

Pavement Manual



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From: James Stevenson, P.E., Director, Maintenance Division

Manual: *Pavement Manual*

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Purpose

The *Pavement Manual* was updated to include a new pavement structural design category, pavement preservation. Information was also added relating to soil k values.

Contents

Chapter 2 – Section 7: Pavement design category added. Caption added to figure 2-10 concerning pavement preservation. Section 7.3 added.

Chapter 8 – Section 3: Adds sentence to section 3.2.7 regarding soil k values.

Contact

Address questions concerning information contained in this Manual Notice to Samuel Glinsky at (512) 968-7385 or samuel.glinsky@txdot.gov.

Archives

Past manual notices are available in a [pdf archive](#).

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Chapter 1: Introduction

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Section 1: Manual Overview

1.1 Purpose

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

- serves as an extensive pavement reference that users can browse to look up typical design 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 for 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.

1.2 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.

1.3 Organization

This manual is organized into thirteen chapters:

- Chapter 1, Introduction. This chapter provides an overview of the manual, policy overview, 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, treatment guidelines, flexible base selection, aggregate types and classes, treated subgrade and base courses, performance graded binders (PG binders), hot-mix asphalt (HMA) pavement mixtures, concrete materials, reinforcing steel, hydraulic cement concrete 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, backcalculation methodology, pavement detours and widening, and perpetual flexible pavement design.
- Chapter 6, Flexible Pavement Construction. This chapter covers subgrade and base construction, surface preparation, plant operation, mix transport, mix placement, and compaction.
- 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, bonded and unbonded concrete overlays and thin whitetopping.
- Chapter 9, Rigid Pavement Construction. This chapter covers concrete mix design, concrete plant operation, delivery of concrete, reinforcing steel placement, paving operations, fixed-form paving, slip-form paving, placing concrete, finishing operations, joints and texture requirements.
- Chapter 10, Rigid Pavement Rehabilitation. This chapter covers the different rehabilitation options for rigid pavements including full-depth repair, half-depth repair, bonded and unbonded concrete overlays, stitching, dowel bar retrofit, joint repairs, diamond grinding, thin HMA overlays, and retrofitting concrete shoulders.
- Chapter 11, Ride Quality. This chapter briefly discusses the units of measurement, equipment for evaluating ride quality, pay schedules, smoothness opportunities, and analysis of ride data.
- Chapter 12, Premature Distress Investigations. This chapter provides an overview of premature pavement failures, investigation team technical assistance and investigation process.
- Chapter 13, Load Zoning and Super Heavy Load Analysis. This chapter covers the procedures for changing load zones on highways (including county roads and bridges), procedure for setting emergency load zones on roads, super heavy load analysis, damage from super heavy load moves and damage claim procedure.

Section 2: Overview of Policy

2.1 Summary

The following table summarizes the pavement design and construction policies required by the department.

Table 1-1: Summary of Pavement Design and Construction Policies

Section	Policy	Additional References
Chapter 2 Pavement Design Process		
Section 2 Pavement Design Standard Operating Procedure (SOP)	District SOP: The district pavement engineer (DPE) will review and update the District Pavement Standard Operating Procedures (SOP) on an annual basis. This SOP shall be reviewed and updated by September 1st annually with a copy emailed to MNT – Pavement Asset Management. If no changes are made from the previous year, send an e-mail to MNT – Pavement Asset Management confirming that no changes were made.	
	Pavement Design Communication: The district engineer (DE) is responsible for documenting communication channels for designing, constructing, and maintaining quality pavements.	
	Pavement Design Approval: Authority for pavement design approval may not be delegated below the DE, except for metropolitan districts. In metropolitan districts, pavement design approval authority may be delegated to the deputy district engineer, district director of construction, operations, or transportation, planning and development for projects with estimated construction costs of less than \$20 million.	
Section 3 District Pavement Engineer's Role	DPE: The DPE is a licensed professional engineer who serves as the district point of contact for the evaluation, preservation, and structural design of pavements.	Section 3 lists general responsibilities for DPE.
	Training: The DPE is required to receive approved training in the use of MODULUS, FPS, AASHTO 1993 CPCD procedure, and TxCRCP ME design software.	Other recommended courses for pavement materials and pavement management are included in this section.
Section 5 Pavement Type Selection	Pavement Type Selection: The decision factors considered for pavement design type shall be included in the pavement design report.	Section 5 includes some discussion on decision factors and recommends using the FHWA software RealCost to perform a life cycle cost analysis if warranted.

Table 1-1: Summary of Pavement Design and Construction Policies

Section	Policy	Additional References
Section 6 Approved Pavement Design Methods	Pavement Design Methods: Use one of the following analytical methods for designing pavements: • FPS 21 for flexible pavements. • Modified Texas Triaxial Design Method for flexible pavements. • TxCRCP-ME for continuously reinforced rigid pavements. • AASHTO design procedure (1993) for CPCD and rigid pavement overlays.	Refer to Chapter 5 for flexible pavement design specifics and Chapter 8 for rigid pavement design specifics.
Section 8 Information Needed for Pavement Design	Evaluate Existing Pavement Condition: The district will take adequate measures to properly characterize the existing functional and structural condition of pavements scheduled for rehabilitation.	Section 8 contains a brief discussion of destructive and nondestructive testing. More information is contained in Chapter 4.
	Moisture Damage Mitigation: Department policy on mitigating moisture damage in pavements is evident in many ways, such as establishing a non-erosive base beneath rigid pavements, and establishing HMA QC/QA density requirements and stripping evaluation.	Guidelines are provided for cases where retro-fitting edge drains may be beneficial. Information about internal (positive) drainage measures is provided in Section 8.
	Ground Water: Another major source of free moisture into the pavement structure is ground water. The department's policy is to intercept ground water outside of the pavement structure to eliminate its impact.	
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), or • unbonded concrete overlays of existing rigid pavements. The following list provides examples of special cases that do not require a full design report but do 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, • bonded concrete overlays on rigid pavements, or • thin whitetopping of flexible pavements.	

Table 1-1: Summary of Pavement Design and Construction Policies

Section	Policy	Additional References
	Completing Pavement Design Report: Follow procedures outlined in Table 2-5. Form 2088: Required to include form 2088, Surface Aggregate Selection Form, as part of the flexible pavement design only. Information from this form will determine the appropriate Surface Aggregate Classification (SAC) of the aggregate used for the final hot-mix asphalt (HMA) riding surface.	
Chapter 5 Flexible Pavement Design		
Section 3 FPS 21 Design Parameters	FPS 21 Design Parameters: The required analytical method of flexible pavement design is FPS 21. Input the Design Parameters for flexible pavement as detailed in this section.	Chapter 2, Section 6, and Table 5-1.
	Post Design Check: Check the design derived by FPS 21 for full-depth shear strength adequacy using the Modified Texas Triaxial Class (TTC) design method contained in the FPS 21 software.	Tables 5-4 and 5-5.
	Traffic Input: Traffic loading must be entered as the 20-yr. cumulative ESALs. A 30-yr. analysis period or longer is allowable, but the designer must still input the projected 20-yr. cumulative ESALs.	
Section 4 FPS 21 Modulus Inputs and Backcalculation	Modulus Inputs: Determine design modulus values as detailed in this section.	Table 5-6 shows a range of typical design values for various new or reclaimed pavement layer materials. Use an input value that is indicative of the material likely to be used. For materials to remain in place, use backcalculated moduli, with adjustments as warranted.
	Backcalculation: Use the backcalculation procedure to determine modulus input values for in situ pavement materials when these materials are used as-is (unmodified) in FPS design.	See list of considerations enumerated in Section 4.
Section 5 Pavement Detours and Pavement Widening	Structural Design of Detours: The falling weight deflectometer (FWD) shall be used to evaluate the adequacy of any existing structure (e.g., shoulders or bypass routes) to carry detour traffic. Design detours using of the following strategies: - FPS 21 and the Modified Texas Triaxial Class (TTC) design procedure (as a standalone design option), - the alternate version of the modified TTC check, or - districts may employ proven design strategies for detours and can develop catalog designs based on traffic levels and subgrade support.	

Table 1-1: Summary of Pavement Design and Construction Policies

Section	Policy	Additional References
	Pavement Widening: Maintain the original cross-section for the widened portion. When the shoulder is to form part of a new lane, perform non-destructive surveys using the falling weight deflectometer (FWD).	Additional detail is given in this section regarding design and location of widening joints for bound, unbound, and dissimilar cross-section widening.
Section 6 Perpetual Pavement Design	Perpetual Pavements: - Use perpetual pavements for traffic levels exceeding 30 million ESALs. - Design perpetual pavements using FPS 21. Conduct a design check of the limiting strain criteria by activating the FPS 21 mechanistic check.	Use limiting strain criteria given. Follow Table 5-6 details for each step.
Chapter 8 Rigid Pavement Design		
Section 1 Overview	Selection of Rigid Pavement Type: The department policy is to utilize Continuously Reinforced Concrete Pavement (CRCP) for new or reconstructed rigid pavements in Texas. The criteria in Chapter 8, Section 1, list the applications where Concrete Pavement Construction Design (CPCD) can be used instead of CRCP, at the discretion of the district engineer.	
	Performance Period: For rigid pavements, the initial pavement structure shall be designed and analyzed for a performance period of 30 yr.	
	Tied Portland cement concrete (PCC) shoulders: Use tied PCC shoulders. If it is not feasible to provide full-width tied PCC shoulders, use a minimum 2-ft. widened outside lane. The PCC shoulders must have the same thickness and the same base layers as the main lane pavement.	
	Concrete Pavement Standards: For CRCP, the steel reinforcement, joints, and other design details are governed by the CRCP standards. For CPCD, the dowel bars, tie bars, joints, and other design details are governed by the CPCD standard.	Link for CRCP and CPCD standards: http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/rdwylse.htm .
Section 2 Approved Design Method	Approved Design Methods: TxCRCP-ME Design program is the approved design method for CRCP. The 1993 AASHTO Guide for Design of Pavement Structures is the only approved design method for CPCD projects.	Chapter 8, Section 3, Rigid Pavement Design Process for CRCP. Chapter 8, Section 4, Rigid Pavement Design Process for CPCD.

Table 1-1: Summary of Pavement Design and Construction Policies

Section	Policy	Additional References
Section 3 Rigid Pavement Design Process for CRCP	Design Parameters: Input the Design Parameters for CRCP as detailed in this section.	
Section 4 Rigid Pavement Design Process for CPCD	Design Parameters: Input the Design Parameters for CPCD as detailed in this section.	
Section 3 & 4 Pavement Design Processes for CRCP and CPCD	Base Layer Requirements: The department requires one of the following base layer combinations for concrete slab support: • 4 in. of hot-mix asphalt (HMA) or asphalt treated base (ATB), or • a minimum 1 in. hot-mix asphalt bond breaker over 6 in. of a cement treated base (CTB). Use item 276, Class L. Width requirement of subgrade/base: The subgrade/base must be designed 2 ft. wider than the concrete slab on each side to accommodate slipform pavement equipment.	
Section 5 Determining Concrete Pavement Thickness	Determining Concrete Pavement Thickness: For CRCP designs, the input thickness should be in 1/2 in. increments. The minimum thickness for CRCP is 7 in., and the maximum thickness is 13 in. For CPCD design, the computed concrete slab thickness should be rounded to the nearest full or half inch. The minimum slab thickness for CPCD is 6 in., and the maximum thickness is 12 in.	
Section 6 Terminal Anchor Joint Selection for Concrete Pavement	Terminal Anchor Joint Selection for Concrete Pavement: Use the transverse expansion joint details at bridge approaches shown in the concrete pavement standards. Districts may develop a Special Specification to use wide-flange systems. The use of anchor lug systems is no longer allowed.	
Chapter 9 Rigid Pavement Construction		
Section 1-7	Rigid Pavement Construction: Construct the rigid pavement in accordance with the department's Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges.	Items 360, 421, and 440. Link for CRCP and CPCD standards: http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/rdwylse.htm
Chapter 10 Rigid Pavement Rehabilitation		

Table 1-1: Summary of Pavement Design and Construction Policies

Section	Policy	Additional References
Section 1-11	Rigid Pavement Rehabilitation: Repair the rigid pavement in accordance with the department's Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges and the relevant Special Specification of the rehabilitation.	Items 361 and 720. Link for Repair of Concrete Pavement Standards: http://www.dot.state.tx.us/insdot/orgchart/cmd/cserve/standard/rdwylse.htm
Chapter 11 Ride Quality		
Section 1 Overview	Ride Quality: Measure ride quality in accordance with the department's Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges.	Items 247, 340, 341, 342, 344, 346, 347, 348, 360, and 585. Table 11-2 can be used to select the appropriate pay adjustment schedule.
Chapter 13 Load Zoning and Super Heavy Load Analysis		
Section 1 Overview for Load Zoning	Executive Order: The department's executive director sets load limits for individual roadway segments by issuing an Executive Order. MNT – Pavement Asset Management prepares and submits proposed Executive Orders to the executive director.	Texas Transportation Code, §621.102.
Section 2 Changing Load Zones on Roads	Changing Load Zones: Follow the procedures in this section for changing load zones.	See Table 13-1.
Section 3 Emergency Load Zones on Roads	Emergency Load Zones: The district shall notify MNT – Pavement Asset Management by telephone or e-mail that an emergency load restriction is required.	See Table 13-2.
Section 4 Changing Load Zones on County Roads and Bridges	County Roads and Bridges: Counties must obtain department concurrence from the district engineer for proposed changes to county road and bridge load limits.	Texas Transportation Code, §621.301. See Tables 13-3 and 13-4.
Section 6 Super Heavy Load Evaluation Process	Super Heavy Load Analysis: All super heavy loads must be permitted through the Motor Carrier Division (MCD) of the Texas Department of Motor Vehicles (TxDMV). Refer to this section for the required evaluation process.	Texas Transportation Code, §623.071 and §623.142. See Figure 13-3. See section 5 for analysis background information.

Table 1-1: Summary of Pavement Design and Construction Policies

Section	Policy	Additional References
Section 8 Damage Claim Procedure	Damage Claim Procedure: Refer to this section for the required procedure.	Financial Management Policy Manual, Chapter 4, Section 10, "Claims by TxDOT Concerning Damage to Highway Property."

Standard Operating Procedures for Responding to PIA Requests for PMIS and/or Skid Data

Pavement Management Information System data and skid data are sometimes requested through the Public Information Act. To be sure each request is handled in the same manner in every district, here are the procedures to be followed:

- Requests for information under the PIA must be in writing and must include the requestor's contact information. E-mailed requests must go through the department's website.
- All requests involving a traffic accident or focusing narrowly on a particular roadway segment or date must be referred to the Occupational Safety Division to determine if the department has received notice of claim threatening a lawsuit.
- When documents are to be withheld, notify GCD and submit a copy of the request with any responsive documents to GCD as soon as possible. Adequate time must be provided to GCD so GCD can prepare a brief and request a ruling from the AG within ten business days.
- When documents are to be released, release them promptly. If you are unable to release the documents within ten business days, you must certify that fact in writing to the requestor and provide a date and hour within a reasonable time when the information will be available for inspection or duplication.

Section 3: Training

3.1 Overview

This manual 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 Maintenance Division, Pavement Asset Management Section and other sources. A partial list of available training courses is as follows:

- Web based training courses through the National Highway Institute (NHI)/American Society of Civil Engineers (ASCE) and the Transportation Curriculum Coordination Council (TCCC). Topical content for these courses includes pavement preservation treatments, in-place recycling techniques, highway materials, and pavement maintenance and rehabilitation. NHI training opportunities can be found at <https://www.nhi.fhwa.dot.gov/default.aspx>. Courses are frequently updated and most courses are free after setting up an account.
- Classroom training courses through NHI. Check the [NHI](#) web page for topics; classes typically require a minimum enrollment and have an associated fee.
- Flexible Pavement Design and Evaluation, Version 21 (FPS 21) and MODULUS Backcalculation Program. Training is presented as a “workshop” by MNT – Pavement Asset Management. Contact MNT – Pavement Asset Management to schedule (see Contacts section below).
- Rigid Pavement Design, DarWin® 3.1 for CPCD and TxCRCP-ME for CRCP. Training is presented as a “workshop” by MNT – Pavement Asset Management. Contact MNT – Pavement Asset Management to schedule (see Contacts section below).
- Flexible Pavement Rehabilitation Strategies Training Course Self-paced training media developed by Texas A&M Transportation Institute (TTI). Although somewhat dated in terms of software versions discussed within, this product offers a comprehensive discussion on evaluating flexible pavement structures using current non-destructive tools and overall design considerations. Please refer to MNT, [Pavement Engineering](#), to download the course.
- Rigid Pavement Rehabilitation Strategies Training Course. Self-paced training media developed by TTI. Please refer to MNT, [Pavement Engineering](#), to download the course.
- Web based training for Modulus 6.0 and the Modulus Temperature Correction program are available on the TxDOT Pavement Design and Analysis Training web site. Web based training is also available for FPS-19W, the predecessor to the current flexible pavement design program FPS 21 (adopted August 2011). Inputs and much of the program functionality have not changed between these programs and there are currently no plans to produce an on line training program exclusively for FPS 21.
- 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) are available on The Center of Transportation

Infrastructure Systems at The University of Texas at El Paso (CTIS) Pavement Design and Analysis Training web site <http://ctis.utep.edu/products/>.

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

Section 4: Contacts

4.1 Contacts for Questions and Comments

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

- Director of MNT – Pavement Asset Management at (512) 416-3288 for general questions or questions or comments about this manual
- MNT – Pavement Asset Management Section, Pavement Analysis and Design Branch at (512) 832-7304, for assistance with premature distress investigations, flexible pavement design, rigid pavement design and rehabilitation
- MNT – Pavement Asset Management Section at (512) 416-3113 for questions about load zoning, and super heavy load analysis
- Flexible Pavements Branch at (512) 506-5836 or 5847 for Flexible Pavement Design, Construction or Rehabilitation
- Rigid Pavement & Concrete Materials Branch at (512) 506-5858 or 5846 for Rigid Pavement Construction
- Geotechnical, Soils & Aggregates Branch at (512) 506-5907 for issues regarding geotechnical investigations, aggregate, and geosynthetics
- 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
- Pavement Preservation Branch (Pavement Asset Management, MNT) at (512) 832-7210 for Pavement Management, Pavement Preservation Techniques, and Pavement Testing
- Research and Technology Implementation Office at (512) 416-4731 for research projects and their implementations.

More information on the MNT – Pavement Asset Management Section can be found at the Maintenance Division website

Chapter 2: Pavement Design Process

Contents:

[Section 1: Overview](#)

[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: Overview

1.1 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. Use 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. Document the pavement design process in a report format as discussed in Section 9, [Pavement Design Reports](#).

1.2 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.

1.3 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.

1.4 Pavement Design Standard Operating Procedure

Each district shall 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 levels, 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)

2.1 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 shall be reviewed and updated by **September 1st annually** with a copy emailed to pavementdesign@txdot.gov. E-mail pavementdesign@txdot.gov if no changes are made from the previous year. 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.

2.2 Conference Participants

Depending on the size or sensitivity of the project, all or some of the following individuals shall 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
- Director of Maintenance
- District Engineer

2.3 Discussion Items

Topics to be discussed during pavement design meetings or conferences are outlined in the district 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 (ATHWLD) review and adjustments. For example, Transportation 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 hot-mix asphalt [HMA]/mixture type, binder type, type of flexible base, treatment of bases, etc.)
- pavement design material properties
- Wet Surface Crash Reduction Program [\(WSCR\)](#)¹
- need for and type of subgrade treatment
- alternate pavement designs (Alternate materials selection - not pavement type. Alternates to limited competition pavement rehabilitation techniques such as Thin-bonded Wearing Courses [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

1. Available through the TxDOT Intranet only.

- typical section, front slope, and ditch geometry
- project drainage
- 2R, 3R, 4R criteria
- special haul routes such as logging, aggregate haul roads, or energy sector site development
- sulfate-bearing and organic rich soils
- use and methodology for potential vertical rise (PVR) design consideration
- pavement design development and approval process

2.4 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, Operations, or Transportation, Planning and Development for projects with estimated construction costs of less than \$20 million.

Section 3: District Pavement Engineer's Role

3.1 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.

3.2 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/soils conditions within the district
- suitability of proposed materials (new and recycled), and
- use of structural evaluation and 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 (PA) 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/materials engineer.

An expanded list of DPE responsibilities is shown below.

JOB FUNCTIONS FOR DISTRICT PAVEMENT ENGINEER
I. Produce cost effective project level designs for new, rehabilitated, and reconstructed pavements based on best practices and life-cycle cost analysis. 1. Perform detailed investigations, data collection, and analysis including ◦ Pavement distress ◦ Structural integrity (deflections) ◦ Roughness ◦ Geology ◦ Subgrade Classification ◦ Confirmation of Percent Trucks in the Traffic Stream ◦ Vehicle/Axle Load Characteristics ◦ Vehicle Configurations ◦ Availability and suitability of local materials ◦ Availability and suitability of recycled materials ◦ Determine available and/or needed funding ◦ Identify alternative designs ◦ Define selection criteria/objectives ◦ Perform life-cycle costs on alternatives ◦ Select most cost effective alternative which meets criteria/objectives 2. Produce typical section and detailed pavement design report 3. Maintain/update the District Pavement SOP by September 1st annually 4. Schedule and Participate in District Pavement Design Concept conferences
II. Recommend pavement preservation policies to maximize condition of the system within District budget constraints. 1. Produce annual report on existing condition of highway system within the District 2. Produce four year budget projection/needs for pavement preservation projects 3. Perform analyses on effect of changes in funding, vehicle weights, axle configurations, suspension types, tire pressures, environmental conditions, legislative mandates, and preservation policies on the highway system, etc. 4. Assist and provide direct input into the development of the District Project Development Plan (PDP) 5. Evaluate effect of current and proposed rehabilitation and maintenance practices, new materials and materials testing specifications, and new construction processes, etc.
III. Identify pavement related research needs. 1. Identify pavement problem areas needing research 2. Submit annual research problem statements 3. Participate in research problem statement ranking process 4. Support statewide pavement related research projects by serving as Technical Chairman or Technical Panel Member when requested 5. Review research projects and reports and assist in implementation of products 6. Participate in research meetings

JOB FUNCTIONS FOR DISTRICT PAVEMENT ENGINEER	
IV. Participate in Technology Transfer and Training Activities.	
1. Participate in annual Short Course Pavement sessions 2. Assist in departmental training in pavements	

3.3 District Pavement Engineer (DPE) Skills

A list of required and recommended training courses is given 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 (e.g., Pavement Design Concept Conference).

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: Required and Recommended Training for the District Pavement Engineer

Training	Category	Comment
MODULUS	Flexible Structure Evaluation	Designed as a workshop and is combined with FPS 21 training. Combined training is usually arranged at district request to MNT – Pavement Asset Management. Hands-on approach used to emphasize evaluation techniques, capabilities, and limitations of the software. A 2 1/2-day period should be scheduled. <i>Required for flexible pavement design.</i>
FPS 21	Flexible Structural Design	As above. <i>Required for flexible pavement design.</i>
Visual Distress Rater's Course CON110 CON111	Identification of visual distress: • concrete distress • flexible distress	The goals are: • understand the Texas Reference Marker System and know how it is used to identify and locate PMIS (PA) section in the field • read a PMIS (PA) 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 (PA). Contact the Maintenance Division, Pavement Asset Preservation Section for assistance.

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

Training	Category	Comment
PMIS Concepts for Administrators CON107	New PMIS (PA)	The goals are: • identify the types of pavement evaluation data available in the new PMIS (PA) • describe the differences between network-level and project-level pavement management, and explain how PMIS can be used to support both • interpret PMIS (PA) data and scores • use new PMIS (PA) program 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	New PMIS (PA)	The goals are: • use PMIS (PA) data to diagnose surface and sub-surface pavement problems • define and interpret the five PMIS (PA) scores • produce three PMIS (PA) analysis reports. *Course material description can be found on the Pavement Management Information System webpage.
Pavement Analyst (PA)	New PMIS (PA)	Web-based software that allows the user to plot various pavement condition indicators on district maps. Formal training is available on request.
Ride Quality Software	Pavement Evaluation – Roughness (IRI)	Software available at http://www.txdot.gov/business/resources/engineering-software.external.html . The DPE may receive guidance on how to use the Ride Quality evaluation software by contacting the Maintenance Division, Pavement Asset Preservation Section.
TxCRCP-ME, DARWin® 3.1	Rigid Structural Design	Designed as a workshop. Contact MNT – Pavement Asset Management, Pavement Analysis and Design Branch. <i>Required for rigid pavement design.</i>
Materials Course	Materials Properties	National Highway Institute (NHI) sponsored web-based course addressing fundamental properties and testing of highway materials (NHI 131117).
TxDOT Materials Academy	Materials properties	Five-week web-based and classroom course. Contact MNT – Pavement Asset Management Section for more information.
Selecting Rehabilitation Strategies for Flexible Pavements	Rehab of existing pavements	Texas A&M Transportation Institute (TTI) administered course taught through an interagency contract (IAC). This course is hands-on oriented – uses department software. Contact MNT – Pavement Asset Management, Pavement Analysis and Design Branch for more information or refer to Crossroads, Maintenance Division, Pavement Engineering link to download the course.

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

Training	Category	Comment
Selecting Rehabilitation Strategies for Concrete Pavements	Rehab of existing pavements	Texas A&M Transportation Institute (TTI) administered course taught through an interagency contract (IAC). This course is hands-on oriented – uses department software. Contact MNT – Pavement Asset Management, Pavement Analysis and Design Branch for more information or refer to Crossroads, Maintenance Division, Pavement Engineering link to download the course.
Other NHI Web-based Opportunities	Multiple (pavement preservation, maintenance, recycling, pavement materials, etc.)	Frequently updated; most courses are free after setting up an account: https://www.nhi.fhwa.dot.gov/default.aspx .
*Available through the TxDOT Intranet only.		

New PMIS (PA) program and pavement design assistance from MNT – Pavement Asset Management will be provided upon request.

The DPE list is also posted on the Maintenance Division website, [Pavement Preservation](#) link.

Section 4: Pavement Types

4.1 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 2-1 shows load distribution for a typical flexible pavement and a typical rigid pavement.

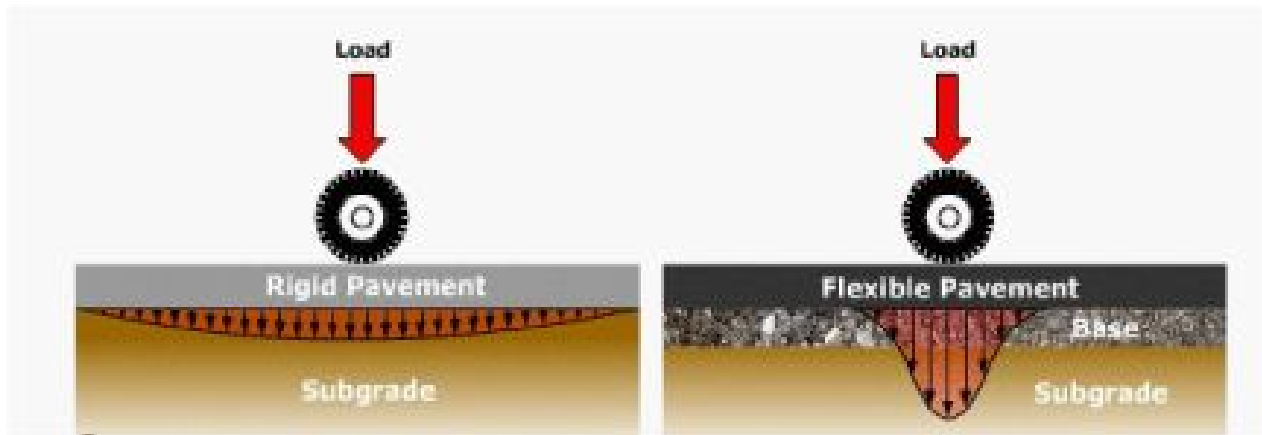


Figure 2-1. Typical stress distribution under a rigid and a flexible pavement.

4.2 Flexible Pavement

A flexible pavement structure is typically composed of several layers of material with better quality materials usually placed 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 subbase courses. Each of these layers contributes to structural support and, ideally, maintains proper drainage.

When hot-mix asphalt (HMA) is used as the surface course, it typically is the stiffest (as measured by elastic modulus) layer 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.

Thicker HMA sections and or sections with stabilized bases behave as a semi-rigid system under traffic loading, whereby loads are spread to a greater degree over the natural subgrade than conventional flexible pavements. See “Rigid and Flexible Pavement Characteristics” above.

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 2-2 shows a typical section for a flexible pavement.

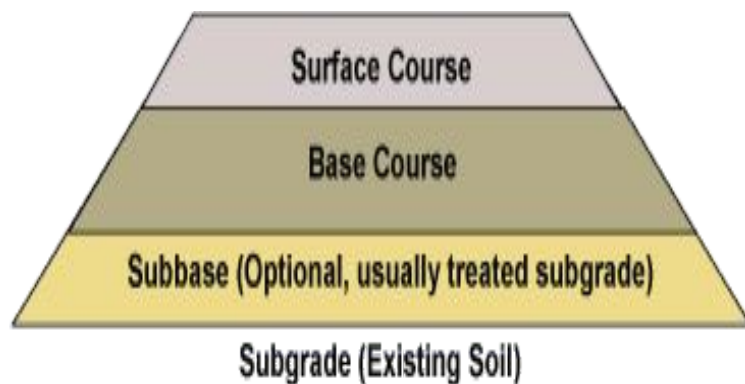


Figure 2-2. Typical section for a flexible pavement.

4.3 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 years or more if properly constructed and maintained.

In conventional flexible pavements, 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 an 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 by using deformation-resistant HMA mixtures. Figure 2-3 shows a generalized perpetual pavement design.

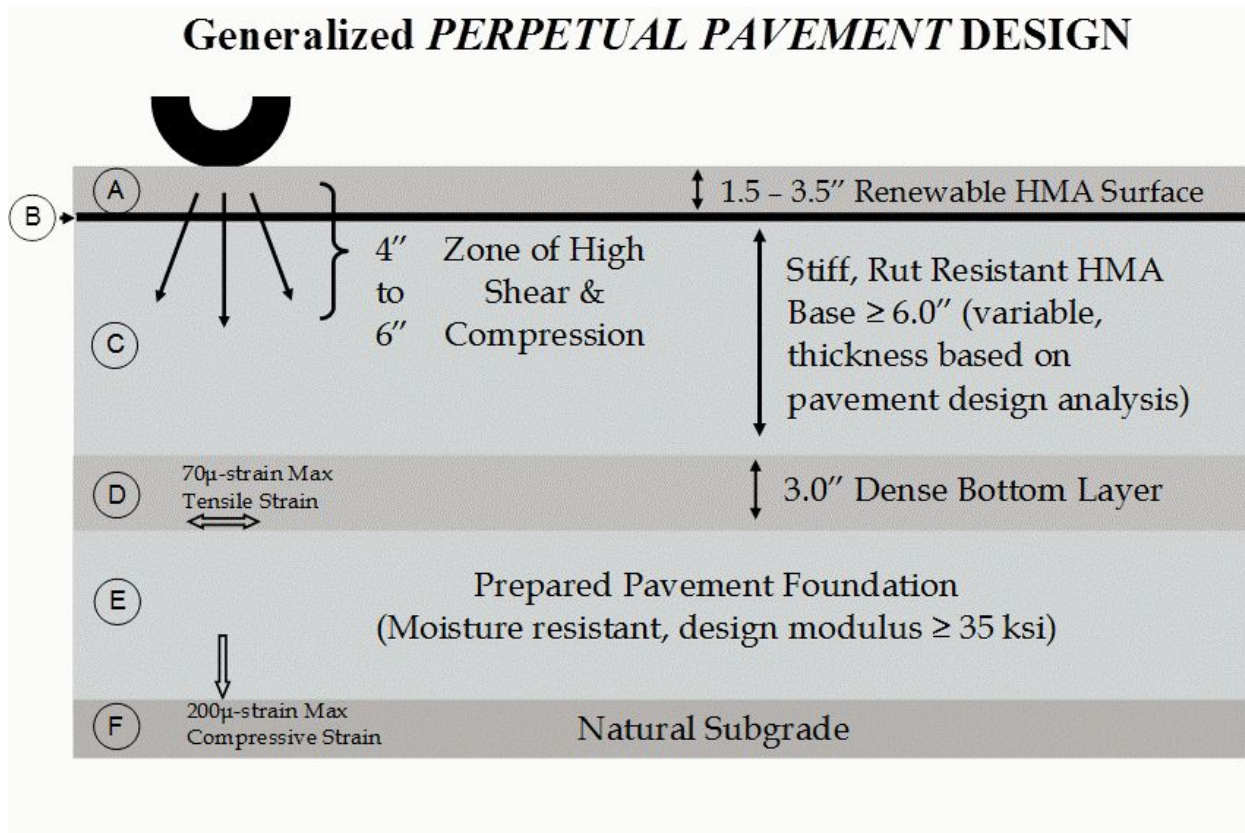


Figure 2-3. Generalized perpetual pavement design.

4.4 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 uniformity of support, 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 2-4 shows a typical section for a rigid pavement.

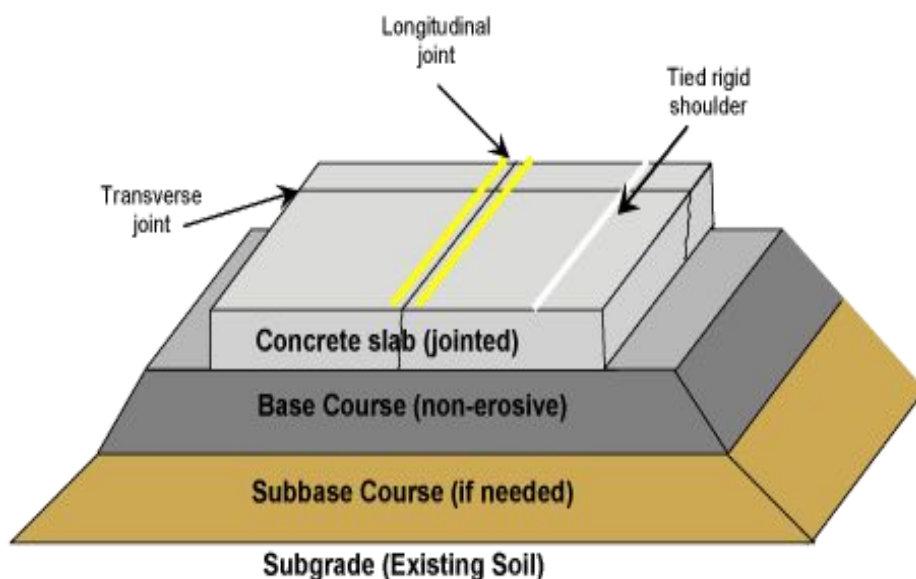


Figure 2-4. Typical section for a rigid pavement.

4.5 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 2-5 shows a typical section of CRCP.

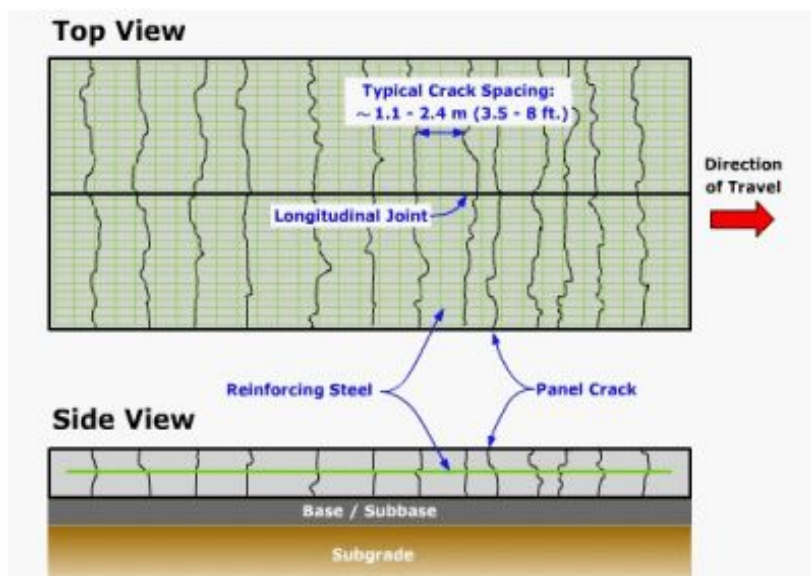


Figure 2-5. Continuously Reinforced Concrete Pavement (CRCP).

4.6 Concrete Pavement Contraction Design (CPCD)

CPCD uses contraction joints to control cracking and does not use any reinforcing steel. An alternative slab design 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 2-6 shows a typical section of CPCD.

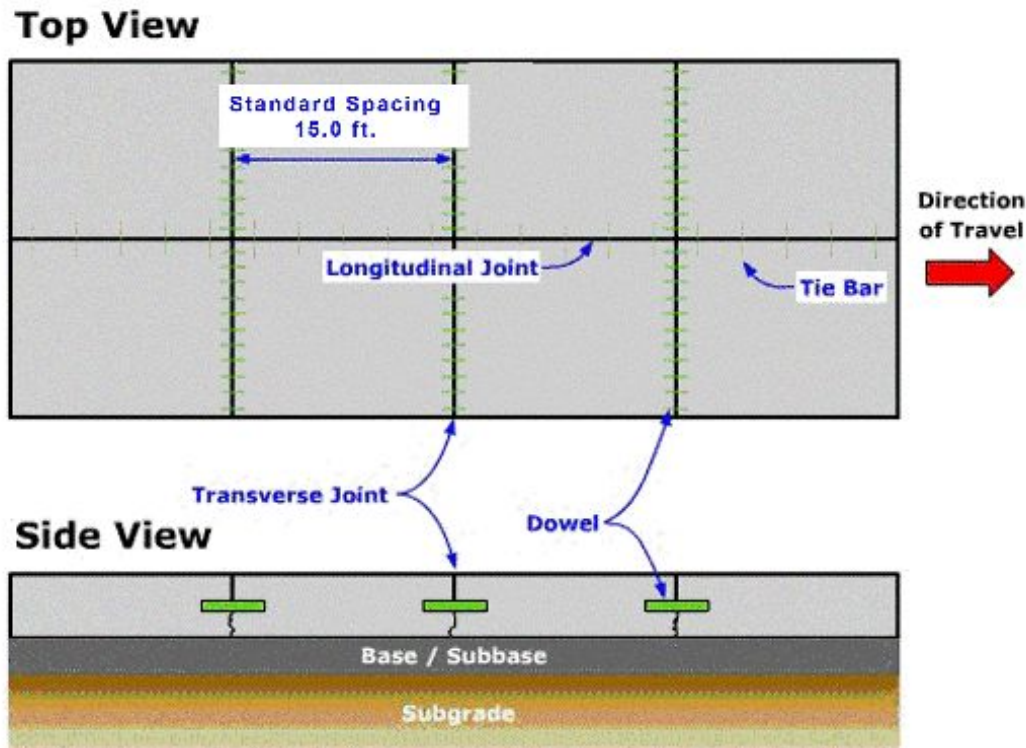


Figure 2-6. Concrete Pavement Contraction Design (CPCD).

4.7 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 2-7 shows a typical section of jointed reinforced concrete pavement.

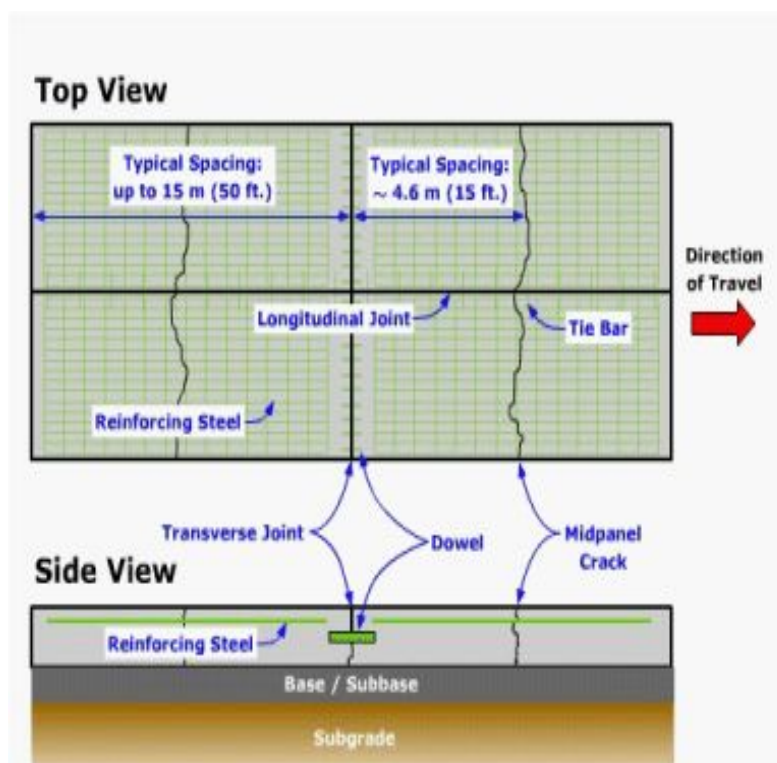


Figure 2-7. Jointed Reinforced Concrete Pavement (JRCP).

4.8 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.

4.9 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.

Section 5: Pavement Type Selection

5.1 Introduction

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. . . .”

5.2 Principal Factors

The principal factors to consider in the selection process are:

- Traffic
- Soils characteristics
- Weather
- Construction considerations
- Recycling opportunities
- Cost comparison.

5.3 Secondary Factors

The secondary factors to consider in the selection process are:

- Performance of similar pavements in the area
- Adjacent existing pavements
- Conservation of materials and energy
- Availability of local materials or contractor capabilities
- Traffic safety
- Incorporation of experimental features
- Stimulation of competition
- Municipal preference, participating of local government preference.

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

5.4 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 throughout the life of the facility, not just the initial investment, and allows for cost comparison of options with varying design lives and potentially differing user costs 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.

TxDOT developed [guidelines for LCCA](#) for pavement type selection.

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 and will be highly dependent upon the selected type and frequency of post-initial construction activities that extend the pavement service life to the end of the selected analysis period. It may be beneficial to evaluate user costs separately from agency costs when looking at different structural options.

Section 6: Approved Pavement Design Methods

6.1 Introduction

Use one of the following analytical methods for designing pavements:

- FPS 21 for flexible pavements
- Modified Texas Triaxial Design Method for flexible pavements
- TxCRCP-ME for continuously reinforced rigid pavements
- AASHTO design procedure (1993) for CPCD rigid pavements.

6.2 Flexible Pavement Design System (FPS 21)

For most flexible pavement design work, especially higher-volume highways (>10,000 ADT, 5 M ESALs), the Flexible Pavement Design System (FPS 21) is the required method for designing flexible pavements. FPS 21 should be used as a check for all flexible designs as described in “Pavement Design Process.” Design procedure training is available to department personnel through MNT – Pavement Asset Management.

- FPS 21 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 21 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 21 program assumes that a smaller deflection means smaller stresses or strains and, therefore, longer pavement life.
- Environmental influences including seasonal changes in material stiffness, frost heave, or moisture susceptibility of materials are not directly considered by the program. Impact of swelling foundation soils is no longer considered in FPS 21. Adding thickness to overcome swelling effects is not encouraged, except in very limited cases. For more information, go to Chapter 3, “Materials Investigation and Selection Information,” Section 2, “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 predictions. 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 21 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 Chapter 5, Section 3, “FPS 21 Design Parameters.”
- The [“Modified Texas Triaxial Design Method”](#) is included in FPS 21 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 HMA layers and full-depth rut life of the structure with options to use several strain-based performance models. It is highly recommended that the results of this check be considered for all pavement designs where the FPS-generated surface bituminous thickness is between 2 and 4 in.
- FPS 21 uses back-calculated modulus to characterize the pavement layer strength (stiffness) based on falling weight deflectometer (FWD) deflection measurements (see Chapter 4, “Pavement Evaluation,” Section 4, [Non-Destructive Evaluation of Pavement Structural Properties](#)). Note that back-calculated modulus used in FPS 21 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 backcalculation.

FPS-19W is the previous design program which has been replaced by FPS 21. Identical inputs used in FPS-19W will generate identical thickness designs in FPS 21, however FPS 21 is the required analytical method for designing flexible pavements.

6.3 Modified Texas Triaxial Design Method for Flexible Pavements

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 to occur in situ 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. This method has been automated and is included as a post-design check module in FPS 21. The method can also be used as a standalone tool for designs where traffic loading cannot 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. When soil testing cannot 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](#)). Also, within the FPS 21 software, the designer can opt to estimate the soil triaxial class if the in situ soil PI is known or if the in situ soil type for the project is known.

6.4 TxCRCP-ME (for Continuously Reinforced Concrete Pavements)

The TxCRCP-ME program is the only approved design method for CRCP projects at TxDOT. This design method was developed under TxDOT research project 0-5832, "Develop Mechanistic/Empirical Design for CRCP." The program performs an analysis of the pavement system for given inputs in estimating the frequency of punchouts, the primary structural distress of CRCP.

6.5 AASHTO 93 Design Procedure (for CPCD 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 conducted to that point. 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).

The 1993 AASHTO Guide for Design of Pavement Structures is the only approved design method for CPCD projects at TxDOT. This 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 design procedure is available in automated form in the AASHTO DARWin® 3.1 program and Web Application at <http://www.pavementinteractive.org/1993-aashto-rigid-pavement-structural-design-application/>.

For more information on using the AASHTO CPCD design procedure, refer to the *1993 AASHTO Guide for Design of Pavement Structures*.

Reinforcing steel design is reflected in the department's recommended CRCP and CPCD standards, found under the Pavements section on the [Roadway Standards webpage](#).

The TxCRCP-ME and AASHTO DARWin® 3.1 programs are available to TxDOT personnel through the district pavement engineer. Consultants may obtain the TxCRCP-ME program from the district pavement engineer or the Pavement Asset Management Section of the Maintenance Division.

Section 7: Pavement Design Categories

7.1 Definitions

There are four pavement design categories:

- “new” pavement design
- pavement reconstruction design
- pavement rehabilitation design
- pavement preservation.

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

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

Table 2-2: Pavement 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.
pavement preservation	Pavement preservation extends highway service life and provides smoother, safer and more reliable roads. Preservation approach selects the most cost-effective action to address a specific condition and performance need, providing the optimal means of minimizing life-cycle costs.
NOTE: These definitions do not necessarily coincide with program definitions.	

7.2 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 that 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.

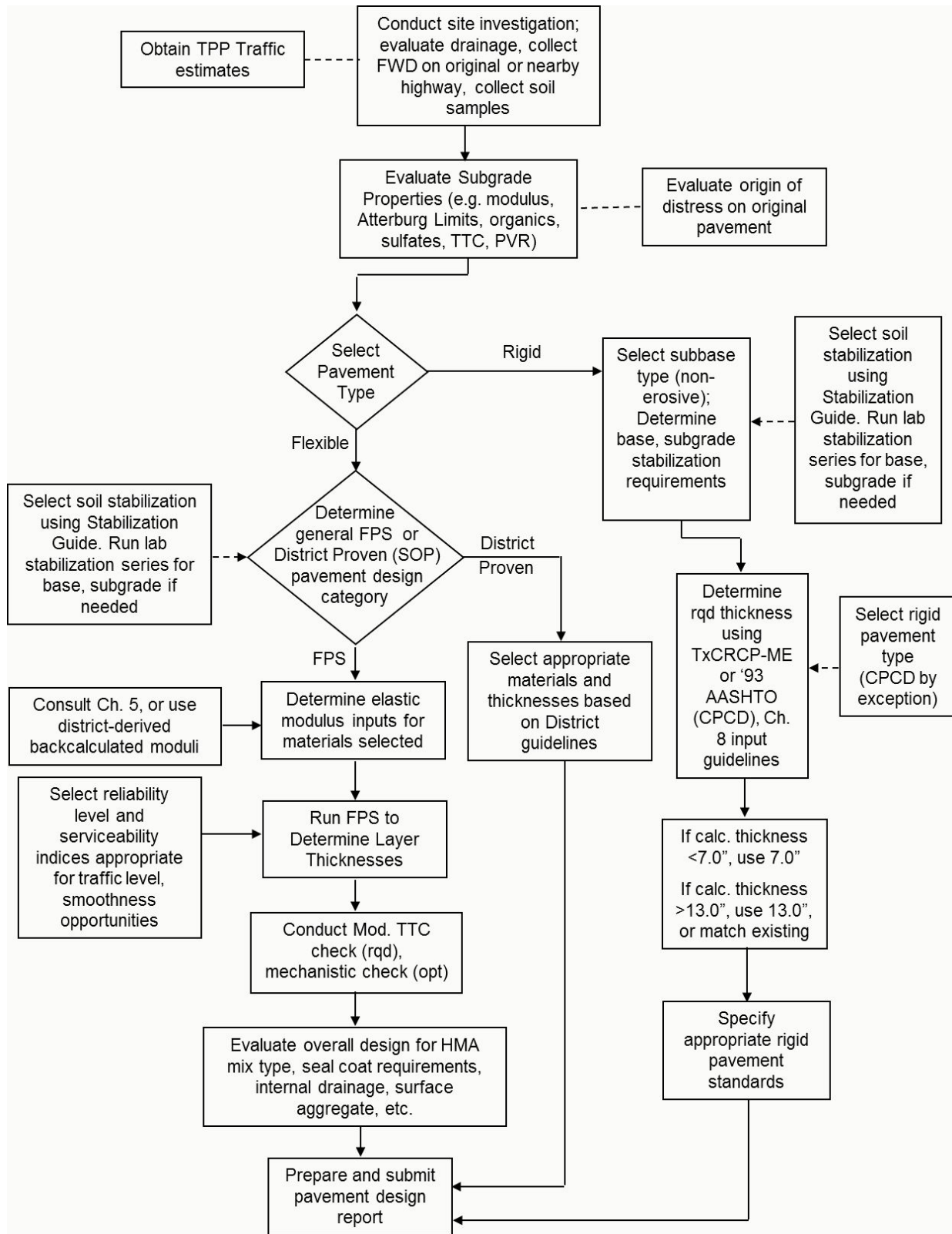


Figure 2-8. Design process for a new pavement/full reconstruction.

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 Courses available on the Maintenance Division website under the [Pavement Engineering](#) link. Refer to [Training](#) in Chapter 1, “Introduction,” for more information.

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

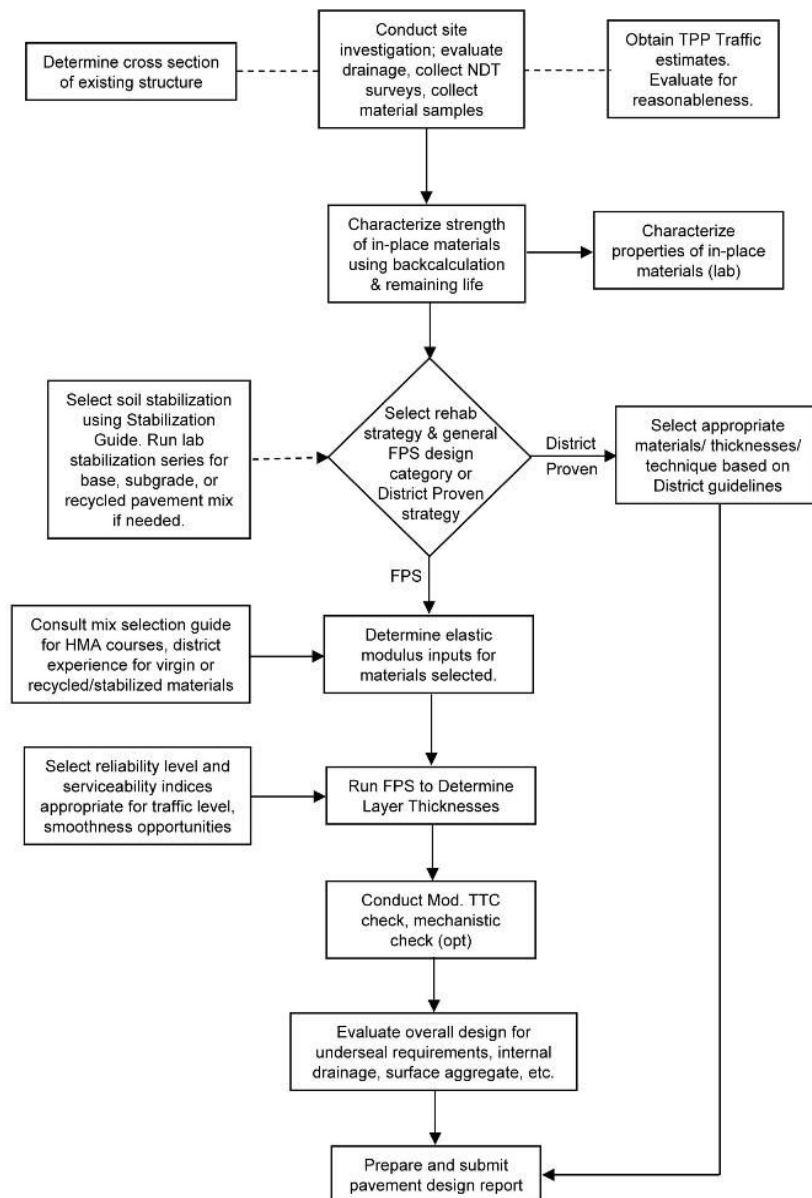


Figure 2-9. Design process for flexible pavement rehabilitation.

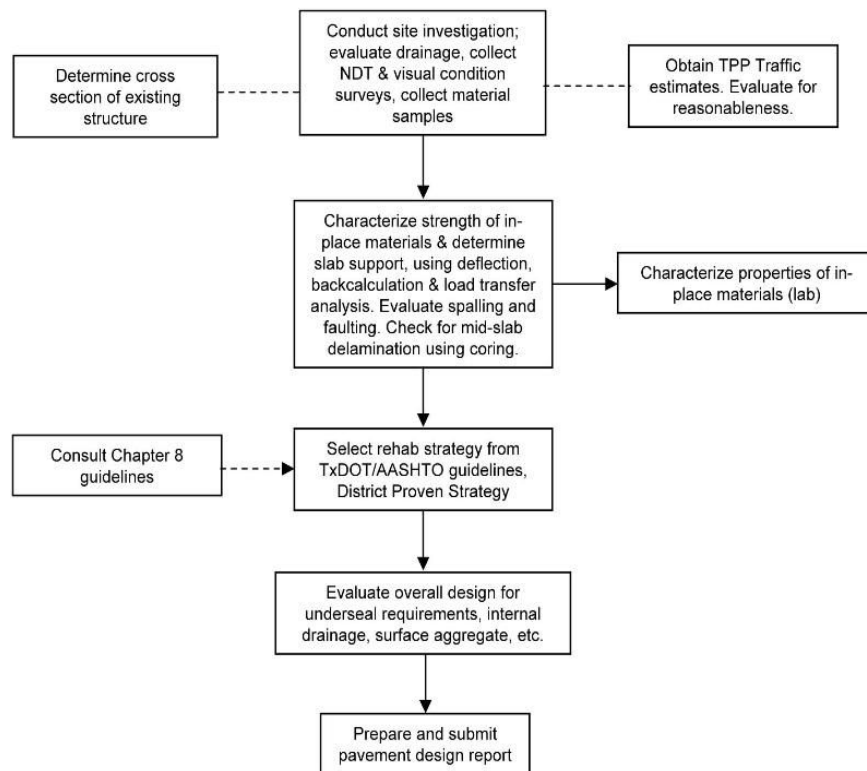


Figure 2-10. Design process for rigid pavement rehabilitation.

Pavement Preservation Design. Pavement preservation is planned and performed to improve or sustain the condition of pavements. Preservation activities generally do not add capacity or structural value but restore the overall condition of pavements. No pavement structural design is required for pavement preservation projects. Examples of preservation projects could include but not limit to seal coat, thin ACP overlays 2" thick or less, mill and inlay 2" thick or less, hot-in-place recycling, microsurfacing/slurry seal, scrub seal, half depth repair/full depth repair, diamond grinding and grooving, joint and/or crack sealing, and slab replacement.

7.3 Federal Aid Eligibility for Preservation and 3R/4R Projects

Preservation (also known as preventive maintenance) and rehabilitation projects start from the planning stage, referred to as the 4-year Pavement Management Plan. Districts plan these projects including timing, locations, treatment types, cost, etc., based on a series of factors such as pavement distresses, ride quality, skid resistance, age and other factors during the 4-year pavement management plan season. These projects are logged into two database systems, TxDOTCONNECT and Maintenance Management System (MMS). Then they are imported into a pavement management system, Pavement Analyst (PA), for evaluation from the network-level perspective. PA can forecast pavement condition, perform advanced analysis to answer what-if scenarios, and recommend optimized pavement work plans for preventive maintenance and rehabilitation to maximize the condition of the network based on constrained funding.

Projects in PA's final 4-year pavement management plan list classified as preventive maintenance would be eligible for federal-aid preservation funding with no pavement design or design exceptions required. Projects not on that optimized project list would be a routine maintenance project not eligible for federal-aid funding. 3R/4R projects are eligible for federal-aid funding but require a pavement design to show that the pavement structure is adequate for at least 10 years of future traffic, and all design exceptions would need to be addressed or formally approved.

Section 8: Information Needed for Pavement Design

8.1 Introduction

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
- existing functional and structural condition evaluation.

8.2 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:

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

8.2.1 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.

8.2.2 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, especially at depth. 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.

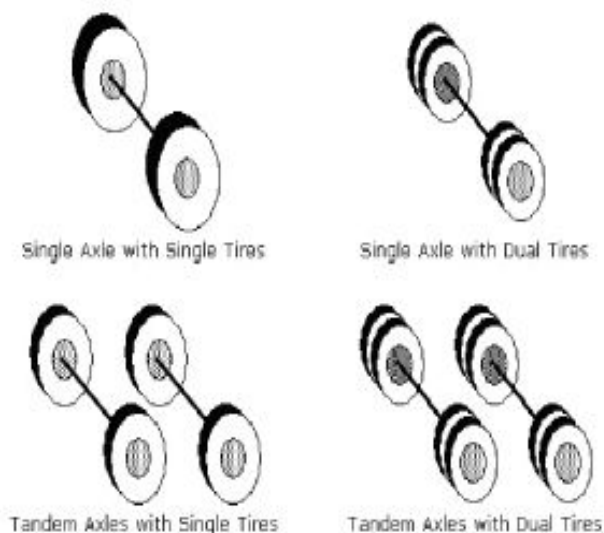


Figure 2-11. Tire axle configurations.

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

8.2.3 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 2-3: Axle Load Limits*

Axle**	Limits (lb.)
Single Axle	20,000
Tandem Axle	34,000

Table 2-3: Axle Load Limits*

Axle**	Limits (lb.)
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.	

8.2.4 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 become complicated to determine and are subject to uncertainties in traffic growth and changes to the traffic stream composition over time. Ultimately, it is not the wheel load, but the damage to the pavement caused by each wheel load that is of 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 18,000 lb. equivalent single axle load (normally designated ESAL). This is the loading format used in FPS 21, DARWin® 3.1, and TxCRCP-ME.
2. **Load spectra.** AASHTOWare Pavement ME (AASHTOWare mechanistic-empirical design system) essentially does away with the ESAL statistic and determines loading effects directly from axle configurations and loads. This is a more precise characterization of traffic but relies on the same raw 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. A separate table with the breakout of trucks by class (classes 4-13) tied to an initial average annual daily truck traffic (AADTT) count and growth rate are also needed. Load spectra data are commonly obtained from weigh-in-motion stations.

8.2.5 Traffic Distribution

Along with load type and repetitions, the load distributions across a particular pavement geometric section 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 and would be the design lane.

The Transportation Planning and Programming Division (TPP) posts a directional distribution statistic 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 as opposed to 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 (Form 2124) submitted to TPP for project level traffic data.

Recommended lane distribution factors for both flexible and rigid pavements designs are:

Table 2-4: Recommended Lane Distribution Factors

Traffic Lanes in One Direction	Lane Distribution Factor
1 or 2	1.00
3	0.70
4 or more	0.60

8.2.6 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 a 30-yr. traffic projection for rigid pavements from the Traffic Section of TPP.

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 Data¹. External requests for traffic data for purposes other than pavement design must be immediately referred to the open records 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. For check box “1” data, time frame to receive data should be no more than 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 2124 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. Currently, 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. The SN is a structural thickness index used in the AASHTO 93 method of flexible pavement design. These structural classes (8-in rigid slab or SN=3 for flexible) 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 when considering alternate pavement types. 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).

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 2124 including:

- District, County, CSJ
- Highway designation (System and Number)

¹. Available internally only.

