Pavement Design Guide

Louisiana Department of Transportation and Development

2013

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Chapter I
Introduction

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Section 1
Guide Overview

1.1 - Purpose

The purpose of this guide is to provide a comprehensive review of pavement design for the Louisiana Department of Transportation and Development (DOTD), consultants, and contractors. This guide will cover the thickness design of new, reconstructed, and rehabilitated rigid and flexible pavements. This guide is expected to:

- serve as an extensive pavement reference that the users can use to look up typical values, methods, practices, and resources;
- provide approved policies and procedures for pavement design for use on DOTD projects;
- provide pavement engineers with a uniform, streamlined process for designing pavements;
- provide pavement engineers with a means for selecting the best performing materials, defining laboratory and field data for design input, and determining the best practices for data collection; and
- serve as guide for selecting rehabilitation strategies.

Section 1.2 - Organization of Contents

This guide is organized into eight chapters:

- Chapter 1, Introduction. This chapter provides an overview of the guide and contacts for pavement and material related topics.
- Chapter 2, Pavement Design Process. This chapter provides definitions of pavement types, basic elements of each pavement structure, and the purpose of each element .
- Chapter 3, Pavement Design Methods. This chapter discusses the approved methods of pavement design and pavement design categories.

- Chapter 4, Information Needed for Pavement Design. This chapter defines the various design inputs discussed in Chapter 3.

- Chapter 5, Geotechnical Investigation of Pavement Structure. This chapter provides laboratory testing of materials (resilient modulus) and non-destructive testing (dynaflect) of pavement structures for input in the AASHTO design equation.

- Chapter 6, Flexible Pavement Design. This chapter provides an overview of types of flexible pavements, approved design methods, typical values of input parameters, and thickness determination.

- Chapter 7, Rigid Pavement Design. This chapter covers the approved pavement design methods for rigid pavements, rigid pavement design process, recommended typical values, and thickness determination.

- Chapter 8, Asphaltic Concrete Overlay of Existing Pavement. This chapter discusses design methods for overlays of existing flexible pavements and rigid pavements with existing asphaltic concrete overlays (composite pavements).

Section 2
Contacts

Contacts for Questions and Comments

The design of pavement structure involves input from several DOTD sections. For specific questions and/or comments, contact the following:

- **Pavement and Geotechnical Design** at [(225) 379-1937] for general questions or comments about this guide.
- **Materials and Testing** at [(225) 248-4131] for issues regarding geotechnical investigation and material testing.
- **Pavement Management** at [(225) 379-4578] for issues relative to pavement testing and evaluation.
- **Traffic Engineering,** at [(225) 379-1924] for issues on traffic data.
- **Research, Training and Technology Implementation** at [(225) 767-9101] for research projects and their implementation as well as training courses on pavement design and evaluation.

Chapter 2
Pavement Design Process

Contents

Section 1
Pavement Types

1.1 Flexible Pavement

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

1.1.2 A typical flexible pavement structure consists of the surface course and underlying base (either stabilized or unstabilized) and subbase (as required) courses. Each of these layers contributes to structural support and drainage. When hot mix asphalt (HMA) is used as the surface course, it is the stiffest (as measured by resilient modulus) and may contribute the most (depending upon thickness) to pavement strength. The underlying layers are less stiff but are still important to pavement strength as well as drainage protection. When a seal coat, such as chip seal, is used as the surface course, the base is generally the layer that contributes most to the structural stiffness. A typical structural design results in a series of layers that gradually decrease in material quality with depth. Figure 2.1 shows a typical section for a flexible pavement.

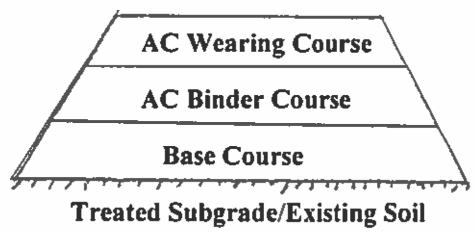

Figure 2.1: Typical Section for a Flexible Pavement* (1/)

1.2 Rigid Pavement

1.2.1 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.

* - Underlined numbers in parentheses refer to list of references on page 9-1

1.2.2 The surface course (concrete slab) is the stiffest layer and provides the majority of strength. The base or subbase layers are orders of magnitude less stiff than the PCC surface but still make important contributions to pavement drainage 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 may have reinforcing steel, which is generally used to handle thermal stresses to reduce or eliminate joints and maintain tight crack widths. Figure 2.2 shows a typical section for a rigid pavement.

