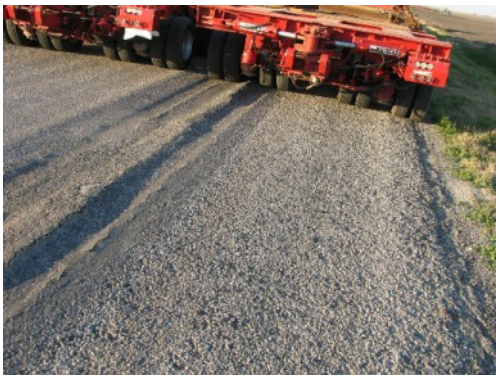

Figure 12-5. Rutting or cracking due to overload of the pavement structure. Recent rainfall or poor drainage conditions resulting in weak or wet subgrade or base materials are often associated with structural damage of this type.

Figure 12-6. Peeling of fresh seal coats (1st photo).

Figure 12-7. Peeling of fresh seal coats (2nd photo).

Figure 12-8. Bleeding of seal coats.

Figure 12-9. Lateral shear failure at the pavement edge (1st photo).

Figure 12-10. Lateral shear failure at the pavement edge (2nd photo).

Section 8 — Damage Claim Procedure

The carrier is liable for any damage caused as a result of the move. It is however, TxDOT's responsibility to prove the damage was caused by the super heavy load. It is recommended that the route be videotaped or photographed as documentation of the condition prior to and after the movement of the super heavy load, if it is not possible to be present during the move. The official TxDOT policy can be found in the *Financial Management Policy Manual*, Chapter 4, Section 11, "Claims by TxDOT Concerning Damage to Highway Property." In general, each district will develop its own internal procedure to augment the policy manual. A typical claim procedure consists of:

1. Maintenance personnel take pictures/videos to document the damage and notify the District Director of Maintenance.
2. District Maintenance Office notifies the company of the damage by telephone and that a letter will follow showing the costs to repair the damage.
3. Maintenance office repairs the damage and documents the equipment used, man hours, and materials needed for the repairs.
4. Director of Maintenance sends the company a certified letter notifying them of the damage and asking for reimbursement. The package includes the cover letter, Form 1494* (Statement of Repair) which is the Summary of Labor, Equipment, and Materials used, a listing of the labor charges by personnel and includes total units (hours), the rate charged and total, and a listing of the equipment charges by type, hours or miles, rate and total.

*Available through the TxDOT intranet only.