- Limits (beginning and ending geographical/offsets in decimal miles from highway intersection)
- 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.
- Include 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/ bypass roadway facility
 - an increase in oil/gas field drilling or wind generator permits along a corridor.
- 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 in. If the slab will be thicker than 8.0 in., consult with MNT – Pavement Asset Management before changing.
 - AASHTO structural number for flexible pavements. NOTE: The default is 3. If the pavement structure will result in a higher structural number, consult with MNT – Pavement Asset Management before changing

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

8.2.7 Adjusting TPP-Supplied Traffic Data

Districts should review the “Traffic Analysis for Highway Design” report to verify reasonableness. Trucks have the largest impact on ESALs and, therefore, the simplest screening check is to verify the percentage of truck traffic in the average daily traffic (ADT) through observation. A simple ratio of actual truck percentages versus those estimated by TPP can be applied to the TPP estimated ESALs as a quick update for design purposes. A 48-hour continuous classification count within the project limits should be considered for jobs where extreme traffic loading is anticipated. Assigning reasonable truck factors (ESALs/truck) and growth rates to project cumulative ESALs over the design period can be performed using the DARWin® 3.1 ESAL generator or similar tool to compare against TPP supplied traffic data.

There may be occasions where the traffic forecast data supplied by TPP becomes slightly dated due to project letting delays or the original data supplied is for the wrong analysis time interval (e.g., 30-yr. data needed, but only 20-yr. data requested). In these cases, the designer may use TRAFFIC6.xls or similar routine to interpolate or extrapolate ADT and cumulative ESAL data using the same formula embedded within FPS. This formula automatically adjusts 20-yr. traffic data to match the user-selected analysis period. Use with caution; for job locations where traffic patterns have recently changed or will change in the near future, major roadways, or other jobs where the original traffic forecasts are more than 5 years old, the designer is encouraged to request an updated forecast from TPP.

For more information on traffic projections, refer to Chapter 1, Section 4 “[10430: Obtain traffic data](#)” in the Project Development Process Manual.

8.3 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 minimum “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.'

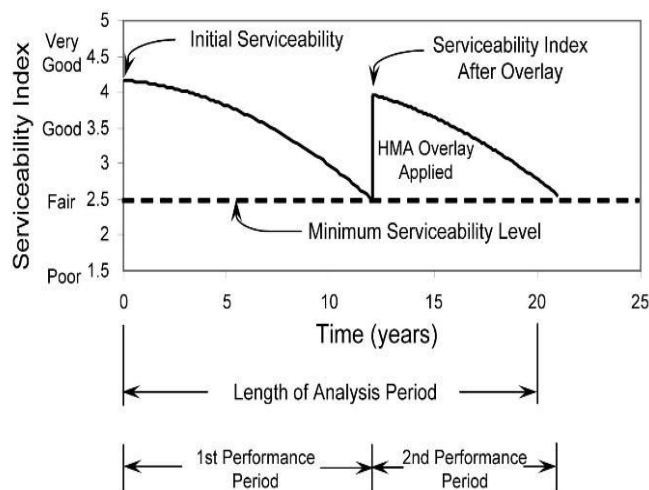


Figure 2-12. Pavement Performance Relationships.

8.4 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 an average modulus (based on specific material type, considering seasonal variation) for all inputs.

8.5 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 21.

Field tests are more practical and preferred because they can be performed rapidly and may be used to test a large volume of material at multiple project locations 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.

8.5.1 Laboratory Measurements of Moduli

The common way to develop the stress-strain relationship is through laboratory tests. The most recognized laboratory test method 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 Transducers (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.

8.5.2 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:

$$M_r = k_1 \sigma_c^{k_2} \sigma_d^{k_3}$$

Where:

M_r = resilient modulus

σ_c and σ_d = 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.

8.5.3 Bituminous Mixtures

Modulus of hot-mix asphalt (HMA) can be determined in several ways. The most common laboratory tests are the resilient modulus and the uniaxial frequency sweep.

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.

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 is an HMA input parameter in the AASHTOWare Pavement ME design process.

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

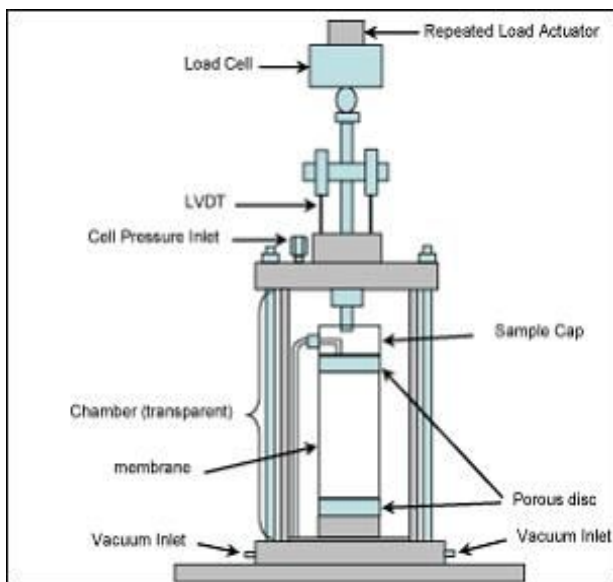


Figure 2-13. Resilient Modulus Testing Apparatus.

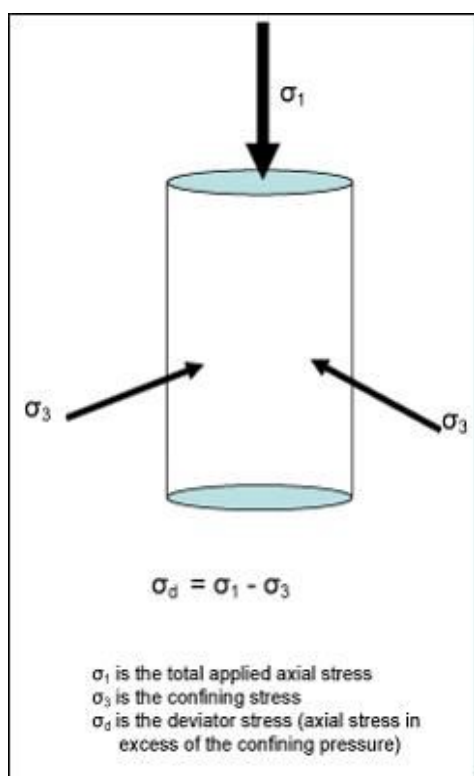


Figure 2-14. Stresses acting on a triaxial specimen.

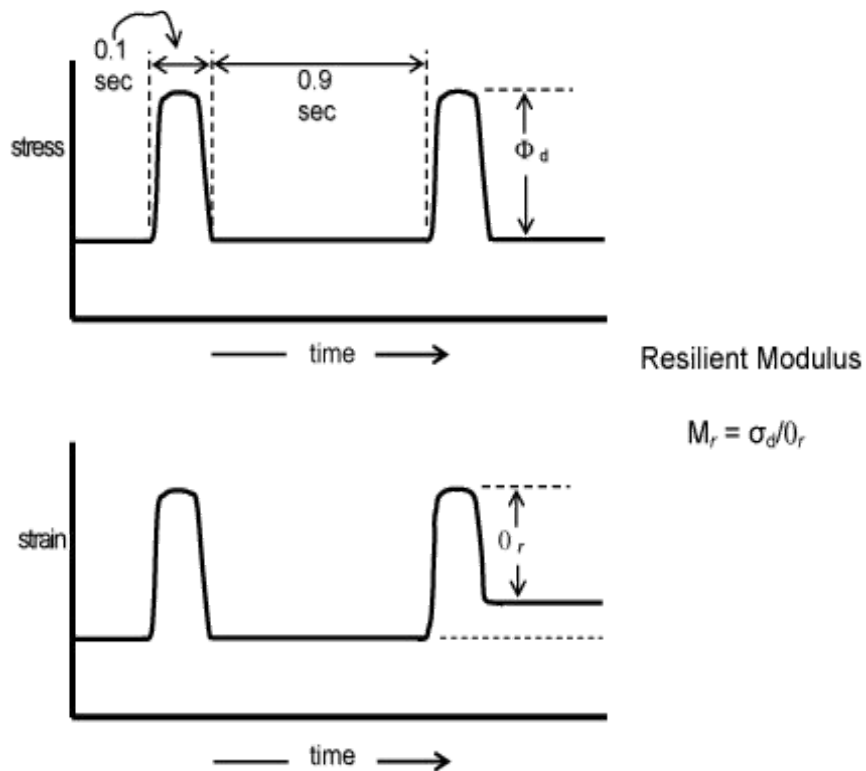


Figure 2-15. Resilient Modulus Test Cycle.

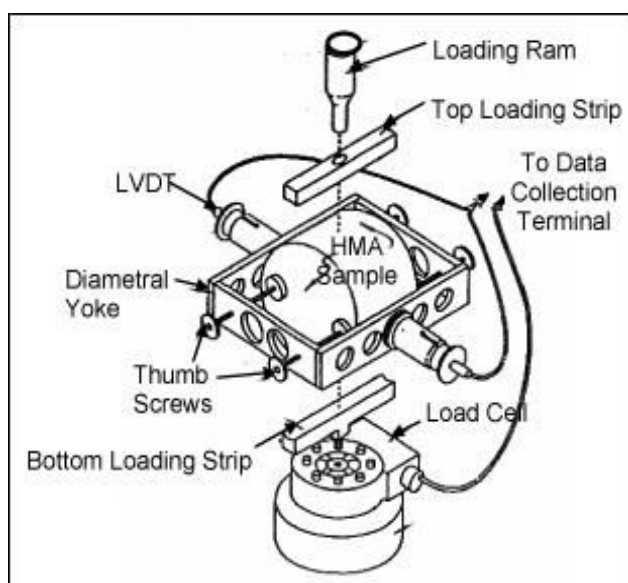


Figure 2-16. Diametral loading of HMA specimen for modulus testing.

8.5.4 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 (structural) 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 21 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 known layer thicknesses and 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 in Chapter 5, Section 4 (see “FPS 21 Modulus Inputs and Backcalculation Methodology”), in the manual.

8.5.5 Other Material Characterization Methods

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

8.6 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 cross slopes, condition of existing culverts, and 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 and may lead to wash outs of fill material behind bridge head walls, MSE embankments, and rip-rapped slopes.

8.6.1 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 is contemplated for use within the pavement structure, the department philosophy since 1994 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 cannot 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 cannot 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 and 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 cannot 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 or deep structures with impervious backfilled side slopes. Diligence in ensuring adequate compaction and thickness of pavement materials to support truck wheel loads above the retro-fitted drain is needed to prevent pavement failure at the lane/shoulder interface for traffic that may occasionally wander from the driving lane.

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

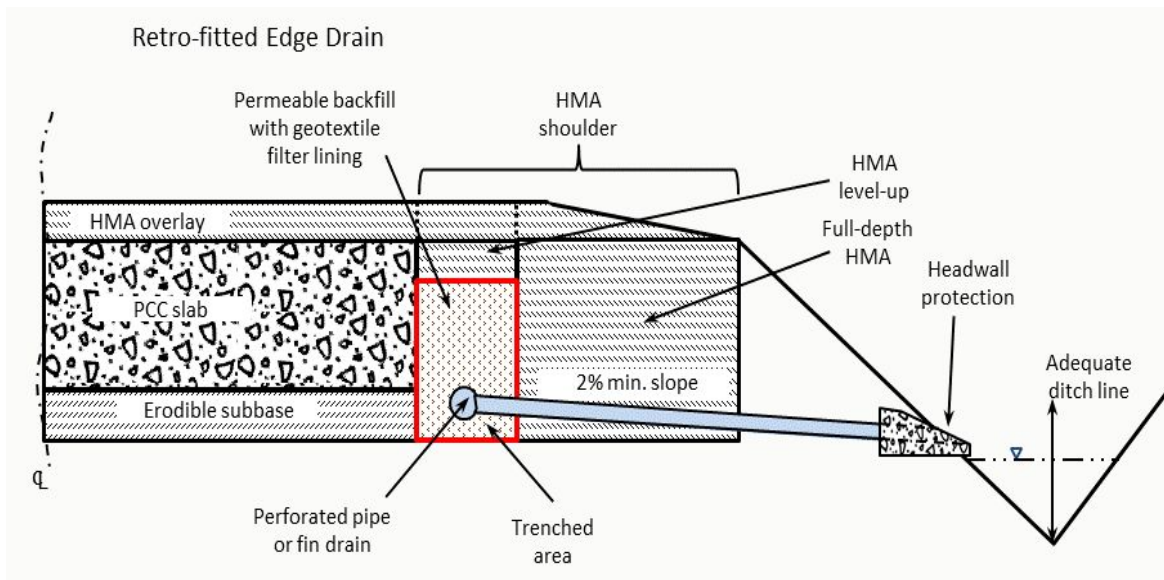


Figure 2-17. Retro-fitted Edge Drain.

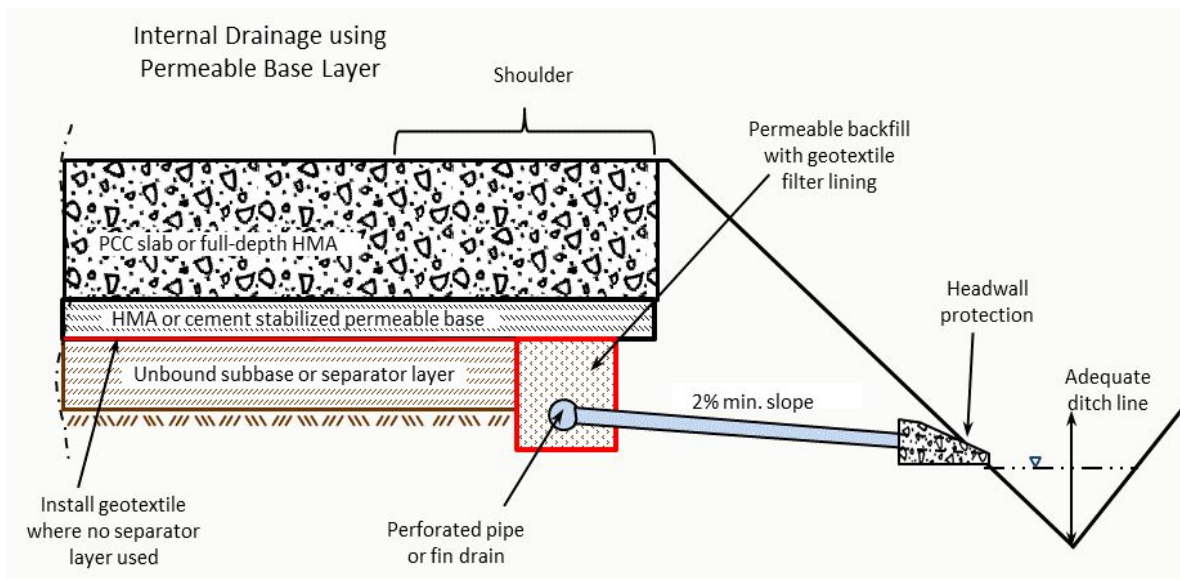


Figure 2-18. Positive drainage using a permeable base layer.

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 1-37A)*, ‘Appendix SS, Hydraulic Design, Maintenance, and Construction Details of Subsurface Drainage Systems.’

Cited references are available upon request from MNT – Pavement Asset Management.

8.7 Evaluating Existing Pavement Condition

The District will take adequate measures to properly characterize the existing functional and structural condition of pavements scheduled for rehabilitation. Evaluating a pavement’s existing condition is necessary for any rehabilitation or reconstruction project as a means to determine the adequacy of past performance, failure mechanisms, and 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, Chapter 4 “Pavement Evaluation.”

Section 9: Pavement Design Reports

9.1 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)
- unbonded concrete overlays of existing rigid pavements.

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

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

HMA overlays, approximately 2 in. thick and less, are considered pavement preservation; 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 manual, industry guidelines, and research findings). A design report may be required for documentation purposes.

The following list provides examples of special cases that do not require a full design report but do 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 TxACOL (Texas Asphalt Concrete Overlay software) developed through research [project 0-5123](#) should be considered when designing these overlays. This process evaluates suitability of proposed overlay HMA mixtures and thicknesses for reflective cracking and rutting performance. Another approach for designing an HMA overlay to an existing rigid pavement is the AASHTO Overlay procedure (automated in DARWin® 3.1). However, this process is highly subjective.

- bonded concrete overlays on rigid pavements (consult with MNT – Pavement Asset Management)
- thin whitetopping of flexible pavements (consult with MNT – Pavement Asset Management).

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

9.2 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 shall include the following when applicable:

- 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), multi-year PMIS (PA) 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. The USDA NRCS website at <http://websoilsurvey.nrcs.usda.gov/app/websoilsurvey.aspx> is an excellent resource for generating maps and soil summaries. 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, Chapter 3, 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 or use of geosynthetics 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.
- Form 2088, Surface Aggregate Selection Form as part of the flexible pavement design only. Information from this form will determine the appropriate Surface Aggregate Classification (SAC) of the aggregate used for the final hot-mix asphalt (HMA) riding surface.
- Results of NDT to characterize the existing structural condition (including the MODULUS backcalculation summary).
- Design input values and output:
 - FPS 21 summary, modified Texas Triaxial check, mechanistic checks, stress analysis, etc., for flexible pavement.
 - AASHTO (DARWin® 3.1) design summary for CPCD rigid pavements.
 - TxCRCP-ME Design summary for CRCP rigid pavements.
 - Alternate pavement design, if appropriate, using past successful practices/district SOP.
- 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 Surface Crash Reduction program, flexible pavements only).
 - Additional appendices (results of borings, material lab tests, raw PMIS (PA) data, life-cycle cost analysis, drainage analysis, design exceptional approvals, etc.), as needed.

For other reporting requirements, contact the DPE for guidance.

9.3 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 content, finalize the report by using the following procedure:

Table 2-5: Completing Pavement Design Report

Step	Action
1	• The district pavement engineer (DPE) reviews the technical content of the draft report and appropriateness of the design for conditions cited within the report. • Submit the draft report to MNT – Pavement Asset Management for review and comment.

Table 2-5: Completing Pavement Design Report

Step	Action
2	• Upon DPE approval of the report, the DPE signs and provides their respective professional 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 MNT – Pavement Asset Management for review and endorsement. • The District Engineer (DE) provides approval of final pavement design. If the report is approved, the approval is indicated by the DE signature and date on the report cover sheet. In metropolitan districts, the DE may delegate final pavement design approval to the District Director of Construction, Maintenance Operations, or Transportation Planning and Development for projects less than \$20 million.
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.

9.4 Pavement Design Report Review and Archive

MNT – Pavement Asset Management archives pavement designs for the state for the purposes of forensics and knowledge-based pavement performance. Submit a scanned copy of completed pavement design reports to pavementdesign@txdot.gov.

Archived reports can be viewed on the TxDOT intranet (site not available to internet users) at the “[Plans Online](#)” page.

Chapter 3: Materials Investigation and Selection Information

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Section 1: Overview

1.1 General

Material selection forms one of the three legs of the high performing pavement "stool," as shown in Figure 3-1. 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 consistency in 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.

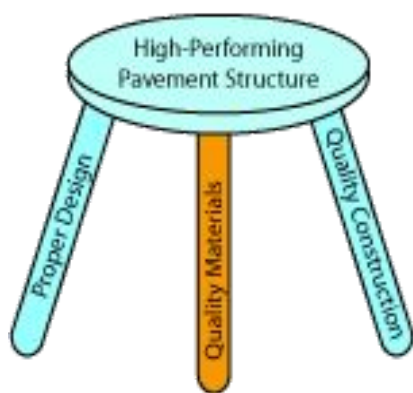


Figure 3-1. High Performing Pavement Stool.

Section 2: Geotechnical Investigation for Pavement Structures

2.1 Introduction

Soil is arguably the most critical component of any transportation system, since all transportation systems are built either on, in, or with soil and products from the ground. The characterization and evaluation of soil is critical to the performance of pavement structures. The guidelines provided herein will only address geotechnical considerations necessary for the design and evaluation of pavement structures.

These guidelines are prepared to provide department personnel, consultants, and contractors with guidance in:

- Determining soil properties and characteristics to be used in pavement design. These properties include, but are not limited to, soil strength, applicable modulus (or stiffness), and volumetric stability of a pavement structure; and
- 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.

2.1.1 Applicability

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

Although intended for all levels of involvement, the decisions that are made from an investigation are critical to the performance of the roadway. Contact the district pavement engineer (DPE) and/or materials engineer for further assistance and recommendations.

2.1.2 Background

It is important to determine the levels of investigation and when to perform them. From project conception to construction and throughout the operation and maintenance phases, geotechnical information is essential. Geotechnical investigations can be very general and cover broad geographic areas, such as an initial site investigation. They can also be very detailed and specific, such as identification of properties and characteristics of a single soil, as is often done in forensic studies.

Some of the 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 locations, reconstruction, reclamation of roadway materials, and resurfacing or overlay, required cuts, or fills).

2.1.3 Scope of Guidelines

These guidelines are 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](#).

2.2 Preliminary Investigation

Preliminary investigations 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:

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

2.2.1 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. Existing data should be reviewed at the beginning of the project as discussed later.

2.2.1.1 Project 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.

2.2.1.2 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 upon 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.
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.

2.2.2 Review of Existing Data

In addition to data available from the department resources and references, a significant volume of information has been compiled by other numerous organizations and agencies over the years, as shown in Table 3-1.

Table 3-1: Existing Data Sources

External Data Sources	Internal Data Sources
1. USDA, Natural Resources Conservation Service 2. Bureau of Economic Geology, University of Texas 3. US Geodetic Survey (USGS) 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. New PMIS (PA) with mapping functions

2.2.3 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 requiring confirmation as shown in Table 3-2.

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, drainage patterns, and water table proximity
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 evaluations

2.2.3.1 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 of soils. 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. 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 pyrite, 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.

2.2.3.2 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.

2.2.3.3 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.

2.2.3.4 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.

2.2.3.5 Vegetation

- Although vegetation is normally beneficial, having large trees or other vegetation requiring a sustainable water supply 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](#), “Calculating the Plasticity Index of Soils”) and large shrinkage potential ([Tex-107-E](#), “Determining the Bar Linear Shrinkage of Soils”).

2.2.3.6 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 critical when materials are to be chemically treated using calcium-based additives. The erosion potential of a soil can also indicate support conditions, existing drainage patterns, and whether specific drainage features will be necessary.

2.2.4 Preliminary Evaluation

Subsequent to the site visit, combine data and information collected to formulate requirements for structural support, subsurface explorations, non-destructive testing, and unique or problematic materials.

2.2.4.1 Structural Support

- From the information gathered in the preliminary stage, it is entirely possible to develop a trial pavement design. Soil maps generated from the Natural Resources Conservation Service [Web Soil Survey](#) may be used to identify the soil series that cross the proposed roadbed alignment.
- From the preliminary 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 in some way to best achieve overall project objectives. Balancing the requirements of the preliminary pavement structure and project constraints can assist in this process.

2.2.4.2 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 GPS (Globing Positioning System), stationing, or some other reference system,

such as TRM (Texas Reference Marker System). Further discussion of sampling requirements may be found under “Subsurface Exploration.”

2.2.4.3 Non-Destructive Testing (NDT)

- This testing may proceed at any point in the preliminary or design stages. The objectives of this testing are to evaluate the existing pavement structure and determine modulus values representative of the entire or discrete sections of the roadbed. The methods and analyses are discussed in Chapter 4, [Pavement Evaluation](#)
- Testing devices applicable to a geotechnical investigation frequently used in Texas are included in Table 3-3.

Table 3-3: Non-Destructive Tests for Geotechnical Investigations

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 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 using lower frequency antennae can penetrate to great depths, but are generally used to investigate unique phenomena, such as utility trench settlement. The nature of “ground coupling” also reduces production speed and is not suitable for project length surveys in most cases.

Table 3-3: Non-Destructive Tests for Geotechnical Investigations

Other Advanced Devices and Techniques:	The Total Pavement Acceptance Device (TPAD) is a multi-function evaluation vehicle combining the attributes of continuous deflection measurement, air-coupled GPR, high definition video, and global positioning. Data are used to determine uniformity of support. The Veris Soil Mapping System is used as a screening tool to measure the soil conductivity covering large areas. High conductivity values may be an indication of soluble sulfates present in the soil. It may be used at any stage of subgrade preparation and embankment construction to measure the conductivity within 2-4ft from the surface. Test results may be plotted on a color-coded map to illustrate the conductivity values. Areas of high conductivity may be selected for sampling and testing of sulfates in accordance with Tex-145-E.

- 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 (existing at the time of testing). To rely completely on a single measurement at one moisture condition may be misleading in determining an appropriate design modulus.

2.2.5 Preliminary Investigation Conclusions

Review of existing documents and information can be as varied as the extent of data obtained. It may or may not yield valuable information. 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 Table 3-4.

Table 3-4: 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

Table 3-4: Project Characterization Parameters Related to a Geotechnical Investigation

Project Characterizations	Soil Characterizations
6. General terrain and some estimate of cuts and fills required.	6. Guidance for subsurface exploration
	7. Plan development for non-destructive testing.

2.3 Subsurface Exploration

2.3.1 General

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.

2.3.1.1 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.

2.3.1.2 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.

2.3.1.3 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.

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.

2.3.1.4 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 listed in Table 3-5.

Table 3-5: Recommended Sampling Frequency

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 and soil organic content	500 ft.

2.3.1.5 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 feet.

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

2.3.2 Material Evaluation

TxDOT's laboratory 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 in Table 3-6 and briefly discussed.

Table 3-6: 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

Table 3-6: Typical Laboratory Soil Test Procedures

Test Category	Test	Test Method	Significance
	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
	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 additives
	Soil Conductivity	Tex-146-E	Field and lab detection of sulfate bearing soils
	Soil Organic Content	Tex-148-E	Determining soil organic content
	Soil pH	Tex-128-E	Determining the alkalinity or corrosivity of soils
	Soil Resistivity	Tex-129-E	Corrosivity of subgrade soils

2.3.2.1 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.

2.3.2.2 Swell potential

- The "Guidance on Potential Vertical Rise" memo (paraphrased below) 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. Consideration of PVR for design purposes is restricted to districts with areas of high soil moisture fluctuation and high plasticity index.

- Test method, [Tex-124-E](#), “Determining Potential Vertical Rise,” is the recommended procedure for determining PVR. A 15 ft. soil column is recommended for the analysis to determine PVR. The maximum allowable amount of PVR for design is 1.5 in. for main lanes (2.0 in. for frontage roads, when allowed), or less conservative (higher allowable swell) as established by individual district standard operating procedures (SOP). NOTE: The lower the PVR, the more conservative the design.
- A pavement structural design proposing to include PVR mitigation strategies will require the approval of MNT – Pavement Asset Management 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 (HMA) pavement > 12 in.

If the proposal meets all of the criteria above, the pavement design with the PVR mitigation strategy will be submitted to the Maintenance 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
 - constructability.
- PVR Treatment Strategies. Where the calculated PVR of the in situ soils exceeds the allowable for sections meeting the above criteria, mitigation of swell may be accomplished by one or more of the following methods:

- Chemical soil stabilization, in accordance with the “[Guidelines for Modification and Stabilization of Soils and Base for Use in Pavement Structures](#).” The target additive content must be designed to provide a permanently stabilized subgrade soil layer in accordance with the applicable test method for the type of additive under consideration.
- Undercut, remove, and replace expansive soils with select fill subbase. Select fill subbase should be placed for a depth of 2 ft. directly beneath the last structural pavement layer. Avoid friable, low plasticity materials, such as sands or loams, since these materials lack shear strength and will act as free moisture conduits that can further exacerbate shrink/swell potential of underlying high PI materials.
- Mechanical reinforcement with geosynthetics, such as geogrid, can be utilized when the top soil strata (1 to 3 ft.) is non-expansive and is underlain by expansive soils. For this situation, practical and economic considerations typically prohibit chemical treatment or undercutting to these depths. Use geogrid in the base course layers and/or use a thicker base course to compensate for any minor movement. Typically, when this occurs, the PVR only exceeds the maximum allowable limit by a small amount.

A design 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** (Comprehensive Development Agreement) firm to address a pavement maintenance clause.

2.3.2.3 Feasibility of Chemical Treatment

- Research has shown the potential for detrimental effects of introducing calcium-based additives into sulfate bearing soils. A protocol has been proposed and is discussed in the “[Guidelines for Treatment of Sulfate-Rich Soils and Bases in Pavement Structures](#).” The protocol evaluates the potential for the occurrence of detrimental reactions after the introduction of a calcium-based additive. If chemical treatment mitigation techniques are not successful, the alternative 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.

- Research has shown soil organic contents above 1% may reduce the effectiveness of calcium-based additives, and long-term strength of the treated soil may be reduced or not achievable using reasonable quantities of additive.

2.4 Treatment Guidelines

Satisfactory pavement performance is largely attributed to a good foundation that provides adequate strength and stability. 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 material engineering properties, ensuring adequate confinement and drainage. When widening or rehabilitating an existing pavement structure, it is essential to match the existing typical section when possible. Frequently, in situ soils and local base materials do not meet the material 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 treatment.

Material properties are improved by incorporating chemical additives, such as lime, cement, fly ash, emulsion, or asphalt. These additives, or a combination of these additives, are 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 of each treatment method.

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 considerations for incorporating chemical additives.

2.5 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 of a geotechnical design report is shown in Table 3-7.

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 summary of the analysis of all 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 3-7: Geotechnical Report

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Section 3: Flexible Base Selection

3.1 Introduction

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 may be used to assist department personnel with the selection of appropriate base grades. The descriptions that follow are contained in this guide; these bases are also most frequently prescribed by the standard specification Item 247. Other information, such as a background for establishing the current specification, structural notes, base selection factors, and base selection example, is also included.

3.2 Flexible Base Description

There are four 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.

Grade 1-2 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.

3.2.1 Grade 1-2

- When there is little confinement, a base material must provide its own stability. Grade 1-2 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 loading 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 traffic loads that are well below legal limits. When thinner (< 3 in.) HMA/bituminous surfacing is used, higher stresses are transmitted into the base material; the likelihood of base rutting and surface fatiguing will increase under these conditions.
- In the case of low confinement or little lateral support, Grade 1-2 is more likely to protect the HMA from failure at all levels of traffic. Grade 1-2 is best used with thin surfaces, little or no confinement (no shoulders), and for moderate to high traffic levels. The gradation for Grade 1-2 will almost always meet the requirements of Grade 5.

3.2.2 Grade 3

1. Available internally only.

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 backfilling pavement edges, rehabilitation, or shoulder work.

3.2.3 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 HMA 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 1-2. A strength requirement can be waived 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. Additionally, 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.

Consult with the Maintenance Division, Pavement Asset Management Section, before adjusting material requirements for reasons other than those listed above.

3.2.4 Grade 5

- Grade 5 material allows harder rock with a lower fines content. Fines may be less cohesive than those found in Grade 1-2. 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 1-2 base and has most of its characteristics, except for the unconfined compressive strength (UCS). 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 1-2 base when confined, a 3 psi lateral 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 recommended for high traffic roadways with thin surfaces or for roadways with no shoulders.

3.3 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 3-8. Grades recommended in a chart similar to Table 3-8 should be chosen based on local experience and historical performance.

Table 3-8: Example Flexible Base Selection Chart

Shoulder Width	HMA Surface Thickness	Traffic (Design ESALs) ¹	Base Grades
< 3 ft.	Surface Treatment	< 500,000	4*
		> 500,000	1-2
	HMA < 3 in.	All traffic levels	1-2
	HMA > 3 in.	< 500,000	4*
		> 500,000	1-2 or 5
> 3 ft.	Surface Treatment	< 500,000	4*
		> 500,000 and < 3,000,000	1-2 or 5
		> 3,000,000	1-2
	HMA < 3 in.	< 500,000	4*
		> 500,000	1-2
	HMA > 3 in.	< 500,000	4*
		> 500,000	1-2 or 5
¹ Percentage of trucks in addition to design ESALS should be taken into consideration. *Grade 1-2 requirements without minimum strengths.			

Section 4: Treated Subgrade and Base Courses

4.1 General

Frequently, local base and subgrade materials do not meet the material and engineering properties required for good pavement foundation performance. A large portion of pavement construction performed today consists of rehabilitating existing roads, which frequently contain reclaimed subgrade, base, and surfacing material layers that are inadequate for current or future traffic loading demands. In order to achieve the needed engineering properties, the base or subgrade frequently requires treatment. In rehabilitation projects, treatment is typically road-mixed in place in accordance with the [Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges](#), Items 260, 265, or 275 (road-mixed) to expedite construction and save cost, but better material uniformity is achieved by plant-mixing. For new and total reconstruction jobs, the designer may choose plant-mixed materials governed by the Standard Specifications, Items 263, 276, 292, 340, 341, or 344.

Where treatment is necessary to meet the engineering requirements, most materials can be 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. Base and subgrade materials are treated with chemical additives to achieve one or more goals when available materials do not meet project-specific requirements. Reasons for treatment include the following:

- increase strength to provide long-term support for the pavement structure,
- reduce the required pavement thickness,
- reduce moisture susceptibility and migration,
- allow for the use of local materials, and
- bind salvaged materials of differing composition for pavement rehabilitation projects.

In addition, goals of subgrade treatment include the following:

- reduce shrink/swell of expansive soils or existing materials, and
- provide a working platform for construction of subsequent layers by drying out wet areas and/or temporarily increasing strength properties.

[“Guidelines for Modification and Stabilization of Soils and Base 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 considerations for incorporating chemical additives.

Typically, permanent treatment is desired for base and subgrade materials, especially over the projected useful life of the pavement structure where modulus values are assumed constant in the design process. Structural credit above the intrinsic capability of the raw base and subgrade material should not be assigned to a treatment process that does not have proven lasting effectiveness. It is incumbent upon the design team to evaluate representative samples of the proposed base and subgrade materials under test methods appropriate for the additives being considered. Follow the recommendations in the [“Guidelines for Modification and Stabilization of Soils and Base in Pavement Structures”](#) or the applicable specification. Furthermore, strength testing under dry and wet conditions should be used to ensure the required properties are reliable under varying environmental conditions.

4.2 Cement, Lime and Fly Ash Treatments

These materials have a long history of use in various parts of the state and are generally the most cost efficient method of treating base and subgrade materials. Construction concerns when using these additives include the extended time necessary prior to opening the roadway to traffic. Additional concerns include inconsistent distribution of additives or mixing additives to the wrong depth when using “road-mixed” procedures. Too much additive, particularly cement, may result in extensive shrinkage cracking that may reflect through the surface of flexible pavements. A mitigation procedure to reduce shrinkage cracking in cement treated base is to microcrack the treated layer with a vibratory roller 24-48 hours after completing compaction and finishing.

Specific design strength requirements when using cement. To ensure long-term strength and stability of cement treated base (CTB) layers, a mix design must be completed to ensure sufficient amounts of cement are added to the mixture to achieve the target strength. Item 276, Cement Treatment (Plant-Mixed), currently designates three classes of cement-treated flexible base, based on 7-day unconfined compressive strength. Class M is intended for use with flexible pavements, Class L is intended for use with rigid pavements, and Class N may be used if the district has successful long-term experience with other strength values. When specifying Item 275, Cement Treatment (Road-Mixed), the laboratory target strength should be comparable to Item 276 requirements when selecting the appropriate cement content.

4.3 Asphalt Treatment (Plant-Mixed) Bases

Asphalt-treated base (ATB) is also known as asphalt-stabilized base (ASB), or simply “black base.” For the HMA options, lower layers are allowed to contain higher percentages of recycled products and a lower quality aggregate with less stringent material specification requirements as compared to surface mixtures. Standard mix design, production, placement, and field acceptance procedures govern these base mixtures in accordance with the applicable specification. These materials are ready to be trafficked as soon as compaction is completed and temperature is below 160°F, but the cost is high compared to other treated base options.

Where long-term moisture susceptibility of HMA and asphalt-treated base (ATB) is a concern, using a plan note to increase the target laboratory density or decrease Ndesign (and thus increase asphalt content) may be beneficial.

4.4 Emulsion and Foamed Asphalt Treatments

Item 314 and various one-time use special specifications allow the use of these additives. These treatment procedures are all intended for reclamation of in-place pavement structures. Application is typically accomplished using a reclaimer/recycler coupled to one or more bulk tankers that supply the liquid treatment agents. Recycling machines have become increasingly sophisticated and more powerful over the years; up to 12-in. of existing pavement depth can be processed with much improved distribution and uniformity of water and bituminous treatment agents. The emulsion content is typically below 4.0%, with the residual typically around 2.5%. Small percentages of lime or cement (typically $\leq 1.5\%$), when required by the mix design, are usually spread on the surface of the existing roadway ahead of the recycling machine. These additives are lifted and mixed together with the reclaimed roadway materials and liquid agents in the on-board mixing drum in a single pass. The lime or cement additives serve the following functions ([Wirtgen Cold Recycling Technology](#), 2012):

- improves adhesion of the bitumen to the aggregate,
- improves dispersion of the bitumen in the mix,
- modifies the plasticity of the natural materials (reduces PI),
- increases the stiffness of the mix and rate of strength gain, and
- accelerates curing of the compacted mix.

Alternately, a lime or cement slurry can be injected into the recycler mixing chamber through a separate spray bar attachment. These processes result in a product that locks up finer particles by encapsulating them in bitumen, causing them to adhere to larger particles that are not coated. The result is a material that remains more flexible than other chemically-treated materials, but significantly reduces the moisture sensitivity. A limitation for using these treatment methods is the lack of knowledge and experience for many in the mixture design and placement operations of these materials. Designing a foamed asphalt mixture requires a specialized laboratory foaming unit. Contact CST's Geotechnical, Soils and Aggregates Branch for assistance.

A significant precaution for these types of treatment options is dredging up high PI subgrade material and mixing it with the reclaimed pavement materials. The high PI materials will not disperse properly and cannot be adequately coated with bitumen; hence the material will remain moisture susceptible and prone to permanent deformation (rutting). This is more likely to occur with thin existing structures and structures with highly variable sectional thickness/composition along the project. Where this condition is anticipated and high PI soils are likely, the design process should include a preliminary recycling pass using lime, or install a lift of flexible base before the recycling pass to include this better material into the mix and to avoid dredging up the subgrade material. Curing emulsion treated layers to achieve strength and stability in a timely manner may be problematic where moisture loss is delayed by environmental conditions, such as

high humidity or unanticipated high in-place moisture contents. On the other hand, curing foamed asphalt treated layers can often be achieved in approximately 2 hours, and the roadway can be reopened to traffic.

Section 5: Performance Graded Binders (PG Binders)

5.1 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 (PG) binders in contrast to the older system of viscosity graded (AC) binders, which are typically used for surface treatments and aggregate precoating (see Item 300). Generally speaking, any paving mix produced hot through a hot-mix plant should be specified with a PG binder.

In the PG binder system, engineering properties believed to be related to the expected performance are measured at temperatures corresponding to the climatic and traffic conditions (maximum 7-day pavement temperature, minimum pavement temperature, loading duration based on truck speed, and traffic volume) of the pavement location. This allows selection of a binder grade that is specifically suited to the particular highway application.

PG binder grade designations are based on climate parameters, as explained below:

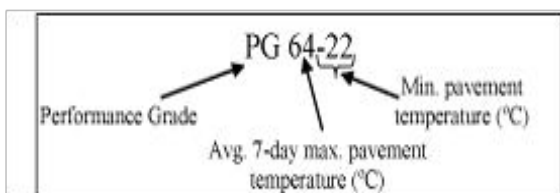


Figure 3-2. Performance Grade Binder.

5.2 Selecting a PG Binder

A “base” PG binder can be established for any region within the United States with sufficient climate data. Initial selection can be made using TxDOT’s computer program, [PG-SPG Grade Selection](#), or the [Federal Highway Administration’s LTPPBind](#) by specifying the geographic location and the desired level of confidence (usually 95% or 98%). TxDOT spreadsheet uses the older version of the LTPP temperature model that was in place when TxDOT implemented PG. The current LTPPbind model is a little different, and may give different grade recommendations. (Usually if there is a difference it’s one grade higher on the top end.). TxDOT has also generated maps ([95% confidence level](#), [98% confidence level](#)) showing the climate based grade for each Texas county.

For Texas, at the 95% confidence level, a PG 64-22 base binder can be used in most locations. It is the most commonly used binder by the department and is frequently used for subsurface mixes and low volume pavements. A few areas of the state with desert conditions may require PG 70 rather than 64. Likewise, some of the northern parts of the state use PG XX-28, rather than -22, due to the colder conditions.

“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 stone-matrix asphalt (SMA) and permeable friction course (PFC) require a minimum PG 76-XX binder) to insure reliable performance. Stiffer PG 70 or PG 76 binders are also commonly used for surface mixes in high traffic areas.

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. Also, the term “grade dumping” (decreasing the high binder grade and, in some cases, the low binder grade) has gained usage with the advent of using higher percentages of RAP (reclaimed asphalt pavement) and RAS (recycled asphalt shingles) in new mixtures, to mitigate the effects of binder stiffening caused by recycled binder substitution.

Where the potential for increased cracking problems exist, such as thin HMA surfaces on resilient bases, use caution when bumping the binder high temperature rating higher to address traffic issues. These conditions would typically require a stiffer binder, but 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 of 92°C or more between the low and high temperature portions of the grade. 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, use caution with grade bumping under these circumstances. Strain levels are typically maximized by traffic loading at the bottom of a 2- to 4-in. thick hot-mix asphalt (HMA) layer. Use of a stiff PG binder (such as PG 70 or higher) with recycled materials in these structures will severely limit the fatigue life of the mix. Chemically treating the lower pavement structure is an alternative if thin ACP surfaces are desired.

Additional guidelines in selecting a PG binder can be found in the document, “[Superpave Binder Materials Selection Procedures](#)” as well as the TxDOT’s [PG-SPG Grade Selection.xls](#) computer program.

Section 6: Hot-Mix Asphalt Pavement Mixtures

6.1 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 traditional 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 or organic binder additives, chemical mixture additives, foaming admixtures, and binder foaming using water-based 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,
- increased haul distance,
- improved workability of harsher mixtures including those incorporating reclaimed asphalt pavement (RAP) and recycled asphalt shingles (RAS), and
- potential lower oxidation/improved fatigue life.

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

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 (Small Quantity) and 341. 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 and low proportion of fines 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 permeable 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, Superpave 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.

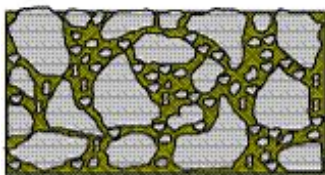


Fig. 3. 3a Dense-graded mix cross section

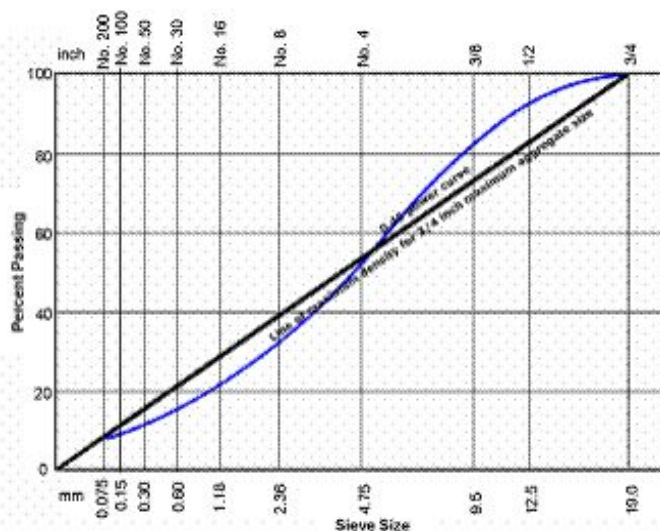


Fig. 3. 3b Typical gradation curve for a dense -graded mix.

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.

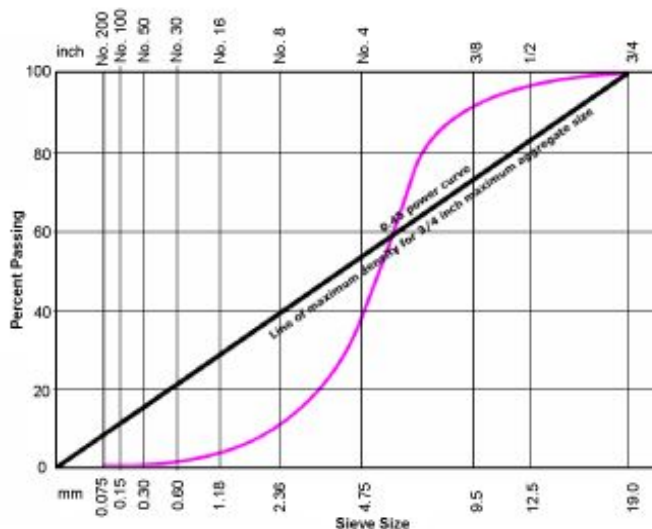


Fig. 3.4b Typical gradation curve for an open-graded mix

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

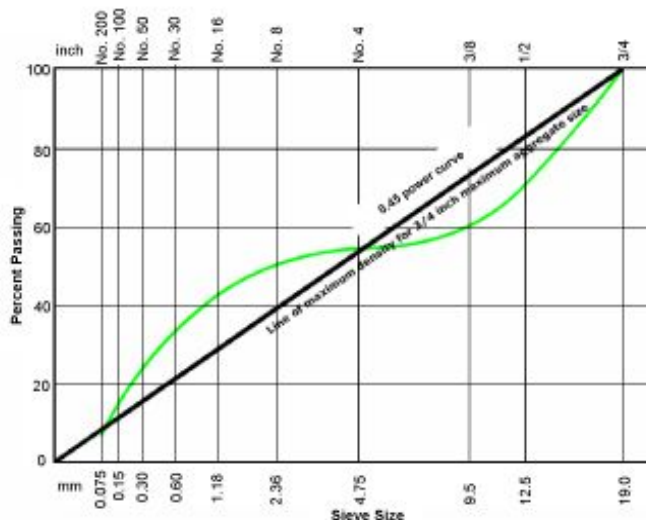


Fig. 3.5b Typical gradation curve for a gap-graded mix.

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

6.2 HMA Mix Design

Mix design is performed in a laboratory using one of the procedures outlined in [Tex-204-F](#), “Design of Bituminous Mixtures,” where the applicable procedure varies 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.

6.2.1 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 under slow or standing truck traffic 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 but proportional to asphalt film thickness. 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 Surface Crash Reduction Program Guidelines \(WSCRPG\)](#)¹, aggregates are classified into three categories (A, B, or C) based on a combination of frictional and durability properties. A friction demand assessment is made by the engineer. The proper aggregate or blend (using categories A and B only) to achieve the assessed rating is then selected.

Design is facilitated by the use of a series of [automated mix design programs](#) in Excel format. Mix designs can be generated in accordance with Tex-204-F by either department personnel or by a consultant/contractor who is certified by the department-approved hot-mix asphalt certification program. Plant mix or raw materials must be furnished by the contractor to the department project engineer to allow verification of the mix design.

6.2.2 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 in accordance with [Tex-206-E](#), "Compacting Specimens Using the Texas Gyratory Compactor [TGC]." Item 347, Thin Overlay Mixture [TOM], often associated with pavement preservation operations, can also be compacted using the 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 the specified target laboratory molded density.

6.2.3 Superpave Gyratory Compactor (SGC)

Superpave mixtures (Item 344), Permeable Friction Course (Item 342), Stone-Matrix Asphalt (Item 346), Thin Bonded Friction Courses (Item 348), and mixture designs used in Hot In-Place Recycling of Asphalt Concrete Surfaces (Item 358) must be compacted using the SGC in accordance with Tex-241-F, "Superpave Gyratory Compacting of Test Specimens of Bituminous Mixtures." Dense-graded mixtures (Items 340 and 341) may be compacted using the SGC with a density requirement of 96.0%.

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 of Tex-204-F require removal of this material because of the smaller TGC mold size. Samples are prepared at various asphalt binder contents around the estimated optimum content. Plots are generated within the design software to evaluate optimum binder content at the specified target laboratory molded 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. Depending upon the mix type, an N design (Ndes) related to minimum

1. Available through the TxDOT intranet only.

asphalt content and design air voids is established in each mixture specification. Ndes 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.

6.2.4 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 mixture design parameter 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.

6.2.5 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 to 93% theoretical maximum density in the Superpave Gyratory Compactor (SGC) using the optimum asphalt content (a requirement for all lab-prepared Hamburg test specimens).

The Hamburg test uses a pair of abutting, trimmed SGC samples placed in a 122°F (50°C) 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.

6.2.6 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.

The Overlay Tester has been implemented for select mixtures using test method [Tex-248-F](#).

Table 3-9: 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; Thin Overlay Mix (TOM)	TGC	Density: 96.5-97.5% Min. VMA by mix type	Indirect tensile strength (Tex-226-F), Hamburg (Tex-242-F), both at optimum AC content at 93 ±1% density; OT (Tex-248-F) for TOM only.	Mix designed by weight of constituent materials
		SGC	N _{des} 50 gyrations; Density: 96% Min. VMA by mix type.		
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	Refer to Part I				Previously Part III used for DG TY A/B mixes designed using the SGC
IV	Superpave SP-A, SP-B, SP-C, SP-D	SGC	N _{des} 50 gyrations; Density: 96% Min. VMA by mix type. Design VFA for SP mixes. By plan note, designate stone on stone contact.	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
V	Permeable Friction Course (PFC); Thin Bonded Permeable Friction Course	SGC	N _{des} 50 gyrations; Min. optimum asphalt content of 6.0% [7.0% for AR mixture]. Lab molded density 78% [PFC-F]–82% [all others]. Max. allowable draindown < 0.1%. No visible stripping by Tex-530-C .	Cantabro Loss (Tex-245-E) at optimum AC content at 78-82% density. Hamburg (Tex-242-F) and OT (Tex-248-F) for PFC-F only.	Mix designed by weight of constituent materials

Table 3-9: Tex-204-F Mix Design Options

Part	Type Mix	Compactor Used	Must Meet	Mix Evaluation	Comment
VI	Stone-Matrix Asphalt (SMA)	SGC	N _{des} 50 gyrations; Density: 96% Min. VMA Min. AC content 6% Must ensure stone-on-stone contact. Max. allowable draindown < 0.1%. No visible stripping by Tex-530-C.	Hamburg (Tex-242-F), OT (Tex 248-F) at optimum AC content at 93 ±1% density.	Mix designed by weight of constituent materials
VII	Stone-Matrix Asphalt Rubber (SMAR)	SGC	N _{des} 50 gyrations; Density: 96% Min. VMA higher for A-R binder Min. crumb rubber modifier content Other requirements as above for SMA	As above	Mix designed by weight of constituent materials
VIII	Thin Bonded Wearing Course	SGC	N _{des} 50 gyrations; Density: 92% Min. VMA Min. AC film thickness 9µm AC content % range: TY A: 5.0-5.8 TY B/C: 4.8-5.6 Max. allowable draindown < 0.1%. No visible stripping by Tex-530-C.	Cantabro Loss (Tex-245-F) at optimum AC content 92% density.	

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 engineer and contractor generally verify the JMF based on plant-produced mixture from a trial batch. The engineer may accept an existing mixture design previously used by the department and may waive the trial batch to verify the JMF. It is recommended that if the trial batch is waived, the mix design should have been developed and verified within the past 12 months.

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.

6.3 Guidelines for Selecting HMA Mixtures

Selection of an HMA mix or combination of mixes to use in a project should be a conscious decision made by the engineer based on mix attributes; evaluating 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.

6.4 Selecting Surface Aggregates to Comply With the Wet Surface Crash Reduction Program (WSCRCP)

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 the [WSCRCP](#)¹.

The department is charged with developing and implementing methodologies for the detection and improvement of locations with a significant incident of wet surface 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 being constructed provide adequate skid resistance,
2. develop an overview of the skid resistant properties of the highway system, and
3. provide information for use in developing safety improvement projects and the implementation of cost effective treatments at appropriate locations.

The WSCRCP 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. WSCRCP addresses three separate but interrelated phases of pavement friction safety. The three phases are crash analysis, aggregate selection, and skid testing.

- Crash analysis is the first phase and it consists of the identification, evaluation, and improvement (as needed) for all wet surface crash locations. The Traffic Operations Division publishes annual crash reports at the following location: <http://crossroads.org/trf/>². Under

1. Available through the TxDOT intranet only.

2. Available through the TxDOT intranet only.

Quick Links, click on “District Wet Surface Crash Reduction Program Location Reports” to see annual crash reports by district.

- 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 Source Quality Catalog](#) (BRSQC) updated (every 6 mos.) by the Geotechnical, Soils & Aggregates Branch of the Construction Division.
- 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 new Pavement Management Information System (PA).

Although the Geotechnical, Soils & Aggregates Branch of the Construction Division has been delegated responsibility for administering [WSCR](#)¹, it is the district’s responsibility to manage frictional properties for their pavements through sound engineering judgment and application of the program.

1.Available through the TxDOT intranet only.

Section 7: Concrete Materials

7.1 Hydraulic Cements

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, calcium hydroxide, and a few other compounds.

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 is 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, a small amount of gypsum is added to control the hydration of aluminates.

The four principal compounds of hydraulic cement are:

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 C150, Standard Specification for Portland Cement, provides for eight types of hydraulic cements; the most commonly used cements are listed below.

- 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 II: When moderate sulfate resistance or moderate heat of hydration is desired.

- 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 V: For use when high sulfate resistance is desired.

Most cement producers in Texas produce a Type I/II cement. This type of cement is widely used in TxDOT projects. This cement meets the requirements for both Type I and Type II cements, and can be used in concrete where either Type I or Type II cement is required.

7.2 Blended Cements

Blended hydraulic cements are produced by intimately and uniformly blending hydraulic cement with other types of fine materials at the cement plant during the clinker grinding process. The primary blending materials are fly ash, slag cement, and limestone. Other pozzolans, such as silica fume, can also be used in blended cements. In Item 421, four types of blended cements are allowed:

- Type IP consists essentially of an intimate and uniform blend of Portland cement and a pozzolan. DMS-4600 requires Type IP cements used for TxDOT projects to contain 20% to 40% of a Class F fly ash.
- Type IS consists essentially of an intimate and uniform blend of Portland cement and slag cement. DMS-4600 requires Type IS cements used for TxDOT projects to contain greater than 35% of slag cement.
- Type IT consists of an intimate and uniform blend of Portland cement and a combination of two other materials such as fly ash and silica fume, or fly ash and limestone.
- Type IL consists essentially of an intimate and uniform blend of Portland cement and a maximum of 15% limestone.

7.3 Fly Ash

Fly Ash is the most used Supplementary Cementing Material (SCM) in Texas due to its availability. Fly ash is a byproduct of the coal burning electric power generating plants. After the ignition of coal in the furnace, the ash residue is carried away by the exhaust gases, and is then collected with electrostatic precipitators or in filter bag houses. As the fly ash travels to the collectors, the material cools and forms spherical glassy particles.

There are two classes of fly ash, Class F and Class C fly ash. Fly ashes are categorized into these classes based on their chemical composition. Class F fly ashes have higher amounts of silica and lower amount of calcium, while Class C fly ashes tend to have lower amounts of silica and higher amount of calcium. Both classes of fly ashes are acceptable for use in concrete pavement provided the total cementitious content of concrete is 520 lb/CY or less.

Class F fly ashes are generally pozzolanic which means they possess little to no cementing properties. However, in the presence of water and calcium hydroxide, they will react and form compounds having cementing properties. Class F fly ashes are excellent in improving the long-

term durability of concrete. Class F fly ashes are effective in reducing permeability and mitigating alkali-silica reactions (ASR), delayed ettringite formation (DEF) and external sulfate attack at relatively low replacement rates.

Class C fly ashes have both pozzolanic and cementing properties. Due to the higher calcium content, Class C fly ashes will react with water and harden. Class C fly ashes are effective in reducing permeability, but not as effective in mitigating alkali-silica reactions (ASR), delayed ettringite formation (DEF) and external sulfate attack. High replacement rate of Class C fly ash are needed to mitigate ASR and DEF, and due to the chemistry of the glass phase, Class C fly ashes are susceptible to external sulfate attack and should not be used in sulfate environments. However, due to the low cementitious content of paving concrete mix designs, both ASR and DEF are not major concerns, and since concrete pavements are not in direct contact with natural subgrades, external sulfate attack is also not of concern; therefore, Class C fly ashes are allowed in paving concrete.

7.4 Aggregates

Coarse and fine aggregates make up 60% to 75% of the volume of the concrete mixtures and can have a strong influence on the fresh and hardened properties of the concrete. Almost any aggregate can be used to produce quality concrete provided the aggregates are durable and clean. Item 421 lists the aggregate requirements.

The particle size distribution or gradation of the aggregate is an important characteristic. Coarse aggregate Grades 2 and 3 listed in Item 421 are typically used in concrete pavements and have large nominal maximum aggregate size. Aggregates are typically gap graded, meaning they are missing sizes throughout the distribution. Gap graded aggregates tend to need more paste (cementitious materials and water) to fill the voids between aggregate particles. As the paste content increases, the amount of potential shrinkage also increases. One method to minimize paste content is to optimize the gradation of the aggregates.

7.4.1 Optimizing Aggregate Gradations

There are several methods that can be utilized to analyze aggregate gradation, and each method has its own pros and cons. The department uses a recently developed percent retained method commonly called the “Tarantula Curve.” In this method, the combined percent retained on every standard sieve for coarse and fine aggregate is used to analyze the gradation. If the percent retained on each sieve is within the limits of the Tarantula Curve, then the gradation is considered optimized. Procedures for analyzing aggregate gradations are outline in Tex-470-A, “Optimized Aggregate Gradation for Hydraulic Cement Concrete Mix Designs.” It is important to note that if an effort is not made to reduce paste content in the mix design, then there are no benefits to utilizing optimized gradation.

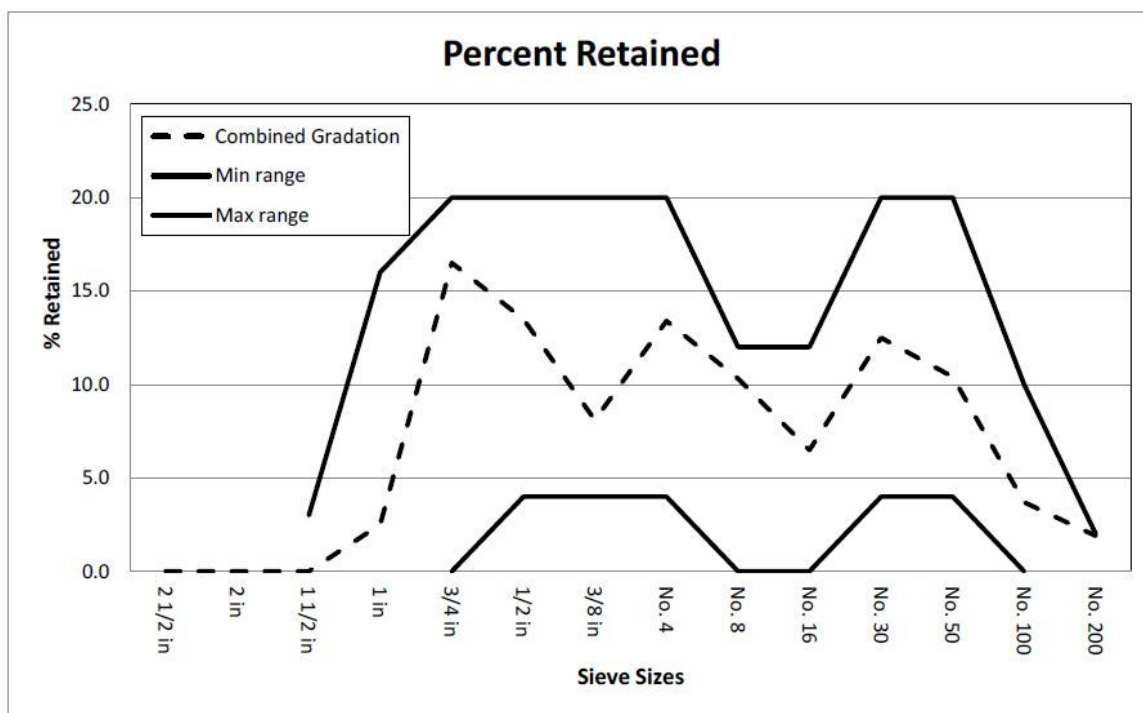


Figure 3-6. Percent Retained Chart.

7.4.2 Coefficient of Thermal Expansion of Coarse Aggregates

Since the coarse aggregate makes up the majority of the volume of the concrete, it has a large impact on the coefficient of thermal expansion (CTE) of the overall concrete. The CTE of the concrete influences the long-term performance of CRCP. CRCP sections constructed with concrete having a high CTE value result in shallow spalling at every transverse crack location causing rough ride of the pavement. Through several years of research, it has been determined that if the CTE of the concrete is limited to not more than 5.5 microstrain/°F, the shallow spalling problem was virtually eliminated. Item 360 limits the CTE of concrete used for CRCP to not more than 5.5 microstrain/°F.