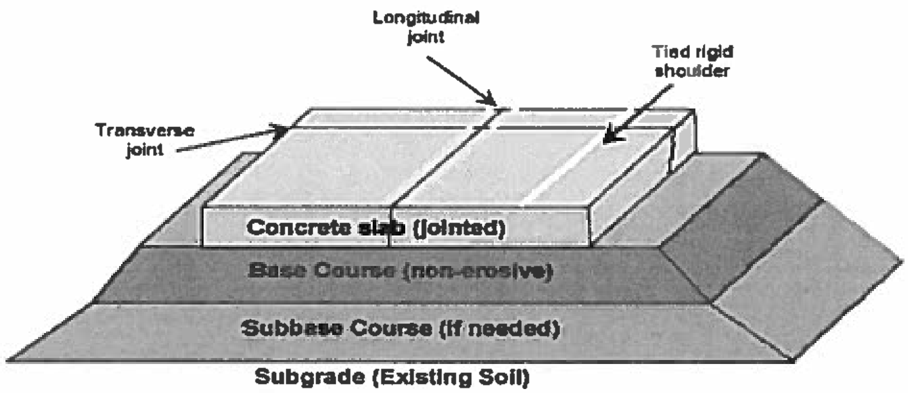

Figure 2.2: Typical Section for a Rigid Pavement (1/)

Although only jointed and continuously reinforced concrete are typically used in Louisiana, rigid pavement designs fall into three categories.

Jointed Concrete Pavement (JCP) Design

This design uses contraction joints to control cracking and does not use any reinforcing steel. 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 Louisiana.

Dowel bars are typically used at transverse joints to assist in load transfer. Tie bars are typically used at longitudinal joints. Figure 2.3 shows a typical section of JCP.

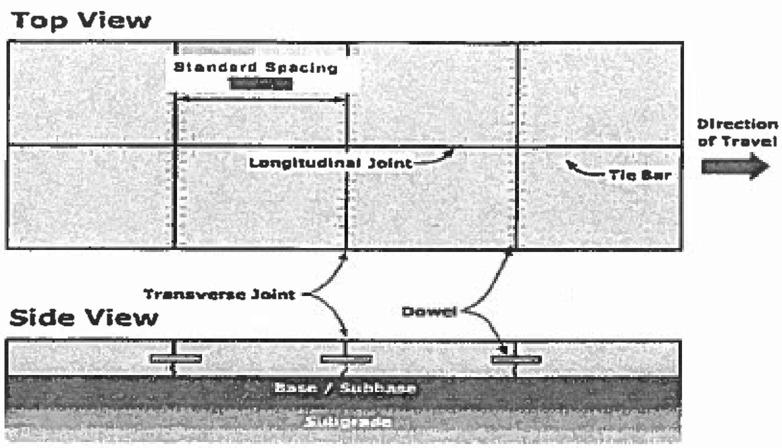

Figure 2.3: Jointed Concrete Pavement (1/)

Jointed Reinforced Concrete Pavement (JRCP) Design

Although DOTD does not typically use this design, it is presented for informational purposes.

JRCP uses contraction joints and reinforcing steel to control cracking. Transverse joint spacing is longer than that for JCP design, and it is typically 50 - 60 ft. Figure 2.4 shows a typical section of jointed reinforced concrete pavement.

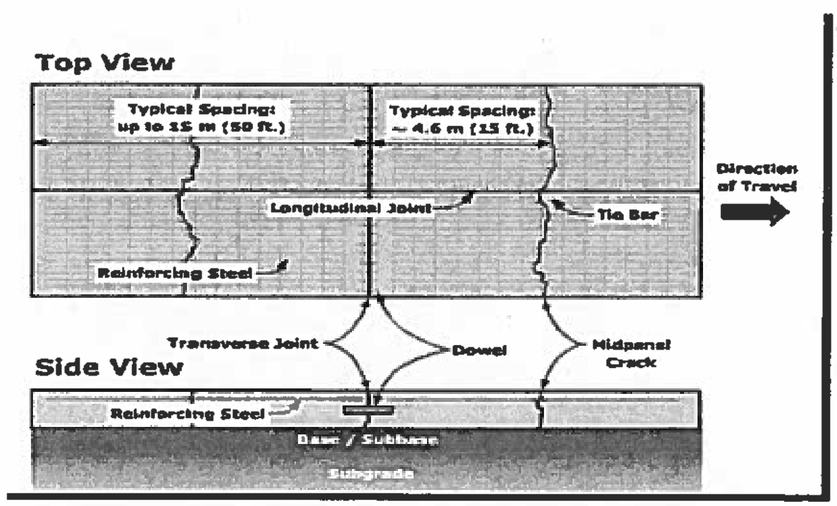

Figure 2.4: Jointed Reinforced Concrete Pavement (1/)

Continuously Reinforced Concrete Pavement (CRCP) Design

CRCP provides a 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 and do not spall excessively, and a uniform non-erosive base is provided.

Figure 2.5 shows a typical section of CRCP.

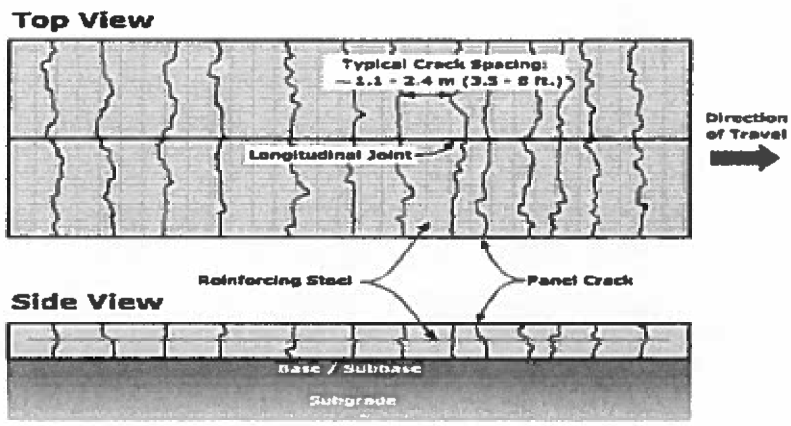

Figure 2.5: Continuously Reinforced Concrete Pavement (1/)

1.3 Composite Pavement

A composite pavement is composed of both hot mix asphalt (HMA) and 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 rehabilitation or safety treatment. Composite pavement behavior under traffic loading is essentially the same as rigid pavement.

1.4 Flexible and Rigid 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 2.6 shows load distribution for a typical flexible pavement and a typical rigid pavement.

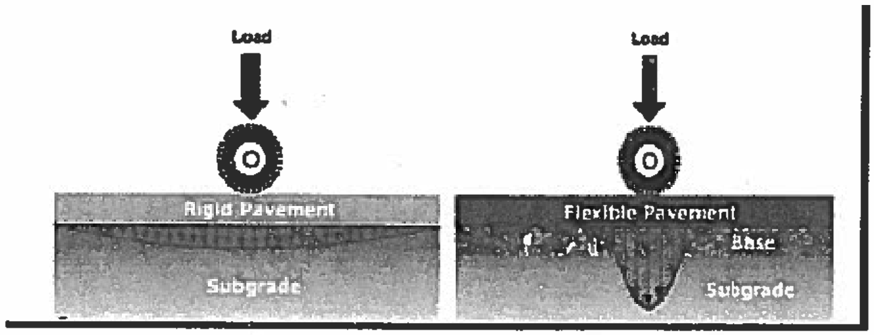

Figure 2.6: Typical Stress Distribution Under a Rigid and Flexible Pavement **(1/)**

Section 2
Pavement Type Selection Criteria

There are several factors that need to be addressed to select a pavement type for a project. The 1993 AASHTO Design Guide(2/) defines some of these in terms of:

- traffic
- subgrade competency
- materials (availability and past performance)
- climate/drainage
- construction considerations
- life-cycle cost analysis

A brief discussion of each follows.

Traffic

This factor should be defined in terms of volume, loading, lane distribution, and impact on construction process.

Subgrade Competency

This factor is critical in selecting the pavement type because of settlement potential of the underlying material such as peat or organic silts. HMA mixes tend to perform better in situations where long-term settlement is expected.

Materials

Selection of materials for pavement design is determined by the availability of suitable material, construction methods, environmental considerations, economics, and past performance.