7.5 Water

Any municipal water source approved by the Department of Health can be used in concrete with no additional testing. Water sources or blends of concrete wash water not approved by the Department of Health can still be used provided they do not contain deleterious amounts of ions such as alkali, chloride, and sulfates, or total solids. Item 421 lists the requirements that water from non-municipal sources must meet. In addition to the chemical compositions of the water, the effect that non-municipal water sources have on concrete setting time and strength must also be evaluated.

7.6 Chemical Admixtures

Chemical admixtures are used to enhance both the fresh and hardened properties of concrete. The department pre-approves several types of chemical admixtures for use. The pre-approved types are listed below.

- **Type A – Water Reducing Admixture:** These admixtures reduce the quantity of mixing water required to produce concrete of a given consistency.
- **Type B – Retarding Admixture:** These admixtures retard the setting of concrete.
- **Type C – Accelerating Admixture:** These admixtures accelerate the setting and early strength development of concrete.
- **Type D – Water-Reducing and Retarding Admixture:** These admixtures reduce the quantity of mixing water required to produce concrete of a given consistency and retard the setting of concrete.
- **Type G – High Range Water Reducing Admixture:** These admixtures reduce the quantity of mixing water required to produce concrete of a given consistency by 12% or greater.
- **Type F – High Range Water Reducing and Retarding:** These admixtures reduce the quantity of mixing water required to produce concrete of a given consistency by 12% or greater and retard the setting of concrete.
- **Air Entraining Admixture:** These admixtures are used to stabilize air mixed into the concrete during mixing and create a system of small, closely spaced air voids within the concrete.

The common types of chemical admixtures used in paving concrete are Type A, Type B, Type D, and air entraining admixtures. Since most concrete pavement are placed with a slip form paver, a high degree of workability is not necessary or wanted; therefore, high range water reducers are not used. Some of the recent admixtures are called mid-range water reducers. There is not an official ASTM classification for these admixtures, but they are usually approved as Type A or Type F admixture. These mid-range admixtures provide slightly more water reduction compared to normal range water reducers, reduce stickiness, and improve finishing, pumping, and placing properties.

Section 8: 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 and CPCD Design Standards](#).

Section 9: Hydraulic Cement Concrete

9.1 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 slag cement), aggregates (coarse and fine), water, and chemical admixtures.

9.2 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, 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.

9.3 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 shrinkage and 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:

- each component material should meet the minimum properties as required in Item 421, and
- the proportions of the component materials should be designed to minimize paste volume.

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. Concrete with gap graded aggregates tend to require more paste to achieve a desired workability. Concrete with more coarse aggregate volume per unit volume of concrete (optimized) tend to need less paste to fill the voids between the aggregate. Having less paste results in less heat of hydration, less drying shrinkage, and less potential for cracking. All these provide for better long-term pavement performance.

9.4 Three Concrete Classes

In the 2014 TxDOT specifications, there are three classes of concrete related to concrete pavement:

- Class P,
- Class K, and
- Class HES.

Table 3-10: 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 • when used, strength requirements must be shown on the plans
HES	• originally 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 • when used, default strength requirements are included in Item 360

Section 10: Geosynthetics in Pavement Structures

10.1 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 this definition was established, geosynthetics have evolved to include many more complex (non-planar) formats. Also, the scope of applications now exceeds 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 address only 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 conclusive 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, the department’s 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.

10.2 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).

10.2.1 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; geotextiles are most frequently used in HMA but do have applications with soil moisture barriers, unbound base confinement, 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 HMA. Geocells can be thought of as a 3-dimensional grid allowing confinement of unbound low cohesion materials in-depth. Geomembranes are low-permeability geosynthetics used as moisture barriers.

10.2.2 Applications

Primarily, there are four applications of geosynthetics typically used by the department within pavement structures as given in Table 3-11:

Table 3-11: 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 unbound bases, subbases, and subgrades. In this application, soils are restrained from movement, thus mechanically stabilizing the layer.
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	

10.3 Geosynthetics for Surface Layer Reinforcement

Geosynthetics used for HMA 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 HMA overlay reflective cracking from:
 - existing layers of HMA, or
 - cracks and joints in rigid pavements, and
- resist moisture intrusion into lower pavement layers.

Department research project 0-1777 investigated the use of several geosynthetics in HMA. 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.

10.4 Geosynthetics for Geotechnical Reinforcement

10.4.1 General

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 has been conducted in pooled fund studies.

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 21 design. An exception to quantifying the structural contribution might be in the use of geocells that confine unbound base material, but no use has been documented in Texas.

10.4.2 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 while preventing subgrade cracking from propagating to the pavement surface.

10.4.3 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.

Geocells have been used to confine low cohesion base materials, allowing these materials to be used more effectively in a pavement structure.

10.5 Geosynthetics for Drainage Applications

Among the products used in drainage applications are:

Table 3-12: 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	• Vertical moisture barrier for pavement edges. • Creation of retention/detention ponds. • Encapsulation provision for confining and waterproofing material in the roadbed.
Geoweb	• 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.

10.6 Specifications and Testing

Table 3-13 shows the applicable [Departmental Material Specifications \(DMS\)](#) and test procedures used to qualify geosynthetics used in the department. Governing construction special specifications

are located at: <http://www.dot.state.tx.us/apps-cg/specs/ShowSD.asp?year=4&type=SS&number=5>.

Table 3-13: Departmental Material Specifications

Specification	Title	Geosynthetic	Application
DMS-6200	Filter Fabric	Geotextile	Geotechnical
DMS-6210	Vertical Moisture Barrier	Geomembrane	Geotechnical
DMS-6220	Fabric For Underseals	Geotextile	HMA
DMS-6230	Temporary Sediment Control Fence Fabric	Geotextile	Drainage
DMS-6240	Geogrid for Base/Embankment Reinforcement	Geogrid	Geotechnical
DMS-6270	Biaxial Geogrid for Environmental Cracking	Geogrid	HMA
DMS-6260	Reinforced Fabric Joint Underseal	Geocomposite	HMA
DMS-6300	Waterproofing	Geomembrane	Drainage/Waterproofing

Chapter 4: Pavement Evaluation

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[Section 6: Geotechnical Investigation for Pavement Structures](#)

Section 1: Overview

1.1 Introduction

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 safety aspects of a highway section (surface texture, cross slope, splash and spray, etc.). 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 and the in situ material properties in order to determine the best rehabilitation strategies.

1.2 Visual Condition Surveys

Visual condition surveys document aspects of both functional and structural pavement conditions, 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.

1.3 Non-destructive Testing (NDT)

NDT 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 pretesting 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. For NDT, data collected in the field is generally objective in nature, but engineer's data analysis and interpretation are subjective.

1.4 Destructive Testing

Destructive testing opens the door to characterization of the pavement's constituent materials not possible to obtain through non-destructive testing alone. Material characterization includes:

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

Section 2: Visual Pavement Condition Surveys

2.1 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 to provide instructions in recording these observations in an “orderly and consistent manner.” To this end, certification training for visual raters is conducted annually in regional sessions.

The designer/planner is interested in the type, extent, and severity of visible distresses or corrective action taken on previous distresses (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 New Pavement Management Information System (PMIS) PA 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 found in the Maintenance Division website under the [Pavement Preservation Section](#) link and the Flexible and Concrete Pavement Rehabilitation Strategies Training courses found in the Construction Division website under the [Pavement Engineering](#) link. These courses discuss typical pavement distresses, their causes, and actions required to remedy.

2.2 Flexible Pavement Visual Survey Condition Categories

2.2.1 Rutting (Shallow, Deep, and Severe)

Rutting is a longitudinal surface depression in a wheel path and is a load-associated distress. Consolidation or lateral movement of the pavement materials due to traffic loads causes rutting. 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 resulting from uncontrolled asphalt cement content (poor plant operations) that may cause shoving and displacement of hot-mix asphalt (HMA),
- moisture sensitive materials (including stripped HMA in non-surface mixes),

- post-construction consolidation of HMA or unbound layers under traffic loads (air void content too high), or
- temperature-sensitive HMA mixtures and inappropriate HMA mix selection for the nature of the traffic loading (free-flowing traffic at highway speeds versus slow moving/accelerating/decelerating environments).

Rutting is rated by area and severity, with the area measured as a percent of the section's total wheel path area. Severity of rutting is described in terms of depth:

- Shallow, 0.25 - 0.49 in.
- Deep, 0.50 - 0.99 in.
- Severe, 1.00 - 1.99 in.
- Failure, 2.00 in. or greater.

At the network level, rutting is evaluated using the automated rut-measuring system on the profiler/rutbar vehicle. Project level surveys can also be conducted using the profiler/rutbar vehicle or manually using a 6.0-ft. straight edge and a steel ruler. Failure rutting (> 2.0 in.) must be measured using a 6.0-ft. straight edge. 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.

2.2.2 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 HMA 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 converted to the percentage of the rated lane's total surface area. 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.

2.2.3 Failures

Failures are localized areas of severe distress where the surface has been severely eroded, badly cracked (where blocks of surface material move about freely), severely faulted, depressed, or severely shoved. A group (3 or more) of small potholes 4-12 in. in diameter and deeper than 2 in., or a single pothole greater than 12 in. in diameter and deeper than 2 in., are classic examples of a failure, 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 have an 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 HMA density, surface HMA lift not bonded to underlying HMA, 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. Failed areas longer than 40 ft. are considered to be multiple failures.

2.2.4 Block Cracking

Block cracking is a climate/materials related distress, where age-hardening of the asphalt coupled with 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 rendered in terms of total feet of full-lane width block cracking converted to 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.

2.2.5 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. Alligator cracks are formed whenever the pavement surface is repeatedly flexed under traffic loads. 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 for the current traffic loads (including thin HMA surfacings), surface layer delaminations, poor construction practices and/or weak materials. Total feet of alligator cracking is converted to 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.

2.2.6 Longitudinal Cracking

Longitudinal cracking consists of cracks or breaks that run approximately parallel to the pavement centerline and may appear anywhere along a shoulder or driving lane. 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. Longitudinal cracking

wider than 2.0 in. or faulted greater than 2.0 in. is rated as a failure. 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.

Longitudinal cracks may occur as a result of poorly constructed HMA mat joints, thermal shrinkage, inadequate support, or reflection from underlying layers. Differential movement beneath the surface is the primary cause of longitudinal cracking outside of the wheel path.

Environmentally-induced longitudinal cracking may originate in desiccated subgrade soils (edge drying), embankment consolidation/slope failures, widening interfaces or active portland cement concrete (PCC) pavement joints reflecting to the surface. 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.

2.2.7 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 frequently associated with environmental surface shrinkage due to temperature cycling, or may result from differential movement beneath the pavement surface. Transverse cracking can deteriorate further under traffic effects and surface moisture infiltration. Movement beneath the pavement surface may be from natural shrinkage of chemically stabilized base and subbase materials, or reflection cracking from underlying PCC joints and active cracks. Material properties can also be a contributing factor, including asphalt cement (AC) binder aging, stiffness of the HMA, or percent stabilization used in the lower layers. The activity of the cracks and their origin must be considered in choosing a rehabilitation strategy.

2.2.8 Raveling

Raveling is the progressive disintegration of the surface due to dislodgment of aggregate particles caused by weathering, traffic, or a combination of the two. In Texas, raveling is mainly associated with seal coats; however, it can occur in HMA 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 (or low seal shot rate/poor chip embedment), stripping of the binder, and HMA segregation/high air voids. Corrective action can include applying a fog seal (short term), seal coat, hot-in-place recycling, or thin overlay.

2.2.9 Flushing

This distress is also known as bleeding and is described as the presence of excess asphalt on the pavement surface. The condition is generally more prevalent in the wheel paths and can be present in both HMA 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 length affected and by degree of severity (low, medium, high). Underlying causes can include high AC content (or seal shot rate too high in the wheel path), 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/use of variable transverse asphalt rates), cold milling with subsequent seal or thin overlay, a permeable friction course, or a thin overlay. Hydroblasting the wheel path using a specialized vacuum recovery vehicle has also been used with success for the removal of excess asphalt.

2.3 Rigid Pavement Visual Survey Condition Categories

2.3.1 Spalled Cracks

A spalled crack is a crack that shows signs of chipping on either side, along some or all of its width. In the TxDOT's new PMIS (PA) 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 1.0-in. wide for CPCD or 3-in. wide for CRCP, and covering more than 1 ft. of the crack length. Spalling on CRCP or CPCD longitudinal cracks are not rated.

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 at 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 HMA overlay.

2.3.2 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 must be at least 12 in. long or wide to be rated as a punchout. For punchouts longer than 10 ft., rate one punchout for each 10 ft. of length. Punchouts are most common in continuously reinforced concrete pavements. Where they 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. Research has shown that punchouts can be partial depth failures, where horizontal cracking initiates at the reinforcing steel plane due to early age stress in the slab. Longitudinal cracking connecting closely-spaced transverse cracks will form along the axis of longitudinal steel due to traffic loading. Furthermore, locations adjacent to transverse construction joints may be more susceptible to this type of failure because of limitations in slip-form paving in their proximity and consolidation practices at the joint. 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 for full-depth punchouts. Half-depth PCC patching is the preferred method of repair for partial-depth failures. Asphalt concrete pavement patches may be used temporarily to address safety issues. Rating is in terms of number of occurrences per mile.

2.3.3 Asphalt Patches

In rigid pavements, full-depth repairs to localized distress are often made using asphalt concrete as an expeditious, temporary fix. Asphalt concrete pavement patches used to address spalls are not counted. 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 new PMIS (PA) 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.”

2.3.4 Concrete Patches

Concrete patches are localized areas of newer concrete intended as “longer lasting” repairs of localized surface or structural 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.

Currently, TxDOT publishes standards for full-depth, full-lane width patches that must be 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. The depth and size of half-depth PCC patches will vary based on the conditions of partial-depth failures. Concrete patches are evaluated in terms of total number observed. Width of the patch is not considered. However, long patches are rated as one patch for every 10 ft. Proper patch construction is discussed in Chapter 10, “Rigid Pavement Rehabilitation.”

2.3.5 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 in terms of ft./crack.

2.3.6 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 in the form of spalling or faulting 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, crescent-shaped hairline cracks which tend to cluster together along joints, slab edges, and larger transverse/longitudinal cracks. They are concave away from the slab corner, joint, or edge, 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/contaminated aggregates subjected to freeze-thaw temperature cycles.

2.3.7 Shattered Slabs

This is a CPCD condition category. A shattered slab is a slab that is so badly cracked that it warrants complete replacement. 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. 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. The current department policy requires a non-erodible base beneath all rigid pavements.

2.3.8 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 (> 1 -in. wide on either side for more than half its length), or are faulted at least $1/4$ -in., are counted as one slab with longitudinal cracks, 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 sawn quickly enough during construction (ideally, such joints should be sawn 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.

2.3.9 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 $1/2$ -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 new PMIS (PA) database is 15.0 ft. Rating should be accomplished by evaluating a 200-ft. section at the beginning and end of each $1/2$ -mi. section and averaging the results in terms of ft./crack (joint).

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

3.1 Introduction

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 routinely evaluated include:

- roughness, and
- skid resistance.

3.2 Roughness

Roughness is the absence or lack of smoothness in the longitudinal or transverse profile of the highway surface, resulting in 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/rut bar vehicle at highway speeds. Scheduling is arranged through the Pavement Evaluation office (Pavement Preservation, MNT) at (512) 832-7210. 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](#) and Chapter 11 of this manual.

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.

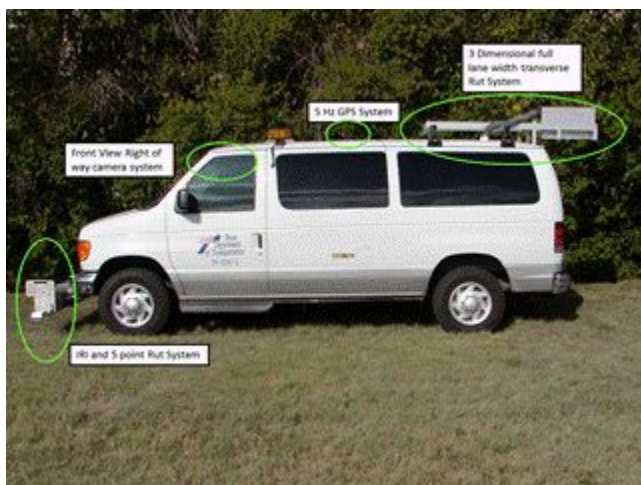


Figure 4-1. Profiler/rut bar vehicle.

3.3 Skid Resistance

Skid resistance is the force developed when a tire that is prevented from rotating slides along the pavement surface. It 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. These systems are scheduled through the Pavement Evaluation office (Pavement Preservation, MNT) at (512) 832-7210.



Figure 4-2. Skid truck.



Figure 4-3. Detail of skid trailer.

Section 4: Non-Destructive Evaluation of Pavement Structural Properties

4.1 Introduction

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 Strategies Training Course located in the Maintenance Division website under the [Pavement Engineering](#) link. Practical exercises in the use of analyses software for data collected by these systems are also included in this course.

4.2 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)*. These systems are scheduled through the Pavement Evaluation office (Pavement Preservation, MNT) at (512) 832-7210. 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 21.
- *Dynamic Cone Penetrometer (DCP)*. Many districts have purchased their own (roughly \$2500 per unit). Units are also available at MNT – Pavement Asset Management. 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 and uniformity of support of unbound layers. It can prove useful in verifying MODULUS layer inputs and moduli outputs. A correlation 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)*. These systems are scheduled through the Pavement Evaluation office (Pavement Preservation, MNT) at (512) 832-7210. When department-operated systems are not available on short notice, the Texas A&M Transportation Institute (TTI) operates a system that may be available through an interagency contract managed by MNT – Pavement Asset Management.

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 HMA 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 HMA 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 from MNT – Pavement Asset Management and through an interagency contract with TTI that is managed by MNT – Pavement Asset Management. GPR testing involves pushing by buggy or towing one or more GPR antennae along the ground at walking speeds. Some antennae operate at a lower frequency than the air-coupled GPR units and, therefore, penetrate much deeper into the pavement and underlying layers. The trade-off with lower frequency antennae is poorer near-surface resolution. Ground-coupled GPR has been used to locate sink holes, search for buried underground objects, such as abandoned storage tanks, or test for water damage, underground utility problems, or other anomalies that are deeper than an air-coupled GPR unit can sense. These systems can explore anomalies under either flexible or rigid pavements.
- *Seismic-based tools (Portable Seismic Pavement Analyzer [PSPA], Dirt SPA [DSPA], V-meter, Free-free Resonant Column)*. Contact MNT – Pavement Asset Management for availability of these devices.

The SPA series of devices are used in the field to measure in situ properties. These devices are used to generate seismic 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.

- *Total Pavement Acceptance Device (TPAD)*. The TPAD was developed under a department research project that concluded in August 2012. Further implementation was authorized under a 2-year project through August 2014. The TPAD combines the capabilities of Rolling Dynamic Deflectometer (continuous deflection measurements) and air-coupled GPR surveys. Additionally, high definition video, linear offsets, high-precision differential GPS coordinates, and surface temperature logging of the pavement section are conducted concurrently. Coordinate use of this device through MNT – Pavement Asset Management.

4.3 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 a minimum of 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. The latest upgrade

to the FWD includes an 8th geophone mounted 12 in. from the load plate opposite sensor #2. This sensor facilitates evaluating load transfer across joints and cracks on rigid pavement. GPS coordinates are also automatically posted to the deflection file for each tested location.



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

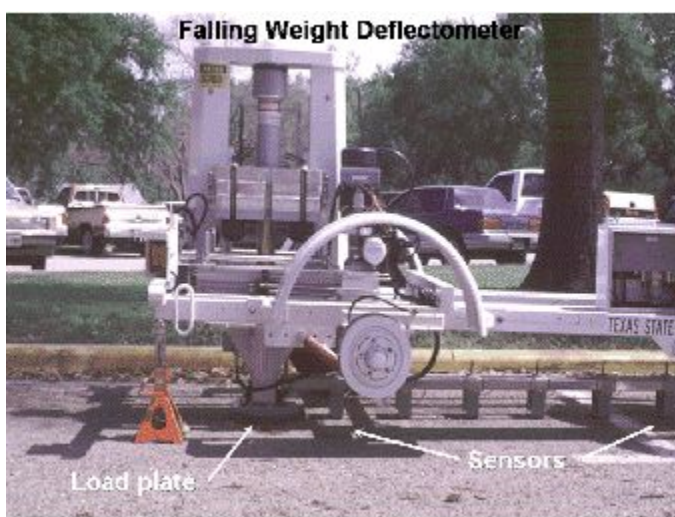


Figure 4-5. Falling Weight Deflectometer, detail view.

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 cases.

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 an infrared sensor to measure surface temperature, and records these temperatures 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 can be used 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. The [Modulus Temperature Correction Program \(MTCP\)](#) can be used for this purpose. This program is available through the department's MNT intranet page, under Engineering Applications.

Liberal use of comments placed in the FWD data file at the time of data collection is encouraged. Comments pertaining to offset from reference markers and bridge abutments, or proximity to patches, cracks, etc., are all important documentation for the individual evaluating the data. Interpretation of collected deflection data can be based on simple comparison of the normalized (for load) maximum measured deflections, or can be much more rigorous, for example, by using backcalculation methods or evaluation of deflection time-histories. Detailed discussion on the backcalculation methodology of layer moduli for flexible pavements is found in Chapter 5, Section 4, [FPS21 Modulus Inputs and Backcalculation Methodology](#)

Description of the FWD with conceptual discussion on basic operation is given in the *Flexible Pavement Rehabilitation Strategies training course* available through the department's MNT intranet page, under [Pavement Engineering](#).

4.3.1 Evaluating Deflection Data for Load Transfer on Rigid Pavements

Efficient load transfer across joints and cracks on rigid pavements is necessary for good performance. Deterioration of cracks and joints through spalling, widening, ruptured steel, erosion of subbase support, and moisture and incompressible materials intrusion will tend to reduce the load transfer efficiency over time. Evaluating the existing load transfer efficiency is necessary to formulate an appropriate rehabilitation strategy. For example, if the level of efficiency is above 70%, an HMA overlay may be an appropriate rehab strategy. Load transfer in concrete slabs is significantly influenced by ambient temperatures; higher temperatures will cause cracks and joints to close, thereby improving aggregate interlock and load transfer. For comparability and veracity of results, all testing should be accomplished at relatively steady temperatures below 80°F. In a typical load transfer setup with the FWD, the FWD is initially positioned with the load plate edge tangent to the crack or joint to be tested, with the W2 sensor positioned on the opposite side of the joint or crack as shown in Figure 4-6.

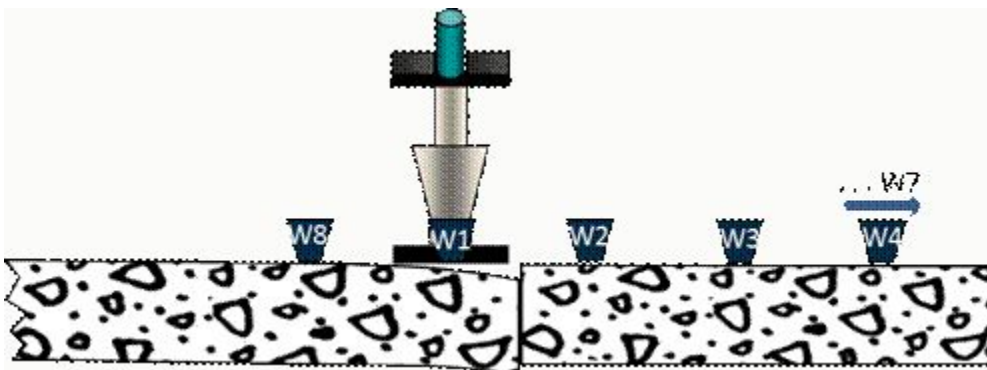


Figure 4-6. FWD setup to measure joint load transfer.

The AASHTO '93 Design Guide defines load transfer efficiency as:

$$d_{je} = d_u / d_l \times 100$$

Where d_{je} is the joint efficiency; d_u is the deflection on the unloaded side; and d_l is the deflection on the loaded side. First, the deflection at W2 is d_u , measuring efficiency from the approach slab perspective. Then, the FWD is pulled forward with the load plate tangent to the joint from the departure slab perspective, and W8 is d_u . W1 will be the d_l in both cases. Whichever measurement indicates the lower efficiency should be used for rehab considerations.

Another methodology for determining load transfer efficiency is to use deflections at the crack or joint compared to that at center slab. First, the FWD is positioned at center slab (away from working joints/cracks) and deflection measurements at the W1 and W2 sensors are noted ($W1_{cs}$ and $W2_{cs}$, respectively). Then, the FWD is moved forward to the joint to be evaluated and the load plate is positioned tangent to the joint on the same slab, with the W2 on the opposite side of the joint as in the AASHTO procedure. Deflection measurements at the W1 and W2 sensors are noted ($W1_{jt}$ and $W2_{jt}$, respectively). Load transfer efficiency is determined as:

$$LTE = (W2_{jt} / W1_{jt}) / (W2_{cs} / W1_{cs}) \times 100$$

Similarly, the evaluation can be performed with the load plate tangent to the joint on the adjacent slab, where the W8 sensor will be used in the above formula in place of the $W2_{jt}$ reading.

The customary Dynatest FWD data file (.fwd file extension) used by TxDOT for backcalculation will only report data for the first seven sensors. However, current FWD survey data protocol saves all sensor data in a small MS Access database, from which data may be extracted to conduct load transfer efficiency calculations.

4.4 Dynamic Cone Penetrometer (DCP)

This portable device complies with ASTM D 6951 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 or shear pin 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.

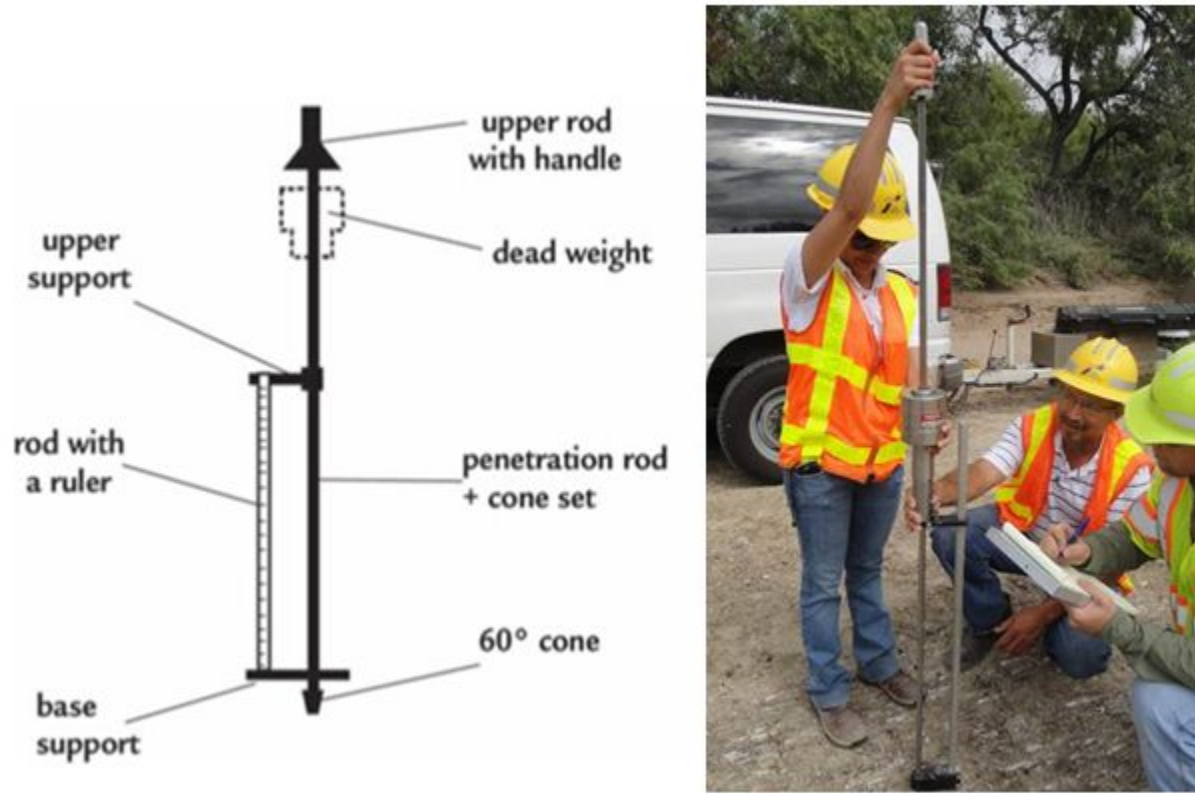


Figure 4-7. Dynamic Cone Penetrometer (DCP).

4.4.1 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 (HMA, PCC, cement-stabilized base) is present above the non-bound layers, a 1.25 in. hole should be first bored through the bound layer(s).

4.4.2 DCP Readings and Data Collection

An initial depth reading is made using a measuring stick/tape 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. Results of testing in flexible base materials with larger aggregates can often be invalidated when the cone attempts to drive an aggregate through the base material matrix. Under

these circumstances, it is better to halt testing and bore/drill down to a lower level, or restart the test adjacent to the initial location. Any time drilling is done before the start of a DCP test, the first data point should be at the actual start depth and the person processing the data will not include the surface (zero depth) as a data point, since this would invalidate the penetration rate (mm/blow) calculation.

At the conclusion of the test, the instrument is extracted by tapping the mass upward (vertically, avoiding an arc) 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, when 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:

$$E = 2550 \times CBR^{0.64}$$

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 Strategies training course*.

4.5 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 scanning 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
HMA	5 - 7	Low

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

Material	Dielectric	
	Typical Range	Signal Attenuation
HMA - stripped	< 4	Low
HMA w/light weight aggregate	3 - 4	Low
HMA - wet (surface or voids)	> 8	Moderate
Open Graded Friction Course, Plant Mix Seal, Permeable Friction Course	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	Moderate
Concrete - new	10 - 20	High
Air	1	Low
Water	81	High
Ice	3	Low

4.5.1 Analysis of GPR Data

Analysis of raw GPR data is accomplished using the [PaveCheck software](#). This program is available through the department's MNT intranet page, under Engineering Applications. PaveCheck has the capability of evaluating and presenting both FWD deflection data and GPR data for the same project simultaneously. The [PaveCheck user's manual](#) is available through the same MNT intranet link. Also, basic concepts of GPR and example problems are presented in the *Flexible Pavement Rehabilitation Strategies training course*. This training aide is somewhat dated in that the older COLORMAP software (not compatible with current computer operating systems) is discussed. 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 HMA, 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 PaveCheck; locating potential coring sites based on radar signature patterns is facilitated by high definition video and GPS coordinates collected in the radar survey. Conclusions drawn from this limited sampling can then be applied with more confidence to the remainder of the survey.

4.6 Ground-coupled Penetrating Radar (GPR)

GPR systems using lower-frequency antennae are able to discern anomalies deeper within the pavement structure. Resolution is improved when the systems are placed in contact with the surface, so the antenna is pushed by buggy or dragged behind a van that houses the data acquisition system. Typical antennae frequencies used by the department range from 200 MHz to 2.5 GHz. Maximum penetration will vary with antenna frequency, soil type, and degree of saturation. Higher antenna frequency will provide lower penetration, but higher near-surface resolution. Under the right conditions, the 200 MHz antenna can penetrate up to 30 ft.; however, with this lower frequency, there is poor near-surface resolution of layers. These systems have been used successfully in locating buried fuel tanks, archeological burial sites, sink holes, rebar during coring, voids under pavement slabs, honeycombing in concrete, pavement thickness above rebar, and aquatic springs within the right-of-way. The systems and the required data analysis remain specialized; services are available through MNT – Pavement Asset Management or through an interagency contract with the Texas A&M Transportation Institute (TTI).



Figure 4-10. 200 MHz Ground-coupled Penetrating Radar.



Figure 4-11. 400 MHz Ground-coupled Penetrating Radar.

4.7 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.

4.7.1 Field Seismic Tools

Portable and Dirt Seismic Pavement Analyzer (PSPA, DSPA). The PSPA has an enhanced capability to measure the seismic modulus, delamination, debonding, and thickness of 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. The foot that contacts the surface is an inactive anvil that is struck with an internal striker; the contact piece acts as a wave transmitter. Specialized acquisition and analysis software is loaded on a laptop computer to retain system portability.

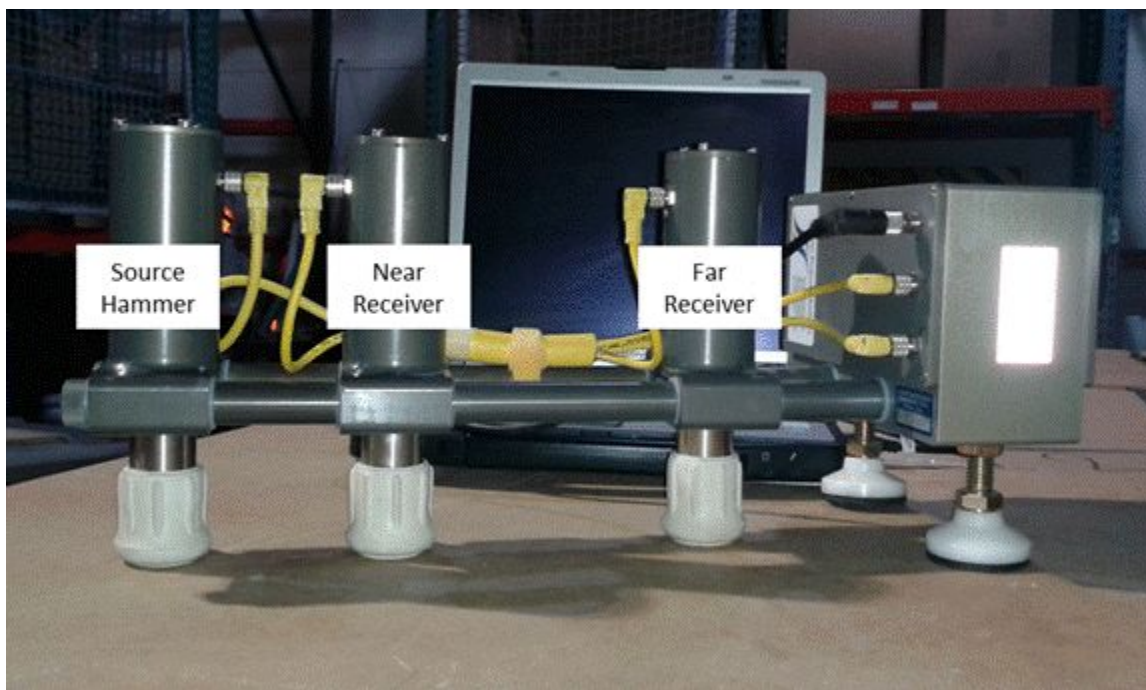


Figure 4-12. Portable Seismic Pavement Analyzer.

4.7.2 Laboratory Seismic Tools

V-meter. The V-meter is an ultrasonic laboratory device that is particularly useful for testing HMA briquettes (lab-compacted specimens or field cores). In this device, a transmitting transducer is securely placed on the bottom 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 top 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 height of the specimen by the travel time, the compression wave velocity and, as such, the *seismic* modulus of the material, is determined.

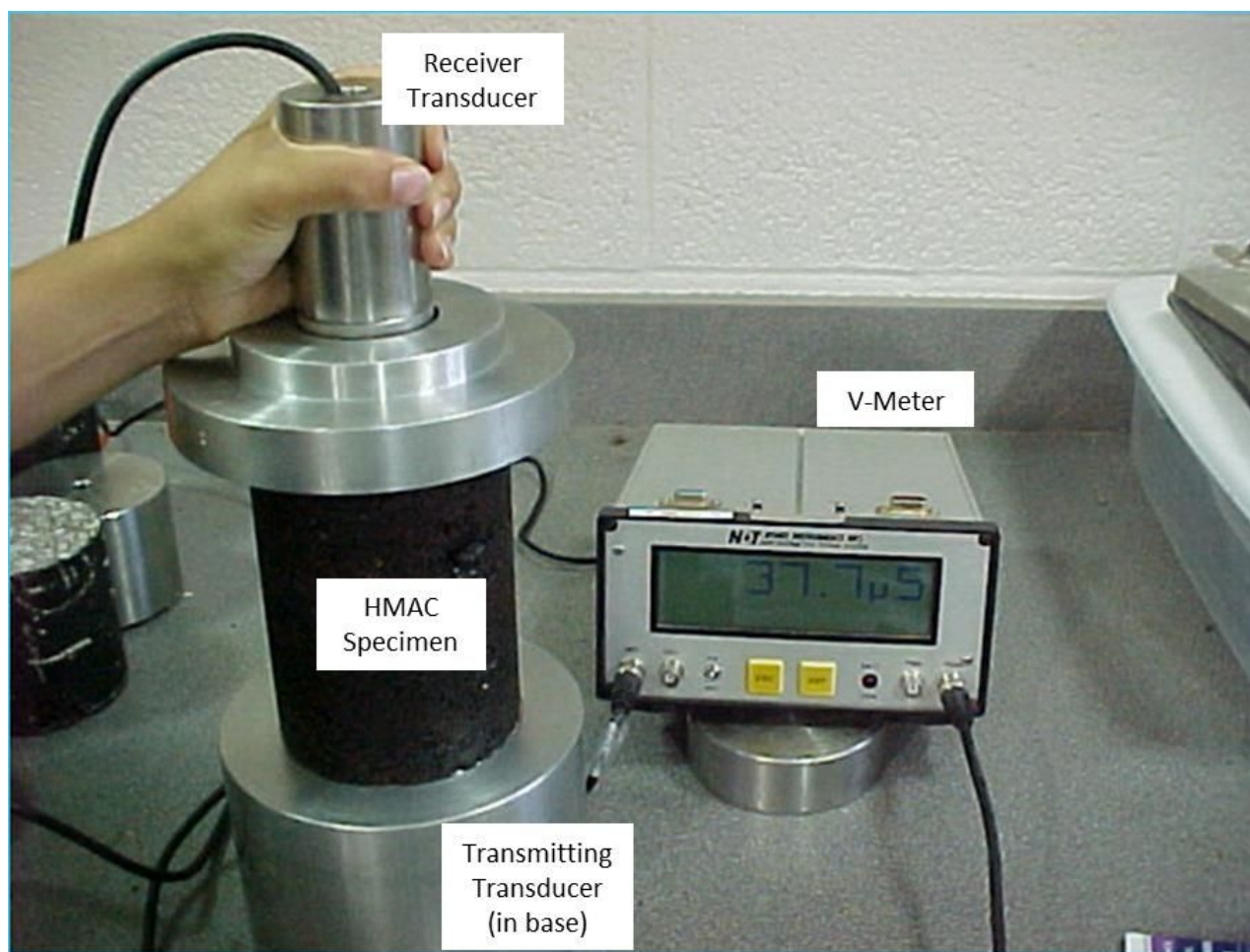


Figure 4-13. V-Meter.

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.

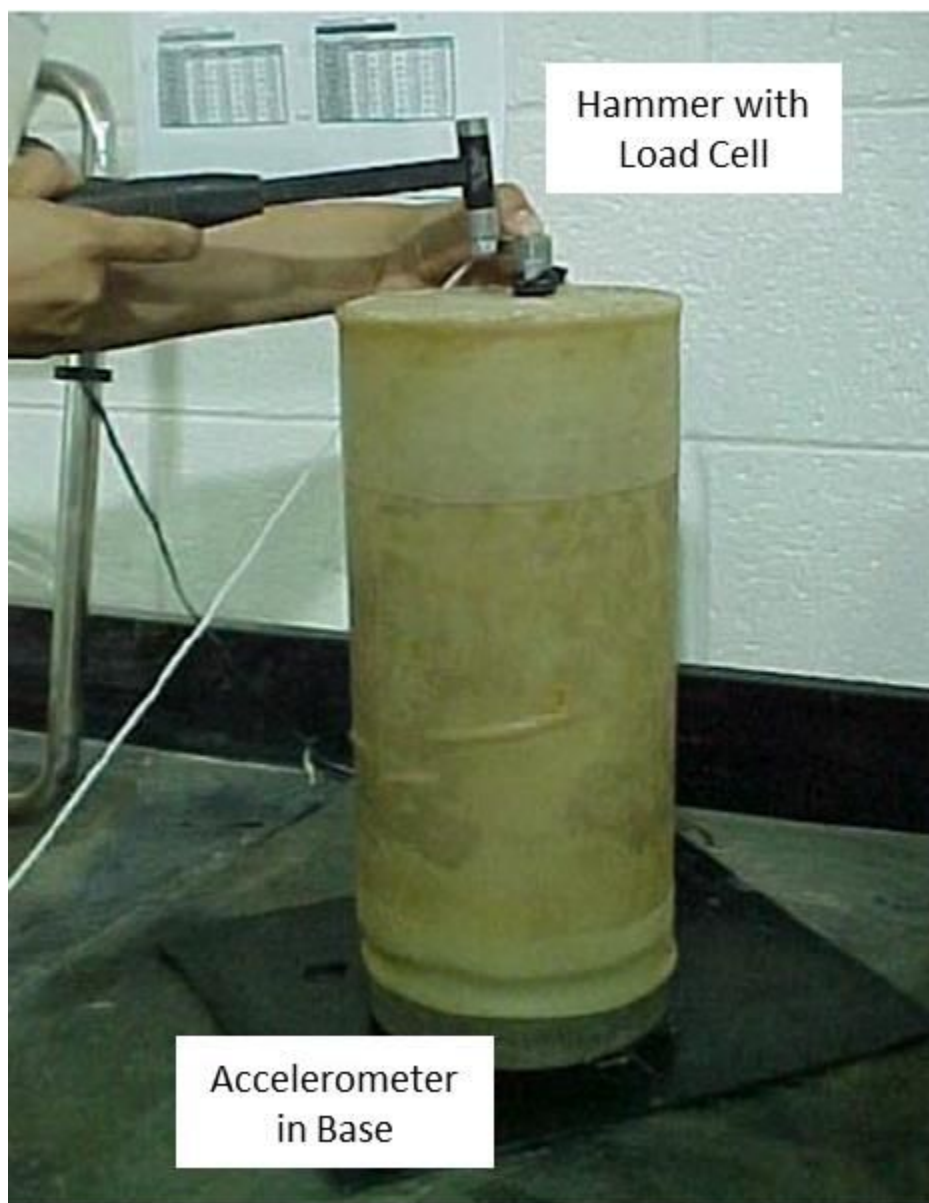


Figure 4-14. Resonant Column.

4.8 Total Pavement Acceptance Device (TPAD)

The Total Pavement Acceptance Device (TPAD) is a truck-mounted system consisting of a rolling deflectometer that applies large cyclic loads to the pavement and measures the induced deflections with rolling deflection sensors as it moves along the pavement, while generating a ground penetrating radar survey coincident with the deflection survey. There are numerous advantages built into this version of a rolling sensor deflectometer, including a low-speed cruise control, improved acquisition software, and survey enhancements, including high definition video, linear distance offsets, high-precision differential GPS coordinates, and surface temperature acquisition. Currently, the TPAD is fitted with three rolling sensors mounted on individual carts connected to

an isolation positioning/lifting frame; one sensor is mounted between the loading rollers, one in front of the loading rollers, and the third a similar distance behind the loading rollers. Rolling sensors have been improved, but data collection speed is currently limited to 2 mph to maintain a reasonable signal-to-noise ratio. The TPAD must be transported to the project site with a semi tractor-trailer. Preliminary software was developed for data analysis of all data collected (deflections, GPR signature tied to deflection location, video, longitudinal offsets, GPS coordinates, and surface temperatures). Because of the added surveying features on the TPAD, it is now much easier to physically revisit a problematic location to conduct further non-destructive testing or material sampling.



Figure 4-15. Total Pavement Acceptance Device (TPAD) with detail of loading rollers and front geophone cart.

Section 5: Destructive Evaluation of Pavement Structural Properties

5.1 Introduction

Trenching, coring, and augering have been used in forensic and routine pavement evaluation to determine the source of the problematic layer or layers and acquire materials for further laboratory testing. For example, a section of US 281 had severe rutting, 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. Figure 4-16 illustrates the pavement section profiles on US 281. Chalk and stringlines were used to differentiate pavement layers, as shown in Figure 4-16 and Figure 4-17. By exposing the pavement layers, the rutting was found to be restricted to the surface HMA layer.

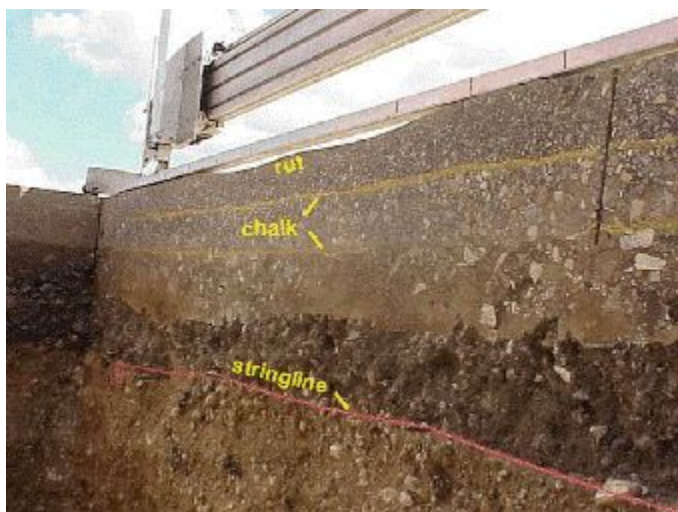


Figure 4-16. Trench sidewalls showing the pavement layer profile.



Figure 4-17. Trench sidewalls showing severe rutting.

5.2 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 4-18. The size of each trench is 3 ft. × 12 ft. Each saw cut is deep enough to penetrate all the way through the HMA layer(s). A strip about 6 in. wide is cut and removed using hand tools from one end of the trench, as shown in Figure 4-19. This creates a slot for the backhoe to reach into and lift the HMA block without major disturbance to the base layer as shown in Figure 4-20. A saw cut is also made across the center of the trench so the HMA blocks can be removed without breaking.

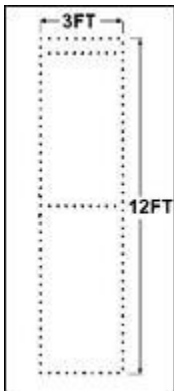


Figure 4-18. Saw cuts for trenching.



Figure 4-19. Removing hot-mix asphalt (HMA) block with backhoe.



Figure 4-20. Flipping hot-mix asphalt (HMA) block out with backhoe.

After the HMA 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. Soils and base materials are bagged and taken for further laboratory evaluation, if needed. 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 HMA 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.

5.3 Coring Procedure

To extract a core sample, the coring rig is placed at the desired location and the barrel-cooling water is turned on, as shown in Figure 4-21. Note that dry ice placed in a modified 5-gal. bucket can be used to cool the core barrel if samples at in situ moisture content are desired. This is particularly advisable if any of the HMA layers to be cored are suspected of being stripping-susceptible since a wet-coring process may severely damage these layers in the coring process.



Figure 4-21. Coring using a water-cooled barrel.

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, metal shims (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.; 6-in. cores are needed if further laboratory evaluation using Hamburg or the Overlay tester is desired. Figure 4-22 shows the extracted core samples with a 4-in. diameter.



Figure 4-22. Extracted core samples.

5.4 Augering

Retrieving samples using an 8- to 12-in. continuous flight auger is useful for determining existing layer thicknesses and for acquiring disturbed samples of all layer materials without resorting to trenching. This sampling technique can be particularly applicable for acquiring materials to determine the type and quantity of stabilizers for full-depth reclamation projects since the consistency of the retrieved materials is similar to those processed through a pavement reclaimer. Figure 4-23 shows a drilling rig with an auger bit attached.



Figure 4-23. Augering a pavement for material samples.

5.5 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. Samples acquired through this procedure are often termed “undisturbed.”

The Shelby tube is a sharpened pipe that is pushed into soil by a hydraulic ram on a truck-mounted boom. These tubes are too fragile to be driven through any layer other than unbound fine grained soils. An auger can be used to remove the HMA and base layers, allowing collection of Shelby tube samples without introducing water in the process. After the HMA 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 is 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.

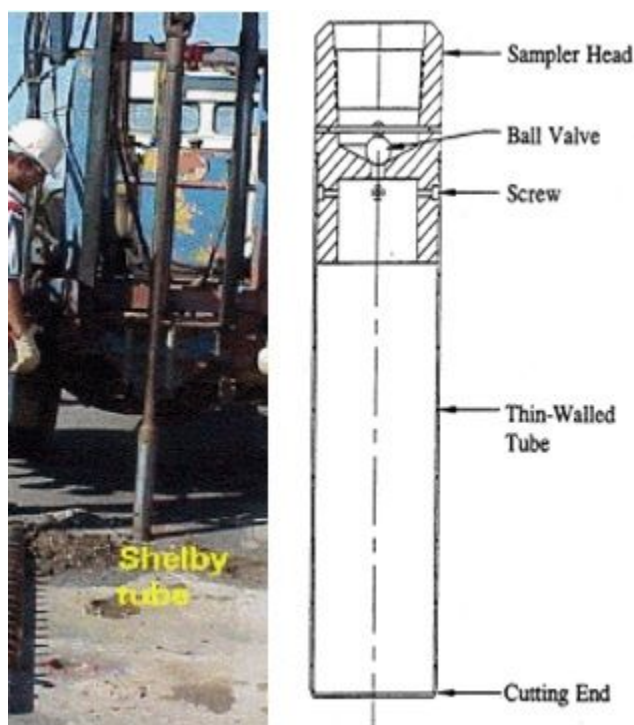


Figure 4-24. Extracting subgrade samples with Shelby tube.



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:

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[Section 2: Types of Flexible Pavements](#)

[Section 3: FPS 21 Design Parameters](#)

[Section 4: FPS21 Modulus Inputs and Backcalculation Methodology](#)

[Section 5: Pavement Detours and Pavement Widening](#)

[Section 6: Perpetual Pavement Design and Mechanistic Design Guidelines](#)

Section 1: Overview

This chapter documents the policy for designing flexible pavements. The following topics are discussed:

- types of flexible pavements,
- design parameters used in FPS 21 (analytical flexible pavement design procedure),
- FPS 21 modulus inputs and backcalculation methodology,
- pavement detours and widening, and
- design of perpetual pavements.

Section 2: Types of Flexible Pavements

2.1 Definition of Flexible Pavement

A true flexible pavement yields “elastically” to traffic loading. It is constructed with a bituminous surface treatment 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/ HMA surface layer must also be resistant to fatigue damage and remain stable under traffic loads when pavement surface temperatures are in excess of 150°F.

In this manual, the term “flexible pavements” is used in a more generalized way to describe any Asphaltic Surfaced structure (other than HMA-overlaid concrete). These pavements range in composition from true flexible pavement to semi-rigid systems (including the full-depth or *perpetual* design). This chapter is applicable to 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 than a true 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 pavement,
2. chemical or mechanical 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.

2.2 Types of Flexible Pavements

These pavements may fall into one of the following categories:

- surface-treatment on a granular base,
- thin hot-mix asphalt (< 2 in.) on a granular base,
- intermediate hot-mix asphalt (2 - 4 in.) on a granular base,
- thick hot-mix asphalt (> 4 in.) (semi-rigid),
- thin hot-mix asphalt 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 be used with any of the above pavement types. Typical stabilizers include asphalt cement (for base only), lime, cement, fly ash, or lime-fly ash combinations.

2.2.1 Perpetual (HMA) 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 rut life, requiring only periodic surface renewal.

For designing perpetual pavements using FPS 21, see Section 6, [Perpetual Pavement Design and Mechanistic Design Guidelines](#)

Section 3: FPS 21 Design Parameters

3.1 Introduction

The required analytical method of flexible [pavement design](#) is FPS 21.

FPS 21 uses a systems approach centered upon deflection measurements from the falling weight deflectometer (FWD). These measurements are then used by the MODULUS computer program to estimate the material properties (modulus) of the pavement layers through a backcalculation process. The material properties of the pavement layers, traffic loading (cumulative ESALs), the change in the serviceability rating, and the desired level of reliability are then used to compute alternative structural designs using FPS 21.

It is mandatory to check the design derived by FPS 21 for subgrade shear strength adequacy using the Modified Texas Triaxial Class (TTC) design method contained within the FPS 21 software.

FPS-19W is the previous design program which has been replaced by FPS 21. FPS 21 has virtually the same input requirements and performance models, and the two programs result in the same solutions when equivalent inputs are used. The [Flexible Pavement Design System \(FPS\) 21: User's Manual](#) was developed jointly between the Texas A&M Transportation Institute and TxDOT and serves as a preliminary reference to the essential operational aspects of the program (TTI Research Report 1869-2 contains information on the models embedded in both FPS-19W and FPS 21). Additional guidance pertaining to recommended inputs and functionality is given in this section.

3.2 Program Tools

FPS 21 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. Some clarification and reinforcement of guidance is provided here.

3.3 Data Input Components

Data input for FPS 21 consists of a Main Menu screen and three pages of inputs. The current screen layout streamlines data input over the older FPS-19W system, but remains very familiar to veteran users of this earlier version.

Project Information inputs (input page 1) will be discussed first; then Basic Design Criteria, Program Controls, and Traffic on page 2; then Construction and Maintenance Date and Detour Design for Overlays on page 3; followed by Design Type 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.

Table 5-1: FPS 21 Design Input Requirements

Parameter	Condition	Recommended Value	Comments
Initial SI*	Surface Treatment	3.8 - 4.0	Using a lower Initial SI has the effect of thickening base.
	Thin/Intermediate ACP (1.5 in. to 4 in.)	4.2 - 4.5	Actual ride for new projects is better than historic values. The lower Initial SI has the effect of thickening base. A 3 to 4 in. surface can have two smoothness opportunities.
	Thick ACP (> 4 in.)	4.5 - 4.8	Should be reduced to 4.5 if the minimum serviceability value is set at 2.5 or less. Thicker pavements have multiple opportunities to achieve higher SI values.
Minimum SI*	Surface Treatment or <1 M ESALs	2.0 - 2.5	Allowing a lower Min. SI has the effect of thinning the pavement structure. Risk is accepted requiring additional maintenance before termination of pavement life.
	Thin/Intermediate ACP (1.5 in. to 4 in.) or Between 1 M and 3 M ESALs	2.5 - 3.0	
	Thick ACP (> 4 in.) Or > 3 M ESALs	3.0	
Confidence Level	All Design Types	C (95%)	A confidence level of C (95%) is recommended for all levels of traffic loading. Attempting to use lower levels will almost always result in a design that will not meet the Modified Texas Triaxial requirement. Using higher levels will result in an overly conservative design.
District Temperature Constant	All	31	Overwriting the default will cause thicker designs for Districts in colder regions, but will not mitigate thermal cracking potential.
Swelling Potential, PVR, Swelling Rate	This analysis has been removed from FPS 21. Consider stabilization of the subgrade to address moderate soil volumetric changes due to moisture. Consult Chapter. 3, Section 2, guidelines for more extreme PVR issues.		

Table 5-1: FPS 21 Design Input Requirements

Parameter	Condition	Recommended Value	Comments
Overlay Cost	Future Cost	Use current design practice	Review the “Detail Cost” sheet in the FPS output 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	Does not change thickness but may sort design output options based on combined user/construction costs as opposed to least construction cost.
Cost per CY	All	District-specific cost should be used.	
* Increased difference between initial SI and minimum SI reduces pavement thickness.			

3.4 General Input Descriptions

3.4.1 Program Main Menu

Access the FPS design modules by clicking on the “FPS Pavement Design” button on the main menu.

3.4.2 Project Information Input Screen *(Input page 1)*

- Page 1 (Project Information) of the inputs covers the administrative aspects of the design, such as job location and identification. An administrative number may be assigned to each “run” in the “problem” field.
- When the designer clicks on the “District” field, dropdown menus appear that allow the designer to select the district and county names/numbers. State and county map groupings also appear. The state map has active links to each district; clicking within the district boundaries will automatically select that district and refresh the map grouping of associated counties. The county dropdown menu must be used to select the actual county location. County selection governs defaults for the subgrade modulus and subgrade soil types accessed in the Modified Texas Triaxial check.
- The current date is automatically refreshed in the “date” field.
- The applicable highway name is placed by the designer in the “highway” field.
- The control-section-job numbers are placed in the appropriate fields.
- Comments may be entered in a five-line field by overwriting any existing comments (no text wrap-around).

- Select the “Use Existing Input File” button if you wish to recall a previously saved file for editing and further analysis.

3.4.3 Basic Design Criteria *(Input page 2)*

3.4.3.1 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 or longer is allowable, but the designer must still input the projected 20-yr. cumulative traffic in the FPS 21 computer program. Adjustments to the traffic are then made internally by the computer program. Similarly, when a very short analysis period (1-2 yr.) is considered for design of short-term detours, the 20-yr. traffic (ESALs) must be used as the traffic input in FPS 21.

CAUTION: When using FPS 21, 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.

3.4.3.2 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 longer than the minimum include: minimizing interruption to traffic and avoiding the necessity for mill and inlay operations where a 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. Because of funding constraints, performance periods are rarely selected to equal the entire length of the analysis period; consequently, the methodology of flexible pavement construction typically used in Texas can be regarded as “staged” construction.

3.4.3.3 Recommended Design Confidence Level

This parameter is meant to address variability in material quality, construction processes, and uncertainties in traffic forecasting 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.

NOTE: Experience has shown that selecting level 'C' (95%) for all design types and traffic loading levels is the best choice for a preliminary FPS run, and typically will be the level submitted for the final design. Selecting a lower reliability level will generally cause the FPS design to fail the post-run Modified Texas Triaxial check; selecting a higher reliability level will result in non-economical design options.

Historic FPS alphabetic codes are tied to a reliability or confidence level as follows:

A 80%

B 90%

C 95%

D 99%

E 99.9%

A reliability level other than "C" is to be used by exception only.

3.4.3.4 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. See [Table 5-1](#) for recommended values.

3.4.3.5 Final 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. See [Table 5-1](#) for recommended values.

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 by performing preventive or standard maintenance, HMA overlays, or other rehabilitation/reconstruction.

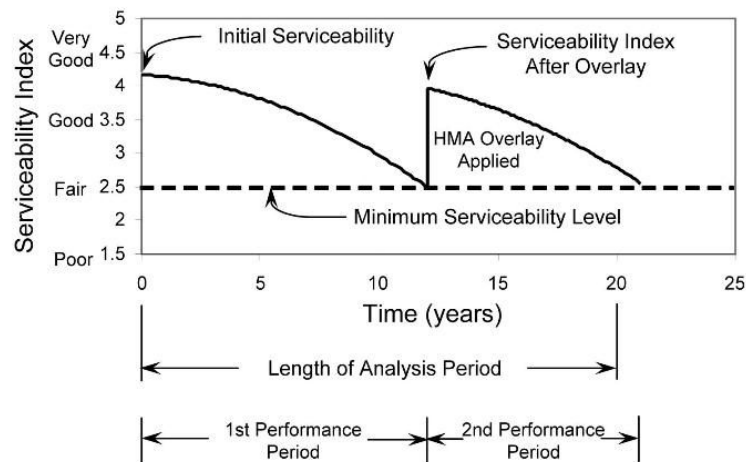


Figure 5-1. Pavement Performance Relationships.

3.4.3.6 Serviceability Index after Overlay

This field is intended as a measure of the pavement condition following an overlay predicted by FPS 21, projected after the initial or subsequent performance period.

NOTE: Overlays will always be programmed unless your time to first overlay is greater than or equal to the analysis period of your design. A planned overlay within the analysis period is equivalent to using “Staged Construction” in the AASHTO 93 design procedure.

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 renewed ride condition (see Figure 5-1).

3.4.3.7 District Temperature Constant

FPS 21 is set to ‘31’ as the default (corresponds to a Central Texas value). Historic temperature constants by district are given in the Help menu, but justification must be cited for their use. Using the values keyed to the project’s home district will generate a steeper performance curve (shorter life) for a colder region than in a warmer region. FPS 21 will not generate an effective design that will counter the effects of thermal cracking (based on thickness alone).

3.4.3.8 Interest Rate (%)

This parameter is used in the life cycle cost analysis to discount future expenditures for overlay and maintenance costs. The default of 7% may be adjusted based on current trends.

3.4.4 Program Controls (Input page 2)

3.4.4.1 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 maximize output design combinations.

3.4.4.2 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.

3.4.4.3 Maximum Thickness of All Overlays

This field can be used to constrain design strategies to meet profile limitations.

3.4.5 Traffic Data (Input page 2)

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” link to Chapter 2, Section 8, for guidance). This TPP report will also contain the Average of the Ten Heaviest Wheel Loads, Daily (ATHWLD) and percent tandems in the ATHWLD, inputs that are both required for the Modified Triaxial Check (Chapter 2, Section 5, “Approved Pavement Design Methods”).