Climate/Drainage

Both surface runoff and subsurface water control must be considered in the design of pavement structures. Effective drainage is essential to proper pavement performance.

Construction Considerations

Selection of pavement type should address construction issues such as:

- pavement thickness, which may impact underground utilities and overhead clearances.

- effects of detours, bypasses, and alternate routes to accommodate rerouted traffic.

- anticipated future improvements and upgrades - pavement type may restrict future improvements/upgrades relative to width, geometry, structural support, etc.

- the impact on maintenance operations.

- any other factors unique to the project or corridor should be evaluated.

Life Cycle Cost Analysis (LCCA)

Life cycle cost analysis provides a useful tool to assist in the pavement type selection process.

Life cycle cost refers to all costs that are involved with the construction, maintenance, rehabilitation and associated user impacts of a pavement over a given analysis period. It is an economic comparison of all feasible construction or rehabilitation alternatives, evaluated over the same analysis period. At a minimum, one HMA and one PCC alternative should be evaluated. Guidelines for conducting LCCA are provided in the DOTD Manual, which is intended for use by the DOTD personnel for the "Alternate Design/Alternate Bid" process. However, this manual is not intended to serve as a standalone guide on conducting LCCA in pavement design. This manual should be used in conjunction with the FHWA publication number FHWA-SA-98-079, *Life Cycle Cost Analysis in Pavement Design*. The FHWA publication discusses many of the factors contributing to the implementation of LCCA on the federal level.

Currently, there are no federal requirements to conduct LCCA on pavement projects. However, Louisiana chooses to conduct LCCA on pavement projects for two primary reasons:

1. LCCA serves as a **tool for pavement type selection**. If the total life cycle costs for competing alternates differ by more than 25%, then the department is confident that the LCCA has indicated the better choice.

2. Both the asphalt and concrete industries in Louisiana have expressed an interest to DOTD to bid projects on an alternate basis, thereby increasing competition between the two industries. LCCA is the mechanism used to **foster competition between both industries**, which in turn will decrease agency costs. **The LCCA program used by DOTD is based on FHWA's Real Cost program.**

Chapter 3
Pavement Design Methods

Contents

Section 1
1993 AASHTO Design (2/)

Background

This chapter provides an overview of the AASHTO design procedure that is currently used by the DOTD to design flexible and rigid pavements.

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

Approximately 1.2 million axle load repetitions were applied to specially designed test tracks in the most comprehensive pavement test experiment design ever conducted. The original AASHTO 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.

The AASHTO 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 AASHTO Road Test was the rendering of cumulative traffic loading in terms of a single statistic known as the 18-kip Equivalent Single Axle Load (ESAL).

Flexible Design (2/, 3/)

The DOTD currently uses the 1993 "AASHTO Guide for Design of Pavement Structures." This guide considers several parameters in its design, such as pavement performance, traffic loading, roadbed soil, construction materials, environment, drainage, reliability, life cycle costs, and shoulder designs.

Flexible design using the AASHTO procedure requires the designer to derive a structural number (SN) that is adequate for the anticipated traffic over the length of a desired performance period. AASHTO defines the SN as *an abstract number expressing the structural strength of a pavement required for given combination of soil support, total traffic expressed as equivalent 18 kip single axle loads, terminal serviceability, and environment. The required SN must be converted to actual thicknesses of surfacing, base, and subbase, by means of appropriate layer coefficients representing the relative strength of construction material.* SN is equivalent to the sum of a layer coefficient (a), layer thickness (D), and layer drainage coefficient (m) for each layer. The SN term is represented by the following equation:

$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$

Determination of SN is presented in the following equation:

$$\log_{10}(W_{18}) = Z_R \times S_0 + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left\{\frac{\Delta PSI}{4.2-1.5}\right\}}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

Where,

SN = Structural Number required to carry projected loading

W_{18} = Accumulated 18-kip Equivalent Single Axle Load over the Pavement Life

Z_R = Standard Normal Deviate

M_R = Resilient Modulus, psi

S_0 = Standard Deviation

ΔPSI = Change in Serviceability Index

Equation 3.1

Though the flexible pavement design equation exists, the pavement designs are usually performed with aid of a computer program called DARWin™ 3.10 software, which utilizes the above AASHTO equations and design methods. However, one aspect that makes using this design procedure somewhat problematic is that layer coefficients are not directly correlated to any universal system of measurement. AASHTO does provide some guidelines for correlation to laboratory-derived resilient modulus.

The flexible pavement design input parameters used in the DARwin program and the AASHTO equation are discussed in the next chapter.

Rigid Design (2/, 3/)

The 1993 AASHTO design guide for rigid pavement structures require the same input factors as defined for design flexible pavements above; namely, pavement performance, traffic loading, roadbed soil, construction materials, environment, drainage, reliability, life cycle costs, and shoulder designs. However, rigid pavement design differs from flexible pavement design in that the SN concept is not used. Instead, specific properties such as the modulus of rupture, joint load transfer, modulus of elasticity of the concrete, and modulus or composite modulus of subgrade reaction must be specified to determine the thickness of concrete pavement.

The formula presented below is used in rigid pavement design:

$$\log_{10}(W_{18}) = Z_R \times S_0 + 7.35 \times \log_{10}(D+1) - 0.06 + \frac{\log_{10}\frac{\Delta PSI}{4.5-1.5}}{1 + \left\{\frac{1.624 \times 10^7}{(D+1)^{8.46}}\right\}} + (4.22 - 0.32 \times p_t) \times \log_{10} \frac{S'_c \times C_d \times (D^{0.75} - 1.132)}{215.63 \times J \left\{ D^{0.75} - \frac{18.42}{(E_c/k)^{0.25}} \right\}}$$

Where,

W_{18} = Accumulated 18-kip Equivalent Single Axle Load over the Pavement Life
Z_R = Standard Normal Deviate
S_0 = Standard Deviation
ΔPSI = Change in Serviceability Index
D = Thickness of the Slab, in
S'_c = Modulus of Rupture (psi) for Portland Cement Concrete
J = Load Transfer Coefficient
C_d = Drainage Coefficient
E_c = Modulus of Elasticity (psi) for Portland Cement Concrete
k = Modulus of Subgrade Reaction (pci)

Equation 3.2

As with flexible pavement design, the rigid pavement designs are usually performed with the aid of the DARWin™ 3.10 software, which utilizes the above AASHTO equations and design methods.

The concrete pavement design input parameters used in the DARwin program and the AASHTO equation are discussed in the next chapter.

Section 2
Mechanistic-Empirical Pavement Design (MEPDG) (4/)

Background

The 1993 (and all previous versions) AASHTO design guide, currently used by the LaDOTD and discussed in the previous section, has some limitations because of its empirical characteristics. The method cannot accurately predict the performance of designed pavement structures. The new Mechanistic-Empirical Pavement Design Guide (MEPDG), developed under the National Cooperative Highway Research Program (NCHRP) Project 1-37A, represent a major change in the current pavement design practices.

The NCHRP 1-37A was sponsored by the AASHTO Joint Task Force on Pavements, NCHRP, and the Federal Highway Administration (FHWA) to develop an Mechanistic-Empirical pavement design procedure. The procedure was published in 2004 for public review and evaluation. This review and evaluation resulted in the development of a software, version 1.1, and an updated design guide document.

In contrast to the existing versions of the AASHTO pavement design guide, which relied heavily on the results of the AASHTO Road Test conducted in Ottawa, Illinois in the early 1960s, the new guide is based on the mechanistic-empirical design procedure. It provides a uniform basis for the design of flexible, rigid, and composite (rehabilitated) pavements and employs mechanistic design parameters for traffic, subgrade, environment, and reliability. It is based on the iterative process - **the outputs from the procedure are pavement distresses and smoothness, not layer thickness as in the AASHTO design**. The current pavement design software can optimize pavement thickness.

The MEPDG design procedure requires consideration of site conditions relative traffic, climate, subgrade, existing pavement condition for rehabilitation, etc. Based on these factors, a trial design is proposed for a new pavement or a strategy for rehabilitation of existing pavement.

The design approach provided in the guide consists of three major stages summarized in **Figure 3.1**. Stage 1 consists of the development of input values for the analysis. During this stage, potential strategies are identified for consideration in the analysis stage. A key step in this process is the foundation analysis. The pavement material characterizations and traffic input are also developed in this stage. The Enhanced Integrated Climate Model (EICM) is used to model temperature and moisture within the pavement layer and the subgrade.

Stage 2 of the design process is the structural/performance analysis. The analysis approach is an iterative one that begins with an initial trial design. If the trial design does not meet the performance criteria, modifications are made and the analyses are re-run until a satisfactory result is obtained.