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 traffic stream against observations along the project corridor. 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 backcalculating the average ESALs per truck from the TPP 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-evaluation of the forecasted traffic, if necessary. Alternately, the methodology cited in Chapter 2, Section 8 (Traffic Projections), may be considered to adjust TPP reported traffic loading.

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

3.4.5.1 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 accumulation 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 6) is placed.

3.4.5.2 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.

3.4.5.3 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, except when standard or project specific adjustments for lane distribution are warranted for multi-lane*

1.Internal access only.

facilities. These factors may be applied to the TPP-supplied figure when at least three lanes exist in the design direction (see Chapter 2, Section 8, [Information Needed for Pavement Design](#)); Lane distribution adjustments to the 20-year cumulative ESALs reported by TPP are as follows:

Table 5-2: Lane Distribution Adjustments

# Lanes in One Direction	Correction Factor Applied to 20-year ESALs
2	Use 100%
3	Use 70%
4 or more	Use 60%

Lane distribution factors can be applied to highways with two lanes in one direction when there are compelling data to warrant reduction of the 100% ESAL loading.

3.4.5.4 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.

3.4.5.5 Average Speed in the Overlay Direction

Figure is entered in miles/hour; the estimate will, in part, depend upon the detour model 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.

3.4.5.6 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.

3.4.5.7 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.

3.4.5.8 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.

3.4.6 Construction And Maintenance Data (Input page 3)

3.4.6.1 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 ½-in. level-up is automatically included for cost purposes in the program, but does not count toward an increase in structure.

3.4.6.2 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.

*3.4.6.3 *ACP Compacted Density, Tons/CY*

See NOTE for parameter influences. Typically the value ranges between 1.90-2.00.

*3.4.6.4 *ACP Production Rate, Tons/Hour*

See NOTE for parameter influences. Typically the value ranges between 150-300 tons/hr.

*3.4.6.5 *Width of Each Lane, Feet*

See NOTE for parameter influences. This value should be equal to the typical lane width.

3.4.6.6 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 6) construction should be tracked at the district level. Values have typically ranged from \$50-\$200/lane-mi.

3.4.6.7 Annual Incremental Increase in Maintenance Cost

This parameter is an adjustment to the baseline 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.

3.4.7 Detour Design For Overlays (Input page 3)

3.4.7.1 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. The model number (1-5) is entered in this field. Clicking on the input field or on the associated up-down arrow buttons will activate a graphic portrayal of each detour model. Clicking on the graphic will hide drawing.

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

These models are not all inclusive of the type of projects underway around Texas. The designer must select the model close to the proposed design, the key requirement is the number of open lanes in the overlay direction.

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.

3.4.7.2 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), unless the highway is restriped/partitioned to cause the lane to be designed for through traffic.

3.4.7.3 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).

3.4.7.4 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).

3.4.7.5 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.

3.4.7.6 Distance Traffic Slowed in Non-overlay Direction

Same as above, except applied to traffic in the non-overlay direction.

3.4.7.7 Detour Distance, Overlay Zone

This field is hidden, unless Detour Model 5 is selected. The distance in miles of the alternate route/special detour is input.

3.4.8 Design Type

The designer must now select which design type will be used for the project. FPS21 uses a menu of seven basic design types:

Table 5-3: FPS21 Seven Basic Design Types

1	2	3	4	5	6	7*
Surface Treatment	HMA	HMA	HMA	HMA	HMA Overlay	User Selected HMA or Surface Treatment
Flexible Base	Flexible Base	Asphalt Stabilized Base/HMA	Asphalt Stabilized Base/HMA	Flexible Base	Existing HMA	User Selected
Subgrade	Subgrade	Subgrade	Flexible Base	Stabilized Subbase/Subgrade	Existing Base	User Selected
-	-	-	Subgrade	Subgrade	Subgrade	User Selected
-	-	-	-	-	-	User Selected
-	-	-	-	-	-	User Selected
-	-	-	-	-	-	Subgrade
* User defined mode. Minimum number of layers is 4; maximum is 7.						

In addition, a previously generated design input file can be recalled (selection made on first input page). *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 composite modulus must be assumed; this procedure would require some discretion to ensure adequacy of the overall design. When using design option 7 (User Defined), the designer must select different materials from the inventory table for each layer to allow the program to consider their unique contribution to the structure. Renaming and editing of the material properties is allowed in the materials table beneath the detour design inputs. There are also unique aspects to the material modulus defaults with each type of design. For example, in design *Types 1 and 2*, 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 modulus ratio of unbound "flex" base sitting on untreated subgrade should not exceed 4.

The *Type 4* 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 for *Type 1* or *2* designs.

The *Type 5* design has no unbound layer modulus adjustments associated with it. When the pavement structure is not clearly a *Type 1, 2, 3, or 4* design, *Type 5* or *Type 7* are recommended. All material names and properties in these pavement types may be overwritten in the materials table beneath the detour design inputs to suit the structure being designed within the limitations already described.

The *Type 6* 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 \(FPS\) 21: User's Manual and thickness design software](#).

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. Click on the “Design Type” button and the seven options will be revealed. Select one of the option buttons then click on the “Exit Pavement Design Type Selection” button to continue. If the design option selected was from 1 to 6, the program will immediately display the materials summary table, at which time the designer can edit all entries (including material description), other than layer number.

If the designer selects option 7 (User Defined), the program displays an interactive building menu where materials are selected by clicking and dragging from the inventory in the table on the right to the blank slate on the left. The default number of layers is 4 (this is an absolute minimum in this mode), but can be increased up to 7 by clicking on the “plus” sign at the top left of the blank slate. Number of layers can be decreased later if desired (“minus” sign), but the designer must check to make sure layers that were desired were not inadvertently removed. Note that layers cannot be dragged backwards from the slate to the tabled inventory, but can be “overwritten” by dragging a new material from the inventory over the undesired material on the slate. Once the designer is satisfied, click on the “Go Back” button at the top of the slate to view the materials summary table. As before, the designer can now edit all entries other than the layer number.

NOTE: When using design option 7 (User Defined), the designer must select different materials from the inventory table for each layer to allow the program to consider their unique contribution to the structure. Each material in the inventory table is given a unique program code. In other words, do not attempt to place the same material selection into multiple layers in the proposed pavement structure. Renaming and editing of the material properties is allowed once you return to the materials summary table beneath the detour design inputs.

3.4.8.1 Material Parameters in the Materials Summary Table

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 well performing, yet economical, structure. Since FPS 21 only gauges a material's contribution on the basis of modulus, other engineering considerations must be applied. Materials selected must be compatible. Failure to properly consider existing material properties, relative stiffness of adjacent layers, HMA mixture-specific rutting and cracking susceptibility, moisture susceptibility of bound and unbound materials, or the adjacent structure composition (in the case of widening) can lead to early failure and/or very high maintenance costs. Also, FPS does not consider the contribution of non-structural components such as prime or tack coats, or underseals, that may be critical to ensuring proper bond between layers and overall moisture resistance of the structure.

3.4.8.1.1 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, nor reorder the sequence. 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.

3.4.8.1.2 Material Name

The designer may edit the name of the material to be used in the layer, if preferred over the default provided.

3.4.8.1.3 Cost per Cubic Yard

Cost for all new materials proposed in a design is in terms of dollars/cubic yard. Even existing or reclaimed layers can have costs associated with them, such as reworking a flexible base or milling an existing HMA layer prior to overlaying. Material costs should be tracked at the district level.

The Average Low Bid Unit Price found on the department's website can be used as a basis for these material costs.

3.4.8.1.4 Modulus, E (ksi)

See Section 4.

3.4.8.1.5 Poisson's Ratio

The designer enters a value typical for the material type used. Recommended values are given in Table 5-6. This input has very little sensitivity in the thickness design process. Selection of a value within the ranges identified is sufficient.

3.4.8.1.6 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 hot-mix asphalt (HMA) layers, minimum practical layer thickness for flexible bases to achieve density, or district policy.

- For existing layers, the average existing layer thickness is input.
- Except in the Overlay Design mode (design Type 6), do not use 'zero' as the minimum thickness. Run a different design option that does not have this layer and compare results of

more than one design option type post-calculation if necessary. A warning message will appear once the program ‘run’ button is activated if zero is placed as the minimum thickness.

- Check district SOP for guidelines on fixed or minimum layer thickness.
- For subgrade, use the average depth to bedrock as determined by MODULUS or the default (200 in.) when the subgrade modulus was not backcalculated.

3.4.8.1.7 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).
- Check district SOP for guidelines on fixed or maximum allowed layer thickness.
- For subgrade, use the average depth to bedrock as determined by MODULUS or the default (200 in.) when the subgrade modulus was not backcalculated.

3.4.8.1.8 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 (design life). Guidelines are given in the built-in help screen for these fields for 10-, 20-, and 30-yr. analysis periods.

3.4.9 FPS Pavement Design Results

Once all inputs are entered, the designer clicks the red arrow button and the output (FPS Pavement Design Results) 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 (when approved as District Policy) to override the outcome of this check if evidence of past performance can be cited. Select the “Check Design” button beneath the design result summary table for each considered design solution in turn. Design check option buttons will appear on the right margin of the follow-on screen. In addition to the Modified Texas Triaxial Design check, running the Mechanistic Check is highly recommended for any FPS design result that produces an HMA surface from 2-4 in thick. This range of HMA thickness is particularly vulnerable to short fatigue life.

3.4.10 Modified Texas Triaxial Check

This design check is accomplished 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” for a selected output structure solution. Once selected, a “pavement plot” window appears with several option buttons.

Table 5-4: Texas Triaxial Design Check

Step	Action
2	Click on “Triaxial Check.” The Modified Texas Triaxial Design check window will appear.
3	Enter the Modified Texas Triaxial Check data.
4	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) <i>Traffic Analysis for Highway Design</i> report.
5	The percent tandem axles in the AHTWLD value is used to trigger a wheel load multiplier of 1.3 times the ATHWLD for percent tandems at 50% or greater.
6	- The Pavement Design Task Force (PDTF, 2009) recommends a wheel load factor of 1.0 be used for all designs where traffic loading is below 3 M ESALs. - The percent tandem axles must be entered as <50% for these designs. or - For designs involving more than 3 M ESALs, use the appropriate percent tandem axles provided by TPP. NOTE: Where heavy industry truck traffic, energy sector development, aggregate pits or concrete plants, etc., are common, special design considerations may be required.
7	Continue with inputs for the modified cohesiometer and subgrade Triaxial Class number as described below.

3.4.10.1 Modified Cohesiometer Value (Cm)

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 stabilized materials used in the highway structure. By allowing for 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/thickness combination used in the proposed structure that has the highest cohesiometer value. A graphic of the FPS-generated design appears at the bottom of the screen to assist the designer in selecting the controlling layer. - The screen will revert back to the input screen with the appropriate value placed in the Cm field.

3.4.10.2 Subgrade Triaxial Class Number

There are three options for supplying the Texas Triaxial Class of the subgrade.

1. Enter the value calculated by performing 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).
2. Determine the controlling project subgrade soil plasticity index (PI). In selecting option 2, a field will appear to allow input of the soil PI. Internal calculations will estimate a correlated Texas Triaxial Class and reveal it in the TTC field.
3. Estimate the TTC based on the project controlling soil type. By selecting option 3, an internal database query will reveal average TTC values for predominant soil types within the project county. Select the project controlling soil type and the corresponding TTC value from the database will be placed in the TTC field.

For all three options, once the TTC value is entered, computations are executed on the upper right-hand side of the screen, and a message will appear indicating whether the design passes or fails the Modified Triaxial check.

Section 4: FPS21 Modulus Inputs and Backcalculation Methodology

4.1 Overview of Modulus Inputs

Each material layer used in the structure will have a modulus input that shall characterize the average seasonal stiffness of that material over the course of the year. The construction process, inherent material variability (initially and over time), and effects of environment (moisture and temperature) and traffic loading will typically introduce considerable variance about the average value. Modulus inputs for HMA are based on a temperature of 77°F.

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 material; the modulus can be significantly influenced by the confinement provided by the layers above or below, 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. Proposed virgin and reclaimed material moduli will require knowledge by the designer (preferably through past use and subsequent evaluation), tempered by the specifics of the current project.

Studies conducted on perpetual pavements indicate that stone-on-stone designed Superpave hot-mixes and thick composite HMA structures, using any type of HMA, have much stiffer in-place moduli values than conventional thinner surfaced HMA 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 backcalculated material modulus values are manually entered into their respective fields (values are not read from the MODULUS backcalculation summary file). Typically, adjustments (see 4.4.1. Backcalculation Limitations and Adjustments below) to the averages produced in the backcalculation process are necessary prior to entry in the FPS program.

4.2 Virgin and Modified-in-Place Materials

Below is a partial listing of typical design moduli by material type for virgin or modified-in-place materials to be used in FPS 21. For materials not listed, contact MNT – Pavement Asset Management for recommendations.

Table 5-6: Recommended Material Design Modulus Values

Material Type	2014 Specifications	Design Modulus	Poisson's Ratio	Comment
Seal Coat	Item 316	200 - 250 ksi	0.35	Considered in the structural design only when placed on the surface. Not considered when used 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-Mix Asphalt	Item 340, 341	Combined HMA thickness: ≤ 4 in. use 500 ksi 4.0 in. < T ≤ 8 in. use 650 ksi ≥ 8 in. use 850 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.
Superpave Mixtures	Item 344	Combined HMA thickness: ≤ 4.0 in. use 650 ksi 4 in. < T ≤ 6 in. use 750 ksi > 6.0 in. use 850 ksi	0.35	
Stone-Matrix Asphalt	Item 346	Same as Item 344	0.35	
Asphalt Treatment (base)	Item 292	250 - 400 ksi	0.35	Use Tex-126-E, "Molding, Testing, and Evaluating Bituminous Black Base Materials," asphalt content.

Table 5-6: Recommended Material Design Modulus Values

Material Type	2014 Specifications	Design Modulus	Poisson's Ratio	Comment
Emulsified Asphalt Treatment (Base)	Statewide special specs	200 ksi	0.35	Contact MTD – Soils & Aggregates section for assistance in establishing optimum emulsion concentration and recommendations for adding cement or other filler material. Humid/wet regions require special considerations to ensure proper curing.
Foamed Asphalt Treatment (Base)	Statewide special specs	200 ksi	0.35	Contact MTD – Soils & Aggregates section for assistance in establishing optimum asphalt content and recommendations for adding cement or other filler material.
Flexible Base	Item 247	If historic data not available, modulus shall be no greater than 3-4 times the subgrade modulus or use FPS default, whichever is lower. 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 Treated 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 Treated 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 <i>minimum</i> 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. Micro cracking is encouraged for higher strengths. Also, very stiff, stabilized bases are not modeled effectively in FPS 21. Higher design moduli shall not be used.

Table 5-6: Recommended Material Design Modulus Values

Material Type	2014 Specifications	Design Modulus	Poisson's Ratio	Comment
Fly Ash or Lime-Fly Ash Treated 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 Treated Subgrade	Item 260, 275	30 - 45 ksi	0.30	Use Tex-121-E or Tex-120-E, Parts 1, 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. For cases when a subgrade will be treated (2-3% lime) to provide a working platform for construction equipment and a platform to improve compactive effort of the overlying layers, <i>this layer shall not be accounted for in the structural design.</i>
Emulsified Asphalt Treatment (Subgrade)	Item 314, various special specs	15 - 25 ksi	0.35	Contact MNT – Pavement Asset Management for assistance in establishing optimum emulsion concentration.
Subgrade	(Existing)	Priority should be to use the project-specific backcalculated 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 backcalculated modulus is preferred. FPS 21 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.

4.3 Modulus Values for Rehab and Reclamation-type Projects

Rehab and reclamation procedures often use materials in non-standard ways that either rely on blending of some in-situ materials, with or without new materials (such as "add-rock", or modifiers), or simply reworking old engineered layers (surface and base) in-place. In many cases, spot repairs (base and surface) may be the only preparation before the existing structure is simply

overlaid with new HMA. The Table below offers some guidance for selecting FPS values for this type of construction.

Table 5-7: Design Modulus Values for Rehab/Reclamation Projects

Material Description	Modulus Value	Poisson's Ratio	Cohesimeter Value (Cm) for Modified Texas Triaxial
Existing Thin Hot Mix	*500 ksi or Back-calculated from FWD data	0.35	Add existing HMA thickness to new HMA overlay thickness; use Cm value for total HMA thickness
Existing Pavement - Scarified, Reshaped and Compacted	~3 times the subgrade modulus	0.35	Use 100 for untreated materials, or select another layer with higher credit
Stabilize Exist Pav/ Subgrade			
a) Mostly granular base (75% or more base)	a) 100 ksi	a) 0.30	a) 800
b) Blend subgrade & base (50% to 75% base)	b) 65 ksi	b) 0.30	b) 650
c) Mostly subgrade (<50% base)	c) 35 ksi	c) 0.35	c) 300
New Flexible Base (on top of existing structure/base)	Gr 1-2: 70 ksi Gr 5: 50 ksi	0.35	Use 100 for untreated materials, or select another layer with higher credit
1st 8" lift of new flexible base (when multiple lifts are req'd)	~3 times the subgrade modulus	0.35	Use 100 for untreated materials, or select another layer with higher credit
* For thin mix, when visually in good condition or repairs will be made to areas in poor condition.			

4.4 Backcalculation Methodology

This procedure is used to determine modulus values for in situ pavement materials when these materials are used as is (unmodified) in FPS design. TxDOT currently uses version 7.0 of the MODULUS software for backcalculation of deflection data collected by the FWD. Version 7.0 is comparable to version 6.0; the main difference is the ability to read current Dynatest R80 formatted data files that have GPS locations embedded. The software (v 7.0) and user's manual (v 6.0) in PDF document format are available through the MNT [Engineering applications](#) link to the TTI on-line pavement design training site. Also, basic operation and discussion of inputs, cautions, and example problems is presented in the *Flexible Pavement Rehabilitation Strategies* training course and *Flexible Pavement Design* workshop.

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.

4.4.1. Backcalculation Limitations and Adjustments

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 veracity of solutions generated. The following are some pointers when using MODULUS 7.0:

- 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 200 ksi. Alternatively, the surface treatment can be combined with the underlying layer as the “surface,” reducing the total number of layers by one. In determining the seed moduli range for the surface, MODULUS assumes the layer is HMA and automatically fills the min/max seed values in accordance with the temperature posted in the Asphalt Temp cell. Where non-bituminous materials are the surface during testing, the user must insert seed values commensurate with the type of material tested.
- The maximum number of layers for which the modulus can be backcalculated is four (one of which is always the natural subgrade) in MODULUS 7.0. 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 HMA, or flex base over reclaimed 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.
- A check of reasonableness in the solutions generated shall be made. Reasonableness is more 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, and the variability of reported moduli may be excessive (coefficient of variation 100% or greater). When there is doubt of reasonableness, the user should perform backcalculation runs using both 3- and 4-layer solutions (employing guidelines given in the third bullet). 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 shall 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 21, adjustment of the average modulus shall 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,” or Item 354, “Planing and Texturing Pavement”) as part of the job.
- Shallow bedrock (typically less than 60 in. deep) will almost always result in underestimation of the subgrade modulus and overestimation of the flexible base modulus and produce very high average error (> 20%). The recommended workaround is to fix the depth to bedrock (DTB) at 120 in. or, alternatively, 240 in. if the solution using the program-generated DTB produces suspect subgrade/base moduli. Another clue that the default solution is suspect would be if the ratio of the flexible base modulus (unstabilized layers only) to the subgrade modulus is very high (> 5). If the user opts to fix the calculated DTB to a value in the 120- to 240-in. range, then this user-selected value must also be used in FPS 21 design.
- MODULUS can perceive a shallow DTB 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 DTB value by using a fixed value of 120 in. Again, check the generated subgrade/base moduli values for reasonableness.
- Soft upper subgrade can also lead to high errors in the backcalculation process. In these cases, use a 4-layer solution where the soft portion of the subgrade is modeled as the subbase layer (fix depth at 12 in.) to provide better fit and more realistic backcalculated values for the base and deep subgrade. A check can be made in MODULUS using the Boussinesq procedure to evaluate how the subgrade modulus varies with depth. Verification with a dynamic cone penetrometer (DCP) may be warranted.

4.4.2 Modulus Correction Factors

In addition to adjustments made to backcalculated average modulus values for outliers in the deflection data set, correction factors must be applied to the backcalculated HMA values. This is particularly important for HMA surfaces greater than 4 inches in thickness. FPS 21 considers the modulus of bituminous materials only at the reference temperature of 77°F. FWD data are rarely

collected at the reference temperature, corrections must be made for FPS input. Since modulus values for surfaces thinner than 3 inches cannot be backcalculated, the designer must use engineering judgement in discerning the design modulus value (at design temperature 77°F, considering condition of existing HMA) of thinner HMA surfaces to be incorporated in the FPS design. Where backcalculation of the HMA surface is possible, modulus adjustments are made using one of the following methods.

Two methods are suggested:

1. Use the formula $CF = T^{2.81}/200,000$, where CF is the correction factor to be multiplied by the backcalculated HMA modulus (adjusted for outliers), and T is the average temperature over the time the FWD survey was made, or
2. Use the Modulus Temperature Correction Program ([MTCP](#)). MTCP can use the surface temperatures measured at each deflection location and, together with the previous day's average temperature (available at weather underground <http://www.wunderground.com/history/>), predict the in-pavement temperature and compute the temperature adjusted modulus. Outliers must still be removed from the calculated temperature-corrected average.

Section 5: Pavement Detours and Pavement Widening

5.1 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 conventional performance period.

Detours can generally be placed into two categories. The first category is the diversion of traffic onto 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 removed.

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 integrated into the permanent structure.

Estimating traffic loading over the time frame for a detour is 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/edge support, and quality workmanship. These points should be emphasized during the pre-construction conference and actively monitored by inspectors in the field.

5.1.1 Structural Design of Detours

The falling weight deflectometer (FWD) shall be used to evaluate the adequacy of any existing structure (e.g., shoulders or bypass routes) to carry detour traffic.

FPS 21 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 TTC 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 21, 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 (as in the post-design check built into FPS) assumes a design life of 20 yr. For detours expected to last less than 2 yr., 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 to be no more than 10 yr.

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 21 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.

When using the Mechanistic Check option in FPS 21 to evaluate detour sufficiency in fatigue cracking and subgrade failure (rutting), the routine will estimate the passes to failure in both these modes of a standard 18-kip axle and compare that against the cumulative 18-kip axles in the first performance period of the FPS design. A “check result” message will appear indicating whether the design passes or fails. However, the FPS-calculated performance period is often much longer than the intended detour life; therefore, estimating the cumulative ESALs to the end of the projected detour life may be desirable. One procedure the designer can use to estimate cumulative ESALs over this short detour life is by calculating ESALs in “TRAFFIC6.xls” (refer to MNT, [Engineering Applications](#), to download the spreadsheet) or similar estimating routine, then compare this value against the predicted crack and rut life numbers generated in the FPS Mechanistic Design Check. Refer to the [User’s Manual](#) for more details on using the FPS Mechanistic Check feature.

5.1.2 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 ensure adequate performance over the expected life of the detour structure. Among these are:

- adequate pavement width to ensure 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, stabilization should be considered for low to moderate strength, moisture-susceptible subgrades. Alternately, a geogrid can be used as long as some separation (e.g., thin granular layer) from a proposed HMA 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 to compact the bottom HMA lift against. Improved density in the HMA will increase fatigue life and moisture resistance;

- adequate drainage by ensuring proper cross slope, ditch lines, and prevention of ponding;
- properly compacted pavement lifts including the HMA surfacing; and
- sufficient HMA density at mat joints and full bonding between HMA lifts.

Proof rolling should be conducted prior to applying surfacing to ensure adequate sheer strength in the unbound layers. Consider the more stringent requirements in the applicable specifications to ensure 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.

5.2 Pavement Widening

Widening is conducted to increase the flow capacity of the highway or to improve safety aspects, such as the inclusion of shoulders, turning bays, etc. Consideration shall 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.

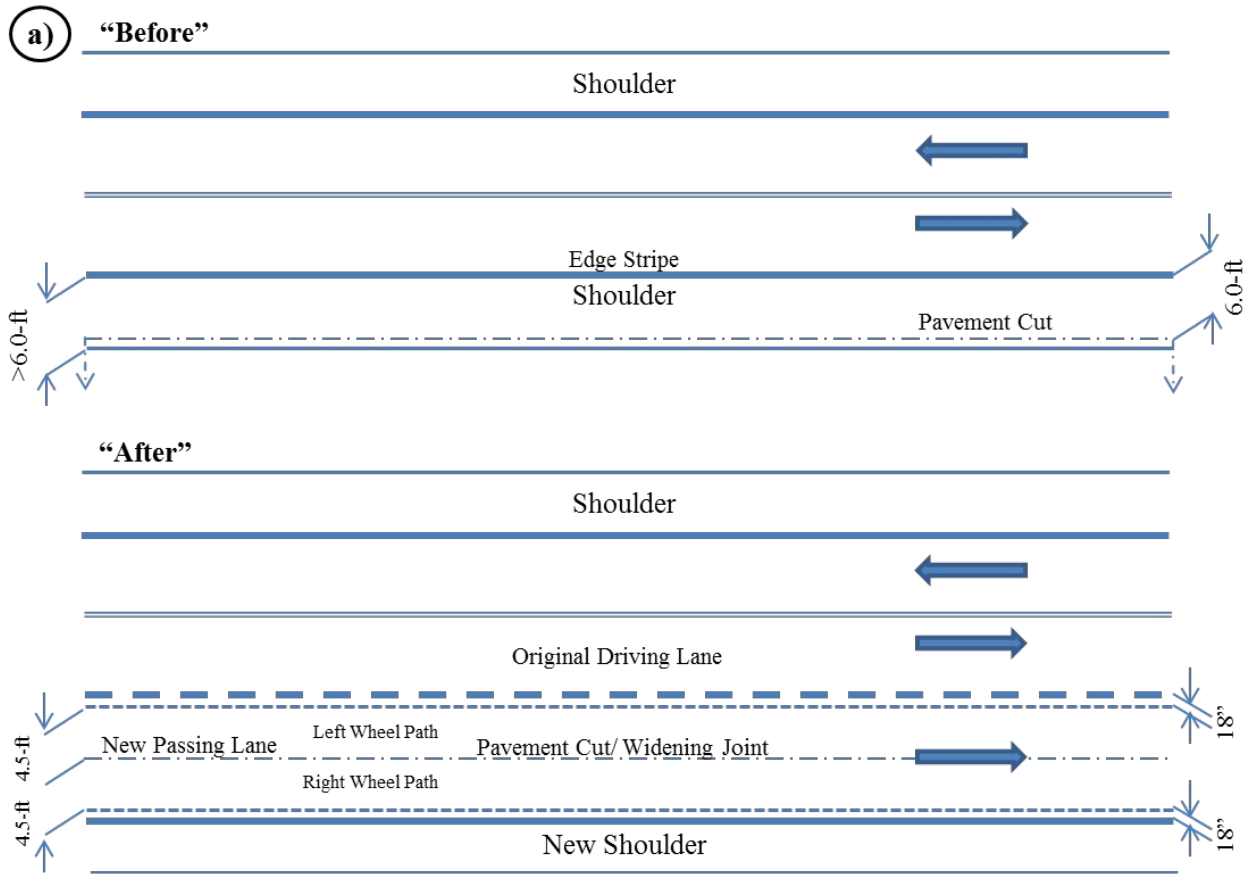
Where traffic control can be facilitated, reworking/widening [add-rock if needed] of the pavement section full-width yielding an upgraded, uniform full-width section can often be obtained for a nominal additional cost over more problematic [poor performing] "scab-on" techniques. Consideration of this technique may be most appropriate for narrower or shoulder widening jobs.

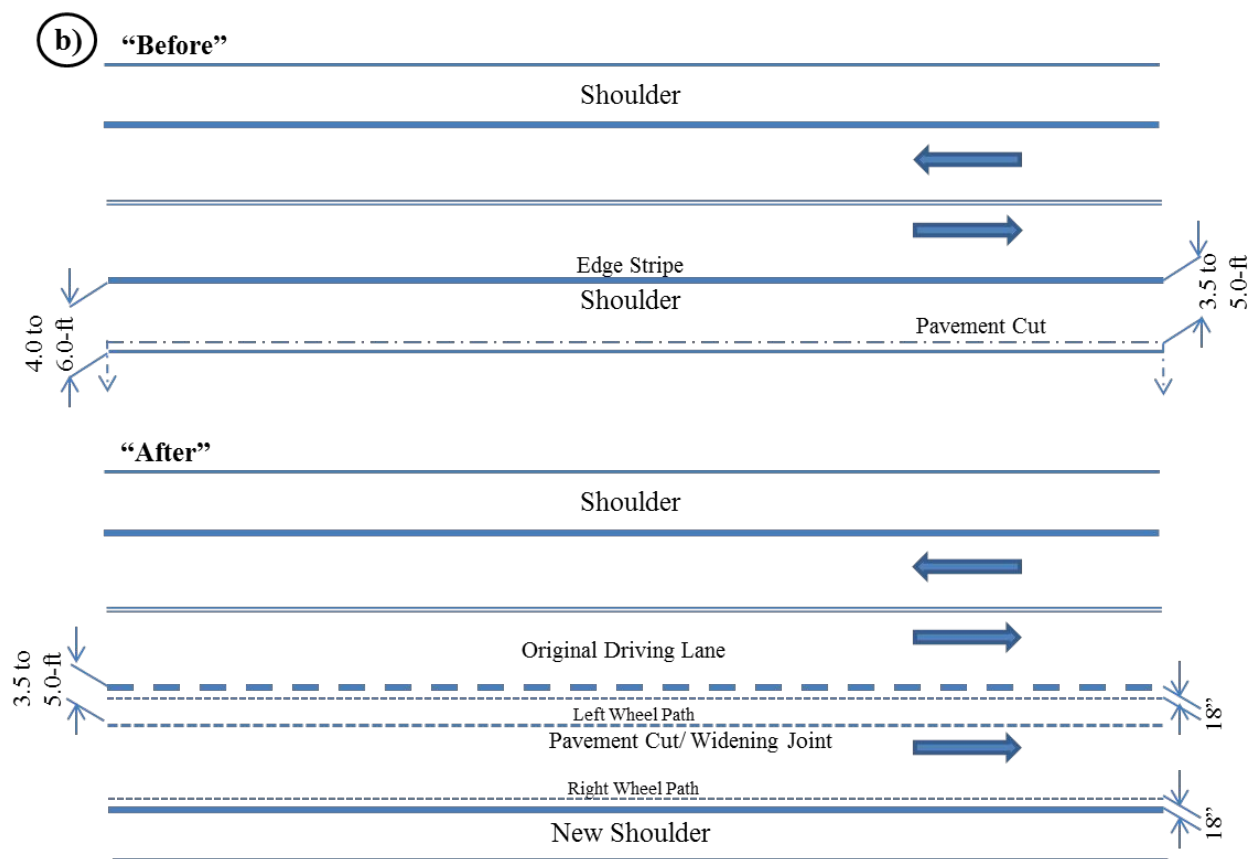
In addition to cross section considerations, full lane-width widening will entail a full-depth joint that is unavoidably a weakened interface in the structure. Compaction against the vertical plane of the old structure will be more difficult to achieve than with full-width construction. Placement of this joint as far from the wheel path as possible will improve performance. Also, applying a final HMA 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 shall 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.

Widening a 2-lane structure for the purpose of developing a Super2 or 4-lane cross section will entail additional considerations for cut locations and shoulder structural analysis when the existing shoulder may form part of a new driving lane. When the shoulder is to form part of a new lane,

non-destructive surveys using the falling weight deflectometer (FWD) are essential. In addition, ground penetrating radar (GPR) may prove to be a useful preliminary survey tool to determine whether the shoulder matches the current driving lane in structure. Recommendations for pavement cut locations for Super2 construction are shown in Figure 5-2.





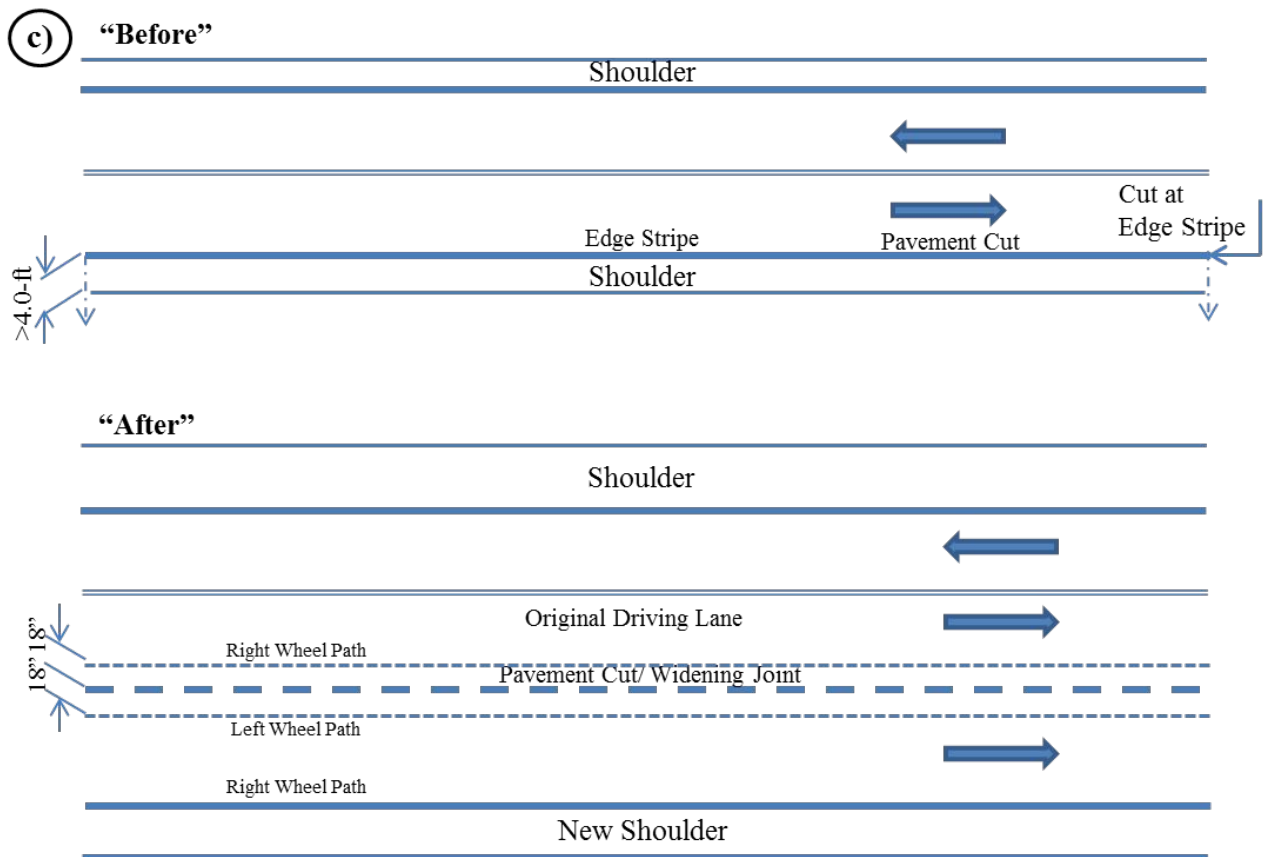


Figure 5-2. Recommended Widening Joint Location: a) 12.0-ft. Lanes, Original Shoulder > 6.0-ft. width, b) 12.0-ft. Lanes, Original Shoulder 4.0- to 6.0-ft. width, and c) 12.0-ft. Lanes, Original Shoulder < 4.0-ft. width.

5.2.1. 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.

5.2.1.1 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 HMA to the outside of the existing lane, also remove an extra 12.0 in. to the inside of the existing lane edge (stripe). The object is to offset the HMA 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 initial curing process.

- Apply level-up to match original HMA surface elevation. Make repairs to original surface as necessary. Apply reflective crack retardant (geotextile, SAMI), if desired.
- Apply overlay across full section. Final longitudinal mat joints should be placed at lane stripes when used.
- Ensure ditch lines are sufficient to prevent hydraulic backflow into the pavement structure. Figure 5-3 depicts an example of an unbound base section.

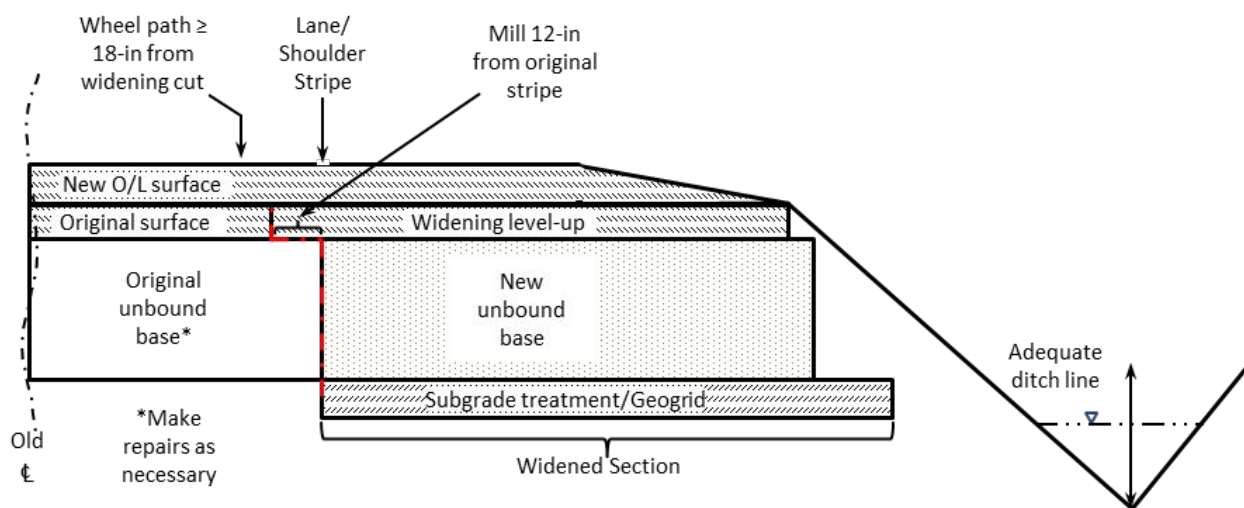


Figure 5-3. Unbound base sections.

5.2.1.2 Bound Base Sections

Other considerations will closely parallel those cited for the unbound base situation. If an original bound base is cement-treated material, there are cases when it may be preferable to use full-depth HMA 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.

5.2.2 Recommendations for Dissimilar Cross Section Widening Strategies

This widening strategy is not encouraged because it often results in lateral subsurface drainage restrictions (see Figure 5-4 “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 (Chapter 2, Section 8). Unfortunately, this remedy is generally

counter to expediting construction. Sources of in-pavement moisture accumulation are shown below.

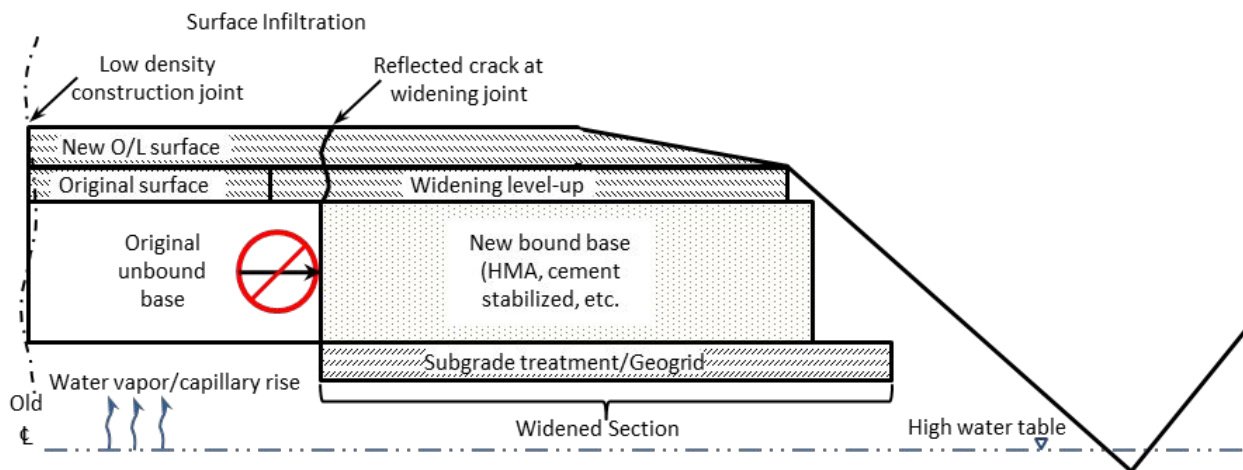


Figure 5-4. In-pavement moisture accumulation, resulting in “bath tub” effect.

Section 6: Perpetual Pavement Design and Mechanistic Design Guidelines

6.1 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 21, 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 incorporate other intrinsic material characteristics, the environment, and their interaction under traffic loading, to the progression of specific forms of distress by using mathematical transfer functions. As a result, multiple performance relationships for any given structure subjected to a given regional environment and regional or site-specific axle loads are needed.

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.

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.

Current guidelines have taken into consideration design and constructability issues experienced in the structures designed under the original guidelines. Recommended structural layer composition for facilities with a projected 20-yr. one direction cumulative loading of at least 30 million ESALs is clarified. When 30 million ESALs are exceeded, premium mixtures, such as Item 346, “Stone Matrix Asphalt (SMA),” and Item 344, “Superpave Mixtures,” should be used in lieu of conventional QC/QA dense-graded specifications (Item 341) to ensure adequate top-down crack resistance in the surface and rut resistance within lower HMA layers. 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 Superpave 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 layers with prior approval by the director, MNT, or designee.

Along with establishing an engineered foundation, using a dense, low air void HMA bottom layer ($N_{des} = 35$ gyrations or 97.5% lab density) shall be considered when:

- full-depth HMA structures have a composite HMA thickness between 8-12 in., or
- there are concerns for bottom-up moisture intrusion into less rich (drier/higher air void) bottom HMA layer.

6.2 Limiting Strain Criteria

While research continues in this area, there is one “special case” that easily lends itself to mechanistic structural evaluation, albeit without robust regard to reliability. Experts have defined *limiting strain 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 strain 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 strain criteria presuppose that quality materials are used and that proper construction procedures are followed. The *limiting strain criteria* reported by experts in the field (such as Nunn and Monismith) are:

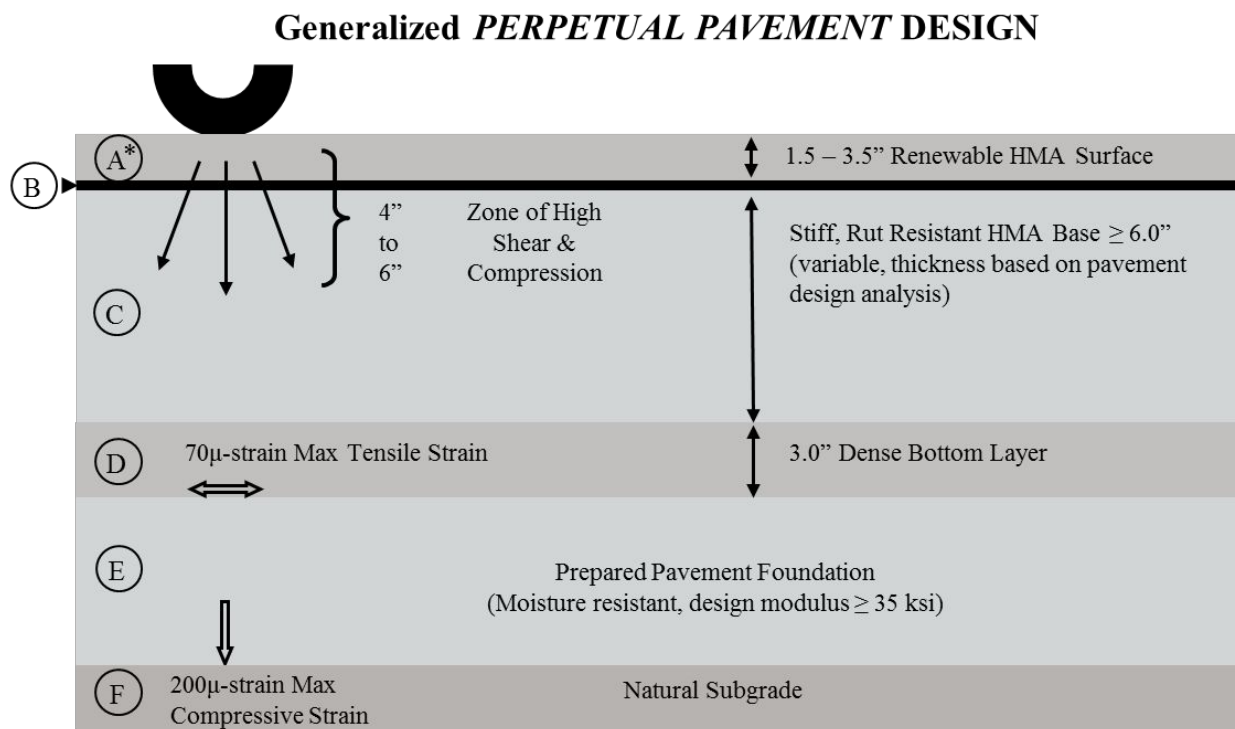
- tensile strain at the bottom of the composite HMA layers: $< 70 \mu\text{-strain}$,
- compressive strain at the top of the subgrade: $< 200 \mu\text{-strain}$.

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 21 to roughly estimate an initial thickness design, then use an analysis program that calculates the primary responses to traffic loading for the heaviest anticipated wheel loads for each season of the year at the critical locations, or
2. the designer speculates at the sufficiency of a desired structure and then evaluates it at the critical locations using a mechanistic analysis program to see what performance life is predicted.
3. In either case, the design would then be modified to achieve the desired or optimized performance.

6.3 Foundation Design

Special attention is required in designing a durable foundation by investigating the underlying soils to determine the appropriate type, level, and depth of stabilization needed. In lieu of subgrade stabilization, a high quality granular base, cement-treated base, or other engineered foundation can be used. Detailed design and construction considerations are provided in Figure 5-5.



* See key on following tables

Figure 5-5. Perpetual Pavement Design.

Table 5-8: Perpetual Pavement Layer Composition

Layer	Layer Composition	Spec Item	Preferred Mix Size	Preferred Lift Thickness	PG Grade	N _{des}
A	Renewable surface	Item 342 (PFC)		1.5 in.	76-XX	50
	SMA (or SP) or two-layer system PFC on top of SMA	Item 346 (SMA)	SMA-D	2.0 in.	76-XX	50
	(Passing Balance Mix Design criteria)	Item 344 (SP)		2.0 in.	76-XX	50
B	Seal Coat	Item 316 or 318	Grade 4 or 4S	--	--	--
C	Rut-Resistant HMA Base	Item 344*	SP-B	4X nominal maximum aggregate size each lift	70-22**	50
D***	Dense Bottom Layer	Item 341	D	2.0 - 3.0 in.	64-22	35 (SGC); or 97.5% lab density (TGC)

Table 5-8: Perpetual Pavement Layer Composition

Layer	Layer Composition	Spec Item	Preferred Mix Size	Preferred Lift Thickness	PG Grade	N _{des}
E	Prepared Pavement Foundation	1. Item 247 2. Item 275 3. Item 260 4. SS 3003 or SS 3063	Grade 1/2 Min 200 psi UCS after conditioning Passing Tex 121E Part 1 Passing Test methods 122E or 134E	6 - 12 in. 6 - 12 in. 8.0 in. 6 - 10 in.	PG 64-22 or CSS 1H	--
F	Natural subgrade	--	--	--	--	--
* Preference should be given to designing above the reference zone. Substitution of Item 344 mixtures using Item 341 is permitted after consulting with MNT – Pavement Asset Management. ** Use PG 70-22 or higher grade for all HMA mixes that fall within the top 6.0 in. of the finished pavement surface. *** Can be eliminated if total HMA layer greater than 10 inches thick. NOTE: See Construction Considerations in figure above, layer E. Must be designed to pass a strength test - contact MTD for details.						

6.4 Designing a Perpetual Pavement Using FPS21

A summary of the steps to design perpetual pavement using FPS 21 follows.

Table 5-9: Designing Perpetual Pavement Using FPS 21

Step	Action
1	Pavement design Type 7 (User Defined) is recommended for this type of structure.
2	Select a 30-yr. analysis period.
3	Use a confidence level of 'C' (95%).
4	Use lane distribution reduction factors when three or more lanes are planned in one direction to adjust the 20-yr. cumulative 18-kip ESALs.
5	Enter the 20-yr. cumulative ESALs (or adjusted ESALs) in the 18-kip ESAL field.
6	Select a "time to first overlay" of 15 yr.
7	Follow general guidelines for all other inputs. Note elevated moduli values are permitted for all HMA layers based on total thickness. Select the red arrow button to run the design.

Steps 1-3: Use pavement design Type 7 for this type of structure since perpetual pavements typically have more than 4 unique layers. Careful attention is emphasized for the selection of structural layers as recommended above under Section 2, *Types of Flexible Pavements*. Select a 30-yr. analysis period. The analysis period will not be critical, since staying reasonably below the limiting strain criteria specified above is the ultimate goal. However, just meeting the criteria does not ensure adequate reliability (i.e., just meeting the 70 μ -strain criterion could mean a high probability of failure when not accounting for poor design, material variability, or construction practices). Set the FPS confidence level to ‘C’ (95%). This confidence level is not tied to the limiting strain criteria, but is useful in ensuring a reasonable beginning thickness to evaluate further.

Steps 4 and 5: Use lane distribution reduction factors when three or more lanes are planned in one direction. Adjust the 20-yr. cumulative ESALs, if needed, with an appropriate reduction factor; then enter the adjusted 20-yr. ESALs into FPS. The FPS program converts the 20-yr ESALs to 30-yr ESALs for the selected analysis period.

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 and still ensure a reasonable reliability. Establishing a “time to first overlay” may not appear to be consistent with “perpetual” design. Actually, the performance module within FPS 21 is inconsistent with perpetual design. The FPS-calculated overlay will not be a structural requirement, but should reasonably mimic an almost certain requirement for “surface renewal” to mitigate the effects of surface wear, oxidation, top-down cracking, etc.

Step 7: Follow general guidelines for all other inputs, with particular attention to serviceability indices and material moduli. Ensure a permanent foundation layer that accounts for environmental variability (including poor soils and moisture fluctuation) is included in the design. Select the *red arrow* 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 entirely pertinent to the development of a perpetual design in FPS, but serves as a rough gauge/reminder that one or more surface renewals will be needed over the design life.) Conduct a design check of the limiting strain criteria by activating the FPS 21 mechanistic check. Figure 5-6 shows a typical perpetual pavement setup in the mechanistic check input screen. Ensure that the horizontal red arrow tensile strain indicator symbol is placed at the lowest HMA interface by dragging it into place. The “vary thickness” green sensitivity button shall be dragged adjacent to the layer that the designer would most likely change the thickness of to optimize the design.

Thick. (in)	Modulus(ksi)	v	Material Name	Vary Thickness
2.00	650.0	0.35	STONE-MATRIX ASPHALT	☐
10.00	650.0	0.35	PERFORMANCE MIX 3/4SF	☒ 0.50
2.00	500.0	0.35	DENSE BOTTOM LVR	☐
6.00	50.0	0.35	FLEXIBLE BASE	☐
6.00	35.0	0.30	LIME(CEMENT) STAB SUBG	☐
200.00	12.0	0.40	SUBGRADE	☐



f1 7.96E-02
 f2 3.291
 f3 .854
 f4 1.37E-09
 f5 4.477

Analysis Mode
☒ Design ☐ User Define

$$N_f = f_1(\epsilon_t)^{-f_2} (E_1)^{-f_3}$$

$$N_d = f_4(\epsilon_v)^{-f_5}$$

Run

Exit

Figure 5-6. Mechanistic Check Input Screen.

By selecting the run button on the input screen, the mechanistic check output screen must show strain results below the limiting strain thresholds previously cited for bottom-up fatigue cracking and compressive failure of the subgrade (rutting) for the design to be valid. Also, the check will report “unlimited” fatigue and rutting life (> 150 million ESALs), as shown in Figure 5-7 below. For a thorough discussion on the use of the FPS Mechanistic Check feature, refer to the [FPS 21 User’s Manual](#).

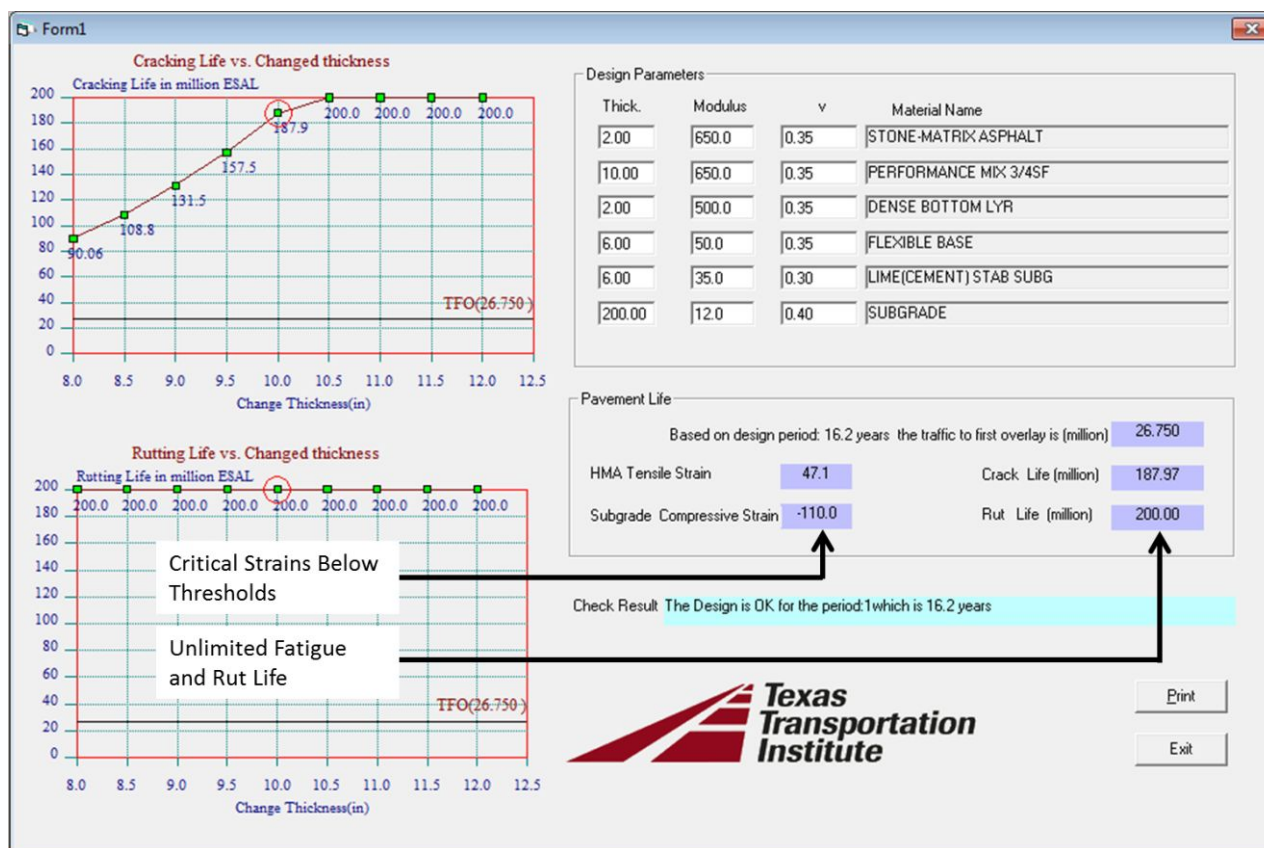


Figure 5-7. Mechanistic Checks Output Screen.

Chapter 6: Flexible Pavement Construction

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Section 1: Overview

1.1 Acknowledgement

Content of this chapter is largely made available from the University of Washington, Pavement Tools Consortium.

1.2 Introduction

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:

- base and subgrade preparation,
- pavement surface preparation,
- special considerations for the construction of perpetual pavements,
- plant operation,
- mix transport,
- mix placement, and
- compaction.

Section 2: Base and Subgrade Preparation

2.1 Introduction

Pavement performance can be largely attributed to the care given in designing and preparing its foundation, which is comprised of the subgrade and base layers. Base and subgrade layers must provide adequate and moisture resistant strength that meets the design 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, and requires considerations beyond basic FPS design inputs to mitigate.

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 or future traffic loading demands. Shortages of locally available high quality soil-aggregate resources are becoming more and more common statewide.

In order to achieve specified properties, subgrade, select fill, and base materials frequently require chemical treatment with additives such as lime, cement, fly ash, and asphalt. Each of these materials must be properly designed to determine the most appropriate additive and concentration to achieve the desired improvement.

Additive selection criteria can be found in the [Guidelines for Chemical Treatment of Soils and Bases 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 method employs. See Chapter 3 or contact MNT – Pavement Asset Management for more information.

2.2 Intelligent Compaction

Intelligent Compaction (IC) is an innovative approach for a continuous quality control process with the objective of achieving uniform and consistent compaction during the placement of soil and aggregate base materials. IC uses vibratory compactors instrumented with global positioning systems, an integrated stiffness measurement system, and real-time computer imaging feedback that plots coverage and compaction progress, allowing the roller operator to make timely tracking adjustments to ensure adequate/uniform compaction coverage. The technology has matured over the past several years to allow inclusion on most jobs for proof rolling. Contact MNT – Pavement Asset Management for more information.

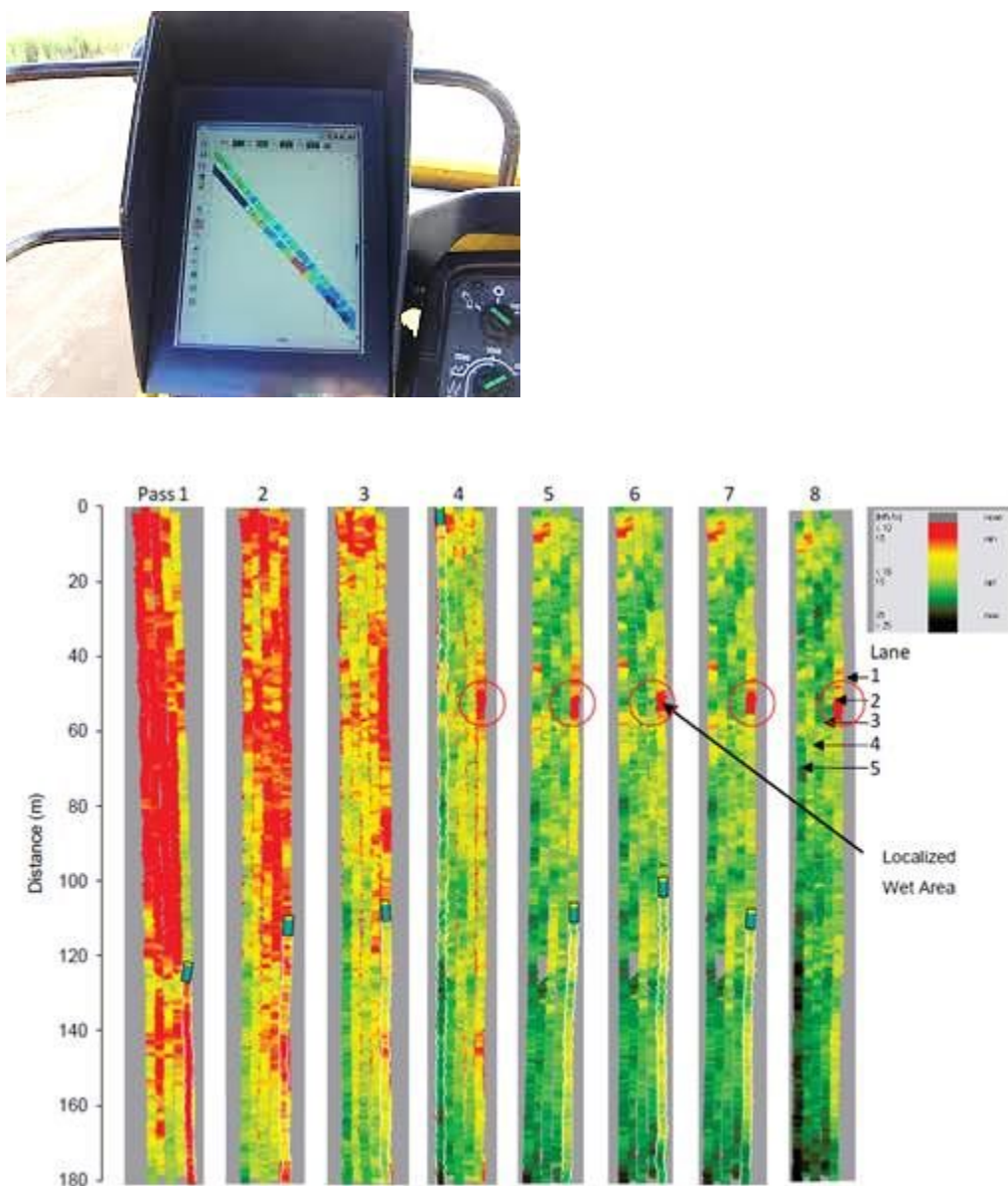


Figure 6-1. Intelligent Compaction Operator's Screen Displaying Relative Degree of Compaction.

Section 3: Pavement Surface Preparation

3.1 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.

3.2 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 asphalt or asphalt emulsion (AE) applied to the surface of untreated subgrade or base layers. The prime coat serves several purposes:

- fills the surface voids and protects the base from weather,
- stabilizes the fines and preserves the base material integrity, and
- 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 a rate between 0.2 to 0.5 gal./SY, and emulsified asphalts are applied between 0.1 to 0.3 gal./SY. For emulsions, it is recommended to cut-in the emulsion with the top 1 in. of the base. Alternately, cutting in 50% of the prime into a surface of cement-treated base, followed by compaction and application of the remaining 50% has shown to be beneficial in bonding of subsequent bituminous layers.

3.3 Underseals

Preparation of a primed soil/aggregate base surface or an existing asphalt-surfaced pavement for an overlay typically includes an underseal which is a sprayed application of asphalt binder (asphalt emulsion or hot applied asphalt binder) immediately covered by a layer of single-sized aggregate. The underseal provides several benefits, such as waterproofing the surface, sealing small cracks, and protecting the underlying surface from solar radiation. 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.

3.4 Seal Coats

For new or reconstructed pavements, the seal coat can be placed on a crushed stone or chemically treated base to provide a roadway with the least expensive permanent type of bituminous surface. Seal coats seal and protect the base and provide strength at the road surface so that the base can resist the abrasive and disruptive forces of traffic. Often these surfaces are placed in multiple treatments of two or three placements of successively smaller rock to enhance durability and reduce tire noise. A prepared road base structure that is to be surfaced using the seal coat concept should always be primed first. The asphalt in the prime should be compatible with the binder used to create the seal coat. Seal coats have been used successfully on both low- and high-traffic volume roadways, but tend to be more successful on low-volume roadways, especially low-volume truck traffic. Use of hot PG, polymer-modified, or asphalt rubber binders with larger grade rock (Grade 2 or 3) can improve the performance of seal coats under higher truck traffic conditions. The use of seal coats in urban areas where accelerating/decelerating traffic and turning movements frequently occur should be approached with caution. Binder application rates vary in accordance with the type/size of the treatment rock being placed, type of binder used, the level of traffic, and the condition/texture of the existing surface. See the [“Seal Coat and Surface Treatment Manual”](#) for guidelines and methodologies for design of binder and aggregate application rates.

3.5 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. 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, applying a level-up course or both), cleaned, and then coated with a binding agent. This subsection covers:

- repair,
- tack coats.

3.5.1 Repair

Generally, pavement overlays are used to restore surface course functional characteristics (smoothness, friction, and aesthetics) or add structural support to an existing pavement. However, even a structural overlay should be placed on a structurally sound base. If an existing pavement is cracked or provides inadequate structural support, these defects will eventually reflect through even the best-constructed overlay and cause premature pavement failure in the form of cracks and deformations. To maximize an overlay's useful life, failed sections of the existing pavements should be patched or replaced and existing pavement cracks should be filled.



Figure 6-2. Repairing Failed Pavement Sections before Overlay.

At most, overlays are designed to add only some structural support; the remaining structural support must reside in the existing pavement. Therefore, small areas of localized structural failure in the existing pavement should be repaired or replaced to provide this structural support (see Figure 6-2). Often, existing pavement failure may be caused by inadequate subgrade or flex base support, or poor drainage. In these cases, the existing pavement over the failed area should be removed full depth and the subgrade should be prepared as it would be for a new pavement (Item 351, “Flexible Pavement Structure Repair”).

Existing pavement crack repair methods depend upon the type and severity of cracks. Badly cracked pavement sections, especially those with pattern cracking (e.g., fatigue cracking) must be patched or replaced because these distresses are often symptoms of more extensive base or subgrade structural failure. Existing cracks other than those symptomatic of structural failure should be cleaned out (blown out with pressurized air and/or swept) and filled with a crack-sealing material when the cracks are clean and dry. Cracks less than about 3/8 in. in width may be too narrow for crack-sealing material to enter. These narrow cracks can be widened with a mechanical router before sealing. Swelling of crack sealant may occur particularly under newly placed thin HMA overlays, causing a bump at these locations. Use of WMA with their lower placement temperatures may mitigate this problem.

If the existing pavement has an excessive amount of fine cracks but is still structurally adequate, it may be more economical to apply a seal coat or slurry/scrub seal instead of filling each individual crack. Also, these situations may lend themselves to using a spray-paver where a relatively heavy emulsion is applied concurrently with the overlay.

In summary, pavement repair should be extensive enough to provide an existing pavement with adequate structural support. Pavement management techniques should provide for overlays before an existing pavement has lost most or all of its structural support capability.

3.5.2 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 or near surface shear 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.

3.5.2.1 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, and
- the type of tack coat according to standard TxDOT specifications on Item 330, “Limestone Rock Asphalt Pavement,” and hot-mix items 334, 340, 341, 342, 344, 346, and 347.

3.5.2.2 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. 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 6-3). Slippage cracking and delamination are distresses typically seen when cleanliness is lacking.



Figure 6-3. Tack coat tracking resulting in no tack coat in the wheel path. © Copyright 2006 University of Washington

3.5.2.3 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 (Flexible Pavements of Ohio, 2001)

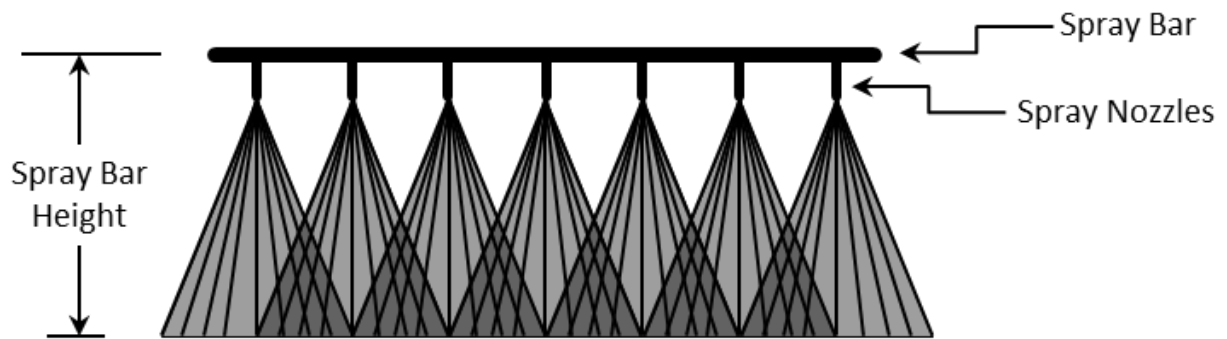
Existing Pavement Condition	Application Rate (gal/SY)	
	Residual ^a	Undiluted ^b
New HMA	0.03 - 0.04	0.05 - 0.07
Oxidized HMA	0.04 - 0.06	0.07 - 0.10
Milled HMA	0.06 - 0.08	0.10 - 0.13
Milled PCC	0.06 - 0.08	0.10 - 0.13
PCC	0.04 - 0.06	0.07 - 0.10
^a Residual: The application rate of just the asphalt binder content of the emulsion.		
^b 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 result 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 6-4, Figure 6-5, Figure 6-6, Figure 6-7, Figure 6-8, Figure 6-9, and Figure 6-10). 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.



Good coverage – double lap

Except for the outside nozzles, each point on the pavement surface is covered by exactly two spray nozzles – a double lap. Usually at least a double lap is needed to meet specified application rates.

Figure 6-4. Tack coat application using double lap coverage. © Copyright 2006 University of Washington



Figure 6-5. Tack Coat Distributor Truck. © Copyright 2006 University of Washington



Figure 6-6. Tack Coat Distributor Truck Spray Bar. © Copyright 2006 University of Washington



Figure 6-7. Tack Coat Distributor Control Panel. © Copyright 2006 University of Washington



Figure 6-8. Jet dryer used ahead of the tack coat distributor. © Copyright 2006 University of Washington



Figure 6-9. Tack coat application. © Copyright 2006 University of Washington



Figure 6-10. Tack coat application showing nozzle coverage area. © Copyright 2006 University of Washington

3.5.3 Other Tack Coat Aspects

3.5.3.1 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).

3.5.3.2 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. More critical to long-term pavement performance is that any tack removed by tracking from construction or other vehicular traffic implies that the tack is no longer available to stick the new overlay to the existing surface. This can become extremely critical in cases of HMA delivery trucks tracking tack out of the wheel paths while in the process of their delivery in front of the paving machine. This sets up the worst possible scenario of removing tack from the most critical part of the pavement structure

directly subject to the shearing action of traffic wheel loads. Allowing tack coats to set (emulsions) before driving on them can substantially reduce tracking. Also, new “trackless” emulsion products that dry very quickly, but are reactivated by the heat of the newly placed HMA, are worth consideration when tracking may be an issue.

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./SY (Flexible Pavements of Ohio, 2001).

Section 4: Special Considerations for the Construction of Perpetual Pavements

4.1 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. A geotechnical investigation of the underlying soils is essential to establish the appropriate type and level of treatment needed to stabilize the foundation layers. Where adequate treatment of the subgrade is impractical, a high quality granular base, cement-treated base, or other engineered foundation should be used.

4.2 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 (surface) 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.

Detailed perpetual pavement design construction considerations are available through this [link](#).

Section 5: Plant Operations

HMA production is the first step in construction of HMA pavement layers. The basic purpose of an HMA plant is to properly proportion, blend, and heat aggregate, additives, 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 & Construction*. National Asphalt Pavement Association Research and Education Foundation. Lanham, M.D.

Transportation Research Board (TRB). 2000. *Hot-Mix Asphalt Paving Handbook 2000*. Transportation Research Board, National Research Council. Washington, D.C.

5.1 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. Batch plants are not widely used in the United States, and the vast majority of newly manufactured plants are drum plants. 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 6-11 shows the basic components of a batch plant.

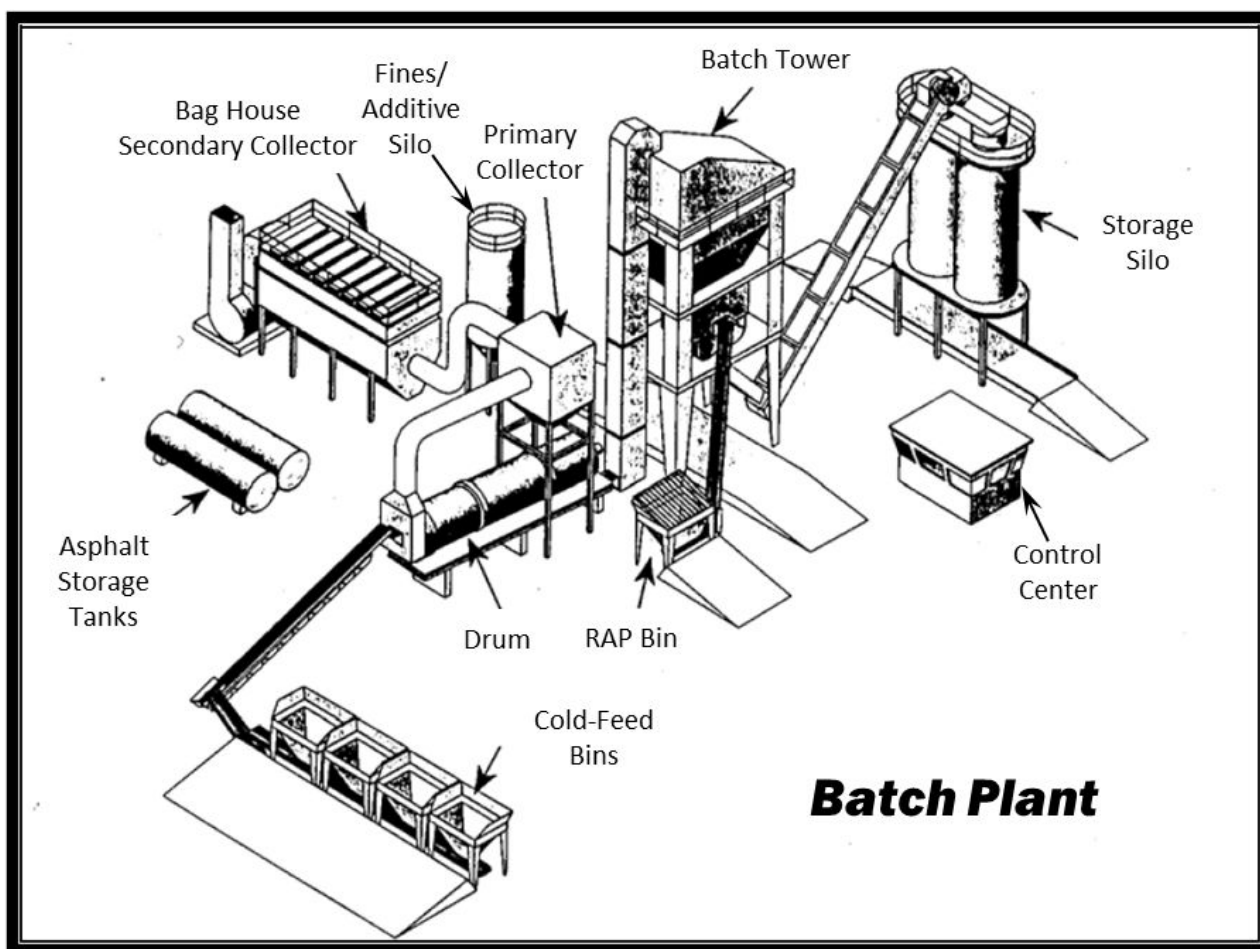


Figure 6-11. Batch Plant. © Copyright 2006 University of Washington

5.2 Drum Plants

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 6-12 shows the basic components of a drum plant and their function.

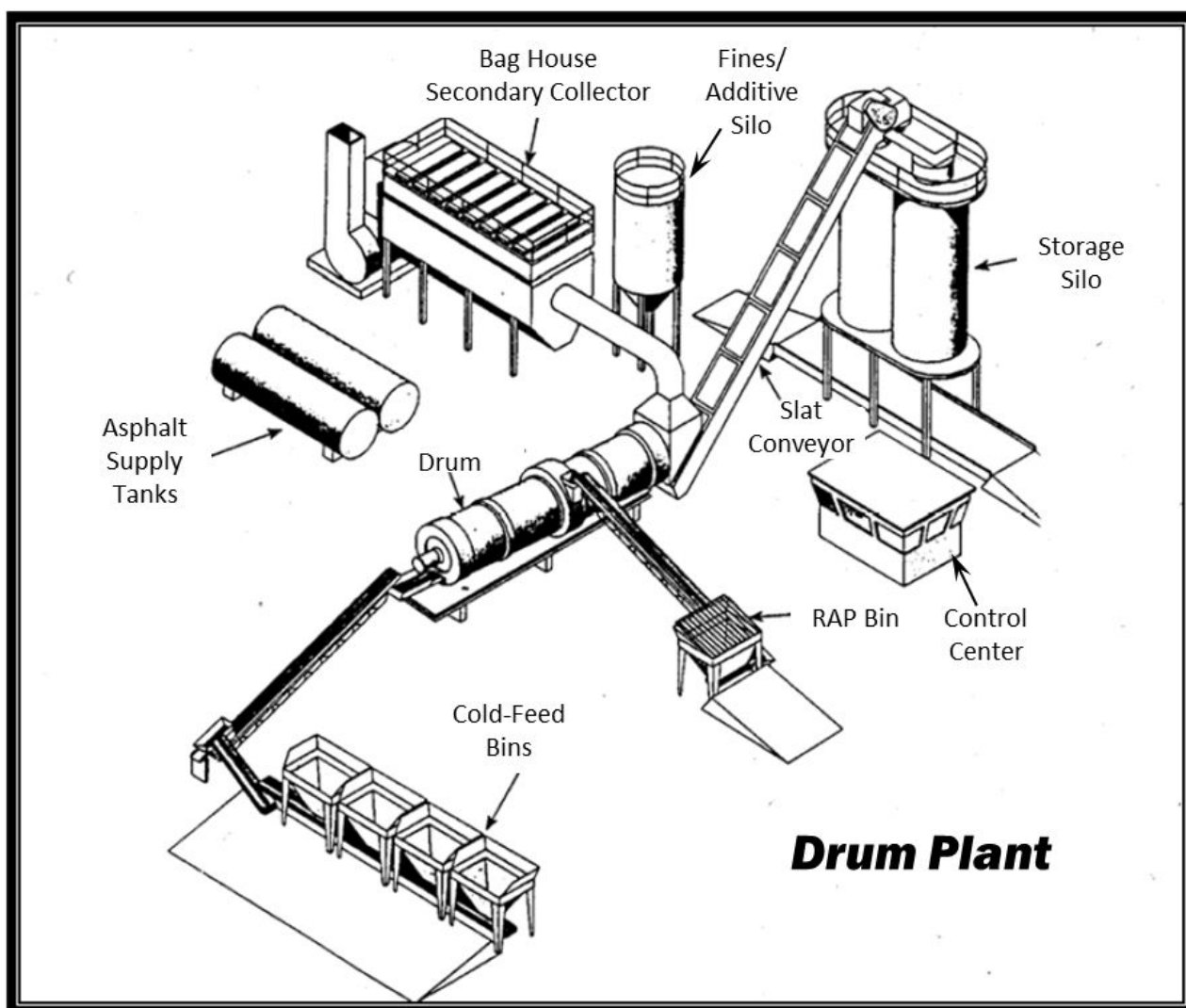


Figure 6-12. Drum Plant. © Copyright 2006 University of Washington

Section 6: Mix Transport

6.1 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, windrow, 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.

6.2 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).

6.2.1 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 6-13). 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:

- *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.
- *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.
- *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.
- *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 6-13. End Dump Truck.

6.2.2 Bottom Dump Truck

Bottom (or belly) dump trucks (see Figure 6-14) 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 6-15), 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 6-14. Bottom dump truck. © Copyright 2006 University of Washington



Figure 6-15. Windrow made by emptying a bottom dump truck. © Copyright 2006 University of Washington

6.2.3 Live Bottom Dump Truck

Live bottom (or flo-boy) dump trucks (see Figure 6-16) 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 6-16. Live Bottom Truck. © Copyright 2006 University of Washington

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 Advantages” below, may make one truck type advantageous over another.

Table 6-2: Truck Type Advantages

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.

Table 6-2: Truck Type Advantages

Situation	Possible Truck Type	Reason
Paving using a mix highly vulnerable to segregation	Live Bottom (Flo Boy)	Live bottom trucks deliver the HMA by conveyor, which minimizes segregation.
Paving on rural highways	Bottom Dump (Belly Dump)	Usually has a larger capacity than end dump trucks (therefore fewer trucks are needed), but requires space and equipment for windrows.

6.3 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:

- loading at the production facility,
- transport within the truck,
- unloading at the paving site,
- operation synchronization.

6.3.1 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:

- *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.
- *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 6-17). 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 6-17. Truck loading under a storage silo. © Copyright 2006 University of Washington

6.3.2 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 6-18). During transport, heat is transferred to the surrounding environment by convection and radiation, and the HMA surface temperature 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 6-19 and Figure 6-20). Factors 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:

- *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 or environmental concerns may prohibit the use of a mobile production facility.
- *Insulate truck beds.* This can decrease HMA heat loss during transport. Insulation as simple as a sheet of plywood has been used.

- *Place a tarpaulin over the truck bed.* A tarp over the truck bed (see Figure 6-21) 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).

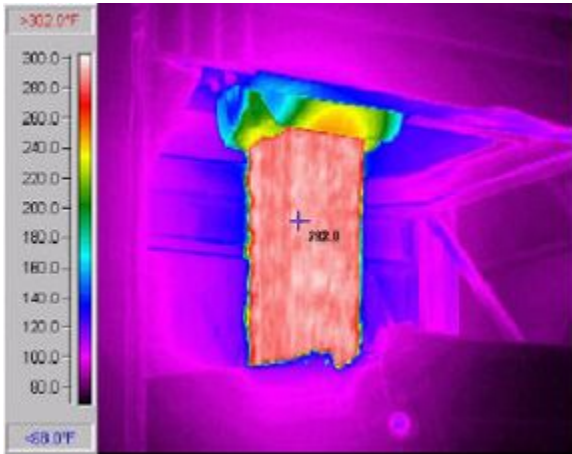


Figure 6-18. Infrared picture of an HMA storage silo loading a truck showing the hot uniform temperature of the mix. © Copyright 2006 University of Washington

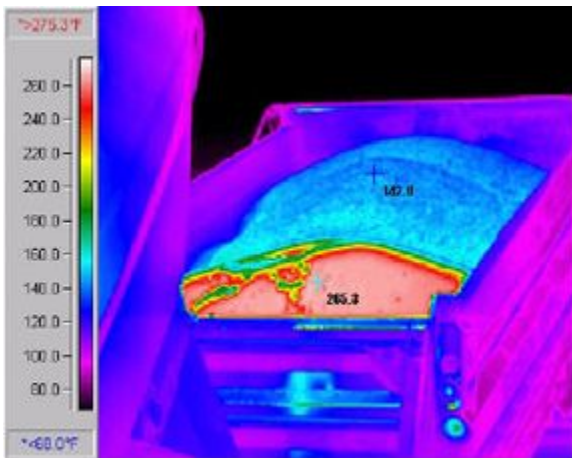


Figure 6-19. Infrared picture of a truck dumping HMA showing the cold surface layer crust (blue) and the hot inner mass (red). © Copyright 2006 University of Washington

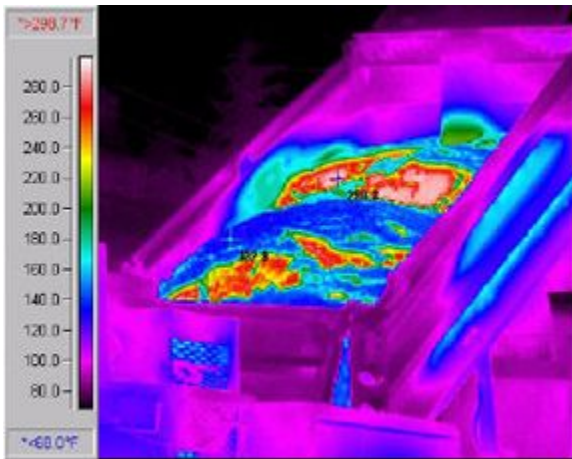


Figure 6-20. Infrared picture of a truck dumping HMA showing the cold surface layer crust (blue) and the hot inner mass (red). © Copyright 2006 University of Washington



Figure 6-21. Tarped Truck. © Copyright 2006 University of Washington

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.

6.3.3 Unloading at the Paving Site

HMA unloading involves those procedures discussed in 'End Dump Truck' under [6.2 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, material transfer vehicle, or dropped in a windrow, 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.
- A material transfer device/vehicle with remixing capability should be considered when mixture type or placement circumstances lend themselves to mixture mechanical or temperature segregation, or a surge volume may be desired to assist in continuous paving operations (see *Operation Synchronization* below) when the transport truck delivery schedule cannot be ensured (see Figure 6-22). However, considerations must be made to ensure sufficient pavement structure is in place to support some of these larger devices. See Section 6, Material Transfer Vehicles, for additional information.



Figure 6-22. Material Transfer Vehicle with surge capacity (Courtesy Roadtec).

6.3.4 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 laydown 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,
- 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.

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.

6.4 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 (belly) 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. Material transfer devices/vehicles can assist in re-establishing mixture uniformity on site.

Section 7: Placement

7.1 Introduction

In concert with the overall goal of operation synchronization, 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 2000](#) (TRB, 2001) and the Asphalt Institute's *HMA Construction Manual* (2001) contain detailed information on asphalt paver components.

7.2 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 Lift Thickness Requirements

HMA Mixture Type		Nominal Maximum Aggregate Size	Minimum Lift Thickness (inches)	Maximum Lift Thickness (inches)	Typical Location of Pavement Layer
Dense Graded (Items 340/341)	TY-A	1-1/2"	3.0"	6.0"	Base
	TY-B	1"	2.5"	5.0"	Base/Intermediate
	TY-C	3/4"	2.0"	4.0"	Intermediate/Surface
	TY-D	1/2"	1.5"	3.0"	Surface/Level-Up
	TY-F	3/8"	1.25"	2.5"	Surface/Level-Up
PFC (Item 342)	PFC-F or PFCR-F	1/2"	0.75"	1.5"	Surface

Table 6-3: TxDOT Lift Thickness Requirements

HMA Mixture Type		Nominal Maximum Aggregate Size	Minimum Lift Thickness (inches)	Maximum Lift Thickness (inches)	Typical Location of Pavement Layer
	PFC-C or PFCR-C	3/4"	1.0"	2.0"	Surface
Superpave Mixtures (Item 344)	SP-A	1"	3.0"	5.0"	Base
	SP-B	3/4"	2.25"	4.0"	Base/Intermediate
	SP-C	1/2"	1.5"	3.0"	Intermediate/Surface
	SP-D	3/8"	1.25"	2.0"	Surface/Level-Up
SMA (Item 346)	SMA-C	3/4"	2.25"	4.0"	Intermediate/Surface
	SMA-D	1/2"	1.5"	3.0"	Intermediate/Surface
	SMA-F	3/8"	1.25"	2.5"	Surface
	SMAR-C	3/4"	2.0"	4.0"	Intermediate/Surface
	SMAR-F	3/8"	1.5"	3.0"	Surface
CAM ^a (SS 3000)	CAM	3/8"	1.0"	2.0"	Base/Intermediate Surface ^a
Thin Overlay Mixtures (Item 347)	TOM-C	1/2"	0.75"	1.25"	Surface
	TOM-F	1/2"	0.5"	0.75"	Surface
Thin Bonded Friction Courses (Item 348)	PFC-C	3/4"	1.0"	2.0"	Surface
	PFC-F	1/2"	0.75"	1.5"	Surface
	TY-A	3/8"	0.5"	0.75"	Surface
	TY-B	1/2"	0.625"	1.0"	Surface
	TY-C	3/4"	0.75"	1.25"	Surface
^a Generally not recommended as a surface mixture for pavements with a posted speed limit > 45 mph.					

- *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, and potentially serve as a conduit for moisture damage to lower layers.
- *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 handworked 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 handworked surface with a straightedge or template before rolling to ensure uniformity.
- *Stone Matrix Asphalt (SMA).* Gap-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 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) 2002. *Designing and Constructing SMA Mixtures – State-of-the-Practice*, Quality Improvement Series 122. National Asphalt Pavement Association. Lanham, MD.
- *Thin Overlay Mixes (TOM); Thin Bonded Friction Courses.* These mixtures are particularly vulnerable to rapid cool-down once placed and must be compacted promptly. Compaction cessation temperature (no further densification possible) is typically reached in less than 10 minutes, even under optimal conditions. WMA additives/processes may be effective in prolonging the compaction window by a few minutes when paving at lower ambient and surface temperature conditions. Using two breakdown rollers in tandem may expedite compaction when the compaction window is limited.

- *Permeable Friction Course (PFC)*. Similar concerns of rapid cooling in compacting permeable friction courses exist as for the thin surface mixtures cited above. In addition, the compactive effort imparted should be the minimum necessary to seat the mixture without causing excessive breakage of the aggregate. A steel-wheeled roller, operated in the static mode, should be used. *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 (checking), fat spots, joint problems, non-uniform texture (including mechanical segregation), roller marks, shoving, surface waves, tearing (streaks), and transverse [screed](#) marks. Compaction can also be compromised when working with certain mixtures that are prone to a “tender zone” phenomenon where the mat will encounter excessive lateral displacement and shoving when subject to roller loads. Any number of parameters can contribute to mixture tenderness, including excessive mix temperature, rounded aggregates, excessive binder content, and excessive moisture. Overcoming this tenderness state may be as simple as waiting several minutes for the mixture to cool slightly, or as problematic as developing a new mixture design.

7.3 Asphalt Paver

In 1934, Barber-Greene introduced the Model 79 asphalt laydown machine, a self-propelled formless laydown machine with a floating screed (Tunncliffe, Beaty and Holt, 1974). Since then, the basic concept of the asphalt paver has remained relatively unchanged:

- HMA is loaded in the front,
- carried to the rear by a set of flight feeders (conveyor belts),
- spread out by a set of augers,
- 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.

7.3.1 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 6-23 and Figure 6-24). They are located on the front of the hopper.



Figure 6-23. Push Roller. © Copyright 2006 University of Washington



Figure 6-24. Truck hitch engaged. © Copyright 2006 University of Washington

- **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 6-25). Hoppers can also be fit with inserts to allow them to carry more HMA (see Figure 6-26). These inserts are typically used in conjunction with a material transfer vehicle (MTV).



Figure 6-25. Hopper with wings folded up. © Copyright 2006 University of Washington



Figure 6-26. Caterpillar AP-1055B with hopper insert. © Copyright 2006 University of Washington

- **Conveyor.** The conveyor mechanism carries the HMA from the hopper, under the chassis and engine, and then to the augers (see Figure 6-27 and Figure 6-28). 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 6-27. Conveyors. © Copyright 2006 University of Washington

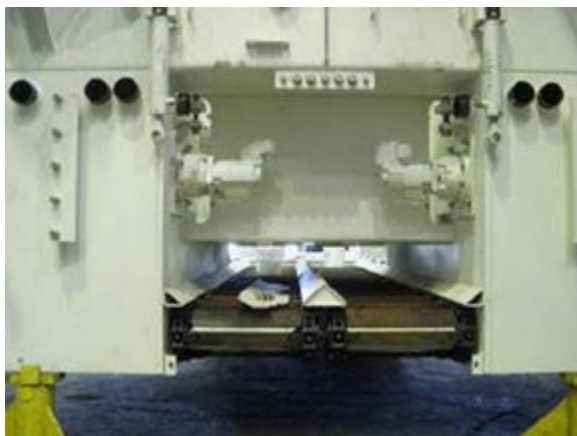


Figure 6-28. Conveyors (seen from the rear of a partially built paver). © Copyright 2006 University of Washington

- **Auger.** The auger receives HMA from the conveyor and spreads it out evenly over the width to be paved (see Figure 6-29). 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 6-30) 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 6-31 and Figure 6-32).



Figure 6-29. Augers distributing HMA. © Copyright 2006 University of Washington



Figure 6-30. Paver Augers (note, gearbox in between augers). © Copyright 2006 University of Washington



Figure 6-31. Gearbox Streak. © Copyright 2006 University of Washington



Figure 6-32. Gearbox streak in a new mat. © Copyright 2006 University of Washington

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).

7.3.2 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, and screed operation summary.

7.3.2.1 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 (see Figure 6-33).
- *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 6-34).
- *Tow point*. Point at which the screed arm is attached to the tractor unit (see Figure 6-35).
- *Depth crank*. The manual control device used to set screed angle and, ultimately, mat thickness.
- *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 6-36 and Figure 6-37). 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.

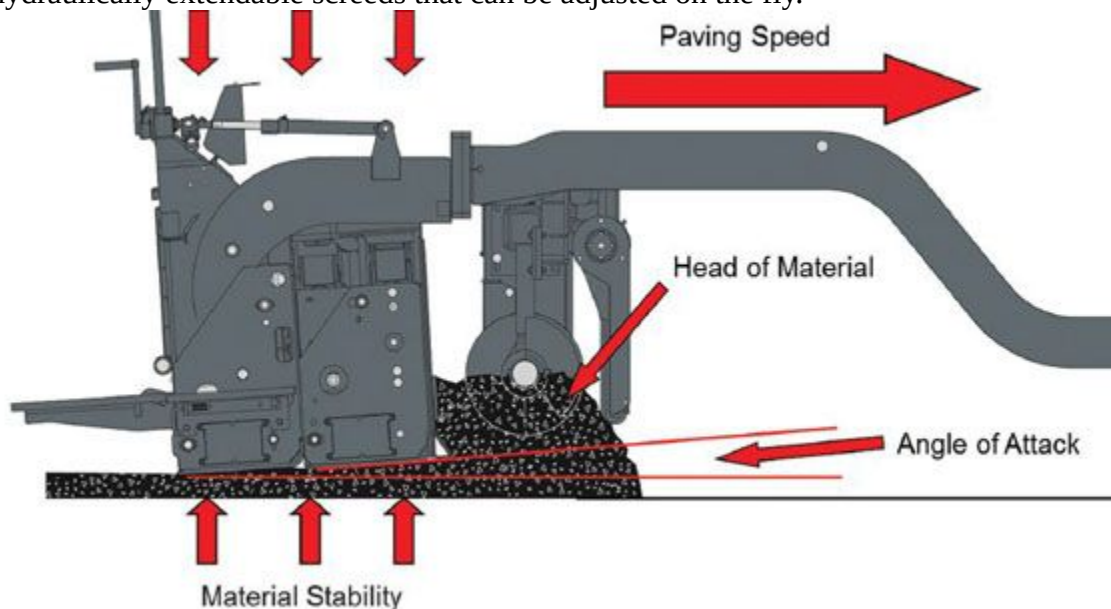


Figure 6-33. Screed angle of attack.

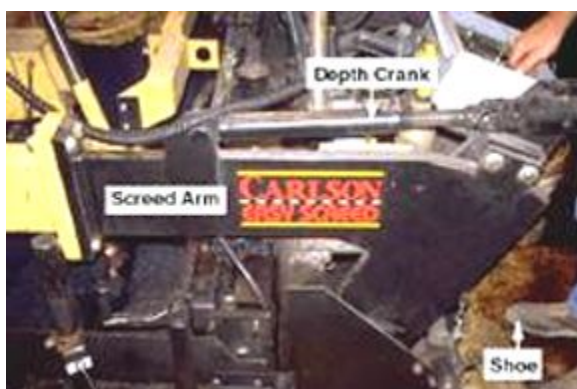


Figure 6-34. Screed close-up showing the screed arm and depth crank. © Copyright 2006 University of Washington



Figure 6-35. Tow Point. © Copyright 2006 University of Washington



Figure 6-36. Hydraulic Screed Extension. © Copyright 2006 University of Washington

(This screed is extended too far [resulting in poor mix delivery and placement] and the tack coat is subpar.)



Figure 6-37. Screed Extension. © Copyright 2006 University of Washington

7.3.2.2 Screed Forces

There are six basic forces (see Figure 6-38) 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.

Screed Components

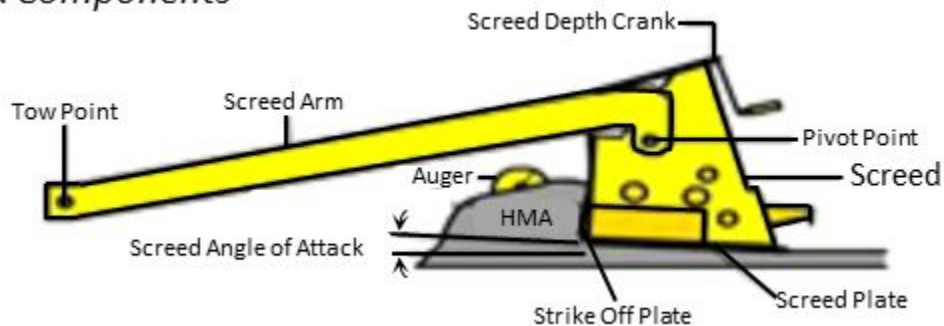


Figure 6-38. Screed Components and Forces. © Copyright 2006 University of Washington

7.3.2.3 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, which increases mat thickness.

- Material Feed Rate.** The amount of asphalt mixture 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 6-39 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.

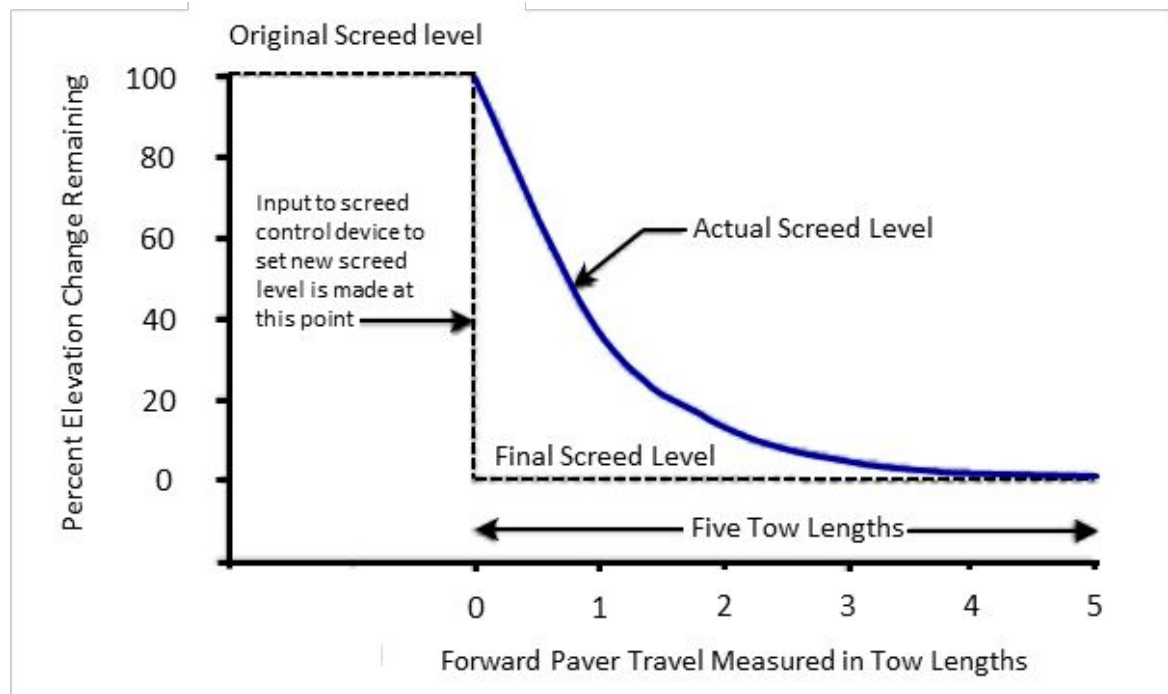


Figure 6-39. Screed reaction to a manual decrease in screed angle, (after TRB, 2000). © 2006 University of Washington

- 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.

7.3.2.4 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 6-40) 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 than 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 length 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 6-40. Automatic grade control using a mobile reference beam. © Copyright 2006 University of Washington

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.

7.3.2.5 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.

7.4 The Spray Paver

The spray paver is an asphalt paving machine that combines the processes of applying tack and paving a bituminous mixture into one machine. The tack emulsion may be applied in a heavy application, sometimes referred to as a membrane, that should be more effective in “sealing” the existing surface than would normally be expected of a tack. Spray nozzles are positioned either just in front of the augurs or in multiple locations on the paver to provide full coverage for the HMA mat placed immediately following the tack application. This process eliminates tracking of the applied tack by construction or other vehicles. Spray pavers can be used with thinner bonded wearing surfaces (Item 348) or conventional HMA applications. An example of a spray paver is shown in Fig. 6-41.

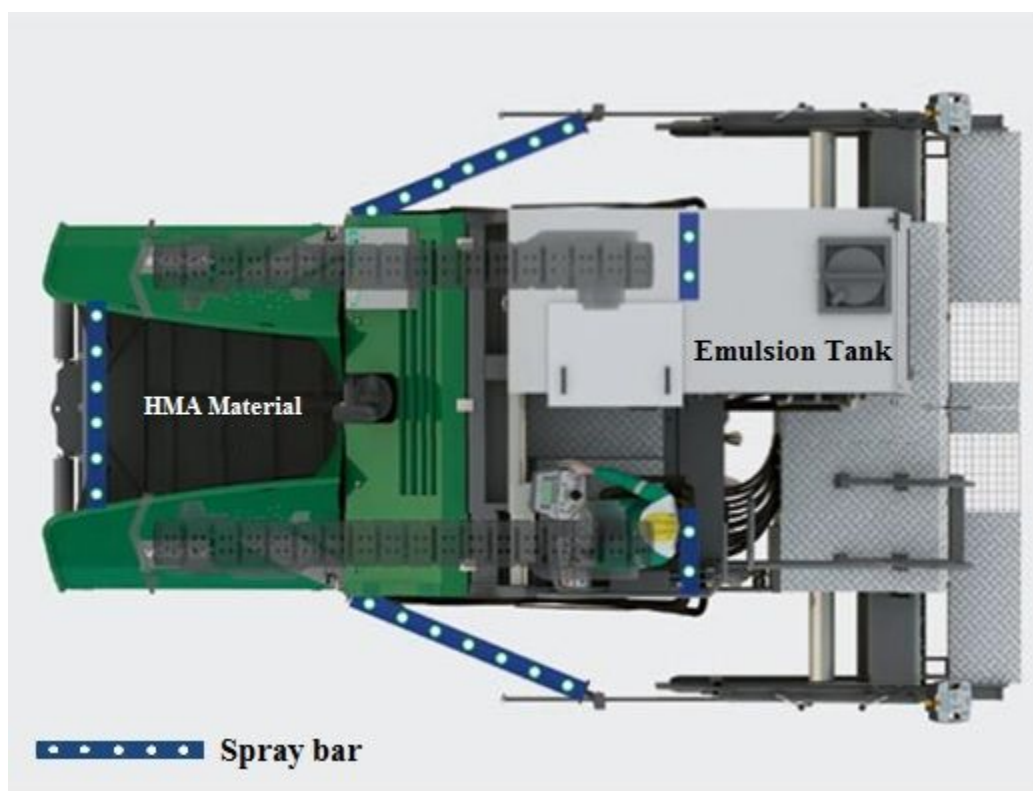


Figure 6-41. Spray Paving Machine (Courtesy of Wirtgen/Vögele).

7.5 Thermal Profiling

Thermal profiles are used to check the level and variability of the mixture temperature behind the paving machine. The applicable test procedure is Tex-244-F. Densification efforts will be impaired

if the mixture has cooled below an appropriate compaction temperature (cessation temperature) for the mat thickness and mixture type. Additionally, if there is a lack of temperature uniformity in the mixture caused by thermal or mechanical segregation, excessive paving machine stop time, etc., compactive efforts will be encumbered. Thermal segregation is a known cause of compaction-related density problems that can lead to moisture infiltration and raveling of the mixture. A mat temperature differential behind the paving machine exceeding 25°F but not exceeding 50°F is classified as moderate thermal segregation. A mat temperature differential exceeding 50°F is classified as severe thermal segregation.

7.5.1 Thermal Camera

Thermal Camera. A thermal camera records a still shot of the HMA mat temperature distribution directly behind the paving machine (and in front of the rollers). The camera must meet requirements in Tex-244-F and have the ability to display the view's high and low temperature on an LCD screen. The contractor is required to conduct a thermal profile for each subplot placed over a test section measuring roughly 150 ft. behind the paver. Corrective action must be taken for recurring moderate thermal segregation, and operations must be suspended to eliminate severe thermal segregation (See Fig. 6-42). Areas detected with severe thermal segregation must be evaluated further using the segregation density profile procedure outlined in Tex 207-F.

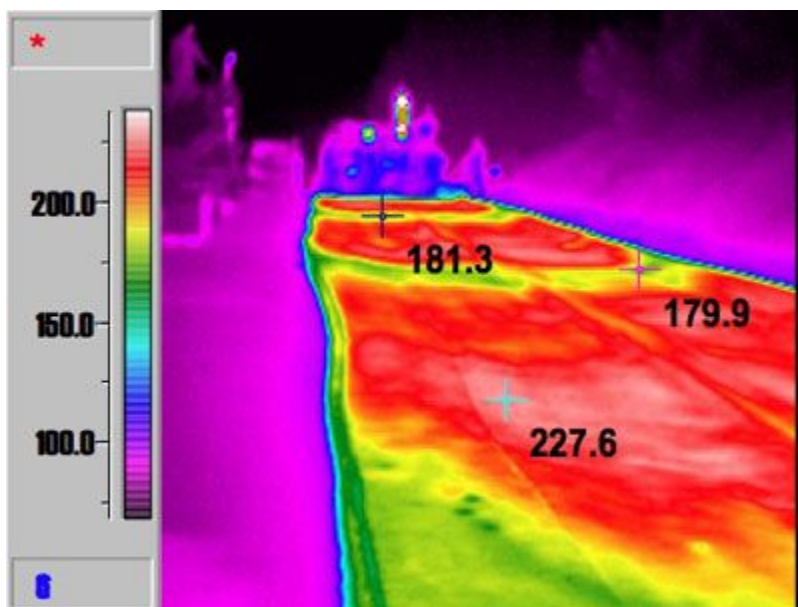


Figure 6-42. Thermal Camera Image Showing Moderate Thermal Segregation.

7.5.2 Thermal Imaging System

Thermal imaging systems are an innovative advancement in the monitoring of the uniformity of the mixture placement temperature behind the paver screed. The system was developed through TxDOT research and has since been commercialized with additional improvements. Figures 6-43 and 6-44 show an early version of the system and an innovative single scanning IR sensor head system that moves the apparatus away from the low mounting position on the catwalk just behind the screed. The ability to monitor the full-width mat temperature distribution in real-time allows the paving crew to make rapid adjustments to the paving operation that improve the uniformity of

the placement temperature, if necessary. An electronic record is generated of the thermal profile tied to longitudinal offset using a distance measuring instrument (DMI) and global positioning system (GPS) coordinates. Electronic copies of the profiles are provided to the engineer; the engineer may suspend operations if the contractor cannot successfully modify the paving process to eliminate recurring severe thermal segregation. Test method Tex-244-F details the apparatus and procedural requirements needed to employ thermal profiling. HMA specifications (Items 341, 342, 344, 346, 347, and 348) offer contractor incentives to use this imaging technique by eliminating the contractor's requirement to run density profiles (not applicable for Items 347, 348), and by relaxing the placement temperature requirements.

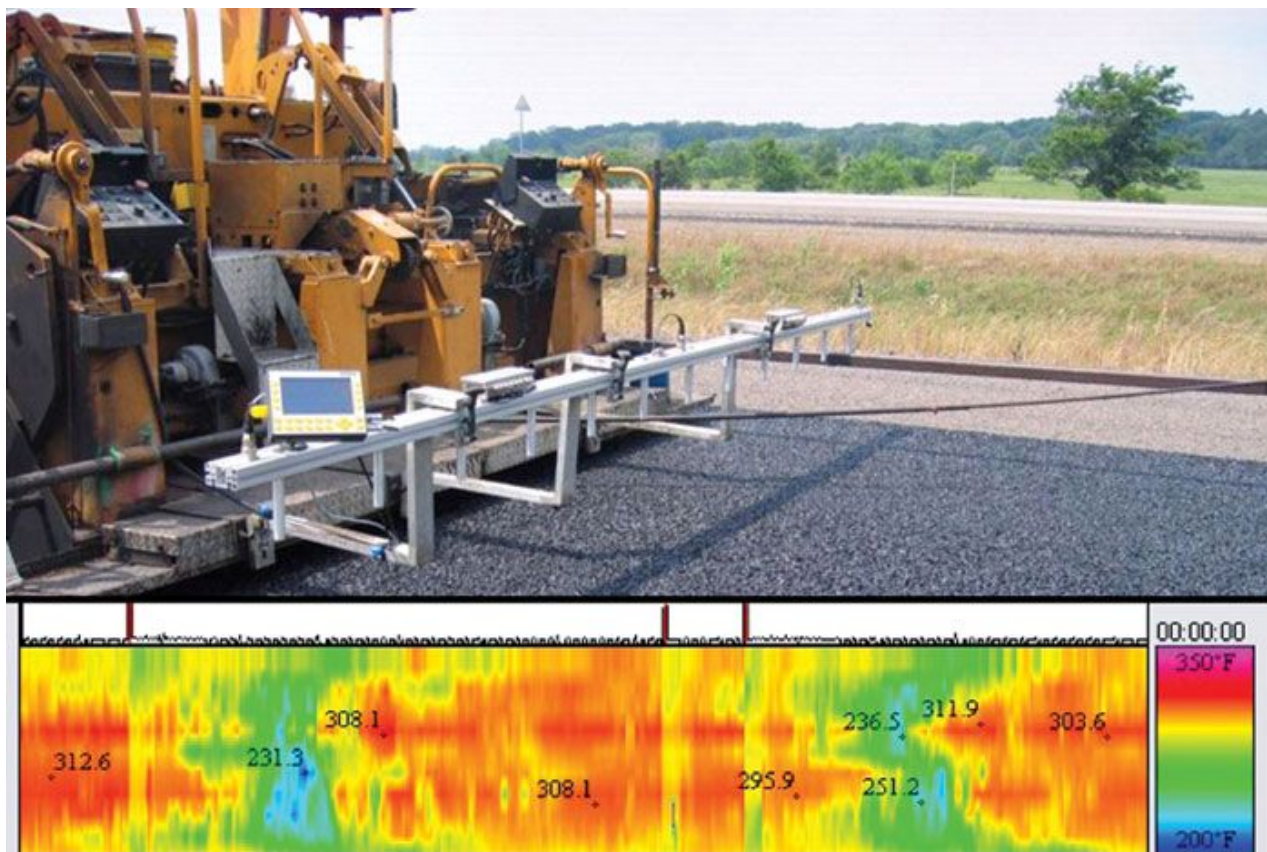


Figure 6-43. Early Version of the Thermal Imaging System.

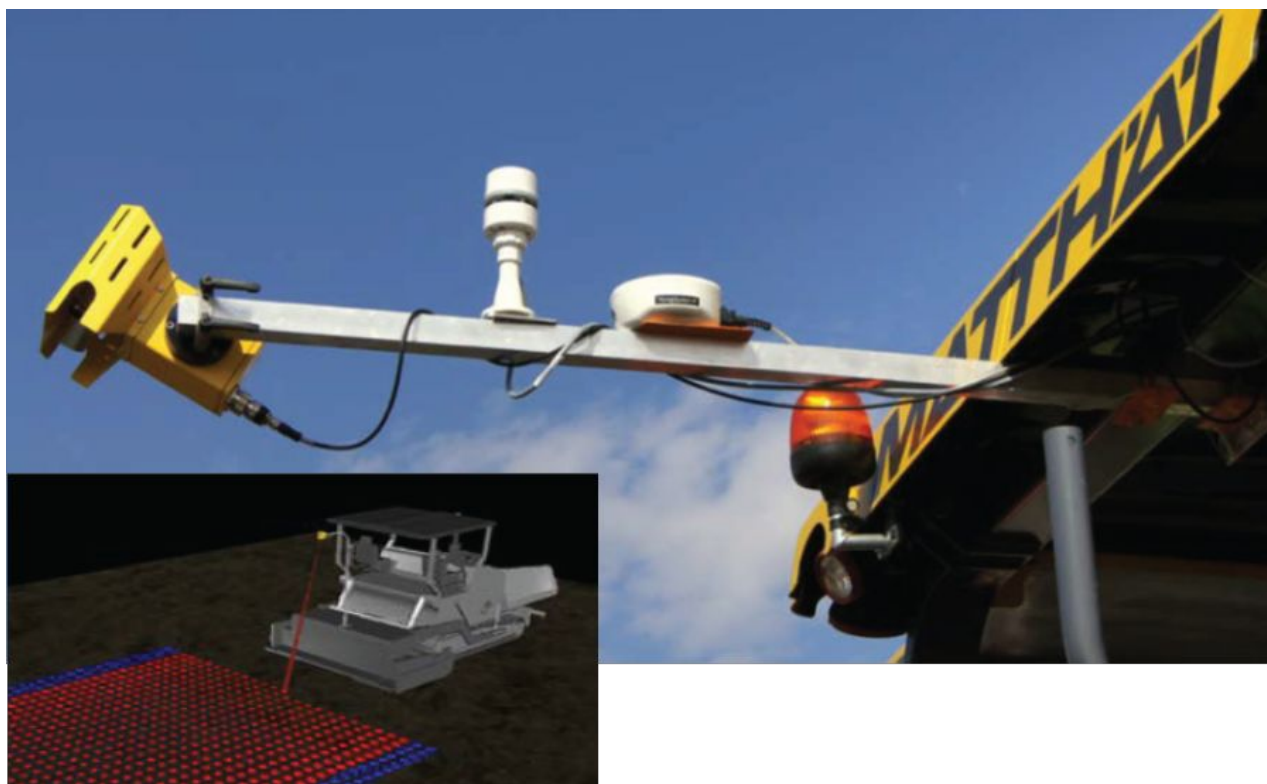


Figure 6-44. Thermal Profiling Using a Single Scanning Sensor Head (Photo courtesy of MOBA).

7.6 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 6-45 and Figure 6-46), 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.

7.6.1 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 6-45. Windrow paving. © Copyright 2006 University of Washington



Figure 6-46. Windrow Elevator. © Copyright 2006 University of Washington

Other MTVs are used to provide an additional surge volume for the paver (see Figure 6-47, Figure 6-48 and Figure 6-49). 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.

7.6.2 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 6-50). 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 6-47. Blaw Knox MC-30 MTV. © Copyright 2006 University of Washington



Figure 6-48. Cedarapids MS-3 MTV. © Copyright 2006 University of Washington



Figure 6-49. Roadtec Shuttle Buggy MTV. © Copyright 2006 University of Washington



Figure 6-50. Roadtec Shuttle Buggy, front view, showing loading hopper for end dump and live bottom trucks. © Copyright 2006 University of Washington

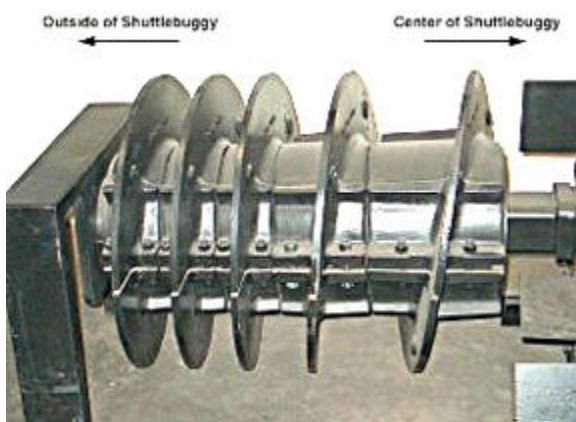


Figure 6-51. Roadtec Shuttle Buggy Patented Remixing Auger. © Copyright 2006 University of Washington

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 6-51). 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

8.1 Introduction

The top three factors in HMA pavement construction are “compaction, compaction, and 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).

8.2 Compaction Measurement and Reporting

Air void content is the measurement that determines the effectiveness of compaction. This measurement is expressed as a volume in terms of percent air voids of the compacted mixture. Percent air voids is calculated by dividing the test specimen’s bulk specific gravity (G_a) by its maximum theoretical specific gravity (G_r). Percent air voids is calculated by using Tex-207-F and Tex-227-F. This procedure uses a bulk specific gravity and a maximum theoretical specific gravity (also referred as Rice gravity) in the following equation:

$$\text{Percent Air Voids} = [1 - (G_a/G_r)] \times 100$$

Where:

- G_r = maximum theoretical specific gravity of the particular HMA in question
- G_a = bulk specific gravity of the HMA in question.

This procedure requires a pavement core (usually 4 - 6 in. in diameter), which is extracted from the compacted HMA (see Figure 6-52 and Figure 6-53). This type of air void testing is generally considered the most accurate method, but is also the most time consuming and expensive.



Figure 6-52. Core extraction. © Copyright 2006 University of Washington



Figure 6-53. Two cores – the core on the right has significantly higher air voids. © Copyright 2006 University of Washington

Gr can also be used with a density gauge to measure the in-place density of the compacted pavement layer. The terms “percent air voids” and “density” are often used interchangeably. Density can be used to calculate percent air voids using the following relationship:

$$\text{Air Voids} = 100 - \text{Density}$$

Contractors can perform quality control of air voids indirectly using a portable density-measuring device such as a nuclear density gauge (see Figure 6-54) or electrical impedance measurement gauge (see Figure 6-55). Accurate calibration of these devices is critical.



Figure 6-54. Nuclear Density Gauge (Courtesy Troxler).



Figure 6-55. Electrical Density Gauge (Courtesy Transtec).

8.3 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% generally produce the best compromise of pavement strength, fatigue life, durability, raveling, rutting, and moisture damage susceptibility.

8.4 Factors Affecting Compaction

HMA compaction is influenced by a myriad of factors including: 1) environment, 2) mix and structural design, and 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
	<i>Asphalt Binder</i>	<i>Other</i>
	• Chemical properties • Physical properties • Amount	• HMA production temperature • Haul distance • Haul time • Foundation support

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: [8.5 Compaction Equipment](#) and [8.6 Roller Variables](#) discuss construction factors.

8.4.1 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:

- *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.
- *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:

- the colder and more viscous the binder, the less actual amount of air void reduction for a given compactive effort, and
- 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. Warm mix additives have been used to extend compaction time resulting from lower temperatures or thinner mix lifts that cool more rapidly.

8.4.2 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):

- *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.
- *Midsized fine aggregate (between the 0.60- and 0.30-mm [No. 30 and No. 50] sieves).* High amounts of midsized fine, rounded aggregate (natural sand) cause a mix to displace laterally or shove under roller loads. This occurs because the excess midsized 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.

- *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. Mixes with low asphalt contents are generally more difficult to compact because of inadequate lubrication, whereas mixes with high asphalt contents will compact more 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 within a limited mixture dependent temperature range known as the tender zone. Mixtures in this zone tend to displace laterally and shove rather than compact under roller loads. It is advisable to cease compaction of these mixtures when the mat temperature is within this zone. Compaction may resume when the mat temperature falls below the tender zone.

8.5 Compaction Equipment

Compaction equipment compacts the HMA by two principal means:

- *By applying its weight to the HMA surface and compressing the material underneath the surface 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.
- *By creating a shear stress between the compressed material underneath the surface 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.

8.5.1 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).

8.5.2 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 6-56 and Figure 6-57).

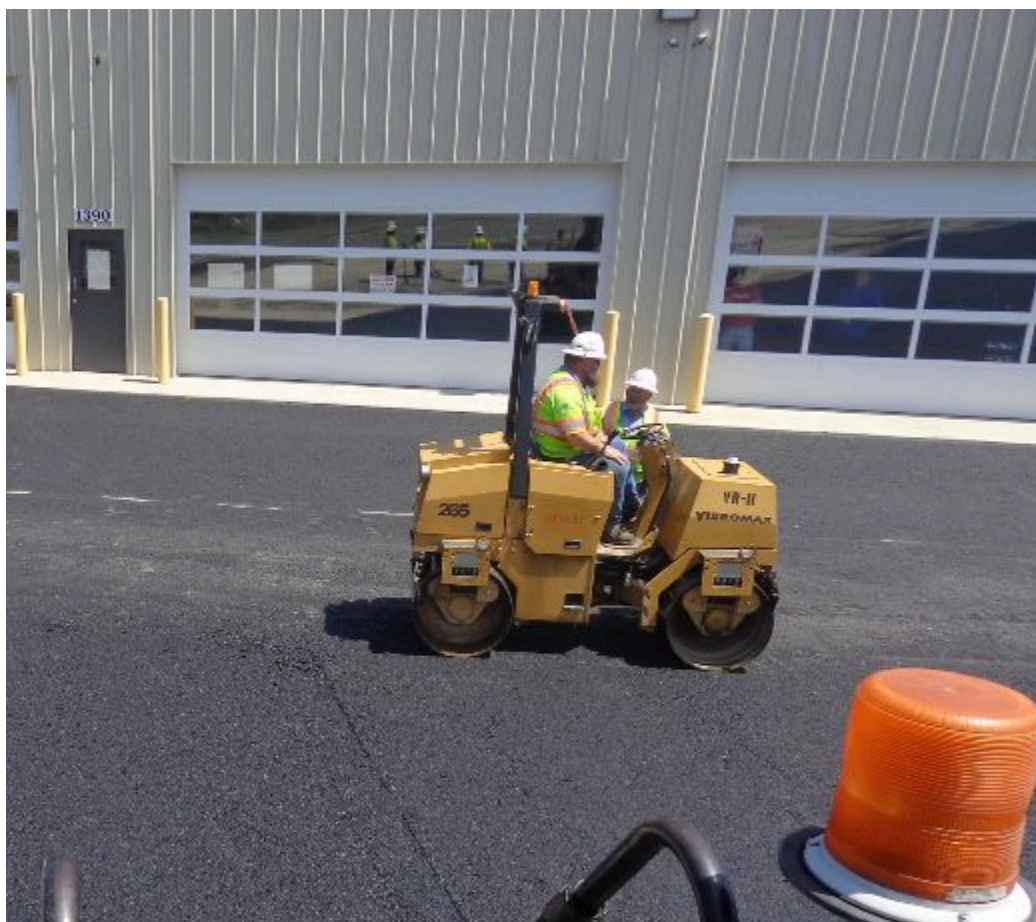


Figure 6-56. Small Static Steel Wheel Roller (1.45 tons 34-inch wide drum). © Copyright 2006 University of Washington



Figure 6-57. Large Vibratory Steel Wheel Roller (18.7 tons 84-inchwide drum). © Copyright 2006 University of Washington

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 6-58), 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 6-58. Filling up with water. © Copyright 2006 University of Washington

8.5.3 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 min	$\text{Frequency} \propto (\text{Dynamic Force})^2$
Amplitude	0.01 to 0.04 in.	$\text{Amplitude} \propto \text{Dynamic Force}$

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 (10 – 12 impacts/ft.). Table 6-6 shows basic guidance for vibratory settings.