Stage 3 of the design process includes those activities required to evaluate the structurally viable alternatives. These activities include engineering analysis and life cycle cost analysis of the alternatives.

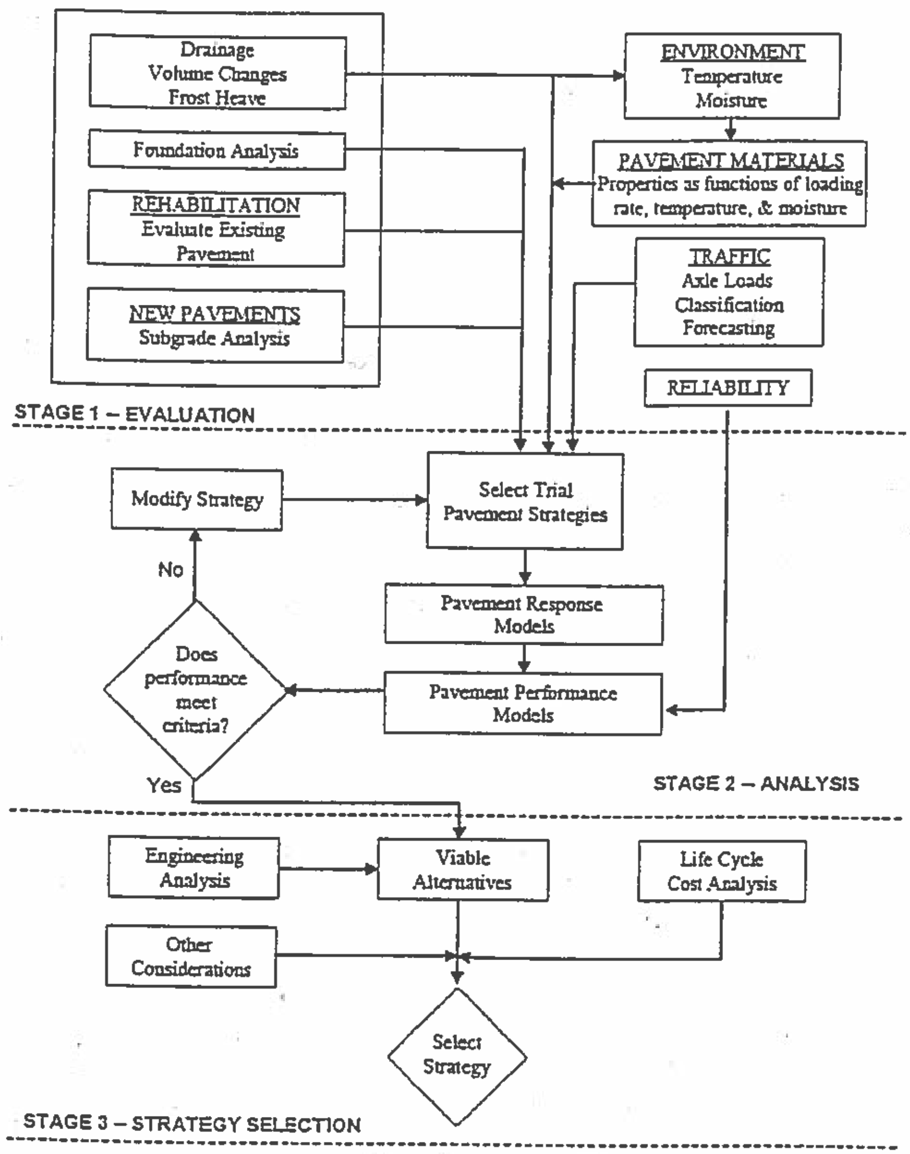

Figure 3.1: Conceptual Schematic of the Three-stage Design Process for MEPDG

The two fundamental differences between the 1993 AASHTO Guide and the MEPDG are that the latter predicts multiple performance indicators and provides a direct tie among materials, structural design, construction, traffic, and pavement management systems. **Figure 3.2** is an example of the interrelationship among these activities for HMA materials.