Table 6-6: Typical Vibratory Settings (from TRB, 2000)

HMA/Mat Characteristics	Frequency	Amplitude
Thin Lifts ($<$ about 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. In some cases, vibratory mode may be allowed to obtain a smooth transition or joint density.	
Lifts between 1.25 and 2.5 in.	High frequency	Low amplitude
Lifts beyond 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 30 – 35 impacts/m (10 – 12 impacts/ft.) is good. At 3,000 vibrations/min., that gives a speed of 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, in 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.

8.5.4 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 60 psi and 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 the mat temperature will not pick up an appreciable amount of asphalt binder. Insulating the tire area with rubber skirts or plywood helps maintain the tires near mat temperature while rolling (see Figure 6-59).



Figure 6-59. Pneumatic Tire Roller (notice rubber skirt insulation around tire area as well as tire marks left in the new mat in front of the roller). © Copyright 2006 University of Washington

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.

8.6 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.

8.6.1 Compaction Sequence

HMA compaction is typically accomplished by a sequential train of compaction equipment (see Figure 6-60). This allows each piece of equipment to be used only in its most advantageous application, resulting in a higher quality mat (both in density and in smoothness) than could be produced with just a single method of compaction.



Figure 6-60. Breakdown and Intermediate Rollers. © Copyright 2006 University of Washington

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.

8.6.2 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; Item 210 gives typical roller compaction parameters by material type for use on TxDOT jobs.

Table 6-7: Typical Roller Speed Ranges (from TRB, 2000)

Type of Roller	Breakdown	Intermediate	Finish
Static Steel Wheel	2.0 – 3.5 mph	2.5 – 4.0 mph	3.0 – 5.0 mph
Pneumatic	2.0 – 3.5 mph	2.5 – 4.0 mph	4.0 – 7.0 mph
Vibratory Steel Wheel	2.0 – 3.0 mph	2.5 – 3.5 mph	not used

As mentioned previously, roller compactive effort comes in two forms:

1. material compression under the surface 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.

8.6.3 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 6-61).

Table 6-8: Reference and Conditions Information for Figure 6-61

Supporting Data: Graph and data taken from Chadbourn et al. (1998).	• Dense graded HMA, • 2.5-in. lift thickness, • 10 mph wind speed, • 67°F air temperature, • Existing surface is milled HMA, • 73°F surface temperature, • 50% cloud cover.

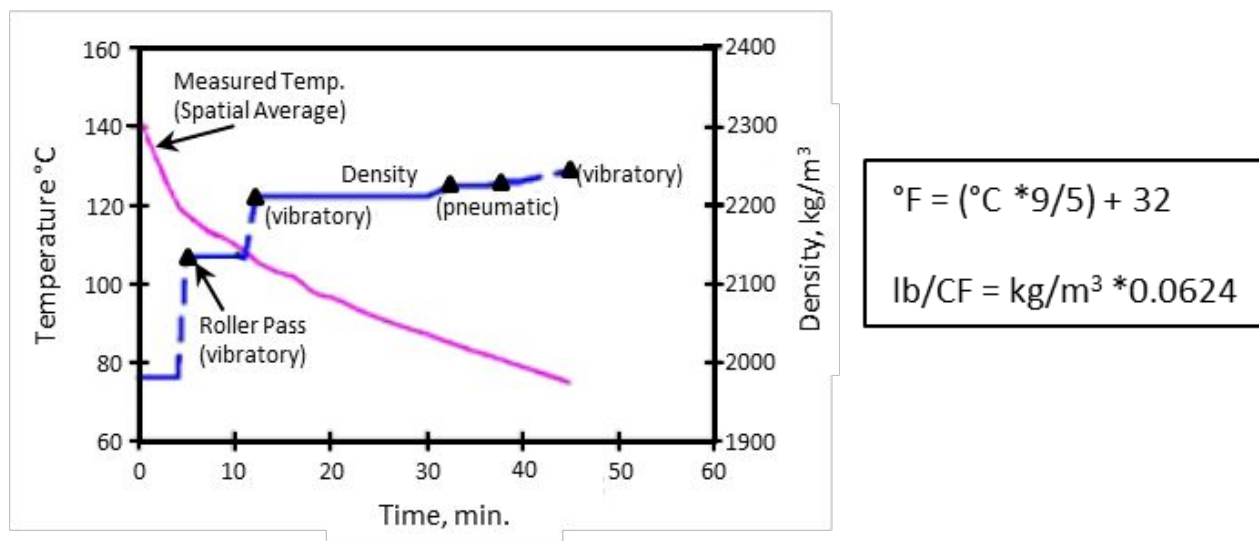


Figure 6-61. Density and Measured Mat Temperature vs. Time (note the increase in density for each roller pass) (Chadbourn et al., 1998). © Copyright 2006 University of Washington

- 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.
- Intelligent Compaction (IC). To date, the benefit of using IC for HMA lifts has been more related to uniform coverage by mapping number of desired passes than any contribution in meeting density. This is because current IC systems lack the necessary sensitivity to distinguish incremental benefit to improving density in typical relatively thin HMA lifts.

8.6.4 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 [8.6 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.

8.6.5 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 guidelines are just general rules of thumb; 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.

8.7 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.

Chapter 7: Flexible Pavement Rehabilitation

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Section 1: Overview

1.1 Introduction

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 tools. 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, and
- meeting all existing constraints of the project.

Section 2: In-place Surface Recycling

2.1 Hot In-place Recycling (HIR)

This rehabilitation technique is meant to address asphalt concrete surface distress and texture issues only; therefore, the underlying base layers must offer adequate support for both the HIR paving train and projected future traffic. Procedures are covered under Item 358 of the Standard Specifications.

The HIR process involves recycling the existing asphalt surface layer by heating, scarifying, and adding a recycling agent. There are three basic HIR processes:

1. Recycling – heating, scarifying, rejuvenating, leveling, reprofiling, and compacting.
2. Remixing – heating, scarifying, rejuvenating, mixing (adding virgin aggregate or new hot-mix), leveling, reprofiling, and compacting.
3. Repaving – combines either recycling or remixing with an overlay of new HMA placed immediately after the recycled mixture. The new HMA layer is placed directly on the recycled layer, and both are compacted simultaneously.

A specialized train (shown in Figure 7-1) is used to heat the surface of the pavement to 200 - 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 cement (AC) binder. Mix design must meet Item 358 requirements.

Pavements with delaminations, especially saturated delaminations, in the top 2 in. should not be considered for HIR projects. Also, the state of practice does not recommend pavements that have been rutted, heavily patched, contain fabric seals, excessively chip-sealed, or rubber chip-sealed as good candidate projects. The 2014 specifications cite a minimum surface temperature for commencement of HIR operations of 60°F, or if the conditions are such that the roadway surface will reach the required temperature within 2 hr. of the beginning of placement operations. 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. Refer to Item 340 for allowances of recycled products in the new 2-in. overlay.



Figure 7-1. Hot In-place Recycling Train.

2.2 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. Figure 7-2 shows a cold in-place recycling train. More recently, foamed asphalt has been used as a binding agent, replacing emulsion on some jobs in other states (Caltrans and others). Other additives, such as lime, fly ash and cement, may be used to improve moisture susceptibility and early strength properties. 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 excessively chip-sealed as good candidate projects. Typically, 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. Application of this procedure has been addressed through special specifications.

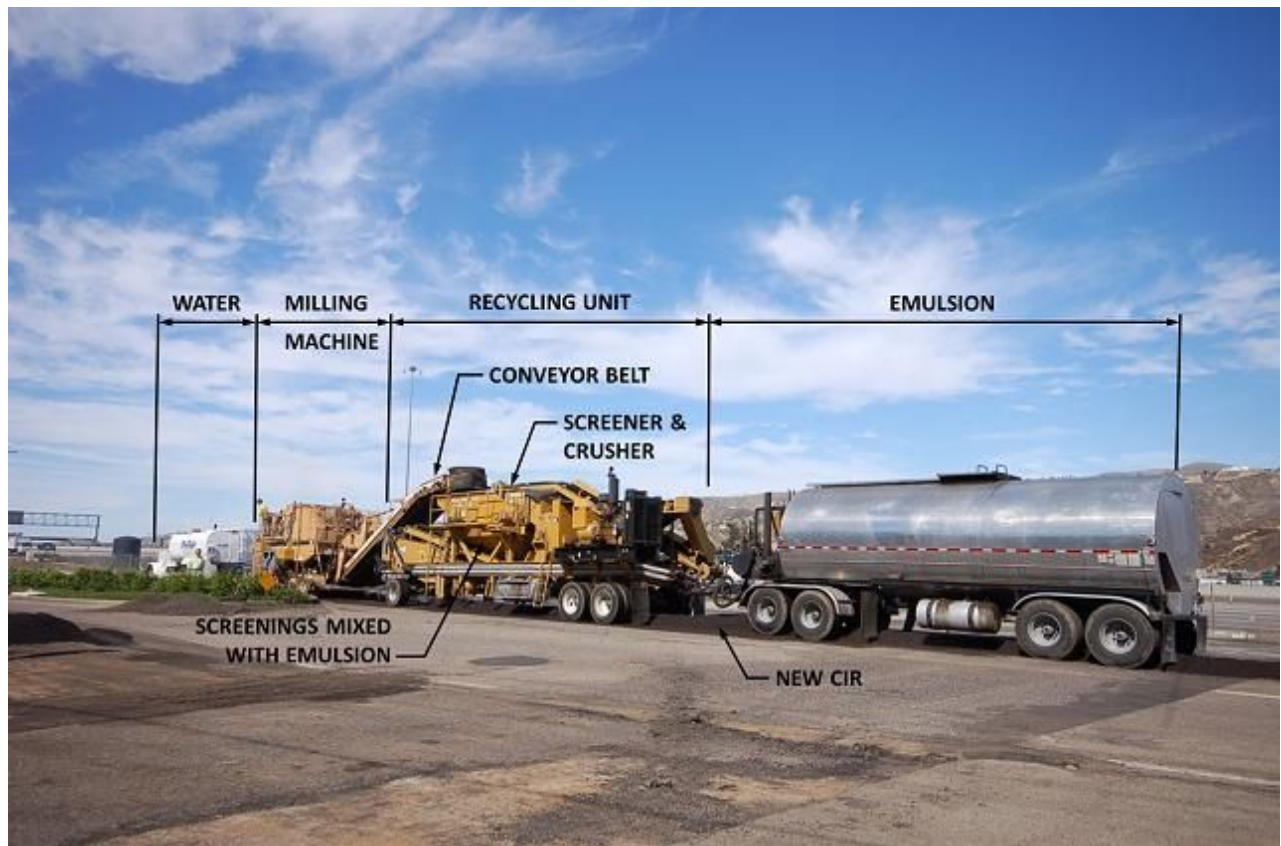


Figure 7-2. Cold In-place Recycling Train.

Section 3: Geosynthetics

3.1 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 geosynthetics in the upper pavement structure 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 asphalt (HMA). 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 HMA layers must avoid RAP contamination and possible fouling of milling equipment.

3.1.1. 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 HMA. 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.

3.1.2. 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 cannot 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 ensure laboratory testing is performed to determine HMA stripping susceptibility of existing mixes (highway cores) and the proposed level-up and overlay mixes.

3.2 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,
- drainage.

3.2.1 Reinforcement

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. Geogrid has also been used in multi-layered repair of roadway embankment slope failures, and is described in the USDA publication [0577 1204 – SDTDC, “Deep Patch Road Embankment Repair Application Guide,” October 2005](#) (see Figure 7-3).

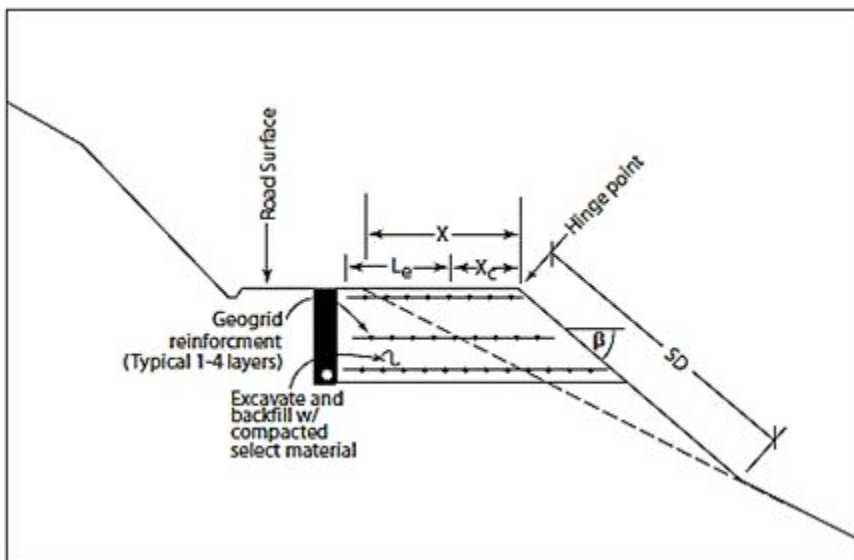


Figure 7-3. Repair of Embankment Failure using Geogrid.

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 Maintenance

Division, Pavement Asset Management Section, recommends that its application be viewed as an “insurance policy” rather than a “modulus- multiplier” or structure-reducing product.

3.2.2 Separation

Geosynthetics used for separation have classically been applied to prevent subgrade soil fines 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 retain moisture and significantly reduce the internal friction angle, rendering 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/subbase. 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.

3.2.3 Filtration

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.

3.2.4 Drainage

Geosynthetics used in pavement drainage have been limited to addressing problematic locations, typically in a reactive manner. Retrofitted pavement edge drains often used when the structure cross section changes transversely (e.g., rigid pavements with flexible pavement shoulders, widening using different structure) is an example of using geosynthetics to expedite lateral drainage of trapped moisture from within the pavement structure. Vertical moisture barriers using an impermeable membrane have been used to prevent moisture ingress through permeable seams from adjacent land into the roadway substructure.

Section 4: Flexible Base Overlay and Flexible Base Thickening

4.1 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 increasing 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.

4.2 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 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 stockpiling, 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 continuous flight auger 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](#), and [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

6.1 Structural Overlays

For flexible pavements, structural hot-mix overlay thicknesses are designed using FPS 21 design, option 6. Currently, the only department-approved rational method for design of structural HMA overlays on rigid pavements is by using the appropriate overlay option in DARWin® 3.1 (AASHTO 93). An M-E based program incorporating findings of the balanced mix design approach (research project 0-5123) is available in consultation with MNT – Pavement Asset Management. 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 ensure 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 4 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 total pavement acceptance device (TPAD) 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, cementitious, or high-density polymer 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 such as the crack attenuating mixture (CAM) may be useful in retarding reflective cracking when overlaying jointed concrete pavements. CAM 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 (2.0 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 ensure 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.

6.2 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 weatherproofing, and is often categorized as "pavement preservation." 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 ensure that a non-structural overlay is appropriate. This would also include evaluating the existing HMA 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 or by using the thin bonded friction course (Item 348) method. Due consideration must also be given to insuring a proper cross-slope by using a level-up course before applying the PFC. 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.

6.2.1 Thin Overlays

Two statewide thin overlay options are available by standard specification: The thin bonded friction courses (Item 348) and thin overlay mixtures (Item 347 [TOM]).

Item 348 covers PFC and wearing course varieties, both of which use a warm polymer-modified asphalt emulsion membrane followed immediately by the application of a hot plant mixed paving mixture. The bonded wearing course is placed in thicknesses from 1/2 to 3/4 in. The thin bonded PFC is placed in thicknesses of 3/4 to 1.5 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. Item 348 allows RAP/RAS and WMA, if desired. 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. The thin overlay mix (TOM, Item 347) uses conventional tacking procedures and allows for a WMA option. However, no RAP/RAS is allowed. The TOM-C is placed in thicknesses ranging from 0.75 to 1.25 in.; TOM-F is placed in thickness ranging from 0.50 to 0.75 in.

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. Use of two rollers in echelon may be necessary.
- 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

7.1 General

For purposes of discussion in this manual, these procedures will include treatments under items 315 (fog seal), 316 (seal coat), and 350 (microsurfacing). Scrub seals have been recently introduced to Texas and can be required by special specification. 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 pavement preservation 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. Scrub seals consist of one or more applications of an asphalt emulsion that is scrubbed with a broom and covered with a single layer of aggregate. The “scrubbing” action is designed to work the emulsion into surface cracks for a better seal. 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](#)

7.2 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. Item 316 is the governing specification. 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 an 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 controlled intersections or along open sections of highway. The process of applying a concrete overlay on an 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,
- thin whitetopping.

Conventional whitetopping will use a slab with a thickness designed by the Rigid Pavement Design procedure (Chapter 8) placed on top of the existing HMA surfaced pavement. Minimum thickness for this overlay option is 7.0 in.

Concrete inlays may be placed on thicker HMA pavements that have been partially milled out. Depending on the proposed slab thickness, requirements might follow either those of conventional whitetopping or thin whitetopping.

Thin whitetopping will use slabs from 4.0 to 7.0 in. thick that are placed on an HMA surface that has been milled or otherwise prepared to enhance the bond. Refer to Chapter 8, Section 8, “Thin Concrete Pavement Overlay (Thin Whitetopping),” for the thickness design, slab size selection, and existing HMA support requirement. Concrete overlays thinner than 4.0 in. are not 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.

Section 9: Reusing Rigid Pavements as Base

Acknowledgement: Content of this section is made available from the University of Washington, Pavement Tools Consortium.

Various procedures can be used to utilize 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 reusing 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 of pavement breakers 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.

9.1 Break and Seat

This process may be used to reduce the size of jointed reinforced concrete pavement (JRCP) to pieces on the order of about 1 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 as shown in Figure 7-4 and Figure 7-5. Broken PCC is then seated into the foundation using several passes of a 35 to 50 ton pneumatic roller.



Figure 7-4. Drop hammer breaking slab for break and seat operations. © Copyright 2006 University of Washington

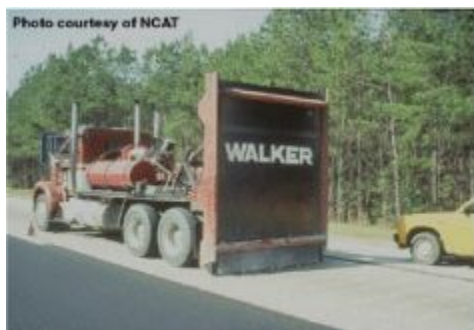


Figure 7-5. Drop hammer “breaking” slab for “break and seat.” © Copyright 2006 University of Washington

9.2 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.

9.3 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 transmitting reflective cracking through the HMA overlay. The fractured layer is typically compacted using a 10 ton vibratory roller. This is the preferred process for reducing JRPC and CRCP pavements. Rubblizing is typically done with one of the following two pieces of equipment:

9.3.1 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 as shown in Figure 7-6. 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 7-7 depicts broken pavement after using the resonant pavement breaker.



Figure 7-6. Resonant Pavement Breaker. © Copyright 2006 University of Washington



Figure 7-7. Rubblized pavement texture following resonant pavement breaking. © Copyright 2006 University of Washington

9.4 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 to 1,500 lbs. Hammers can be dropped from variable heights 1 to 5 ft. to create impact energies between 2,000 to 12,000 ft.-lbs. Hammers cycle at a rate of 30 to 35 impacts per minute. MHBs can rubblize an entire lane up to 13 ft. in a single pass (Antigo Construction, 2001). Figure 7-8 shows a typical multi-head breaker.



Figure 7-8. Multi-head Pavement Rubblizer. © Copyright 2006 University of Washington

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),
- thin bonded friction courses (PFC or wearing),
- reflective crack relief interlayer (RCRI).

10.1 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 surface. 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 1 in. of the existing top layer and replacing it with 2 in. of new overlay. RAP content in accordance with Item 340 should be allowed in the new 2-in. 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 1 in.), mixing a recycling agent. A new overlay is placed on top of the new recycled surface using standard paving operations.

10.2 Alternate Options to Thin Bonded Friction Course

The thin bonded friction course is used as a pavement preservation treatment to restore skid resistance, improve ride quality, and arrest surface oxidation.

The thin bonded friction course is an open-graded mix (PFC or denser wearing course) placed on polymer modified rapid-setting emulsion membrane. Placement of a thin bonded PFC surface is intended to provide increased safety and better ride quality on a high speed roadway. These mixtures are placed before the emulsion breaks using a specialty paver. Note: Membrane rate must meet the requirements of Item 348.

The thin bonded friction course may also be used when there are restrictions on overlay thickness due to clearance requirements.

The alternate option to this process is placing a tack coat and thin overlay using a mixture such as a Thin Overlay Mix (TOM, Item 347), dense-graded TY F mix (Item 341), or an underseal (seal coat) plus a 3/4-in. thick PFC (Item 342). These options require two separate operations/pieces of equipment.

10.3 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 test. The mix is typically placed 1-in. 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

Contents:

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[Section 2: Approved Design Method](#)

[Section 3: Rigid Pavement Design Process for CRCP](#)

[Section 4: Rigid Pavement Design Process for CPCD](#)

[Section 5: Determining Concrete Pavement Thickness](#)

[Section 6: Terminal Anchor Joint Selection for Concrete Pavement](#)

[Section 7: Bonded and Unbonded Concrete Overlays](#)

[Section 8: Thin Concrete Pavement Overlay \(Thin Whitetopping\)](#)

Section 1: Overview

1.1 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 drying shrinkage of the concrete, environment changes, and traffic loads. Forces due to shrinkage and environment changes are large enough in the concrete pavement to cause cracks to form without any traffic loading.

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 continuously reinforced concrete pavement (CRCP) and concrete pavement contraction design (CPCD), which is also called jointed concrete pavement (JCP).

1.1.1 Continuously Reinforced Concrete Pavement (CRCP)

CRCP contains both longitudinal and transverse steel. CRCP does not contain transverse joints except at construction joints. Figure 8-1 shows the typical reinforcing steel layout for CRCP.

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 for CRCP](#). Steel reinforcement and other design details are governed by the CRCP standards. The latest CRCP standards can be obtained from the department’s internet website at the following address: <http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/rdwylse.htm>.

Some districts have district-wide CRCP standards.

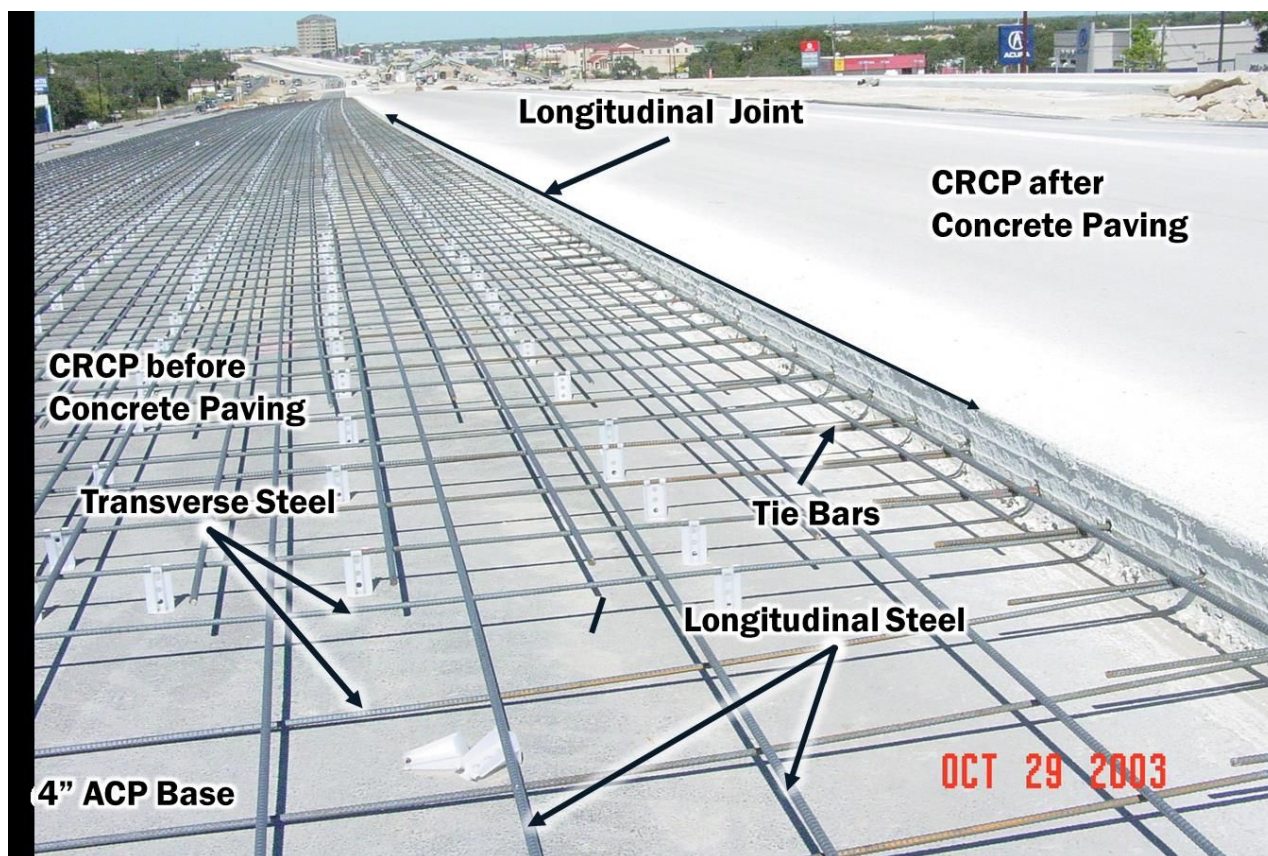


Figure 8-1. Continuously Reinforced Concrete Pavement.

1.1.2 Concrete Pavement Contraction Design (CPCD)

CPCD 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. intervals. Longitudinal joints are used to control random longitudinal cracking. Longitudinal joints are tied together with tie bars. Figure 8-2 shows the typical layout for CPCD.

The thickness design of CPCD is detailed in Section 2, [Approved Design Method](#), and Section 4, [Rigid Pavement Design Process for CPCD](#). Other CPCD details are governed by the concrete pavement contraction design (CPCD) standards that can be obtained from the department's internet website at the following address: <http://www.dot.state.tx.us/insdot/orgchart/cmd/cserve/standard/rdwylse.htm>.



Figure 8-2. Concrete Pavement Contraction Design.

1.2 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. Although CRCP is the preferred concrete pavement type, the department has expanded the applications where CPCD can be used as an acceptable alternative to CRCP. The following criteria list the applications where CPCD can be used instead of CRCP at the discretion of the district engineer:

- For roadways with design traffic of 40 million ESALs or less,
- For frontage roads where CRCP is difficult to construct due to numerous leave-out sections,
- For roadways controlled and maintained by another government entity,
- For parking areas or roadways with crosswalks, adjacent parking, or sidewalks,
- For railroad crossings, approaches to structures, or to widen existing jointed pavement, or
- 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.

Table 8-1 recommends applicable concrete pavement types for particular situations.

Table 8-1: Selection of Rigid Pavement Type

Factors	Where/When	CPCD	CRCP
Traffic Level	> 40 million ESALS		√√√
	10 - 40 million ESALS	√√	√√√
	< 10 million ESALS	√√√	√
Constructability Under Traffic	Frontage Road with Numerous Leave-outs	√√√	
	Intersection/Crossings	√√√	
Material Sources	Use Local Coarse Aggregates with High COTE for Traffic Levels with ≤ 40 million ESALS	√√√	
Support Condition	Natural Subgrade with Higher Risk for Heaves		√√√
	On Embankment with Higher Risk for Voids under Slab		√√√

Consult the Maintenance Division, Pavement Asset Management Section, staff when considering CPCD pavements for situations not covered by the above criteria.

Rigid pavement types other than CRCP and CPCD may be appropriate for a wide variety of situations. Refer to Section 7, “Bonded and Unbonded Concrete Overlays,” and Section 8, “Thin Concrete Pavement Overlay (Thin Whitetopping),” for a description of other rigid pavement applications.

1.3 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.

1.4 Tied Concrete Shoulders

There is substantial evidence that tied Portland cement concrete (PCC) shoulders improve PCC pavement performance significantly. Therefore, use tied PCC shoulders. If it is not feasible to provide full-width tied PCC shoulders, use a minimum 2-ft. widened outside lane.

PCC shoulders should be tied to the main lane pavement by tie bars or by the main lane's transverse steel. The PCC shoulder must 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.

Section 2: Approved Design Method

2.1 Introduction

The TxCRCP-ME design program and the AASHTO DARWin® 3.1 program are available to department personnel through the district pavement engineer. Consultants may obtain the TxCRCP-ME program from the district pavement engineer or the Pavement Asset Management Section of the Maintenance Division.

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 MNT – Pavement Asset Management for further assistance.

2.2 TxCRCP-ME Design Program for CRCP

The design method was developed under department research project 0-5832, “Develop Mechanistic/Empirical Design for CRCP.” An elaborate three-dimensional finite element analysis was conducted to identify the mechanisms of punchout distress in CRCP, and the critical component that may cause the punchout distress was mechanistically evaluated. A full factorial parametric study was performed for significant input variables to compile the database of the analysis results. A program was written using Microsoft Excel to perform the analysis of the pavement system for given inputs in estimating the frequency of punchouts, the primary structural distress in CRCP. The conversion from mechanistic structural responses to pavement distress is achieved by a transfer function determined empirically, utilizing data collected from the department’s rigid pavement database project. The final results of the software are presented in the form of charts and tables.

2.3 AASHTO Rigid Pavement Design Procedure for CPCD

The 1993 AASHTO *Guide for Design of Pavement Structures* is the only approved design method for CPCD projects. This design procedure is available in automated forms, the AASHTO DARWin® 3.1 program, and at <http://www.pavementinteractive.org/1993-aashto-rigid-pavement-structural-design-application/>.

Section 3: Rigid Pavement Design Process for CRCP

3.1 TxCRCP-ME Design Program

CRCP design consists of two elements: slab thickness design and steel reinforcement design. The first national CRCP design procedures for slab thickness were developed with information from the AASHO Road Test and were included in the 1972 AASHTO *Interim Guide for Design of Pavement Structures* (AASHTO, 1981). However, the AASHO Road Test only included jointed concrete pavement sections and not CRCP sections. Distresses in jointed concrete pavements (CPCD) are quite different from the distresses observed in modern CRCP. In CRCP, transverse cracking is normal behavior and does not contribute to degradation of serviceability. The behavior of CPCD and CRCP and their effect on pavement performance is quite different from each other, so the use of the AASHO Road Test data for the development of CRCP design procedures is not rational. In some sense, state DOTs reverse-engineered the AASHTO design equations for CRCP design by selecting reasonable values for selected input variables. In 1986 and 1993, extensive revisions were made to the 1972 Interim Guide, and newer versions of the design guides were published. However, very little effort was made to improve the CRCP design portion, except that steel design equations were incorporated.

The department has used the AASHTO 93 Guide for the design of CRCP, and it has served the department well for the design of CRCP, despite its limitations. In March 2004, the [NCHRP 1-37](#) report and the mechanistic-empirical pavement design guide software (MEPDG) were released. In 2005, the department initiated [Research Project 0-4714-1](#) to evaluate the MEPDG for potential implementation. The study recommended, for various reasons, not to implement the MEPDG as a replacement for the design methods used at that time.

In 2007, the department initiated a research study, [0-5832](#), to develop its own mechanistic-empirical CRCP design procedures that would model the performance of the department's typical concrete pavement structure and performance. Three-dimensional analysis was conducted for in-depth analysis of mechanistic behavior of CRCP, including the interactions between longitudinal steel and concrete. The project produced a simple Microsoft Excel spreadsheet to perform the design.

The following Figure 8-3 summarizes the CRCP design process.

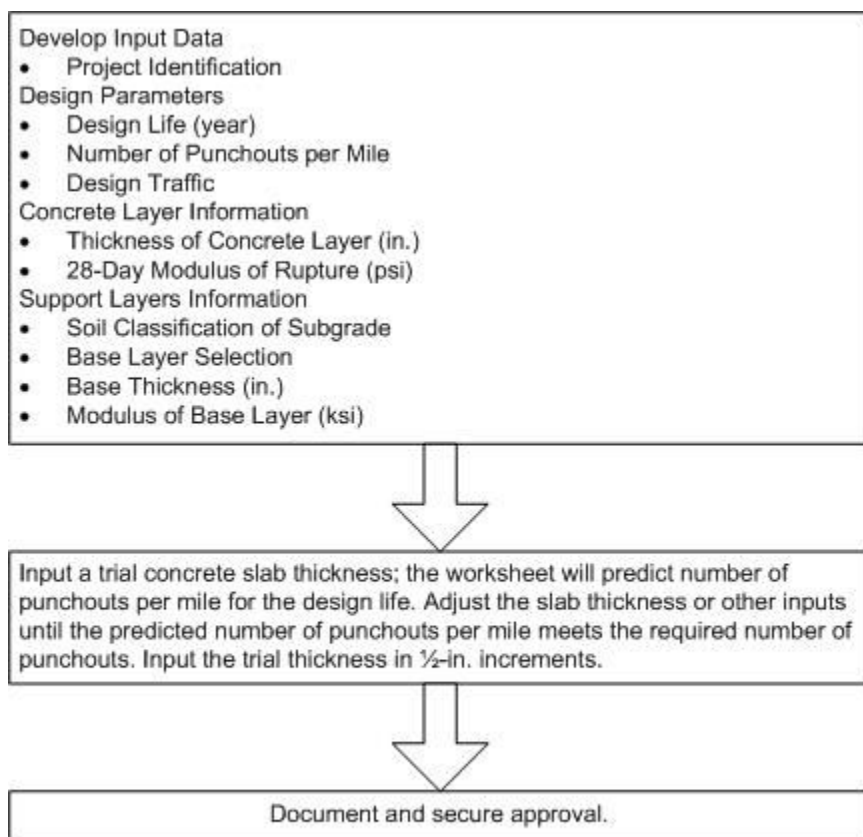


Figure 8-3. CRCP Pavement Design Process.

3.2 TxCRCP-ME Design Input Values

The following input variables are needed for the TxCRCP-ME pavement design procedure:

3.2.1 Project Identification

The district is the only required input in this section. By selecting the district, the program will determine the environmental conditions for this pavement design. All other information is not required to complete the thickness design, but they are required for the final design submittal.

3.2.2 Design Life (year)

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.

3.2.3 Number of Punchouts per Mile

Provide a number of punchouts per mile that is considered the terminal condition of CRCP you are designing. Traditionally, 10 per mile has been the number used for CRCP design. For a higher class

of highway where the number of punchouts may be minimized, contact MNT – Pavement Asset Management for further assistance.

3.2.4 Design Traffic

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 Division (TPP). This report is requested during the design phase of a project and, upon receipt, should be evaluated for reasonableness.

Input the one-way total 18-kip ESALs from the TPP traffic analysis report into the design worksheet. The worksheet will calculate the design lane ESALs based on inputted number of lanes in one direction.

Local conditions may cause the directional distribution of heavy vehicles to be unequal. An example is 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.

3.2.5 Thickness of Concrete Layer (in.)

Input a trial concrete slab thickness; the worksheet will predict number of punchouts per mile for the design life. Adjust the slab thickness or other inputs until the predicted number of punchouts per mile meets the requirement in “B. Design Parameters.” Input the trial thickness in 1/2-in. increments.

3.2.6 28-Day Modulus of Rupture (psi)

The Modulus of Rupture (Mr) of concrete is a measure of the flexural strength of the concrete as determined by breaking concrete beam test specimens. Use a 28-day Mr of 570 psi. If the engineer selects an alternate value for Mr, it must be documented with an explanation. Also, if a higher Mr is used, it should be required in the plan to use a higher concrete strength than what is required in Item 360.

3.2.7 Soil Classification of Subgrade

Select the soil classification of subgrade from Unified Soil Classification system in Table 8-2. Selecting the appropriate soil classification will assist in determining the composite k-value of the support layers. The soil k values used in the TxCRCP-ME program are used for thickness design purposes only and shall not be used for field verification of subgrade design during construction.

Table 8-2: Subgrade Soil Classification

Soil Description		Soil Classification System
		USCS
Coarse Grained Soils	Gravel	GW or GP
	Coarse Sand	SW

Table 8-2: Subgrade Soil Classification

Soil Description		Soil Classification System
		USCS
	Fine Sand	SP
Granular Materials with High Fines	Silty Gravel	GM
	Silty Sandy Gravel	
	Silty Sand	SM
	Silty Gravelly Sand	
	Clayey Gravel	GC
	Clayey Sandy Gravel	
	Clayey Sand	SC
	Clayey Gravelly Sand	
Fine Grained Soils	Silt	ML or OL
	Silt / Sand / Gravel Mix	
	Poorly Graded Silt	MH
	Plastic Clay	CL
	Moderately Plastic Elastic Clay	CL or OL
	Highly Plastic Elastic Clay	CH or OH

3.2.8 Base Layer Requirements

Select ATB for asphalt treated base, HMA for hot-mix asphalt, or CTB for cement treated base.

Field performance evaluations of concrete pavement have revealed that the use of a durable, stabilized, and non-erodable 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.

The department recognized this and requires one of the following base layer combinations for concrete slab support:

- 4 in. of hot-mix asphalt (HMA) or asphalt treated base (ATB), or
- a minimum 1 in. hot-mix asphalt bond breaker over 6 in. of a cement treated base; use Item 276, Class L.

Approval from the Pavement Analysis & Design Branch of MNT – Pavement Asset Management is required for use of base layers other than those listed above.

Selection of CTB Class

To ensure long-term strength and stability of cement treated layers, sufficient cement must be used in the mixture. Item 276, "Cement Treatment (Plant-Mixed)," currently designates three classes of cement treated flexible base, based on 7-day unconfined compressive strength. Class M is intended for use with flexible pavements. Class L is intended for use with rigid pavements. Class N can be used if the district has successful long-term experience with other strengths.

Use of Bond Breaker

A bond breaker should always be used between concrete pavement and cement treated base. There have been several instances across Texas where excessive cracking and premature failures occurred when a concrete slab was placed directly on cement treated base. These problems occur because concrete slabs tend to bond directly to cement treated 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.

The department recommends a minimum of 1 in. asphalt concrete stress-relieving layer be used between cement treated base and the concrete slab. A polyethylene sheet is not recommended for use as a bond breaker, due to construction problems evident from past 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 the Geotechnical, Soils and Aggregates Branch of CST for further assistance.

The subgrade/base must 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 the Geotechnical, Soils and Aggregates Branch of CST personnel is required. Refer to Chapter 2, Section 7, for an example of a typical drainable base system.

3.2.8.1 Base Thickness (in.)

Input the proposed base layer thickness. For cement treated bases, ignore the 1-in. thick bond breaker.

3.2.8.2 Modulus of Base Layer (ksi)

Input the modulus of elasticity of the base layer. Typical values of ATB vary from 100 ksi to 400 ksi. Use a value of 400 ksi for HMA and ATB.

The modulus of elasticity for cement treated bases (CTB) varies from 100 ksi to 700 ksi. Use a modulus of 500 ksi for cement treated bases.

Composite k-value will be calculated by the TxCRCP-ME design program based on the inputs of thickness of the stabilized base, elastic modulus of stabilized base, and subgrade soil classification.

Research project 0-5832 developed the composite k-value table by the following process:

1. Concrete stresses due to wheel loading were estimated by two-dimensional Finite Element Model analysis for a wide range of soil and base conditions. In the modeling, soil stiffness was characterized by modulus of subgrade reaction (k) and that of base by modulus of elasticity.
2. A factorial was developed for various subgrade k and base modulus of elasticity and thickness. Concrete stress was estimated for each cell of the 106 factorial (specific combination of subgrade k and base modulus).
3. For each cell, the “equivalent” k value was derived from FEM analysis that would provide the same concrete stresses.
4. A table, called “k-Table” was developed and included in the TxCRCP-ME program.

For a factorial mentioned in step 2 above, the following levels were selected:

1. Subgrade k: 7 levels (25, 50, 100, 150, 200, 250, and 300 psi/in.).
2. Base thickness: 5 levels (2 in. – 6 in. with 1 in. increment).
3. Base modulus: 34 levels (50 ksi to 100 ksi with 10 ksi increment, 100 ksi to 1,000 ksi with 50 ksi increment, 1,000 ksi to 2,000 ksi with 100 ksi increment).

A total of 1,190 combinations were analyzed, and the k-Table was developed.

Section 4: Rigid Pavement Design Process for CPCD

4.1 Introduction

The following Figure 8-4 summarizes the CPCD design process.

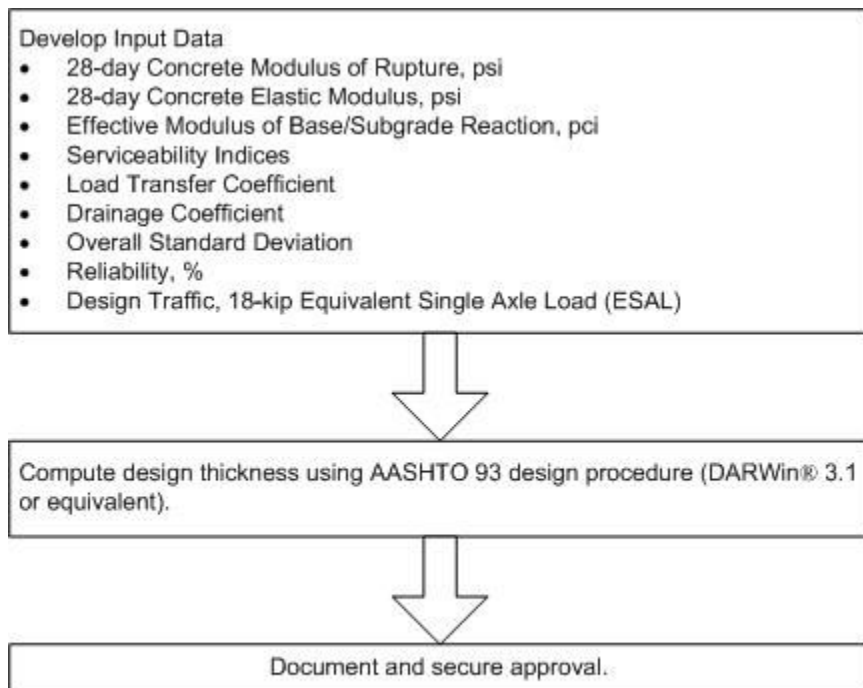


Figure 8-4. CPCD Pavement Design Process.

4.2 Input Values

The following input variables are needed for the AASHTO rigid pavement design procedure.

4.2.1 28-day Concrete Modulus of Rupture, Mr

The Mr of concrete is a measure of the flexural strength of the concrete as determined by breaking concrete beam test specimens. An Mr 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 Mr, it must be documented with an explanation.

4.2.2 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.

4.2.3 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.

A k-value of 300 psi/in. should be used in the CPCD design procedure with one of the stabilized base layer combinations listed below.

Field performance evaluations of concrete pavement have revealed that the use of a durable, stabilized, and non-erodable 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.

The department recognized this and requires one of the following base layer combinations for concrete slab support:

- 4 in. of hot-mix asphalt (HMA) or asphalt treated base (ATB), or
- a minimum 1 in. hot-mix asphalt bond breaker over 6 in. of a cement treated base. Use Item 276, Class L.

Approval from the Pavement Analysis & Design Branch of MNT – Pavement Asset Management Section is required for use of base layers other than those listed above.

Selection of CTB Class

To ensure long-term strength and stability of cement treated layers, sufficient cement must be used in the mixture. Item 276, “Cement Treatment (Plant-Mixed),” currently designates three classes of cement treated flexible base, based on 7-day unconfined compressive strength. Class M is intended for use with flexible pavements. Class L is intended for use with rigid pavements. Class N can be used if the district has successful long-term experience with other strengths.

Use of Bond Breaker

A bond breaker should always be used between concrete pavement and cement treated base. There have been several instances across Texas where excessive cracking and premature failures occurred when a concrete slab was placed directly on cement treated base. These problems occur because concrete slabs tend to bond directly to cement treated 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 treated base and the concrete slab. A polyethylene sheet is not recommended for use as a bond breaker, due to construction problems evident from past 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 the Geotechnical, Soils and Aggregates Branch of CST for further assistance.

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 Geotechnical, Soils and Aggregates Branch of CST personnel is required. Refer to Chapter 2, Section 7, for an example of a typical drainable base system.

4.2.4 Serviceability Indices

For concrete pavement design, the difference between the initial and terminal serviceability is an important design consideration. 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.

4.2.5 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 AASHTO Road Test.

The department requires the use of tied concrete shoulders, tied curb and gutter, or a widened lane, and the use of load transfer devices, so the load transfer coefficient to be used for CPCD design is 2.9.

4.2.6 Drainage Coefficient

The drainage coefficient characterizes the quality of drainage of the base 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.

The department has not had much experience with positive drainage systems. As stated in Chapter 2, the department's philosophy on this issue is to prevent water intrusion and pumping by using bases that are dense-graded, non-erosive, and stabilized. The department has had good performance with such bases, and it is believed that the bases recommended

earlier in this section provide performance equivalent to the AASHTO '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 8-3 Drainage Coefficients" shows the recommended drainage coefficients for Texas. The coefficients are selected based on the annual rainfall in the project area.

Table 8-3: 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.	

4.2.7 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 CPCD design.

4.2.8 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.

4.2.9 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 TPP. This report is

requested during the design phase of a project and, upon receipt, should be evaluated for reasonableness using guidelines discussed in Chapter 2, Section 8.

The predicted 18-kip ESALs is multiplied by a lane distribution factor (LDF). Table 8-4 lists the lane distribution factors. 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 8-4: 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	

Section 5: Determining Concrete Pavement Thickness

5.1 Introduction

For CRCP design, the required pavement thickness is the thickness in which the predicted number of punchouts per mile is less than the design requirement. For CRCP designs, the input thickness should be in 1/2-in. increments. The minimum thickness for CRCP is 7 in., and the maximum thickness is 13 in. Use 13 in. for slab thicknesses greater than 13 in. Districts wanting to use thicker pavements should submit designs greater than 13 in. to the district engineer for approval along with their justification for doing so.

For CPCD design, the computed concrete slab thickness should be rounded 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 of 10.24 in. The minimum slab thickness for CPCD is 6 in., and the maximum thickness is 12 in. It is recommended to use CRCP for sections where the CPCD design thickness is greater than 12 in.

Section 6: Terminal Anchor Joint Selection for Concrete Pavement

6.1 Terminal Joint Design Recommendation

Concrete experiences volume changes due to temperature and moisture variations. Normally, concrete pavement is abutted to bridge structures via approach slabs. To prevent damage to the bridge structures, some form of system is installed at the bridge and pavement interface. There were three systems in use at the department: expansion joint system, wide-flange system, and 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.

The 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.

An anchor lug system tries to contain the concrete movement at the interface between bridge and pavement by providing five anchor lugs.

The department has investigated the movements of CRCP near bridges and the effectiveness of these three terminal systems in Research project 0-6326, "Rational Use of Terminal Anchorages in Portland Cement Concrete Pavements."

The research data has shown that the anchor lug system is not effective. The stresses generated in soil due to slab expansion at lug walls are large enough to result in permanent deformations in soils. The soil does not retract with the lug when the pavement contracts. The permanent deformations result in voids between soil and lug walls.

Field measurements have indicated that the base friction restrains slab movements effectively. Using coarse-textured base such as HMA might be the most effective tool to control slab movements. Simple expansion joint systems or wide-flange systems are effective in accommodating slab movements. Expansion joint systems should cost less than wide-flange systems and attain comparable performance.

The use of anchor lug systems is no longer allowed, and the standard TA(CP)-99 has been deleted. The transverse expansion joint details at bridge approaches are shown in the concrete pavement standards.

Districts may develop a Special Specification to use the wide-flange system.

Section 7: Bonded and Unbonded Concrete Overlays

7.1 Introduction

Bonded concrete overlay (BCO) consists of a 2-in. to 8-in. thick concrete layer placed on top of the existing concrete pavement with operations conducted to ensure full bond between new and old concrete layers. A BCO with a minimum thickness of 4 in. is one of the most cost-effective ways of enhancing structural capacity of under-designed pavements by reducing deflections and extending service life. A BCO with a thickness of less than 4 in. is typically used to restore pavement surface characteristics, such as ride and friction.

The department maintains many miles of thin PCC pavement that have exceeded their design traffic projection and are still in reasonably good condition. The use of BCO is based on the fundamental design assumption that 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 as discussed in Chapter 10.

Bonded concrete overlays over jointed concrete pavements are difficult to construct because all joints must be matched. CRCP-bonded concrete overlays have been constructed and have performed successfully in several districts but have not been used widely throughout the state. Districts considering a bonded concrete overlay can contact MNT – Pavement Asset Management, Pavement Analysis & Design Branch, for assistance.

Unbonded concrete overlay consists of a concrete layer (5 in. or greater) on top of an existing concrete with a HMA interlayer 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 because a reduced amount of repairs were made to the existing pavement prior to constructing the unbonded concrete overlay.

Unbonded CRCP concrete overlays may be used over CRCP, jointed concrete pavement (CPCD), 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.

7.2 AASHTO Overlay Thickness Design for Bonded Overlays

The equation below is used to calculate the overlay thickness to increase structural capacity to carry future traffic. The designer can also use the DARWin® 3.1 program and select the “Overlay Design” Module and “Bonded PCC Overlay of PCC Pavement.”

$$D_{ol} = D_f - D_{eff}$$

Where:

D_{ol} = required slab thickness of overlay, in.

D_f = slab thickness to carry future traffic, in.

D_{eff} = thickness of existing slab, in.

The slab thickness, D_f , is determined for the design traffic as if it were built as a new pavement on the prepared base. Use the design procedure in Sections 3 or 4 for the pertinent pavement type. Some existing pavements might not have stabilized bases, and the k-value should be evaluated using FWD or DCP.

Determination of existing Effective Slab Thickness by Condition Survey Method

$$D_{eff} = F_{jc} * F_{dur} * F_{fat} * D$$

Where:

F_{jc} = joints and cracks adjustment factor

F_{dur} = durability adjustment factor

F_{fat} = fatigue adjustment factor

D = thickness of existing slab, in.

For BCO design, the condition of the existing pavement is one of the most important factors. If the pavement condition 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 12 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 transverse cracks may appear to be deteriorated, quite often they are structurally 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, along the entire length of the transverse crack and across the entire lane width, should be counted as “deteriorated transverse cracks.”

The number of patches per mile and evidence of pumping should be recorded. In Texas' old concrete pavements, typically, the base was not stabilized and pumping resulted. Durability related problems, such as D-cracking and ASR cracking, should be noted and their severity recorded. Overall, it has been quite rare to observe durability-related problems in concrete pavement in Texas.

7.2.1 Joints and Cracks Adjustment Factor, F_{jc}

The Joints and Cracks Adjustment Factor, F_{jc} , 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 cracks and any other major discontinuities in the existing pavement prior to overlay. The target F_{jc} 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 F_{jc} from Figure 8-5.

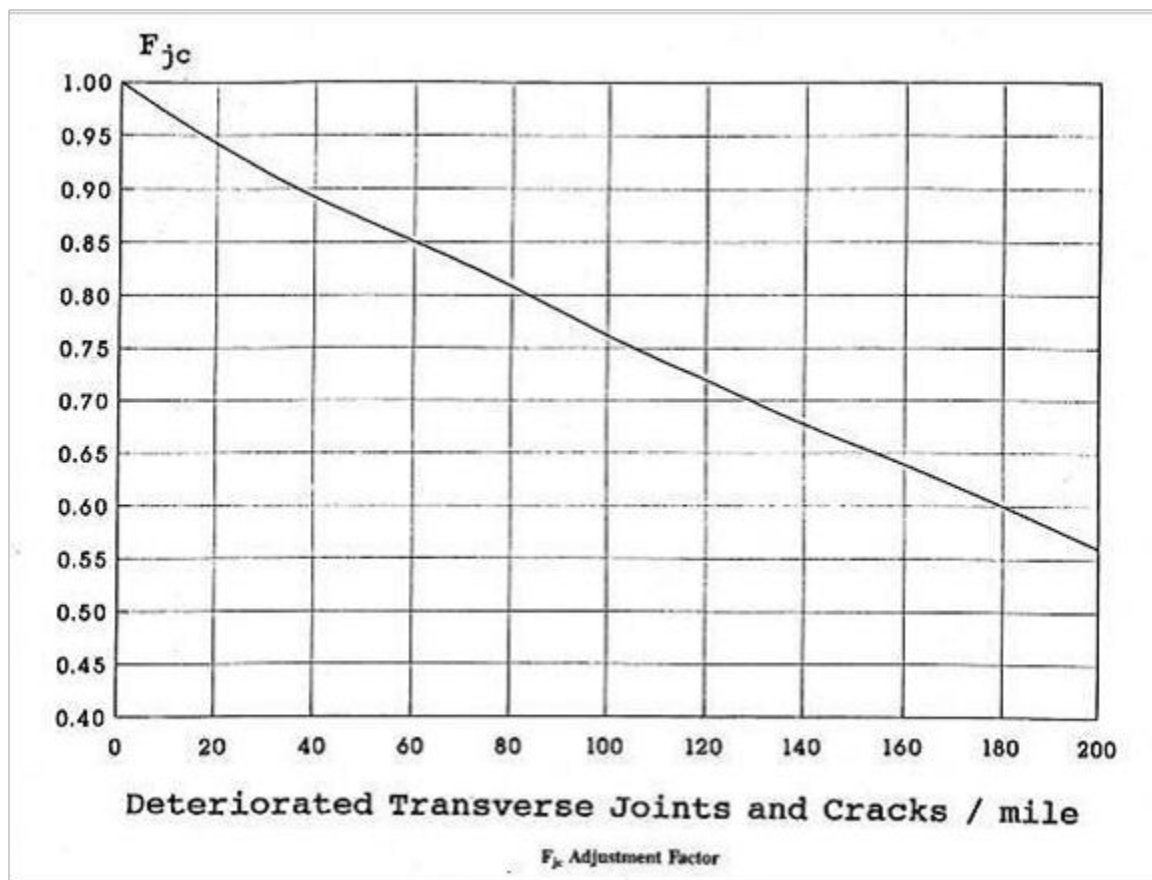


Figure 8-5. Joints and Cracks Adjustment Factor for Bonded Overlays, F_{jc} . Source: AASHTO Guide for Design of Pavement Structures (1993)

7.2.2 Durability Adjustment Factor, F_{dur}

The Durability Adjustment Factor, F_{dur} , 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. F_{dur} is determined using historical records and condition survey data. Table 8-5 shows the durability adjustment factor.

Table 8-5: Durability Adjustment Factor

Durability Adjustment Factor, F_{dur}	Historical Records and Condition Survey Data
1.00	No evidence or history of PCC durability problems.
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).

7.2.3 Fatigue Damage Adjustment Factor, F_{fat}

The Fatigue Damage Adjustment Factor, F_{fat} , 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. Table 8-6 shows the fatigue damage adjustment factor.

Table 8-6: Fatigue Damage Adjustment Factor

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.

7.2.4 Reinforcement Design

Reinforcement 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 recommended. For thinner overlays, fibers have been successfully used.

The design engineer will determine the reinforcement bar size and number of longitudinal steel, tie bars, and transverse steel. The performance of CRCP depends on the percentage of longitudinal steel. Failing to place longitudinal steel in a BCO 4 in. or thicker will effectively reduce the percentage of longitudinal steel in the combined slab, which could increase steel

stresses and make transverse crack widths larger. See the recommended reinforcing steel percentage and vertical location in Table 8-7.

Table 8-7: Reinforcement Requirements

BCO Thickness	Longitudinal Steel (%)	Vertical Location	Fibers
≤ 3 in.	N/A	N/A	Yes
≤ 5 in.	0.6%	Bottom of BCO	No
> 5 in.	0.6%	Middle of BCO	No

7.3 AASHTO Overlay Thickness Design for Unbonded Concrete Overlays

The 1993 AASHTO Guide for Design of Pavement Structures is recommended for unbonded concrete overlay thickness design. The designer can use the AASHTO DARWin® 3.1 program and select the “Overlay Design” Module and “Unbonded PCC Overlay of PCC Pavement.”

7.3.1 Overlay Thickness Design

$$D_{ol} = (D_f^2 - D_{eff}^2)^{1/2}$$

Where:

D_{ol} - required slab thickness of overlay, in.

D_f = slab thickness to carry future traffic, in.

D_{eff} = effective thickness of existing slab, in.

Determination of Effective Slab Thickness by Condition Survey Method

$$D_{eff} = F_{jcu} * D$$

Where:

D = existing slab thickness, in. (Use 10 in. when existing $D > 10$ in.)

F_{jcu} = joints and cracks adjustment factor

F_{jcu} 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 F_{jcu} from Figure 8-6.

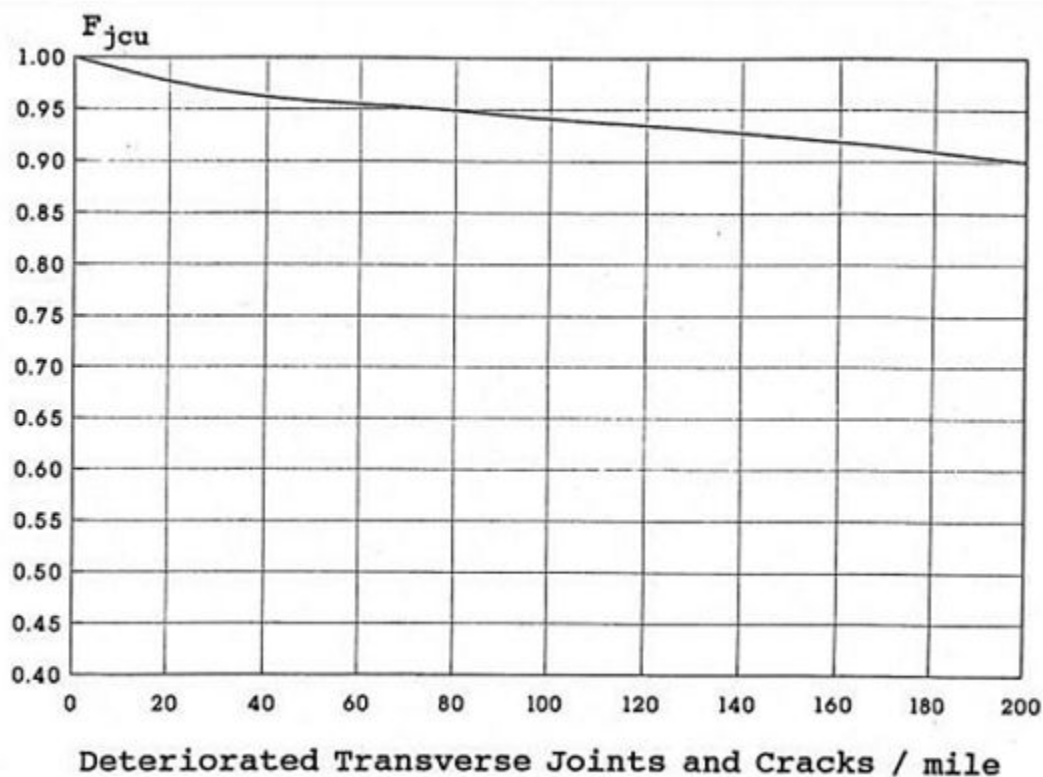


Figure 8-6. Joints and Cracks Adjustment Factor for Unbonded Overlays, F_{jcu} Source: AASHTO Guide for Design of Pavement Structures (1993)

7.3.2 Reinforcing Steel Design

The reinforcing steel placements are the same as new CRCP when unbonded overlay is 7 in. or thicker. When unbonded overlay is less than 7 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, tie bars, and transverse bars, and consult with the Pavement Analysis & Design Branch of MNT – Pavement Asset Management.

More information about bonded and unbonded concrete overlays is detailed in “Bonded Concrete Overlay” and “Unbonded Concrete Overlay” in Chapter 10, “Rigid Pavement Rehabilitation.”

Section 8: Thin Concrete Pavement Overlay (Thin Whitetopping)

8.1 Introduction

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 (see Figure 8-7). 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 areas. 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 reduce time and delays accompanying the frequent maintenance of an asphalt surface.



Figure 8-7. Whitetopping.

8.2 Guidelines for Thin Whitetopping (TWT)

The following guidelines provide the recommendations for TWT design thickness, joint spacing, 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, and thinner slabs require shorter joint spacing, often encroaching on the wheel path.
- Designs for > 7 in. TWT were not included because our standard concrete pavement designs should be used along with load transfer devices.
- Contraction joint spacing is set at 6-ft. intervals 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 $\geq 1/2$ 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 end sections.
- At bridge approaches, an approach slab is recommended with an expansion joint as shown in existing standards.
- The department has yet to establish when expansion joints are needed for wide pavements.
- The use of steel fibers is not recommended at locations where de-icing salts may be used.
- Apply governing Special Specification, Thin Whitetopping (Concrete Overlay).
- Use Thin Whitetopping Details, TWT-04 Standard Sheet.
- Use Table 8-8 as a general guide for determining proper TWT thickness. More research will be conducted to improve the design procedure.

Table 8-8: Thin Whitetopping Thickness (in.) Design

Trucks per Day per Lane	Design Life (yr.)					
	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

Table 8-8: Thin Whitetopping Thickness (in.) Design

Trucks per Day per Lane	Design Life (yr.)					
	5	6	7	8	9	10
350	4	4	5	5	5	5
400	4	5	5	5	5	5
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 TPP.

$$\text{Trucks per day per lane} = \text{ADT} \times \text{PDD} \times \text{PT} \times \text{LDF}$$

Where:

- ADT = average daily traffic for the first year of design period
- PDD = percent directional distribution of traffic (assume 50% split, unless observation proves otherwise)
- 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 = $13,500 \times 0.63 \times 0.03 \times 1.0 = 255$
- 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

1.1 Introduction

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 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 slabs. For subgrade construction, base construction, and surface preparation, refer to Chapter 6, [Flexible Pavement Construction](#).

Section 2: Concrete Mix Design

2.1 Introduction

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 concrete properties are strength, air content, slump, and 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 on mix design, production considerations, and testing requirements. 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.

2.2 Job Control Testing

In addition to the required mix design strength, the job control strength shall 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.

2.3 Opening to Traffic

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.

2.4 Maturity Method

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

3.1 Introduction

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. A batch of concrete is the amount of material mixed at one time. 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.

Common methods of mixing concrete are:

- central-mixed,
- shrink-mixed,
- truck-mixed,
- volumetric mixer-mixed, or
- hand-mixed.

Volumetric mixer-mixed 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.

3.2 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-mixed plants are sometimes referred to as wet batch plants. Figure 9-1 shows a diagram of 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.



Figure 9-1. Central-mixing plant.

In this central-mixing plant, aggregates (coarse and fine) are fed by conveyer belts from 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 content of the coarse and fine aggregate and proper batch water content adjustment is critical to the production of consistent concrete. In normal operations, moisture contents of fine aggregates are evaluated using moisture probes located in the weigh bins. Figure 9-3 shows a typical moisture probe used for fine aggregate.



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 amp meter reading is an indirect indicator of the consistency, which is derived from the correlation between consistency and electrical 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.

Mix times are dependent on rated capacity and speed of rotation of the mixer. Concrete should be mixed until a uniform mass is attained. Mixer performance can be tested in accordance with "Tex-472-A, Uniformity of Concrete," to indicate the mix uniformity.

Central-mixed concrete is transported in non-agitating trucks, truck mixers, or agitating trucks. In most of TxDOT's concrete paving 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 over 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 (see 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.

3.3 Shrink-mixed Concrete

Shrink-mixed concrete is used to increase the truck's load capacity and retain the advantages of truck-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.

3.4 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.

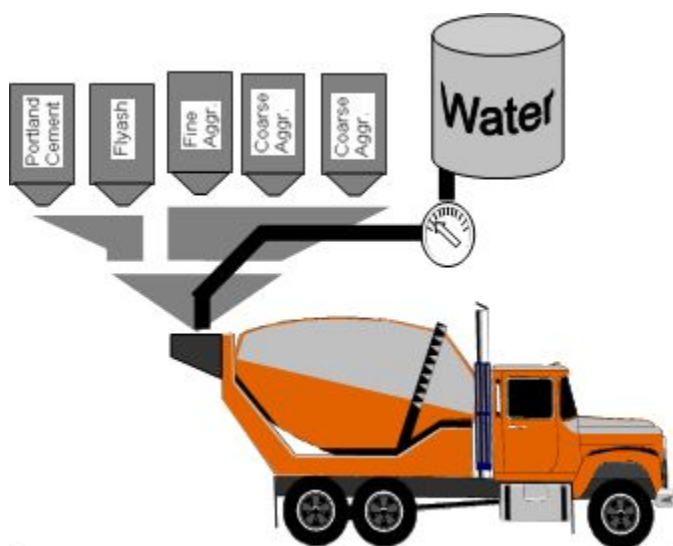


Figure 9-10. Truck-mixed or dry batching of concrete.

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 batch plant site. When the mix is correct at the plant, it will most likely be correct when it arrives at the job site.

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

4.1 Introduction

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.

4.2 Concrete Mixing 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 delivery.

4.3 Concrete Delivery Time

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.

4.4 Water Additions

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.

4.5 Wash Water

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: Reinforcing Steel Placement

5.1 Introduction

In this section, only the placement of longitudinal and transverse rebar is described. The placement of other rebar, such as tie bars and dowel bars, is described in Section 7, “Joints.”

5.2 Reinforcing Steel

The longitudinal steel keeps the naturally occurring transverse cracks in continuously reinforced concrete pavement (CRCP) tight, thereby providing a 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 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.

Grade 60 or above deformed steel bars that meet the requirements of 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 a 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 pavements 13 in. or less, all the steel is placed into one mat or layer at the mid-depth of the pavement.

For 14- and 15-in. thick concrete pavements, two layers of longitudinal 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.

5.2.1 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, reinforcing steel shall be free from dirt, paint, grease, oil, or other foreign materials. Reinforcing steel shall be free from defects, such as cracks and laminations.

5.2.2 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 longitudinal 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 bar is effectively transferred to the next steel bar 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 longitudinal steel.

5.2.3 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 any given 2-ft. longitudinal length and 12-ft. width of pavement. Splicing longitudinal steel is not allowed within 10 ft. of construction transverse joints.

Following are drawings (see Figure 9-15, Figure 9-16, Figure 9-17, and Figure 9-18) that show acceptable and non-acceptable splice locations or patterns.

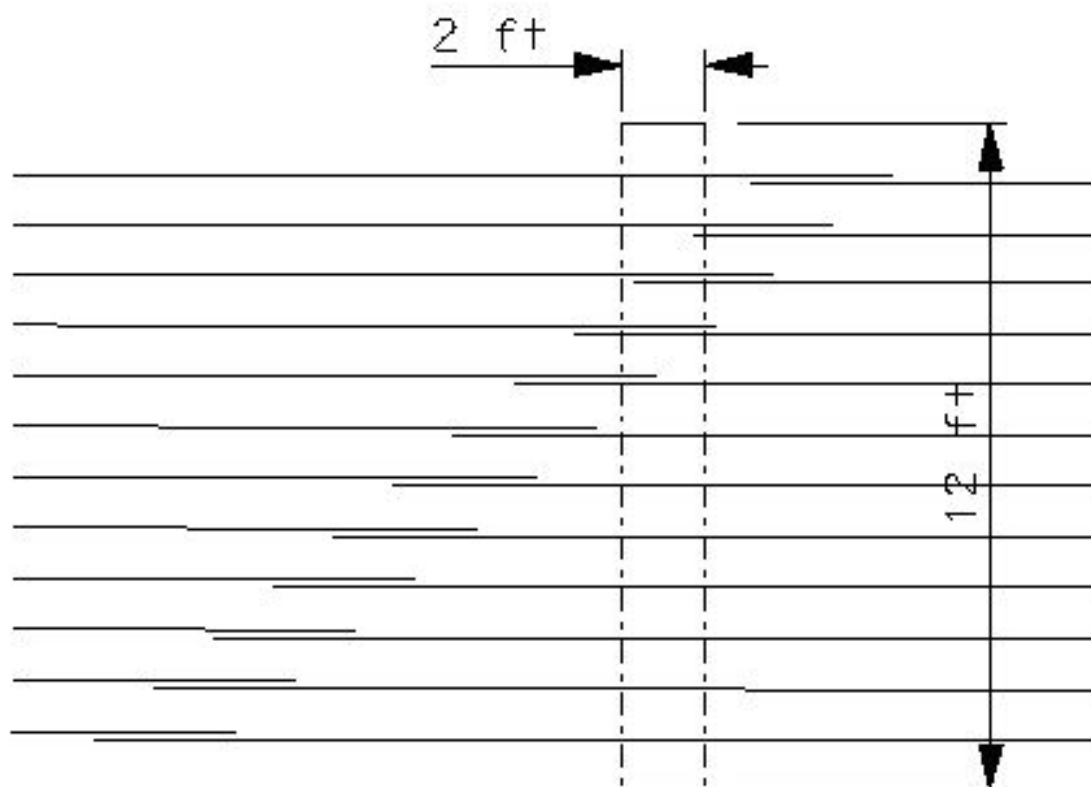


Figure 9-15. Acceptable splicing pattern- 33% (4 of the 12 bars) within the box are spliced.

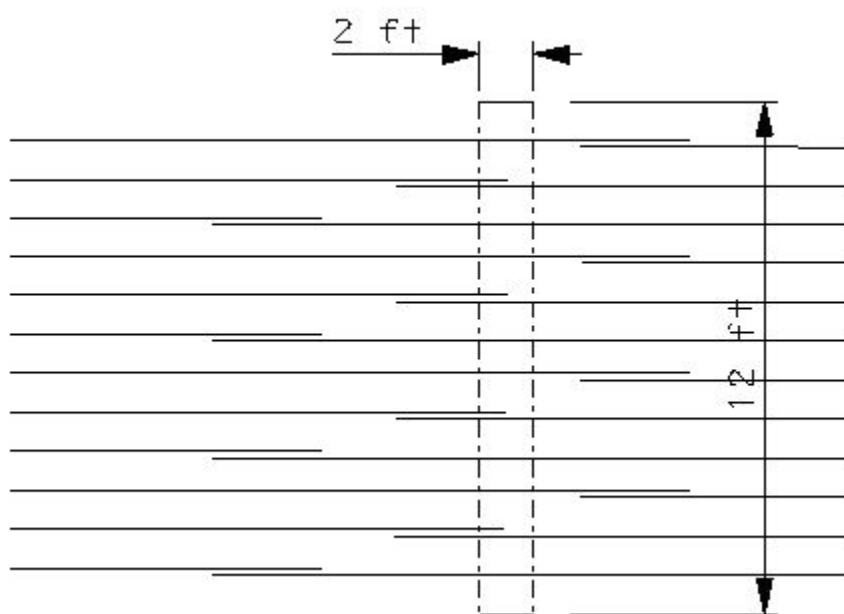


Figure 9-16. Acceptable splicing pattern.

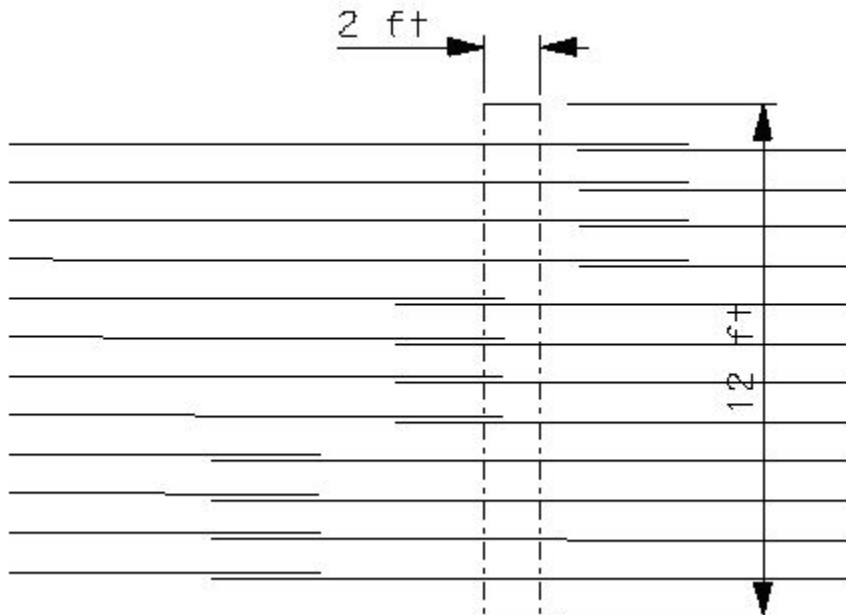


Figure 9-17. Acceptable splicing pattern.

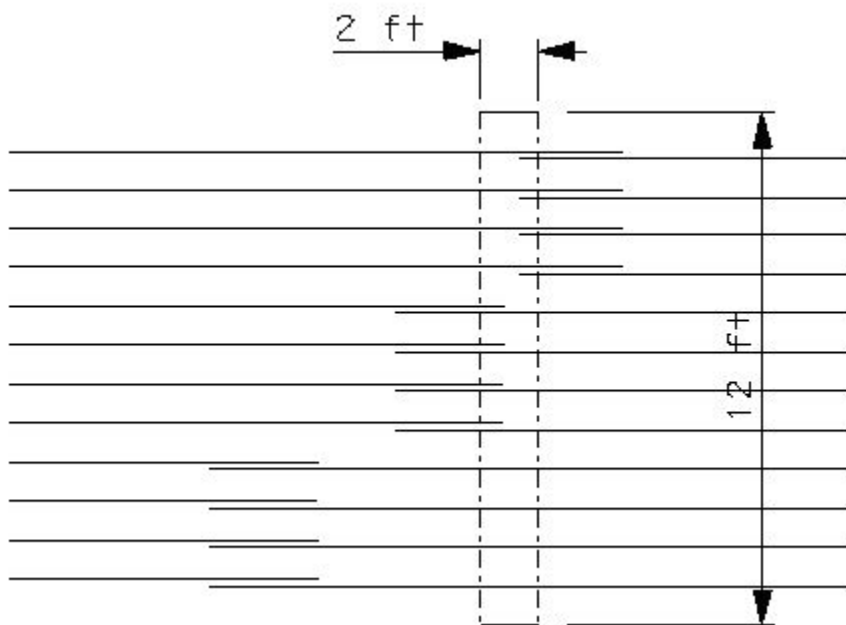


Figure 9-18. Non-acceptable splicing pattern – 66% of bars in the box are being spliced.

5.2.4 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 removed before the concrete pour.



Figure 9-19. Pinning the mat.

Reinforcing steel depth has an effect on CRCP performance. CRCP Design Standards require the tolerance of longitudinal steel at ± 1 in. horizontally and ± 0.5 in. vertically. Figure 9-20 illustrates the checking location of the steel.



Figure 9-20. Checking steel location.

Section 6: Paving Operations

6.1 Introduction

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.

6.2 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 base.

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 screed smooths 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 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.

6.2.1 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 of metal with a minimum thickness of at least 0.2 in. and are 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 while in place.
4. Flange braces extend outward from the base not less than $\frac{2}{3}$ the depth of the form.

The final grade and smoothness of the concrete surface is determined, to a large extent, by how secure and how close 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.

6.2.2 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 for paving.

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.

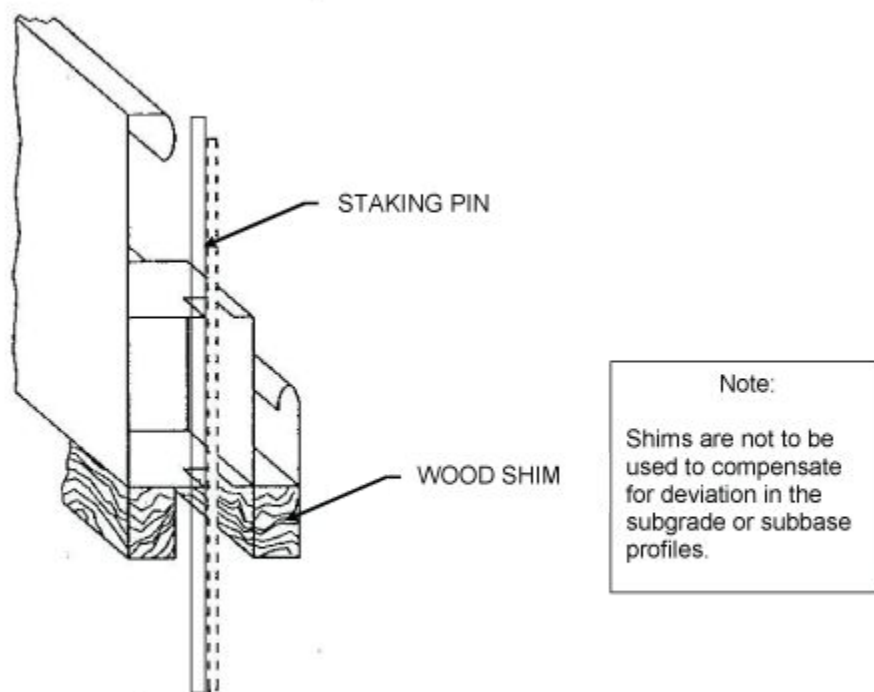


Figure 9-25. Side form.

6.2.3 Checking Forms

Unlike a slip-form paving operation, where the thickness of concrete pavement can be adjusted for “knots” or rises in the base 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 base 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 base, 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 base and “scratch” the surface, that would indicate an area with a rise in the base and where the concrete pavement would have insufficient thickness. If there is a high spot in the base, 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. Base planers were

used in the past with side form paving to excavate excess base material. These planers were suitable for use on non-stabilized base materials. Any milling of stabilized base should be approved by the engineer to ensure that no damage to the base or deficient base thickness result.

6.2.4 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, Figure 9-27, and Figure 9-28 show 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.

6.2.5 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.

6.3 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.

6.3.1 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.

6.3.2 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 smooths 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. The bigger machines with four tracks, two tracks on each side, are steered 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 make up for trash, such as spilled concrete or tie bars, in the track path.



Figure 9-34. Track path.

6.3.3 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). Figures 9-39 and 9-40 show 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.

6.3.4 Horizontal Alignment

The alignment or steering of the paver can be either sensor-controlled or operator-steered.

6.3.5 String-less Concrete Paving

The following information is taken from the FHWA Concrete Pavement Road MAP. Conventional concrete paving with a slip-form paver requires the installation of a stringline and support posts adjacent to the roadway to establish the correct pavement alignment and profile. The stringline adds several additional feet (+/- 6 ft.) of required clearance to the paving envelope, which is already wider than the pavement due to the tracks of the slip-form paver. In addition, the stringline becomes an obstacle for equipment, concrete delivery trucks, and finishing crews. If equipment access across the stringline is required, the stringline must be lowered and reset, resulting in delays and introducing the potential for errors.

String-less paving is a technology that eliminates the installation and maintenance of stringlines and has the potential to decrease the need for surveying and increase the smoothness of the pavement profile. The benefits that can result from string-less paving include increased production, decreased construction time, and reduced potential for errors.

Several companies have developed string-less equipment control and guidance systems using technologies such as global positioning systems (GPS), robotic total stations, and laser positioning. String-less technology replaces the traditional stringlines with an electronic tracking process that controls the horizontal and vertical operation of the slip-form paver. The construction industry has

been using string-less technology for elevation and steering control of equipment for a number of years. To date, the extensive use of this technology has been applied to grading operations. However, string-less paving is an emerging technology for concrete paving because it has the potential to allow contractors and owner/agencies to receive production benefits (e.g., reduced survey costs, fewer construction hours) while still meeting smoothness requirements. Figure 9-41 shows the string-less paving machine.

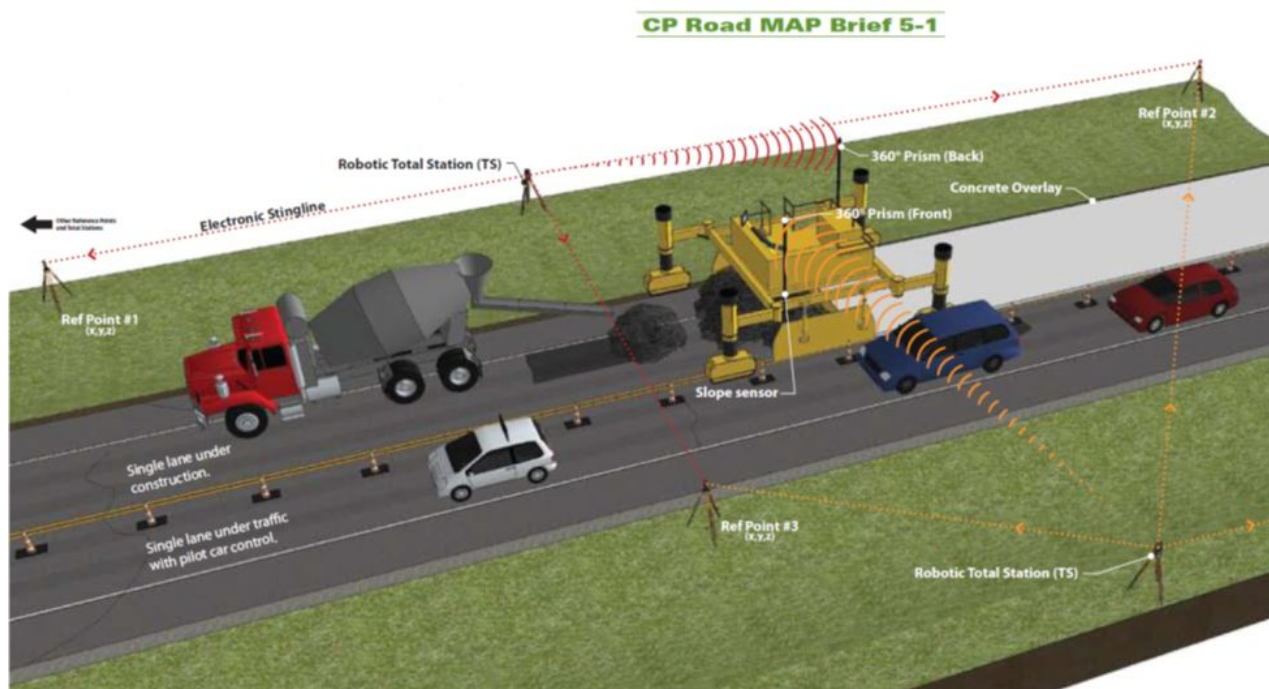


Figure 9-41. String-less paving equipment.

Contractors using string-less paving equipment are still required to establish control points at maximum intervals of 500 ft. and use the control points as reference.

6.3.6 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 5 seconds. Continued running of the vibrators will result in segregation of the concrete, forcing water and fines to the surface.

6.3.7 Augers

Rotating the augers moves the concrete sideways across the entire lane being paved. The augers are controlled by the operator and can be turned either clockwise or counterclockwise as needed. Figure 9-42 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 9-43 shows the spreading of the concrete.



Figure 9-42. Close-up of an auger.



Figure 9-43. Auger and strike-off spreading the concrete.

6.3.8 Vibrators

The frequency in air of the immersion vibrator units shall not be less than 8,000 cycles per minute. Figure 9-44 shows immersion of vibrators. Handheld vibrators are driven by an electric motor.

Vibrators on a slip-form paver are hydraulically driven. Figure 9-45 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, and
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 9-44. Immersion vibrators.



Figure 9-45. Underside of slip-form paver.

Figure 9-46 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 9-47 shows the vibrated zones.

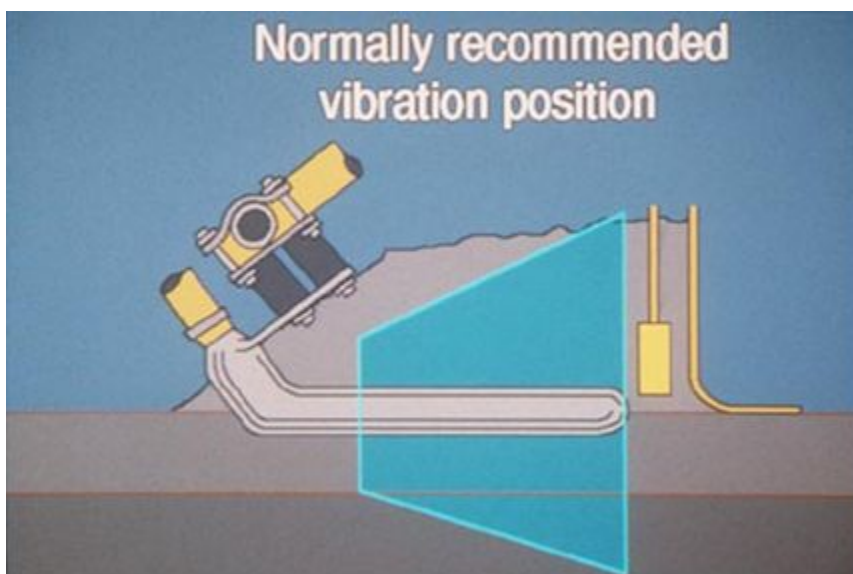


Figure 9-46. Normally recommended vibration position.

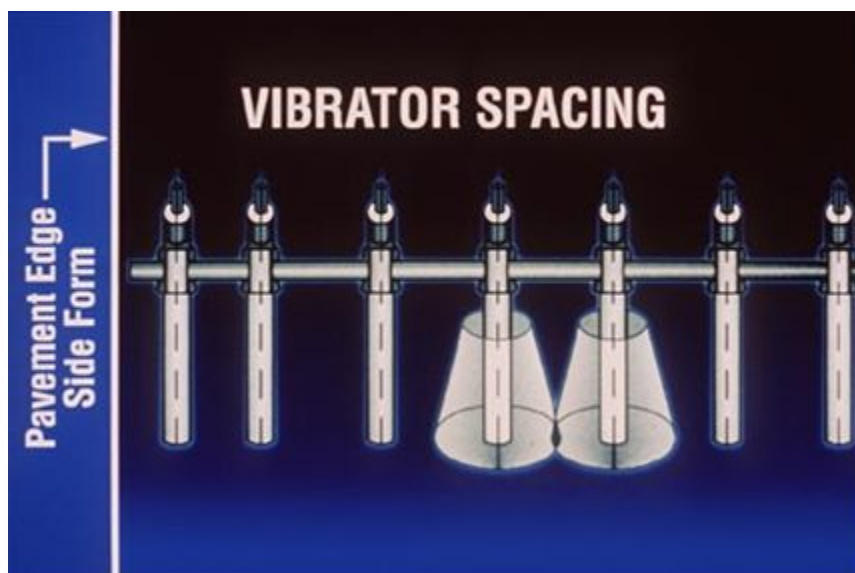


Figure 9-47. Drawing showing the vibrated zone.

No vibrator streaks or trails should be apparent behind the paver. Figure 9-48 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 9-48. Unacceptable – vibrator streaks.

6.3.9 Pan Float

After screeding, the concrete surface is finished with a pan float that further smooths and consolidates the concrete. Pan floats are solid plates anywhere from 18 to 60 in. wide and slightly narrower than the pavement width. Figure 9-49 shows pan floats on an idle paver.



Figure 9-49. Looking up at a pan float on an idle paver.

6.4 Placing Concrete

As mentioned above, concrete can be placed in front of the paving operations using a number of methods. Regardless of which method is used, concrete should be placed as near as possible to the final location. Concrete should never be moved using rakes or vibrators, as these methods cause segregation. Concrete should only be moved by the use of shovels or the augers mounted on the spreader and paver.

In order to promote proper hydration, concrete placed should have a temperature between 40°F and 95°F. It is allowed to use the loads that are already in transit when the temperature was found to exceed the allowable limits. Immediate action must be taken or paving operations ceased until the concrete temperature is corrected.

When the ambient temperature in the shade is 40°F and falling, concrete should not be placed. Typical concrete paving mix designs have relatively low cement contents and usually contain fly ash. This combination of materials tends to make the concrete susceptible to prolonged set times and slow strength gains in cold weather conditions, which can lead to plastic shrinkage cracking. If the ambient temperature is expected to drop below freezing, precautions must be taken to ensure the freshly placed concrete is protected against freezing.

Placing concrete during hot weather can also lead to plastic shrinkage cracking if proper curing is not performed as soon as possible after texturing operations. If there is a delay in application of the curing method, measures should be taken to ensure that the unprotected pavement surface is kept from drying. The use of fog sprays, wind screens, or evaporation retardants are acceptable methods to prevent the surface from drying.

6.5 Finishing Operations

6.5.1 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 straightedged, or both.

6.5.2 Floats

A float may be used only when using a finishing machine. Figure 9-50 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 9-50. 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 9-51. Notice the surface marks left by the float.

Another type of float is the drag float, an example of which is shown in Figure 9-52. 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 9-51. Longitudinal float.



Figure 9-52. "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 cannot reach to the edges, the edges are floated with hand trowels. Such a process is shown in Figure 9-53.



Figure 9-53. Hand trowel.

Care must be exercised while floating to avoid distorting the surface. A bull float (Figure 9-54) 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 9-54. Bull float.

6.5.3 Straightedge

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 9-55 and Figure 9-56. A straightedge flattens the concrete surface and in the case where floats were not previously used, can close surface tears produced by a screed. A straightedge is better at flattening surface bumps and seams than a float, but using a straightedge takes longer than floating because it cannot be mechanized. The straightedges should be parallel to the centerline of the pavement.



Figure 9-55. Straightedge finishing.



Figure 9-56. Straightedge finishing behind a longitudinal float.

Notice in Figure 9-56 that the surface no longer shows the irregularities left by the longitudinal float.

6.5.4 Bleed Water and Free Surface Water

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. The use of supplementary cementitious materials, such as fly ash and slag cement, 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 is not allowed to be applied from a nozzle or thumb over a garden-type hose as shown in Figure 9-57. Adding water to the surface makes the finishing easy to perform. However, this practice dilutes and washes away the surface mortar, resulting in poor performance of concrete pavement in terms of skid resistance, scaling, and durability.

If excessive mortar slurry is present during finishing operations (see Figure 9-58) without the addition of water, this generally indicates an issue with the concrete mix. Batch plant operations should be reviewed to correct this situation.

Finishing operations should not be conducted when free water is on the surface of the concrete. 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.



Figure 9-57. Unacceptable - Water being added.



Figure 9-58. Finishing with excessive mortar slurry.

6.5.6 Evaporation Retardants

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 finishing before the texturing can be performed. This delay may increase if the concrete temperature is cool or a high dosage of a set-retarding admixture is used in the concrete mix.

One method of keeping the surface from drying is the use of an evaporation retardant. The evaporation retardant is applied in a fine mist to uniformly cover the pavement surface. An important property, in addition to reducing evaporation, is that it is specifically formulated to not harm the concrete when final texturing 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.

6.6 Texturing Operations

Surface texture is required to provide skid resistance and to prevent hydroplaning. Skid resistance is achieved from a combination of the fine aggregate and carpet drag. Hydroplaning is minimized by tining. A carpet drag texture and a metal-tine texture finish are required for all areas with a posted speed limit in excess of 45 mph. When shown on the plans, a carpet drag texture can be the only surface texture required for areas with a posted speed limit less than 45 mph.

When carpet drag is the only surface texture required by the plans, ensure that adequate and consistent coarse texture is achieved by applying sufficient weight to the carpet and keeping the carpet from getting plugged with grout. The target carpet drag texture is 0.04 in., as measured by Tex 436-A. Any location with a texture less than 0.03 in. must be corrected by diamond grinding or shot blasting. The engineer will determine the test locations at points located transversely to the direction of traffic in the outside wheel path.

Figure 9-59 shows an upside-down artificial carpet pulled longitudinally on the plastic concrete. This gives the concrete surface a rough texture similar to a broom finish.



Figure 9-59. Artificial carpet drag.

The carpet shall be a single piece long enough to reach across the width of the plastic concrete and having enough longitudinal length in contact with the concrete to produce the desired texture. The engineer may allow changes in the length and width of the carpet to accommodate specific applications.

To keep the mortar from hardening on the carpet, the carpet 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 carpet is caused by hardened grout on the carpet. The contractor should stop the drag as soon as the streaking is observed and remove any dried grout from the carpet.

Figure 9-60 shows the mounting of a carpet. The carpet shall be mounted on a movable support system capable of varying the area of carpet in contact with the pavement and can be pulled either mechanically or manually.

To keep the carpet in contact with the concrete, the trailing end of the carpet may be weighted down with something that will provide enough weight to produce the desired texture.



Figure 9-60. Mounting of artificial carpet.

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.

After the carpet drag operation is completed and the concrete surface has set enough, tining operations can begin. The default tining is transverse (see Figure 9-61); however, longitudinal tining is acceptable when shown on the plans. The metal tines should make grooves in the plastic concrete which are $\frac{3}{16}$ in. deep and $\frac{1}{12}$ in. wide. The minimum groove depth is $\frac{1}{8}$ in. The grooves shall be spaced approximately 1 in., center-to-center, for transverse tining, or $\frac{3}{4}$ in.,

center-to-center, for longitudinal tining. If noise is a concern, longitudinal tining has been shown to be quieter than transverse tining.



Figure 9-61. Transverse 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 9-62 shows a mechanical tining device.



Figure 9-62. Mechanical tining device.

6.7 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.

6.7.1 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 tie 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 (see Figure 9-63) is essential when more than one lane of pavement is placed. Note that the machine is applying curing compound to the vertical edge of the concrete as well.



Figure 9-63. Mechanized spraying unit.

Figure 9-64 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 9-64. Mechanized spraying unit.

Figure 9-65 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 9-65. Unacceptable – non-uniform application of curing compounds.

Plastic shrinkage cracks occurred in the pavement section shown in Figure 9-66 within 5 hr. of concrete placement due to poor curing compound application. This will cause durability problems.



Figure 9-66. Plastic shrinkage cracks.

Figure 9-67 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 to ensure proper bonding of the steel to the yet-to-be-placed concrete.



Figure 9-67. Preventing curing compounds from dropping on tie bars.

Section 7: Joints

7.1 Introduction

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 that could be avoided if properly 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 held tightly closed by longitudinal reinforcing steel. Figure 9-68 shows an example of random cracks in CRCP. These random cracks do not cause distress.



Figure 9-68. Random cracks in CRCP.

There are three types of joints in rigid pavements:

1. Contraction joint,
2. Construction joint,
3. Expansion joint.

7.2 Contraction Joints

Contraction joints are used to relieve tensile stresses resulting from temperature drops and moisture variations in concrete. The contraction joints can be longitudinal contraction joints or transverse contraction joints.

A longitudinal contraction joint is required when the concrete placement width is more than 15 ft. because the chances of cracking, in the longitudinal direction, increase. In normal concrete pavement construction, the width of the concrete placement varies and can be as wide as 50 ft. The saw-cut depth of a longitudinal contraction joint should be $\frac{1}{3}$ of the slab thickness. For CPCD, single piece tie bars are spaced at 24 in. to keep the two adjoining lanes together. For CRCP, in addition to the transverse steel, single piece tie bars are spaced at 48 in. to keep the two adjoining lanes together.

Transverse contraction joints are used only in plain-jointed pavement (CPCD). An example is shown in Figure 9-69. This type of joint is sawn into the green concrete (concrete will not spall or ravel at the cut edges). CPCD standards require transverse contraction joints to be sawed every 15 ft.



Figure 9-69. Transverse contraction joint.

Dowel bars are provided at the transverse 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 detailed requirements for dowels, such as dowel bar size, length, and spacing, are shown in the CPCD Standards. Figure 9-70 shows a picture of a dowel bar assembly.



Figure 9-70. 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 bar is tack-welded to the wire basket. The wire basket is nailed to the base to prevent the basket from being pushed along the base 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.

Problems can occur when longitudinal and transverse contraction joints are sawn too late or too shallow. Attempts to saw-cut too early might cause raveling of aggregates, while saw-cutting too late might result in uncontrolled 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. 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 causing severe raveling. Item 360 allows some minor raveling due to early saw-cutting.

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. Most concrete saws use diamond-tipped saw blades which require water to cool and lubricate the blade. Some saws are riding saws, like a riding lawnmower (see Figure 9-71), and others are self-propelled walk-behind saws (see Figure 9-72). Some newer saw models do not require the use of water and can actually saw the concrete earlier than the concrete saws shown here. These saws are typically called “early entry” saws and are specially designed to minimize raveling.



Figure 9-71. Riding saw.



Figure 9-72. Walk-behind saw.

Figure 9-73 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 if the crack is sealed and allowed to stay in place, the concrete between the crack and the saw-cut will eventually spall out. To prevent uncontrolled cracking, every second or third transverse joint should be sawed first then the intermediate joints. This is called skip-sawing.

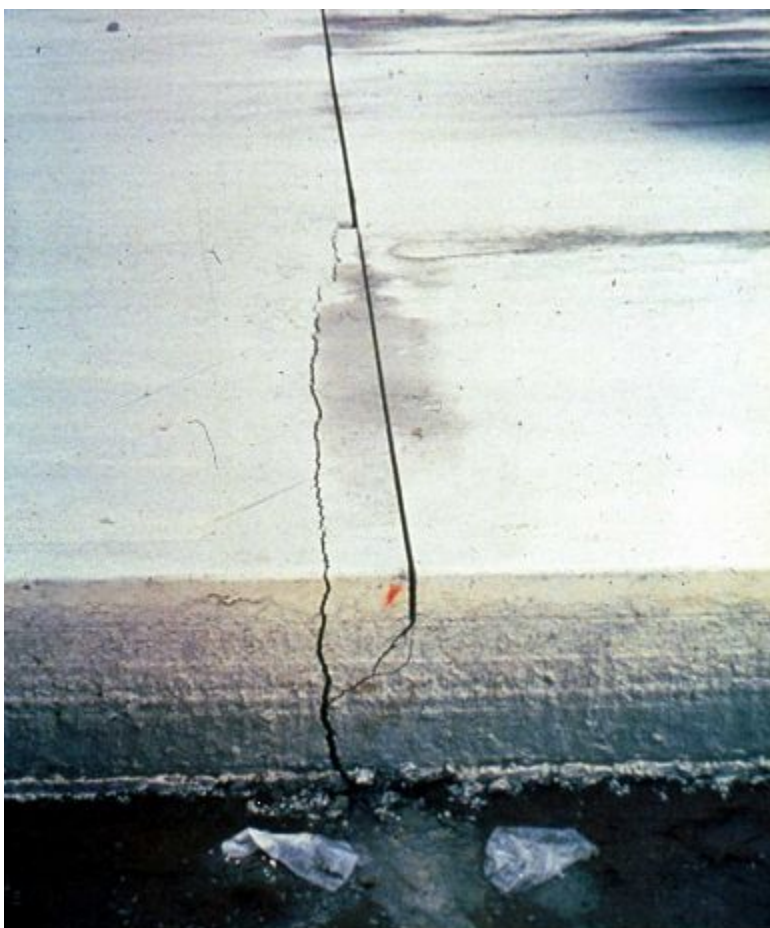


Figure 9-73. Uncontrolled crack developed while initial sawing. The left side of the slab needs to be removed and replaced.

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 the pavement cracking somewhere else. The saw cut depth must be a minimum of $\frac{1}{3}$ of the slab thickness for CPCD pavements and longitudinal contraction joints in single mat CRCP. For longitudinal contraction joints in double mat CRCP, saw cuts are a depth of 4 in. Saw cutting should be accomplished as soon as possible without damaging 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's standard for sealing joints can be found in <http://www.dot.state.tx.us/insdtdot/orgchart/cmd/cserve/standard/rdwylse.htm>.

Figure 9-74 shows a 12-in. thick pavement with an actual saw-cut depth of approximately 2.5 in. The required saw-cut depth is 4.0 in. As a result of the shallow saw-cut depth, a crack was not induced at the desired joint.



Figure 9-74. Crack was not induced due to shallower saw-cut depth than required.

7.3 Construction Joints

Construction joints can be longitudinal or transverse. A transverse construction joint is formed when the paving operation is stopped because of the end of the workday or because of a mechanical breakdown. A longitudinal construction joint is formed when two adjoining lanes are placed with separate concrete placements.

For transverse construction joints, provide a bulkhead (header) that is 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 (see Figure 9-75).

For CPCD, install a transverse construction joint either at a planned transverse contraction joint location or mid-slab between planned transverse contraction joints. Install tie bars of the size and spacing used in longitudinal joints for mid-slab transverse construction joints. For CRCP, additional longitudinal steel is required (see Figure 9-76). Protect the reinforcing steel immediately beyond the transverse construction joint from damage, vibration, and impact. Splicing of longitudinal steel is not allowed within 10 ft. of a transverse construction joint.

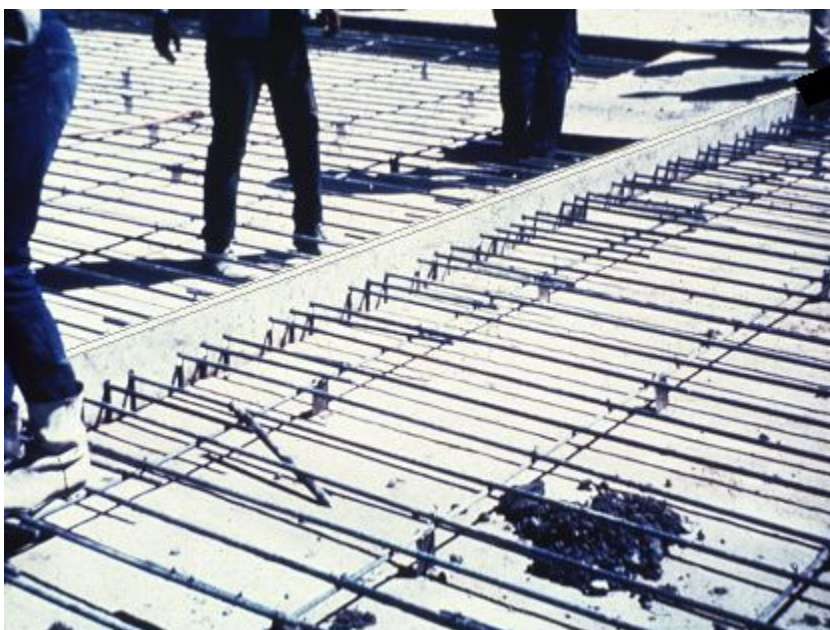


Figure 9-75. Bulkhead (header).



Figure 9-76. Additional steel is required at header.

For longitudinal construction joints, the short pieces of reinforcing steel across the joints are 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.

One-piece tie bars may be used when they do not interfere with traffic or construction conditions. The tie bars are inserted manually or mechanically in the concrete that is still confined by the slip-form paver. Larger finishing machines have the capacity to mechanically insert the tie bars evenly with little disturbance to the plastic concrete. When inserting one piece tie bars, caution is needed

to avoid hitting the reinforcing mat with the tie bars. It is recommended to insert tie bars a few inches away from the transverse bar locations to avoid displacing the mat. Insertion by hand often causes the edge of the plastic concrete to drop down and requires constant remedial work.

Figure 9-77 shows a surface drop down due to hand insertion.



Figure 9-77. Surface drop down due to hand insertion.

Check tie bar insertion depth and locations during paving operations to ensure proper placement. Care must be taken to prevent any movement of the tie bars after placement. Figure 9-78 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 9-78. Wood side form at a transverse construction joint.

Multiple piece tie bars are threaded in the middle to allow the two halves of the tie bar to be separated and then reconnected prior to the casting of the adjacent pavement. Finish multiple piece tie bar assemblies from the approved list of DMS – 4515, “Multiple Piece Tie Bars for Concrete Pavements.”

In the past, bent tie bars were commonly used to keep the tie bars from interfering with traffic or construction. After the concrete set, the bent tie bars were straightened to the proper position. This practice is no longer allowed, since there were problems due to the weakening of the tie bar which leads to steel rupture and lane separations.

Figure 9-79 shows the spacing of the transverse steel and the spacing of the tie bars in a CRCP section.



Figure 9-79. Tie bars.

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 base. These supports need to be sturdy enough to hold the tie bar in position when concrete is placed and consolidated over it.

Tie bars are required at longitudinal widening joints for CRCP and CPCD. Provide #6 bars at 24 in. spacing for 8-in. and thicker slabs. Use #5 bars at 24 in. spacing for less than 8-in. thick slabs. Refer to the CRCP standard for the longitudinal widening joint details.

Mark the tie bar locations and drill holes into the existing concrete at least 10 in. deep. Use a drill bit with a diameter that is 1/8-in. greater than that of the tie bars. Figure 9-80 shows the drilling operation being performed at a widening joint. When manufacturer's instructions are available, precisely follow the requirements for cleaning the holes as outlined. Clean the holes with a wire brush and compressed air to remove all the dust and moisture. Follow the epoxy manufacturer's instructions to apply the epoxy. Insert the tip of the epoxy cartridge or the tip of the machine

applicator to the end of the tie bar hole and inject Type III, Class C, epoxy to fill the entire hole, then insert the tie bars.

Demonstrate, through simulated job conditions, that the bond strength of the epoxy-grouted tie bars meets a pullout strength of at least $3/4$ of the yield strength of the tie bar when tested in accordance with ASTM E 488, within the Epoxy manufacturer's recommended curing time. Increase embedment depth and retest when necessary to meet testing requirements. Perform tie bar testing before starting widening work.



Figure 9-80. Marking and drilling holes at widening joint.

7.4 Expansion Joints

Concrete experiences volume changes due to temperature and moisture variations. When the temperature changes, concrete contracts and expands. The resulting expansion of the concrete pavement must be taken into account. Normally, concrete pavement is abutted to bridge structures via approach slabs. If the pavement expansion is not accounted for, the approach slabs will be pushed into the bridge and can cause serious damage to the bridge structures. To prevent this problem, a mitigation system is installed at the bridge and pavement interface to account for the expansion. There were three systems in use by the department:

- 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.

The wide-flange system is similar to the expansion joint 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.

An anchor lug system tries to restrict the concrete movement at the interface between bridge and pavement by providing several anchor lugs.

TxDOT has investigated the movements of CRCP near bridges and the effectiveness of these three terminal systems in Research project 0-6326, “Rational Use of Terminal Anchorages in Portland Cement Concrete Pavements.”

The research data has shown that the anchor lug system is not effective in the long run. The stresses generated in soil due to slab expansion at lug walls are large enough to result in permanent deformations in soils. The soil does not retract with the lug when the pavement contracts. The permanent deformations result in voids between soil and lug walls.

Field measurements have indicated that the base friction restrains slab movements effectively. Using coarse texture base, such as HMA, might be the most effective tool to control slab movements. Simple expansion joint systems or wide-flange systems are also effective in accommodating slab movements. Expansion joint systems should cost less than wide-flange systems with comparable performance.

The standard for the anchor lug system was deleted and not replaced. The transverse expansion joint details at bridge approaches are shown in sheet 2 of 2 of CRCP and CPCD standards.

Item 368, Concrete Pavement Terminals of 2004 Standard Specification, was removed in the 2014 Standard Specification. Districts may develop a Special Specification for using wide-flange systems.

7.5 Joint Sealing

The purpose of joint sealing concrete is to reduce infiltration of moisture and incompressible materials into joints for improved pavement performance. Infiltration of water through unsealed joints is the main source of surface water into the pavement structure. Moisture in the pavement foundation can allow loss of slab support from base and subgrade erosion and pumping, which causes concrete pavement distress. Sealing also can prevent incompressible materials from entering joints. Incompressible materials lock joints and create excessive stresses that may cause spalling, blowups, or shattering.

There have been considerable national debate and research efforts on the issues and cost-effectiveness of joint sealing and resealing. The department requires all joints to be sealed in concrete pavement per Item 360, “Concrete Pavement,” and the joint sealing standard.

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 occur 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 in close proximity, 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.

Section 2: Full-Depth Repair

2.1 Introduction

In this chapter, Portland cement concrete (PCC) pavement distress types that require full-depth repair are discussed, followed by the repair procedures.

2.2 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 that extend through the depth of a slab occur due to temperature/moisture variations and/or wheel load stress and require FDR. Transverse cracks in CPCD that extend through the depth of a slab are caused by design issues, such as inappropriate slab lengths and deficient slab thickness, or construction issues, such as non-uniform or insufficient base support.

Plastic shrinkage cracks occur when the rate of evaporation from the surface exceeds the rate at which the bleed water is available. Shallow plastic shrinkage cracks (approximately 1 to 2 in. from the surface) are not candidates for FDR unless they occur throughout the slab.

Shattered slabs and corner breaks in CPCD result from design issues, such as deficient slab thickness, or construction/design issues, such as non-uniform or insufficient base support.

In continuously reinforced concrete pavement (CRCP), the following distresses require FDR:

- punchouts,
- deep spalling.

Punchouts in CRCP are caused by design issues, such as deficient slab thickness, or construction/design issues, such as non-uniform or insufficient base support. It is manifested by depressed block(s) of concrete bordered by transverse and longitudinal cracks. Longitudinal steel crossing transverse cracks at the punchouts will eventually rupture, resulting in further deterioration of the punchout. Punchouts are the most serious distress type in CRCP. Improved design and construction practices by the department over the past several decades has significantly reduced the frequency of punchouts. Figure 10-1 shows a typical punchout. Note the asphalt patch was applied to restore the surface elevation, implying that the concrete block was pushed into the base or raveled out.



Figure 10-1. Typical punchout 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 this type of spalling. In Texas, spalling is more prevalent when coarse aggregates with high coefficient of thermal expansion 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 which can be repaired by half-depth repair (HDR). A series of deep spallings cause substantial structural damage to the pavement and should be repaired by FDR. Based on the severity, durability, and cost, the engineer will determine using FDR or HDR to repair the deep spallings. Figure 10-2 shows deep spalling.

Unlike punchouts, it is 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), portable seismic pavement analyzer (PSPA), or ultrasonic tomography device (MIRA).



Figure 10-2. Deep spalling.

2.3 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, remove damaged base,
5. Drill holes for longitudinal and transverse tie bars or dowel bars,
6. Provide longitudinal and transverse steel continuity (CRCP only),
7. Place and finish concrete, and
8. Restore existing joints.

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. The 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. 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. Figure 10-3 shows an example of a repair where the limits should have extended farther to the left side.



Figure 10-3. Full depth repair of punchouts in CRCP.

For FDR of extensive transverse cracking and shattered slabs in 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 punchouts, visual observation of pumping or the use of falling weight deflectometer (FWD) testing can be used to determine the limits of the repair. For deep spalling, coring or non-destructive testing is the most efficient evaluation method. It is recommended that the limits of FDR are 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-blade saws through the full depth of the concrete slab. Saw-cutting with diamond-blade 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 should not 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 base but also in the concrete, which might increase the repair area needed.

3. Remove the concrete slab

Once the repair limits are saw-cut, the concrete is removed in one of two ways. One method is to lift out large pieces of the slab, and the other method is to break up the slab. Item 361 requires the lift-out method to be used for slab removal. The breakup method is not allowed because it could cause damage to the 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 lifting.

Once the lift pin arrangements are complete, cranes or front-end loaders lift the slab 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, Remove Damaged Base

After the slab removal, remove loose or damaged base material completely, leaving no loose base material. Recompact base materials to the satisfaction of the engineer. When shown on the plans, level the base layer with cold-mix asphalt to the original bottom line and grade of the concrete slab before repair concrete is placed. Place concrete directly onto the compacted base layer unless otherwise directed. Also, if the distress in the base 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 base beyond the perimeter of the FDR is not repaired, the same type of pavement distress will take place.

5. Drill holes for longitudinal and transverse tie bars or dowel bars

If tie bars installed at the repair perimeters are not performing well, the potential problem of lane separation at the longitudinal joint or distresses at the transverse joints will increase.

To achieve the optimum bond between concrete and tie bars inside drilled holes, the holes must be of adequate length, and all dust and moisture inside the holes should be completely removed. The most commonly used method to clean the holes is to use a wire brush and apply compressed air. 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 the epoxy. To check whether there is oil vapor, shoot compressed air into white paper for a few seconds and check the paper for oily residue.

Once the drilled holes are cleaned, completely fill with epoxy. The epoxy used in this operation has a low viscosity, and it is difficult to fill the holes completely.

In CPCD, dowels need to be provided at all transverse joints when removing whole slabs. A minimum of four dowels are recommended for each wheel path. Dowel bars can be provided by either drilling and epoxying or by performing a dowel bar retro-fit procedure.

When removing two adjacent half slabs, tie bars must be provided at the transverse construction joint by drilling and epoxying. Provide dowel bars at the new transverse contraction joint and saw-cut and seal the new joint.

Regardless of which repair type described above is used, provide tie bars at the longitudinal construction joint by drilling and epoxying. In CRCP, provide continuity of the reinforcing steel at the transverse construction joints by drilling and epoxying tie bars. Provide tie bars at the longitudinal joint by drilling and epoxying. The spacing and depth of the holes are specified in the concrete pavement repair standard. 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 department requires a minimum length of the drilled hole of 10 in. and requires the contractor to demonstrate that the installation process meets the specified pull-out strength. If the specified pull-out strength is not met, increase the embedment depth and retest. Other variables can be modified to increase the pull-out strength such as epoxy application and hole cleanout. Failure of drilled and epoxied bars to achieve proper pull-out strength is generally related to inadequate filling of drilled holes with epoxy rather than the length of embedment.

6. Provide longitudinal and transverse steel continuity (CRCP only)

For FDR of CRCP, this step of providing longitudinal continuity of tie bars is of utmost importance.

The concrete repair standard requires a minimum lap length of 25 in. for #6 bars. To prevent pull-out failure, the steel embedment 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, and the lap length will need to be increased.

Figure 10-6 shows a full-depth repaired slab that was removed to repair additional distresses that occurred in the adjacent slab. The additional distress occurred because the continuity of the longitudinal steel was not provided in the original repair. 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 the tie bars are not fully bonded to the concrete and the holes are not filled completely with epoxy. If pull-out testing had been performed, it would have indicated a possible failure. The inability of the tie bars to provide continuity of the longitudinal steel caused a wide transverse joint, which resulted in additional distress.



Figure 10-6. Previously repaired slab.

7. Place and finish 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 provide 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. 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 is normal practice to use a high cement content as well as Type III cement. This practice could be beneficial if the repairs are done in winter when the ambient temperature 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. This practice could also result in larger drying shrinkage of the concrete, especially near the surface. As concrete temperature increases, the evaporation of water from the surface also increases. Proper curing is essential to minimize map cracking caused by large temperature gradients and high evaporation. Figure 10-7 shows map cracking caused by improper curing.



Figure 10-7. Map cracking in FDR slab.

If class HES concrete cannot achieve strength within the required time frame, the use of materials meeting the requirements of DMS-4655 is allowed.

For FDR projects, 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. 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.

8. Restore existing joints

Restore all the existing joints within repair area in accordance with the relevant standard joint details and Item 360, “Concrete Pavement.” Especially, perform a timely saw-cut over the dowel bars and restore the transverse contraction joints for CPCD.

Section 3: Concrete Pavement Repair

3.1 Introduction

Use Item 361, “Repair of Concrete Pavement,” for situations where the deteriorated concrete extends to the mid-depth of the slab. Use Item 720, “Repair of Spalling in Concrete Pavement,” or Item 721, “Fiber Reinforce Polymer Patching Material,” for situations where the deteriorated concrete is shallower than the mid-depth of the slab. For emergency work or temporary repair, Item 700, “Pothole Repair,” may be utilized.

3.2 Repair Procedures

The following procedures need to be followed when repairing concrete pavement:

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

It is important to properly evaluate the extent of the deteriorated concrete and determine the limits of the repair as shown in Figure 10-8. For example, when spalling occurs, the damage often extends beyond the visible spalled area. Since most spalls are caused by delaminations, the extent of delaminations should be identified. One of the most efficient ways of evaluating the extent of delaminations is the use of a sounding test. A hammer or steel rebar can be used for sounding testing by tapping 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 10-8. Identify the repair limits.

Figure 10-9 shows an example of limits that were not properly identified.



Figure 10-9. Coring adjacent to the spalled repaired area.

The repair material applied (left side of the picture) has a greenish color. The core, taken several inches away from the limits of the spall repair, revealed several delaminations at various depths. The top portion of the core came out disintegrated. If a sounding test had been conducted, areas with delamination would have been identified and the areas could have been 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. Ground penetrating radar (GPR), portable seismic pavement analyzer (PSPA), and ultrasonic tomography device (MIRA) are additional methods that can be used to determine the limit of deterioration. Contact MNT – Pavement Asset Management for assistance with these methods.

2. Remove deteriorated concrete

After the repair limits are determined, remove the deteriorated concrete as shown in Figure 10-10. Saw-cut the perimeter of the repair area to a minimum depth of 1-1/2 inches and then chip the concrete out using light pneumatic tools.



Figure 10-10. Removing deteriorated concrete.

3. Clean the repair surfaces

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. After deteriorated concrete is removed, the concrete surfaces should have a rough texture. Clean the repair area with water to remove any dust to improve bonding of the patching material as shown in Figure 10-11.



Figure 10-11. Clean repair surfaces.

4. Place repair materials

The department has several repair material options that can be used which are listed in Items 361, 720, and 721.

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.

When using ready-mixed concrete, the repair surface should be saturated with no ponding water prior to placing the concrete. The concrete must be cured in accordance with Item 360.

When using repair products listed on the department's Material Producer Lists, prior to placing repair materials, carefully read and follow the manufacturer's recommendations for surface preparation, mixing, placing, and curing.



Figure 10-12. Place repair materials.

5. Finish the surface

Finish the repair surface to conform roadway surface. Match the repair surface texture with adjacent concrete as shown in Figure 10-13.

Good curing is essential. Because the repair areas and depths 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 not properly cured. 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 bleed water no longer comes to the surface. However, repair materials have low water-cement ratios and tend not to bleed. In that case, curing operations should begin as soon as possible.



Figure 10-13. Finish the surface.

Section 4: Bonded Concrete Overlay

4.1 Introduction

This section describes the construction of bonded concrete overlays (BCO) on both CRCP and CPCD pavements.

Many of the older concrete pavements in Texas were designed and constructed with insufficient thicknesses for today's traffic demand. This insufficient thickness often results in pavement distresses such as punchouts for CRCP and mid-slab cracking or joint faulting in CPCD. However, if the Portland cement concrete (PCC) pavement is still structurally sound, does not show significant signs of distresses, but has insufficient thickness, BCO can provide cost-effective rehabilitation strategies to extend the pavement life.

When constructing a bonded concrete overlay, a 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 for various reasons.

The most important factor for the success of a BCO is having a good bond between a new and old concrete layers. If a good bond is provided, the new composite slab, old and new concrete layers, will behave monolithically as a thicker slab. On the other hand, if the two layers are not properly bonded together, the two layers will behave as independent slabs, which will result in high wheel load stress within the new concrete layer, and the pavement performance will be compromised.

4.2 Bonded Concrete Overlay (BCO) Procedures

The construction of bonded concrete overlay (BCO) involves the following procedures:

1. Repair distresses in the existing pavement.
2. Prepare surface of existing pavement for overlay.
3. If needed, place steel.
4. Place and cure concrete.
5. Saw-cut and seal joints.

Each step is explained in more detail.

1. Repair distresses in the existing pavement

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.

Deep spalling, delaminations, punchouts, and deteriorated patches must be repaired. Existing asphalt concrete (AC) patches should be removed and replaced with PCC patches so the existing pavement is made structurally sound. Concrete repairs should be performed in accordance with Sections 2 and 3. Working longitudinal cracks may be repaired by stitching, as described in Section 6, [Stitching](#)

It is common practice to remove and replace the large deteriorated areas when structural distresses are extensive. When the distress is caused by a localized foundation weakness, it is necessary to ensure that the weak base layer materials are removed and the remaining base is well compacted during FDR, as detailed in Section 2. When voids are detected under existing slabs, grout should be injected to stabilize the pavement.

When constructing a BCO over a CPCD section, it is necessary to ensure that the sections have adequate load transfer efficiency. CPCD sections built without dowels will need to have dowel bar retrofits done prior to constructing the overlay. Section 7, “Dowel Bar Retrofit,” details these requirements.

2. Prepare surface of existing pavement for overlay

As described previously, the critical factor for the good performance of a BCO is the bond between the existing concrete and overlaid concrete. One of the requirements needed to ensure a 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 layers were a single structure.

There are several surface preparation methods that can be used to achieve a roughened surface. The most common are:

- shotblasting,
- sandblasting, and
- cold milling and shotblasting.

Shotblasting involves a spinning drum equipped with compressed air that blasts tiny steel balls (shot), which impact the surface at an angle to scarify the surface. A vacuum collects both the shot and the dust. The shot is separated from the dust by magnetic action for continuous reuse. Regulating the speed allows the level of scarification to be controlled. Slower speeds yield a higher depth 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 clean the surface to remove debris prior to paving. The sandblasting surface finishing is not as uniform as 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 10-14. 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 requiring deep scarification. However, it generates a high amount of dust and contamination, which must be removed prior to overlaying.



Figure 10-16. 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 are detrimental to its structural integrity. Shotblasting is often required after cold milling to remove the loose fractured remnants on the pavement.

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.

If the pavement has been overlaid with HMA layers, these layers should be completely milled off during the surface preparation. Remnants of HMA will hinder the bonding of both PCC layers and are likely to trigger delamination. Complete milling of these layers will ensure that all surface contaminants such as oil, carbonates, and acids are removed.

Once the surface is roughened, the section should not be opened to traffic until the overlay is completed and cured. Allowing traffic on the roughened surface allows the opportunity for contaminants to be deposited on the new prepared surface, which may cause debonding issues.

Air blasting is not capable of removing paint stripes, tire marks or the grout matrix. It should be used only as a supplementary cleaning procedure to remove loose material and debris from the surface after milling, shotblasting, or sandblasting. Air blasting is also used just before overlaying to thoroughly remove debris from the prepared surface. It is important to minimize the time between the final surface cleaning and paving in order to prevent the contaminants from resettling.

3. If needed, place steel

When constructing a CRCP BCO, reinforcing steel is needed within the new layer of concrete to maintain tight cracks. Steel should be placed at a depth that provides a minimum concrete cover of 3 in. If the overlaid thickness layer will not provide sufficient coverage, reinforcement steel bars can be placed directly over the surface of the existing pavement as shown in Figure 10-15, rather than at mid-depth of the overlay. For thinner overlays, steel placed at mid-depth may preclude the use of a slip-form paving machine 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 chairs are not needed, and the steel will restrain concrete volume changes at the interface, which will prevent or minimize debonding. The only disadvantage is the reduction of the interface area between the new and old concrete. Refer to Chapter 8, Section 7, for the steel percentage and vertical location.

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 10-15.



Figure 10-15. Airblasting to clean the concrete surface.

4. Place and cure concrete

The materials selected for use in the concrete mixture must be carefully selected. The aggregates of the 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 equal to or 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 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 also has a significant impact on the concrete's coefficient of thermal expansion (CoTE). Concrete has a CoTE ranging from 4 to 6 microstrain/°F. Large differences in the CoTE between the existing and new concrete result in increased stresses at the interface, which will impact the bonding. It is recommended that the coarse aggregate in the BCO should have a thermal coefficient that is lower than the CoTE of the existing concrete and must not exceed 5.5 microstrain/°F.

The maximum aggregate size of the BCO concrete should be 1/3 of the overlay thickness. This will ensure a uniform distribution of the concrete constituents when placing the BCO. If the aggregate is larger than 1/3 of the BCO thickness, segregation of the oversized aggregates is likely to occur, especially in areas where it is difficult to consolidate around reinforcing steel.

The required concrete strength of the BCO concrete is the same as the strength required in Item 360. Type I or I/II cements are the 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. Special attention should be paid when Type III cement is used. For a BCO, the use of Type I or I/II cements is recommended, as they produce less heat from hydration than a Type III cement and, therefore, reduce the development of thermal stresses.

Typical placement methods used for concrete pavement construction are also used to construct BCOs. BCOs can either be slip-formed or fix-formed placed. Chapter 9 details these placement techniques. When constructing a BCO over a CPCD, prior to placing the overlay concrete, the location of the existing joints should be clearly marked so that the joints in the overlay concrete can be sawn directly over the existing joints.

Special attention should be given to adverse environmental conditions during paving. A combination of high wind velocity, high air temperature, low relative humidity, and high concrete temperature is the most harmful paving conditions. These conditions promote high water evaporation rates from the 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. Under these conditions, necessary precautions need to be taken to prevent any adverse effects. Chapter 9 details necessary steps to take during adverse weather conditions to protect the pavement.

Curing is a key component for preserving satisfactory moisture content and temperature in the concrete during its early stages so that desired properties may develop. Curing is critical 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. A fast-track BCO takes this concept further; by utilizing special materials, the road can be opened to traffic in a minimal time after placement. A fast-track 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, using Type III cement, as opposed to Type I cement.

5. Saw-Cut and Seal Joints

For CPCD sections, thinner overlays have a greater potential for rapid shrinkage and contraction, and, therefore, joint construction should take place as soon as possible. For CPCD, joints in the overlay should be sawn directly over joints in the existing pavement to prevent reflective cracking of the existing joints through the BCO. The depth of saw-cutting for transverse joints should be full depth plus 1/2 in. Longitudinal joints should be full depth.

Transverse and longitudinal joints should be sealed according to the joint sealing standard. <ftp://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/standard/roadway/js14.pdf>.

Section 5: Unbonded Concrete Overlay

5.1 Introduction

This section describes the construction of unbonded concrete overlays over both CRCP and CPCD. 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 an unbonded concrete overlay (UBCO).

UBCOs have been used successfully in many parts of the country. UBCOs are typically thicker and will generally cost more than a BCO. While the existing PCC pavement should be in good condition for a BCO to work, UBCO can be successfully used where the existing pavement is in poor condition. This is a significant advantage of UBCO over BCO or other rehabilitation methods. Also, very little preparation work is needed, except for repairing shattered slabs in the existing CPCD or punchouts in CRCP.

The performance of an UBCO is highly dependent on 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 against distresses in the existing pavement affecting the overlaid concrete. The recommended mixtures for interlayer are Dense-Graded item 340/341 TY-C and TY-D, and Superpave mixtures item 344 SP-C and SP-D.

If the existing concrete pavement is overlaid with an HMA layer already, the existing HMA layer can be utilized as the interlayer. The minimum thickness of structurally sound HMA required for bonding is 3 in. Milling can be used to remove the surface distresses of existing HMA. The amount of HMA removed depends on the types and severity of distresses and the thickness of the HMA. If a stripped or debonded layer of HMA is encountered, it must be completely removed to provide a sound structural layer for bonding. All unsound areas should be removed prior to performing any further operations.

5.2 UBCO Procedures

Even though UBCO is a rehabilitation method, the construction methods are very similar to that of new concrete pavement construction. The UBCO construction procedures are as follows:

1. Repairing distresses in the existing pavement.
2. Placing the interlayer.
3. Constructing the UBCO.

1. Repairing distresses in the existing pavement

If the UBCO is placed over CPCD, shatter slabs must be removed and replaced. If the UBCO is placed over CRCP, punchouts should be repaired with full-depth repair (FDR) methods. Spallings,

whether they are shallow or deep, do not need to be repaired. Sections 2 and 3 detail the methods that should be used to properly repair these distresses.

2. Placing the interlayer

Place the interlayer in accordance with HMA specifications and the details of construction method in Chapter 6.

3. Constructing the UBCO

Once all the distresses requiring repairs are completed and the interlayer is placed, the UBCO is constructed using the same methods and following Item 360 and the pertinent pavement standards as when constructing a new concrete pavement. Chapter 9 discusses these construction methods.

Section 6: Stitching

6.1 Introduction

Stitching is performed at longitudinal cracks to maintain aggregate interlock and provide additional 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.

6.2 Pavement Distresses that Require Stitching

In CPCD, **stitching should never be used at transverse cracks**. Stitching transverse cracks in CPCD will cause stress buildup from the volume changes due to temperature variations somewhere else and result in additional cracks or spalls. Transverse cracks in CPCD section should be repaired with either FDR or dowel bar retrofit. Longitudinal cracks in CPCD do not necessarily deteriorate at the same rate as transverse cracks, so the slabs do not need to be removed and replaced. Longitudinal cracking in CPCD is most often caused by shallow and possible late saw-cutting of longitudinal contraction joints. For longitudinal cracks that are not continuing to widen, only sealing of the crack may be needed. For longitudinal cracks that are continuing to widen, stitching is a good repair option to keep the cracks tight. Longitudinal cracks should be repaired as soon as possible after identification to prevent further deterioration and separation. Repairing the cracks early saves money in the long term.

In CRCP, the only time stitching is utilized is where lanes are separating at the longitudinal construction joints. In CRCP, transverse and longitudinal reinforcements are provided and cracks are held tight and typically do not cause pavement distress. When longitudinal cracks extend through the full slab depth, punchouts will result and full-depth repair of the pavement is the most appropriate repair option.

CPCD and CRCP have reinforcing steel crossing the longitudinal construction and contraction joints which keep the lanes from separating. However, for various reasons, lanes do separate at longitudinal joints, primarily longitudinal construction joints. In some projects where bent tie bars were used, the tie bars were not straightened after the slip-form paving machine passed. In other projects, corrosion of the tie bars was noted. Corrosion and shear were found in association with tie bar failures and lane separations. These construction errors and steel deterioration resulted in lane separation. Lane separations over 2 in. wide have 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 conditions cause higher wheel load stresses than interior loading conditions do, and increases the potential for longitudinal and/or corner cracks in CPCD and punchouts in CRCP, which reduces the pavement life.

There are other possible causes for lane separations, such as weak base layers. In Project 0-5444, there was no direct correlation found between DCP readings and the likelihood of longitudinal cracking or lane separations, but when lower modulus values of base were found, the possibility of problems with longitudinal cracking and joint separations was greater.

If voids are present under faulted slabs, the pavement should be undersealed to re-establish uniform support for the slabs prior to performing stitching operations.

Falling weight deflection testing is often used to measure load transfer efficiency (LTE) to determine if stitching is warranted. When taking load transfer efficiency (LTE) readings, the deflections associated with the LTE test locations need to be known to determine the condition of the pavement. A high or low LTE reading can be misleading; a high LTE reading does not necessarily mean the pavement is in good condition. However, high measured deflections always means the slabs have low LTE. Stitching has been rarely used by the department. However, lane separations have occurred at the longitudinal joints and, lately, stitching has been used on several projects. When considering the use of stitching, contact MNT – Pavement Asset Management for assistance.

6.3 Types of Stitching

There are three types of stitching methods: cross-stitching, slot-stitching, and U-bar stitching. Cross-stitching is the most widely used method. Project 0-5444 investigated lane separations and the best repair methods.

Cross-stitching should be used to repair cracks/separations that are fairly tight. 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. Dust is removed by compressed air, and epoxy is injected into the holes. Tie bars are inserted, and excess epoxy is removed.

For wider cracks/lane separations, slot-stitching should be used. Slot-stitching is the most economical repair method for restoring load transfer, preventing separations, and improving performance of longitudinal joints and wide cracks. In slot-stitching, slots with lengths no shorter 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 a repair mortar 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 repair mortar with good bond and to have good consolidation around the bars.

In U-bar stitching, slots are cut using a slot-cutting machine, and concrete is broken and removed by pneumatic hammer. Figure 10-16 shows the slots and U-bars. In this method, anchoring action by the U-bars provides most of the restraining force. Use of proper repair materials and consolidating around the ends of the U-bars is important.



Figure 10-16. Slots and U-bars.

Project 0-5444 investigated what causes lane separations and the best repair methods. The following conclusions regarding longitudinal cracking and joint separations in concrete pavements have been made upon completion of the research project.

Section 7: Dowel Bar Retrofit

7.1 Introduction

This rehabilitation technique is only applicable to CPCD, not CRCP. Load transfer is defined as the ability of a joint or crack to transfer wheel load from one slab to the next. Good load transfer at the joints improves the performance of CPCD, and the majority of load transfer is achieved by dowels which cross the transverse joints. 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. To ensure adequate load transfer for the life of the pavement, dowels must be used. However, there are jointed concrete pavement sections that were built without dowels. The absence of dowels along with deficient base or subgrade support results in faulting and cracking problems at joints. Figure 10-17 shows faulting in CPCD.



Figure 10-17. Severe faulting at the transverse joint.

The pavement shown in Figure 10-17 does not have dowel bars, and the base support was not adequate. The lack of load transfer in CPCD sections without dowels increases the dynamic wheel loading, which results in increased faulting. 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, repair mortar is placed in the slots, 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. The department has developed a special specification and design standards for DBR.

7.2 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. Place repair mortar in the slots.

1. Identify the need for DBR

Any of the following conditions warrant the use of DBR:

1. load transfer efficiency (LTE) of 60% or less,
2. faulting greater than 0.1 in., or
3. differential deflection of 10 mils or more.

If the faulting is due to deficient base support, DBR alone should not be used to restore LTE. In this case, base repair should be done in addition to DBR.

For pavements where the pavement condition is satisfactory but dowels were not used, DBR can be used to strengthen the pavement's structural condition as a preventive maintenance operation.

2. Cut the slots

Once the need for DBR is established, the next step is to cut the slots using a diamond saw slot cutter. It is customary to provide three dowel bars in each wheel path. The diamond saw slot cutter shown in Figure 10-18 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 repair material to flow around the bars and consolidate properly.



Figure 10-18. Diamond saw slot cutter.

Figure 10-19 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.

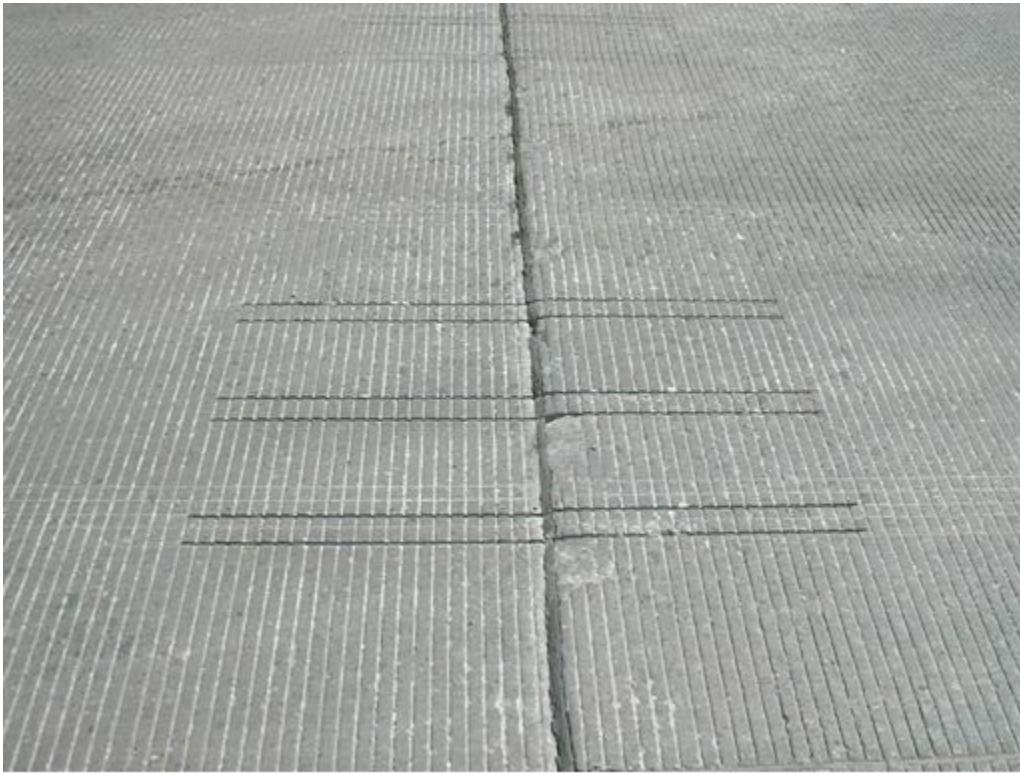


Figure 10-19. Cuts made for slots.

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 10-20. 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 10-21. Figure 10-22 shows the flat bottom after being water cleaned. Maintaining clean slots is important since the bond between repair material and slots is critical to providing good load transfer. Joint caulking is necessary to prevent patch material from entering the joint.



Figure 10-20. Removing concrete fins.



Figure 10-21. Sandblasting the slots.



Figure 10-22. 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 10-23 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 repair mortar to surround the dowels, and should fit snugly in the slot to keep dowels properly aligned.



Figure 10-23. Dowel bar in place for backfill material application.

5. Placing repair mortar in the slots

Once the dowels are placed, the repair mortar is applied. The repair mortar should have thermal properties similar 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 prepackaged mixes have been used successfully. High-early-strength concrete usually contains Type III cement and accelerators. Accelerators decrease set times and increase rate of strength development. Aggregates in the mix should be small enough to allow the concrete to flow around the bar and consolidate properly. Consolidation of the repair mortars is done with a small spud vibrator. Care should be taken not to hit the dowels with the vibrator, knocking the dowel out of alignment.

Once the repair mortar is applied, the surface is finished flush with the surrounding surface. Curing compound should be applied as soon as practical. Prepackaged repair materials should be cured according to the manufacturer's recommendations. Sawing is done over the joint reformer, and the entire project is diamond ground, with resealing the joints as a final step. Figure 10-24 shows the section shown in Figure 10-17 after DBR, slab jacking, and diamond grinding (DG). Refer to Section 9 about the details of diamond grinding.



Figure 10-24. After DBR, slab jacking, and diamond grinding of the section shown in Figure 10-17.

Section 8: Joint Repair

8.1 Introduction

In PCC pavements, joints are provided to accommodate concrete volume changes due to temperature and moisture variations. In CPCD, both transverse and longitudinal joints are used. Only longitudinal joints are used in 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 the weakest areas in PCC pavement. There is a discontinuity in the 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 during high seasonal/daytime temperatures 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 10-25.



Figure 10-25. 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 10-26 illustrates a misaligned dowel bar.



Figure 10-26. Misaligned dowel bar.

8.2 Pavement Distresses that Require Joint Repairs

In CPCD, any cracking, breaking, or spalling of slab edges, on either side of a transverse joint, need to be repaired. The repair can be full-depth repair (FDR) or half-depth repair (HDR), 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 10-27. Any repair of the transverse construction joints requires FDR.



Figure 10-27. 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.

8.3 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

9.1 Introduction

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 has not been extensively used by the department 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, to achieve ride quality specifications. 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 10-28.



Figure 10-28. 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 quiet.

9.2 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. 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 life. 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.

9.3 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. The department 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 department developed special specifications for diamond grinding.

Section 10: Thin HMA Overlays

10.1 Introduction

The service life of an older existing CRCP pavement can often be extended many years by the addition of a thin hot-mix asphalt (HMA) overlay, approximately 2 in. thick. This treatment can be applied to pavements that are beginning to show deterioration, are getting rough, are experiencing an increasing number of punchouts, or have experienced a loss of skid numbers.

Thin HMA overlays can dramatically improve the skid resistance of concrete pavements that are still in very good structural condition. The PFC hot-mix is an excellent choice for improving skid resistance and reducing the potential for hydroplaning.

10.2 HMA Overlay on CRCP

Punchouts and loss of skid resistance have been treated successfully through the use of thin HMA overlays. 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 HMA helps keep water from penetrating to the base. These reasons may be true, but it is essential to keep surface water from infiltrating into a CRCP that has 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 HMA 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 base has greatly contributed to the formation of the punchouts. For the overlay to be successful, the underlying factors causing the distress need to be treated. 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.

10.3 HMA Overlay on CPCD

HMA overlays have also been used on structurally deficient CPCD. However, the success of those overlays has been marginal to poor because they are an expensive treatment with a relatively short service life. CPCD that has experienced pervasive transverse contraction joint failures with faulting require thicker HMA. As a rule of thumb, reflective cracking progresses at a rate of 1 in. per year. The loss of load transfer at the transverse joints results in independent movement of the CPCD slabs, which causes a crack to form in the HMA that reflects through to the surface. HMA spalls from either side of the joint. If the joint in the CPCD is open wide enough, two cracks will reflect through, one from each side of the joint. Large pieces of HMA can break off and leave an opening above the joint. Rubblization of the existing concrete pavement combined with a thick

HMA 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 has 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 base. Construction procedures for retro-fitting a pavement with concrete shoulders are identical to procedures for widening an existing concrete pavement. For more information refer to Chapter 9, Section 7.

Chapter 11: Ride Quality

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Section 1: Overview

Ride quality has been a fundamental concern of highway users since the earliest days of modern era highway construction. From the vehicle operator's perspective, the concern is predominantly functional (comfort and safety oriented), but, indirectly, there is a component of ride quality that eventually affects the structural performance of the pavement. Smooth pavement mitigates the magnitude of dynamic wheel loading; the smoother the pavement, the lower the dynamic loading and the resulting harmful responses to loading (deflections, stresses, and strains) within the structure. Ride quality has been integrated into the department's highway construction procedures as a Standard Specification Item since 1993. The ride quality goal is to begin a pavement structure's performance period (see Chapter 2, Fig. 2-12) at a high level of smoothness, and maintain it at an acceptable level throughout the design life by judicious use of pavement preservation and rehabilitation efforts. Pavements that begin their performance life in a very smooth condition tend to maintain acceptable smoothness over a longer period of time, all other things being equal. To assess ride quality, a longitudinal profile is evaluated in both wheel paths. The pavement profile is a measure of the variation of elevation differentials of the pavement surface from a reference plane.

Section 2: Ride Quality Measurement

2.1 Introduction

The International Roughness Index (IRI) is the statistic used to measure how smooth or rough a pavement surface is. 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/mi, 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. A relative appreciation for qualitative smoothness descriptions versus numerical values of IRI is given in Table 11-1.

Table 11-1: FHWA IRI Categories

Roughness Category	IRI Value
Good	≤ 95
Acceptable	≤ 170
Unacceptable	> 170

2.2 Equipment for Measuring Ride Quality

Item 585 specifies two methods for measuring surface profile: Surface Test Type A and Surface Test Type B. Surface Test Type A typically uses a 10-ft. straightedge, but may use high-speed or lightweight inertial profilers where allowed. Surface Test Type A is intended for use on short projects (< 2,500 ft.) and specific pavement segments, such as ramps, leave-outs, etc. Surface Test Type B requires the use of a high-speed or lightweight inertial profiler and is the default method for measuring surface profiles. Inertial profilers must be certified annually for use on department projects in accordance with Tex-1001-S. Profiler operator certification is required every 3 years; operator testing guidelines that include both a written and practical examination are outlined in Tex-1001-S.

The inertial profiler is a device that uses laser sensors and accelerometers to measure the profile of the pavement surface. There are generally three types of inertial profilers:

- one that can travel at highway speeds, a high-speed inertial profiler,
- a lightweight inertial profiler that can travel at a minimum speed of 12 mph, and
- a portable profiler.

Figure 11-1 and Figure 11-2 are pictures of a high-speed inertial profiler; Figure 11-3 is an example of a lightweight profiler. Figure 11-4 shows a picture of a portable profiler. Portable profilers are typically mounted on a trailer towing hitch and can be moved between housing vehicles or shipped independently of the housing vehicle for use on jobs in another geographical

location. However, the system (housing vehicle and portable profiler) must be certified as a unit in Texas.



Figure 11-1. High-speed Inertial Profiler.



Figure 11-2. High-speed Inertial Profiler with Rut Bar Setup.



Figure 11-3. Lightweight Profiler. © Copyright 2006 University of Washington



Figure 11-4. Portable Profiler. (Courtesy International Cybernetics Corp.)

Section 3: Ride Requirements for Flexible Base

For pavement sections where the final riding surface is a 1- or 2-course surface treatment on flexible base, the ride quality criterion established in Item 247 limits the base surface roughness for each 0.1-mi section to no more than 100 in/mi. for each wheel path unless otherwise shown on the plans. There is no pay adjustment schedule. Areas exceeding the requirement must be corrected unless otherwise shown on the plans.

Section 4: Surface Test Type A for Concrete and Hot-Mix Asphalt Surfaces

Surface Test Type A is the default for service roads and ramps, bridge structures, short projects, leave-out sections (such as driveways, intersections, etc.), ends, shoulders, and other areas, including intermediate layers. Use a general note if you want to use surface Test Type B rather than Surface Test Type A for all except leave-out sections and ends. There is no pay adjustment schedule for Surface Test Type A. Areas exceeding the requirement must be corrected and retested.

Section 5: Surface Test Type B for Concrete and Hot-Mix Asphalt Surfaces

Surface Test Type B is the default for the final riding surface of travel lanes with the exceptions noted in Section 5 above. The ride quality specification for concrete and hot-mix asphalt surfaces includes incentives and disincentives based on the contractor's performance in constructing a smooth surface. One of three pay adjustment schedules in Item 585 is selected by the engineer to determine the level of bonus or liquidated damages for each 0.1-mi. section for these projects. Pay adjustment schedules 1 and 2 have the same bonus schedule; pay adjustment schedule 3 has a lower bonus schedule than pay adjustment schedules 1 and 2. The liquidated damages are greater for schedule 1, less for schedule 2, and there are no liquidated damages when schedule 3 is selected.

Item 585 specifies Surface Test Type B with Pay Adjustment Schedule 3 as the default. However, Schedule 1 and 2 could be more appropriate for many construction scenarios. Use the guidelines in Table 11-2 and engineering judgment to determine the appropriate Pay Adjustment Schedule based on what is achievable. Factors to consider include the existing IRI, class of the road, posted speed, previous experiences on similar projects, the ability to improve the existing ride with the number of smoothness opportunities specified, and the need for higher ride quality. General notes or notes on typical sections are required to change the Pay Adjustment Schedule. It is strongly advised that the designer show on the plans which pay schedule they wish to use. Note that Table 11-2 does not cover all possible construction scenarios. For further assistance, please contact MNT – Pavement Asset Management.

Table 11-2: Guidance for Selecting Pay Adjustment Schedules

Project Description				Recommended Pay Adjustment Schedule
New Construction or Major Rehabilitation (IH, US, Multilane divided highways)	Rigid Pavements	CRCP, CPCD and Unbonded Concrete Overlay		2
		Bonded Concrete Overlay		3
		All roads with posted speed < 45 MPH		3
	Flexible Pavements with total HMA thickness > 1.5"			1
Overlay or Minor Rehabilitation	Flexible Pavements with total HMA thickness < 1.5" such as an overlay with a Permeable Friction Course (PFC). Note that in some cases Surface Test Type A may be more appropriate for this application.			3*
	Flexible Pavements Total HMA thickness > 1.5"	All roads with posted speed < 45 MPH		3*

Table 11-2: Guidance for Selecting Pay Adjustment Schedules

Project Description				Recommended Pay Adjustment Schedule
		When there are 2 or more smoothness opportunities (see Section 7)	All highway classifications other than 2-lane undivided	1*
			2-lane undivided highways	2*
		When there is only 1 smoothness opportunity (see Section 7)	All highway classifications other than 2-lane undivided	2*
			2-lane undivided highways	3*

* It may be appropriate to increase or decrease this number depending on the ride quality of the existing pavement. For example: if the ride quality of the existing pavement is poor (IRI > 170), it may be appropriate to increase this number if applicable. Conversely, it may be appropriate to decrease this number if applicable and if the ride quality of the existing pavement is good (IRI < 95).

The pay adjustment schedules listed in Item 585 are shown graphically in Figure 11-5.

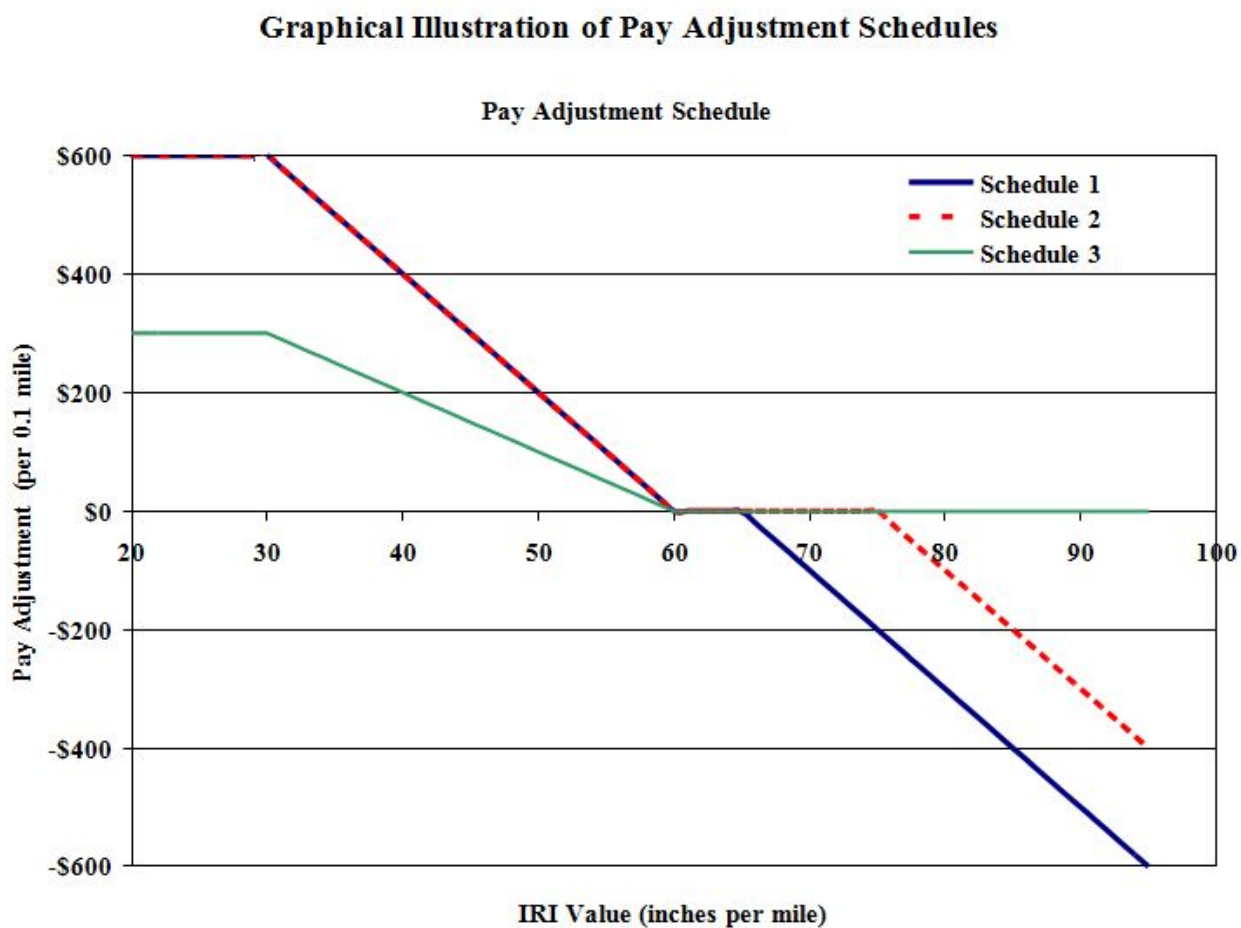


Figure 11-5. Graphical Illustration of Pay Adjustment Schedules.

Section 6: Considerations for Improving Ride Quality

As a general rule, the roughness (IRI value) can be reduced approximately 50% with each lift of hot-mix; however, there is a point of diminishing returns once the IRI values get below 60. Typically, an IRI value less than 60 is considered and an IRI greater than 95 requires corrective action. Note that the most recent international roughness index (IRI) values are stored in the department's Pavement Management Information System (PMIS) PA database. It is recommended that these values be obtained early in the decision-making process.

Smoothness opportunities shown in Table 11-2 are defined as a continuous level-up regardless of the thickness, a specified lift of 1.0 in. or more of HMA, in-place recycling, and motor-grading flexible base courses. Spot level-ups, milling operations, and seal coats will not be considered as smoothness opportunities. Mill and fill operations that require matching the existing pavement are not considered smoothness opportunities.

Note that diamond grinding is the default method (on both flexible and rigid pavements) for removing localized roughness (bumps and dips). There are several exceptions to the requirement for addressing localized roughness. These exceptions are spelled out in detail in Item 585.

In some cases where only a single lift of hot-mix is specified, it may be advantageous to diamond grind some of the larger bumps and dips prior to the hot-mix overlay. In such cases, diamond grinding should be set up as a separate bid item and the roadway should be profiled in advance to identify the existing bumps. Note that diamond grinding is an effective method of removing bumps, yet somewhat less effective at removing dips.

On projects that have 3 or more proposed lifts of hot-mix, the designer should consider adding a plan note requiring the contractor, at his own expense, to profile the pavement and diamond grind areas of localized roughness prior to placing the final lift of hot-mix.

Section 7: Analysis of Ride Data

Ride data is collected in accordance with Tex-1001-S. The ride data is analyzed using TxDOT's [Ride Quality computer program](#). The analysis output includes a description of the project and profile summaries as follows:

District, county, highway, beginning reference marker, end reference marker, CSJ, lane designation, IRI values every 0.1 mi., pay, locations of bumps or dips, and total pay adjustment.

A sample output for a hot-mix asphalt surface profile follows:

Ride Quality Analysis Rel 2013.01.03

TxDOT Smoothness Specification 585 Pay Schedule 1

Report run on Tuesday Nov 17 2015 9:30:50AM

Input profile data file created Wednesday Jul 25 2012 6:06:56AM

District 14 Highway SL1PFV1
 Area Office TRAVIS Beg RM 0434 +00.000
 County 227 Beg Station 0000+00.0
 CSJ 1234-34-333 Lane roadbed R1
 Phone 000-000-0000 Name jeff
 Input file c:\txdot\engapps\rideqc\sampladata\sampleprofile.pro

Bump penalty fine -\$500 each one.
 Bumps tagged if greater or equal to 0.1505 inches.
 Bonus paid for average IRIs of 30(\$600) to 60(\$0)
 Penalty assessed for average IRIs of 65(\$0) to 95(-\$600)
 Bonus paid in sections with bump.

Profile Length(Miles) 2.0506 Length(Station Units) 0108+27.2ft.

Distance	Station	Type	Width(feet)	Elev(inches)	Penalty
01.1080	58+50.2	Dip	2.7	-.18	-\$ 500
01.1735	61+96.1	Dip	.4	-.15	-\$ 500
01.3402	70+76.3	Bump	.4	.16	-\$ 500
01.3413	70+81.9	Bump	.2	.15	-\$ 500
Bumps/dips detected 4					Bump Fines -\$ 2000

Separate penalties for
 localized roughness
 per Item 585, pay
 schedule 1

***Notes: Bumps closer than 5 feet are not shown or counted.**

**Bump widths shown are not a true indication of the actual bump width but rather
 the portion of the bump exceeding 0.1505 inches from the moving average of the profile.**

Bumps not calculated on margins of profile where filtering is tapered on and off.

Distance	Station	PSI	IRI(L)	IRI(R)	Avg IRI	Pay*SectLen	Pay
00.1000	5+28.0	4.38	59.51	70.84	65.00	\$ 0*(0.1000/0.10)	\$ 0
00.2000	10+56.0	4.22	69.22	75.52	72.00	-\$140*(0.1000/0.10)	-\$ 140
00.3000	15+84.0	4.62	54.90	56.96	56.00	\$ 80*(0.1000/0.10)	\$ 80
00.4000	21+12.0	4.08	73.15	84.38	79.00	-\$280*(0.1000/0.10)	-\$ 280
00.5000	26+40.0	4.68	47.52	59.68	54.00	\$120*(0.1000/0.10)	\$ 120
00.6000	31+68.0	4.69	47.67	58.35	53.00	\$140*(0.1000/0.10)	\$ 140
00.7000	36+96.0	4.74	47.48	52.42	50.00	\$200*(0.1000/0.10)	\$ 200
00.8000	42+24.0	4.63	55.47	55.38	55.00	\$100*(0.1000/0.10)	\$ 100
00.9000	47+52.0	4.17	74.25	75.05	75.00	-\$200*(0.1000/0.10)	-\$ 200
01.0000	52+80.0	4.40	56.24	72.60	64.00	\$ 0*(0.1000/0.10)	\$ 0
01.1000	58+08.0	4.68	49.61	57.38	53.00	\$140*(0.1000/0.10)	\$ 140
01.2000	63+36.0	3.65	108.24	93.45	101.00	-\$	Corrective Work
01.3000	68+64.0	4.49	67.04	54.30	61.00	\$ 0*(0.1000/0.10)	\$ 0
01.4000	73+92.0	3.85	85.25	94.24	90.00	-\$500*(0.1000/0.10)	-\$ 500
01.4803	78+16.0	4.31	70.50	65.63	68.00	-\$ 60*(0.0801/0.10)	-\$ 48
Pay Adjustment Subtotal							-\$ 388

Penalties in
 red per Item
 585, pay
 schedule 1

Avg. > 95
 in/mi

01.4803 78+15.8 *** Leave Out Start
 01.5142 79+94.7 *** Leave Out End Total Leave Out = 179.0ft

Distance	Station	PSI	IRI(L)	IRI(R)	Avg IRI	Pay*SectLen	Pay
01.6000	84+48.0	4.19	69.61	77.25	73.00	-\$160*(0.0861/0.10)	-\$ 138
01.7000	89+76.0	4.19	73.38	73.83	74.00	-\$180*(0.1000/0.10)	-\$ 180
01.8000	95+04.0	4.49	60.04	61.89	61.00	\$ 0*(0.1000/0.10)	\$ 0
01.9000	100+32.0	4.17	72.49	76.29	74.00	-\$180*(0.1000/0.10)	-\$ 180
02.0000	105+60.0	4.16	78.08	71.87	75.00	-\$200*(0.1000/0.10)	-\$ 200
02.0505	108+26.6	4.20	76.47	69.48	73.00	-\$160*(0.0502/0.10)	-\$ 80
Pay Adjustment Subtotal							-\$ 778
Ave Left IRI 66.2		Ave Right IRI 69.4		Ave IRI 67.8			
Total IRI adjustments							-\$ 1166
Total Bump adjustments							-\$ 2000
Total adjustments							-\$ 3166

Total Penalties

Figure 11-6. Sample Ride Quality Software Output.

Chapter 12: Premature Distress Investigations

Contents:

[Section 1: Overview](#)

[Section 2: Investigation Team](#)

[Section 3: Investigation Process](#)

Section 1: Overview

1.1 Introduction

Premature pavement failures typically occur within the first half of a rigid pavement's design life and within 5 years after completion of the project for flexible pavements. Premature pavement failures can be attributed to any number of factors or combination of those factors, including inadequate structural design or incorrectly addressing existing structural deficiencies, inadequate consideration of design compatibility with the abutting pavement, poor quality or non-uniform materials, inadequate surface preparation, or poor construction. Despite advancements in pavement technology in past decades, premature failures and chronic pavement distresses continue to occur. Experience has shown that the majority of premature pavement failures in Texas are related to material and construction deficiencies. Material deficiencies can be related to inadequate assessment of existing materials and what measures are needed to enhance their attributes for better performance. Although improvements have been made to construction specifications, equipment, and construction processes, poor quality construction or use of substandard materials can occur due to a number of complex and sometimes competing variables, such as:

- reduced inspection staffing,
- employee turnover,
- fluctuations in levels of experienced inspectors, project managers and engineers,
- incompatibilities between new admixtures and construction materials,
- implementation of new technologies, construction methods, and best practices,
- environmental constraints and recycled materials,
- issues unforeseen during design and construction phases.

To prevent or reduce the probability of premature pavement distress 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 distress. A pavement distress investigation involves a thorough review and analysis of existing construction quality control records and tests, and nondestructive testing such as ground penetrating radar (GPR), falling weight deflectometer (FWD), and dynamic cone penetration testing (DCP). These are essential to identify problematic areas and probable causes. Additional field testing, such as portable FWD, dirt seismic pavement analyzer (DSP), coring, trenching, and laboratory testing, may be conducted to confirm the initial hypothesis. The outcomes from investigations can be used to validate or modify the existing design plan and to resolve the disputes involving construction claims or change orders.

1.2 Technical Assistance

For technical assistance requests, contact the Pavement Asset Management Section of the Maintenance Division. Requests for technical assistance are best made as early as possible after

becoming aware of the distress to allow opportunity to correct problems if the project is still under construction, prevent further pavement distress, obtain samples of materials actually used on the project for lab testing, and to allow time for appropriate remediation.

Section 2: Investigation Team

2.1 Objectives

An investigation team provides technical assistance to the districts and divisions to determine the causes of and recommend solutions to premature or recurring pavement distresses. The investigation team's involvement is strictly by request. The team will act as a consultant to the districts and divisions.

The investigation team's objectives are to:

- enhance the district/division's problem-solving capabilities,
- identify, document, and communicate causes of premature pavement distresses so that they may be prevented in the future, and
- provide remedial solutions for premature or recurring pavement distresses.

The director of MNT – Pavement Asset Management will serve as the Premature Distress Investigation Team Coordinator. The director is responsible for:

- assigning project leader and team members to specific projects, and
- reviewing progress on individual projects and ensuring timely completion.

Section 3: Investigation Process

3.1 Overview

Responsibilities for MNT – Pavement Asset Management and the requesting district/division will be determined during the preliminary project meeting. Table 12-1 describes the investigation process.

Table 12-1: Investigation Process

Step	Required Action
1	Conduct preliminary project meeting and perform on-site investigation. The district should provide the following information: • District/division contact(s), • 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.), and • Soil and geologic information.
2	Formulate action plan for detailed investigation based on background information, the preliminary meeting, and on-site investigation. Detailed condition survey may include: • Surface distress condition, • Ground Penetrating Radar, • Dynamic Cone Penetration, • Falling Weight Deflectometer (FWD), • Total Pavement Analysis Device (TPAD), • Core Rig, • Materials sampling and testing, and • Other testing as required.
3	Coordinate the following activities: • Review and analysis of relevant project records. Sampling and laboratory testing of materials. • Traffic control. • Equipment and personnel to acquire material samples. • Equipment and personnel for repair of cored or trenched sections.
4	• MNT – Pavement Asset Management will analyze data and identify most likely causes of problem.

Table 12-1: Investigation Process

Step	Required Action
5	MNT – Pavement Asset Management will present findings/recommendations to the district through a technical report and a presentation. The report will consist of the following: • Project history and background, • Review of project records, • 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 distress mechanism, and • Project recommendations prioritizing corrective strategies and necessary changes to process and procedures.
6	• Monitor project subsequent to corrective measures to assess effectiveness, and • Provide periodic feedback to MNT – Pavement Asset Management regarding the effectiveness of the repair strategy.

If you have any questions, need additional information, or copies of investigation reports, contact the director of MNT – Pavement Asset Management.

Chapter 13: Load Zoning and Super Heavy Load Analysis

Contents:

[Section 1: Overview of 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 of Load Zoning

1.1 Background

There are approximately 16,000 miles of load-restricted highways in Texas. These facilities were generally constructed prior to the late 1950s and were designed for lighter axle configurations and wheel loads than are currently allowed by law. In addition, environmental effects have weakened some structures.

The Maintenance Division, Pavement Asset Management Section, manages a database for load-restricted highways. The load-restricted highways can be accessed through the following link: <http://www.txdot.gov/business/resources/construction/load-zoning.html>

The Bridge Division manages a database for load-restricted bridge structures. Refer to <http://apps.dot.state.tx.us/apps/gis/lrbm/> for the load restricted bridge application that includes both on and off-system load restricted bridges.

1.2 Executive Orders

In an effort to protect facilities that were originally designed for lighter wheel loads from accelerated deterioration, a provision exists in the Texas Transportation Code §621.102. The law authorizes the executive director of the department to set load limits. The executive director must accomplish all actions to revise, post, or remove postings from restricted facilities by issuing a legal document called an Executive Order. MNT – Pavement Asset Management is responsible for preparing and submitting, to the executive director, proposed Executive Orders involving load zoning.

1.3 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

2.1 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. Refer to the step-by-step instructions in “Table 13-1: Changing Load Zones on Roads” for adding a load zone.

If the deficiency is severe enough to cause potential for accident or injury, then an emergency posting for 120 days may be utilized. See Section 3 for procedures to implement an emergency Executive Order. If repairs or upgrades are not anticipated for more than 120 days, restrictions must be authorized and promulgated by a subsequent "permanent" Executive Order.

2.2 Removing

Removing a load zone may be requested if rehabilitation or reconstruction has been performed, or the load-zoned road 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 shall be made by Executive Order. 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 13-1: Changing Load Zones on Roads” for removing a load zone.

2.3 Changing

The following table lists the steps, responsible party, and required action for changing load zones on roads:

Table 13-1: 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, subgrade soil lab test reports, or other pertinent information, such as a copy of the pavement design 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 Maintenance Division, Pavement Asset Management Section.
2	MNT – Pavement Asset Management	Analyzes data, estimates the remaining life of the pavement section, and makes recommendation to district.

Table 13-1: Changing Load Zones on Roads

Step	Responsible Party	Required Action
3	District	Makes decision on load requirements based on recommendation from MNT – Pavement Asset Management.
4	MNT – Pavement Asset Management	- Prepares Executive Order for executive director approval; - Notifies by email affected district upon executive director approval of load zone revision. A copy of approved Executive Order will accompany district notification; - Updates the publicly accessible load zone map.
5	District	Erects or removes signs consistent with the proper load limits once approved Executive Order has been received.
6	MNT – Pavement Asset Management	Provides consultation to the Department of Motor Vehicles for the issuance of permits.

Section 3: Emergency Load Zones on Roads

3.1 Setting Emergency Load Zones on Roads

The district shall notify the Maintenance Division, Pavement Asset Management Section, by telephone or email 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 13-2: 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	MNT – Pavement Asset Management	- Evaluate information received from district; - Prepares letter for the director of MNT authorizing the emergency load restriction for 120 days (director can also authorize 120-day extension of emergency load zone restriction, if necessary); - Notifies district and the Department of Motor Vehicles, Motor Carrier Division (MCD), of approval; - Updates the publicly accessible load zone map.
3	District	Erects signs indicating the emergency load limits once approval from the director of MNT has been received.
4	MNT – Pavement Asset Management	Provides consultation to MCD as needed.

Section 4: Changing Load Zones on County Roads and Bridges

4.1 Law Ruling

Texas Transportation Code §621.301 requires counties to obtain TxDOT concurrence for proposed changes to 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 Executive Order.

4.2 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 shall take when a county submits a proposed change in load limits to the district engineer.

Table 13-3: 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 13-4: Required Information and Supporting Documentation."
3	Evaluate the request. • If the district does not have the resources or expertise to perform the review, contact MNT 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 13-3: 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.

4.3 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 13-4: Required Information and Supporting Documentation

Roads	Bridges
- A cover letter that includes: - Name, phone number, email address for a county contact person. - Route name/number, location including limits of proposed load zoning, 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. - Date of the last rehabilitation or reconstruction performed on the road. - Description of pavement typical sections and section limits, including layer thickness, material types, lane and shoulder widths, and curb and gutter locations, if applicable. - Engineering Analysis - Supporting documentation that has been sealed by a professional engineer. The engineering analysis shall follow one of the methods described below.	- 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 <i>AASHTO Manual for Condition Evaluation of Bridges</i>, Chapter 6.

Engineering Analysis

An engineering analysis is required for load posting roadways. The analysis and recommendations are required to be signed and sealed by a professional engineer. The analysis must demonstrate that the recommended gross vehicle weight or limiting axle load group is expected to preserve the pavement structure. TxDOT recommends a remaining life of approximately 10 years; however, the analysis may result in less when accompanied by supporting justification.

At a minimum, an engineering analysis must include the following information and analysis supporting the load zoning recommendation:

- Location map
- Pavement visual distress and condition evaluation
 - Extent (location and severity)
 - Distress analysis (identification and causes)
- Traffic load study data supporting a 10-year analysis including:
 - Average Annual Daily Traffic
 - Percent Trucks
 - Equivalent Single Axle Loads
- Characterization of pavement layers
 - 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 appropriate for pavement analyses.
- Analysis of pavement structure and proposed load limit determination using one or more of the following methods:
 - Mechanistic analyses (determination of maximum allowable load using stress and strain calculations)
 - Analyses of deflection data using a Falling Weight Deflectometer or other similar measurement equipment
 - The Texas triaxial method by classification of soil presented in “Determining Allowable Pavement Load Limit Using the Texas Triaxial Classification Procedure” at http://ftp.dot.state.tx.us/pub/txdot-info/cst/county_load_zone.pdf.
 - Other appropriate analytical methods based on accepted engineering practice

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. Some of the loads 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 super heavy loads must be permitted through the Motor Carrier Division (MCD) of the Texas Department of Motor Vehicles. Empirical information obtained during field studies of super heavy load moves has provided the basis for the current evaluation threshold criteria established by the Maintenance Division, Pavement Asset Management Section.

Super heavy loads with GVW above 500,000 lb. and/or maximum trailer tire load above 6,000 lb. are analyzed by MNT – Pavement Asset Management. The super heavy load analysis threshold criteria for evaluation were 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 MNT – Pavement Asset Management. Emphasis has been placed on analyzing super heavy loads with a greater potential for pavement damage. As depicted in Figure 13-1, 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 trailer tire loads greater than 6,000 lb. when the GVW is less than 500,000 lb.

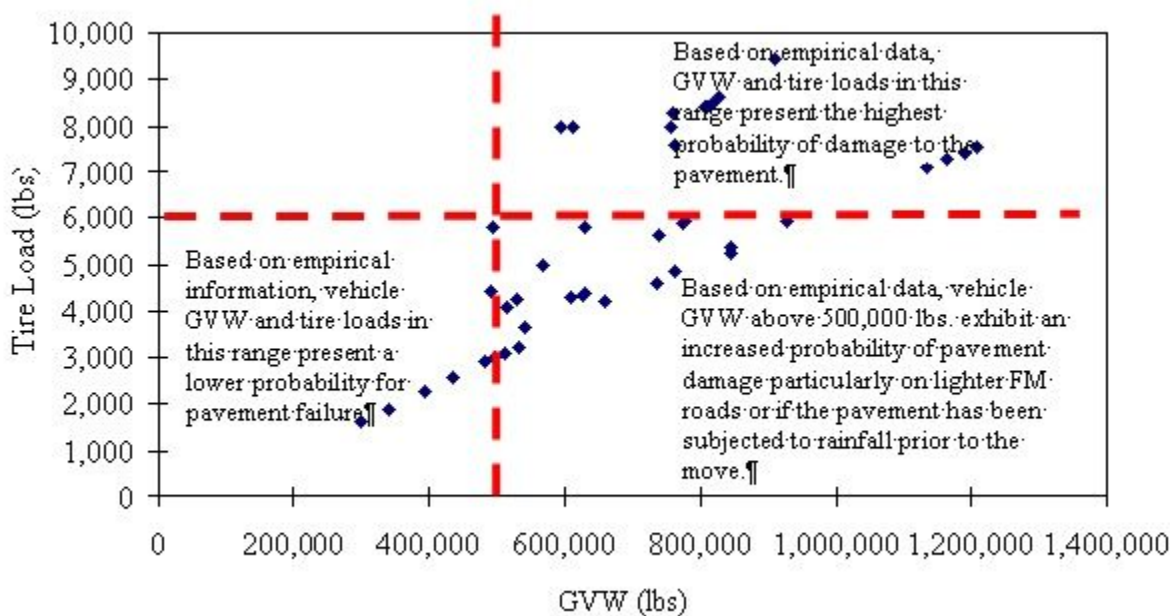


Figure 13-1. General relationship in gross vehicle weight (GVW) and increased trailer tire load based on loading diagrams.

Flexible pavements, which constitute approximately 93% of the Texas 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 13-2, as

tire loads increase, 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.

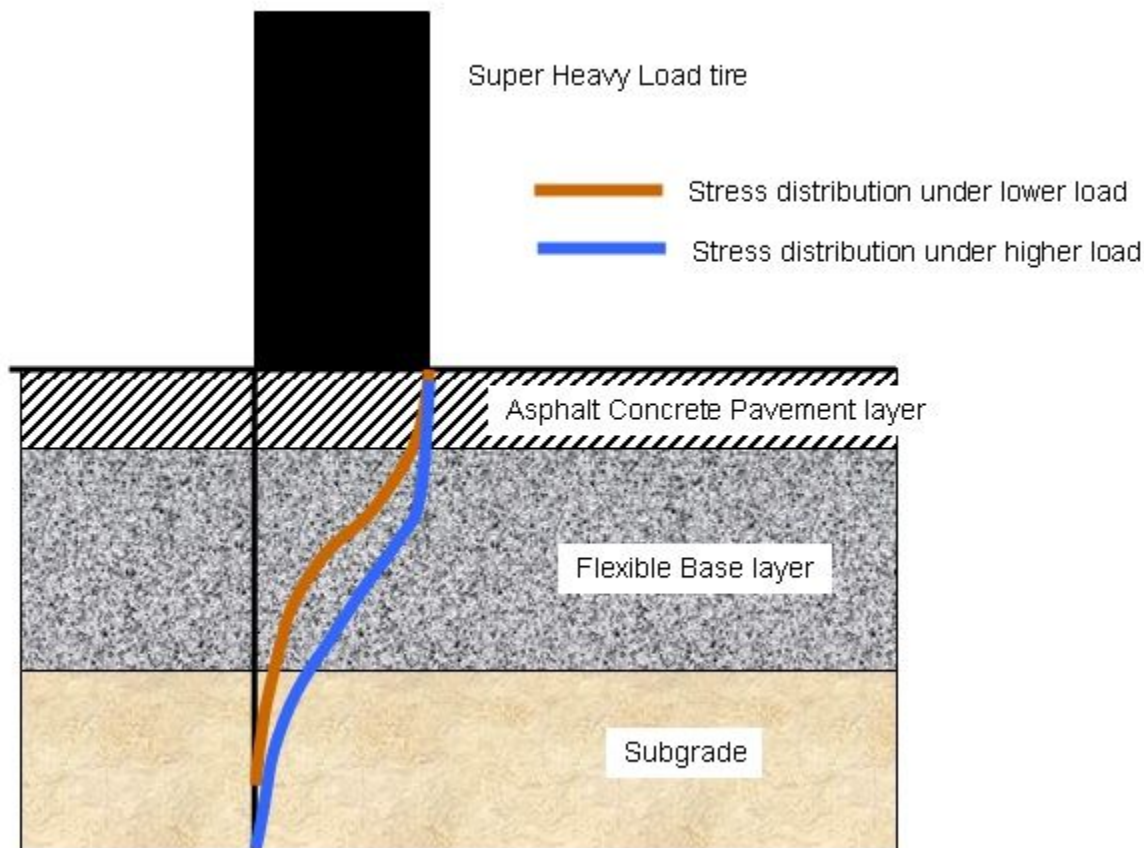


Figure 13-2. Stress distribution within a flexible pavement structure for a low and high tire load.

As mentioned previously, the trailer tire load limit of 6,000 lb. was established based on empirical information related to observed distress which has 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 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 also damage (by peeling up) new pavement surfaces (overlays and chip seals) that are less than 5 weeks old. It is the district's decision to place a "no travel" restriction for newly placed seal coats 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

super heavy 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. The Texas Department of Motor Vehicles, 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, spraying water or some other method to reduce the possibility of seal coat damage may be considered.

Upon receiving an email from MCD or MNT – Pavement Asset Management, the district pavement engineer will have three business days to alert MCD or MNT – Pavement Asset Management if a reroute is needed. MCD and MNT – Pavement Asset Management maintain a contact list provided by districts on who should be notified. If, after three business days, no reroute requests have been sent to MCD or MNT – Pavement Asset Management, the route will be considered approved and the permit application will continue to be processed. The exact date and time of the super heavy load move is difficult to know due to numerous factors. District personnel may take a proactive role and contact the carrier to obtain the date(s) of movement. For super heavy loads with a GVW over 500,000 lb., MCD requires the carrier to notify district personnel 48 hours prior to the move. Districts need to provide contact information for at least three of their employees to MCD and MNT – Pavement Asset Management to keep the list updated.

It is 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 section employee can check the axle configuration to ensure that it 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

The Texas Department of Motor Vehicles, Motor Carrier Division (MCD), will create a proposed route for super heavy loads with GVW over 500,000 lbs. or for tire loads exceeding 6,000 lbs. MCD then submits the proposed route to the customer for review. If the customer accepts the proposed route, MCD submits a letter, including the proposed route, load configuration diagrams, and proposed move dates, and control section numbers to MNT – Pavement Asset Management. MNT – Pavement Asset Management reviews proposed route using the data and information supplied by MCD and evaluates the pavement condition using PMIS distress and condition scores. The proposed route is submitted to the affected TxDOT districts for review. The districts are given three business days to complete their review. MNT – Pavement Asset Management may conduct additional testing as needed, using data supplied by the affected district (pavement typical sections, visual distress evaluation, FWD deflections, and construction information). Based on the analysis, MNT – Pavement Asset Management provides MCD recommendations on the proposed route. Figure 13-3 shows the flow chart of the super heavy load evaluation process.

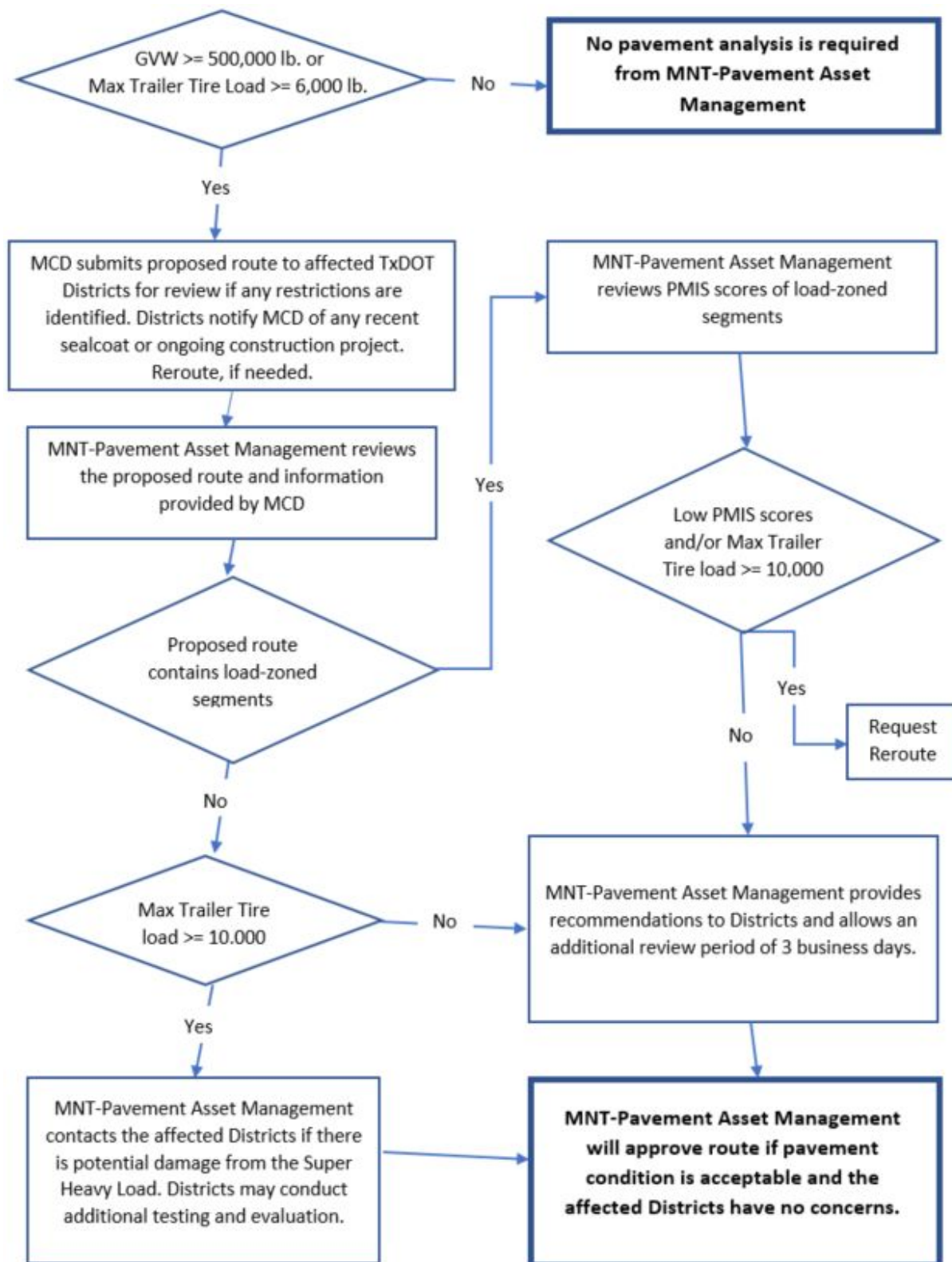


Figure 13-3. Flow chart showing the super heavy load evaluation process.

Section 7: Damage from Super Heavy Load Moves

The five typical types of damage related to super heavy load moves are given as follows:

1. Shearing of the pavement surface during turning movements (see Figure 13-4). The figure emphasizes 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, the mover must take effective measures to protect the surface at the turn.
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 (see Figure 13-5). 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 carrier must effectively protect the shoulders where the trailer is going to track.
3. Peeling of fresh seal coats or asphalt concrete pavement overlays (see Figure 13-6 and Figure 13-7). Moves should be avoided on fresh seal coats. If there is no other alternative, the mover can take effective measures to protect the pavement.
4. Bleeding of seal coats or asphalt concrete surfaces (see Figure 13-8).
5. Lateral shear failure at the pavement edge (see Figure 13-9 and Figure 13-10).



Figure 13-4. Shearing of the pavement surface during turning movements.



Figure 13-5. 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 13-6. Peeling of fresh seal coats (1st photo).



Figure 13-7. Peeling of fresh seal coats (2nd photo).



Figure 13-8. Bleeding of seal coats.



Figure 13-9. Lateral shear failure at the pavement edge (1st photo).



Figure 13-10. 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 that 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 10, "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 these actions:

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. The 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 1494¹ (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.

1. Available through the TxDOT intranet only.