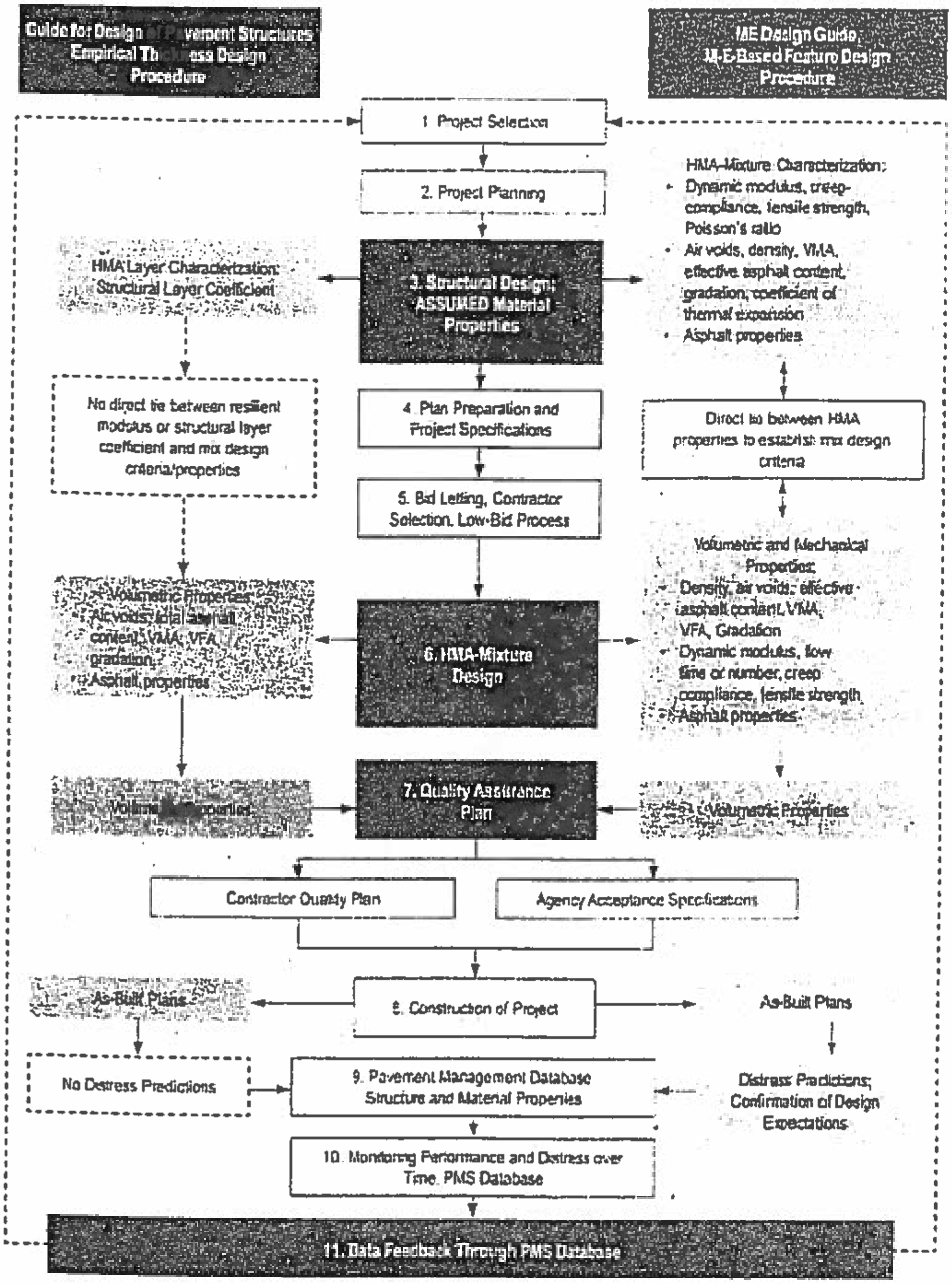

Figure 3.2: Typical Difference between Empirical Design Procedure and an Integrated M-E Design System of HMA-Mix Characterization

Design Inputs

The approach to design inputs in MEPDG is hierarchical, a feature lacking in the existing version of the AASHTO Guide. This approach is employed with regard to traffic, materials, and environmental inputs. Three levels of inputs are provided depending on the level of accuracy desired.

Level 1 inputs provide the highest level of accuracy and would have the lowest level of uncertainty or error. It would typically be used for designing heavily trafficked roads or wherever there are dire safety or economic consequences of early failure.

Level 2 inputs provide an intermediate level of accuracy and would be closest to the typical procedure used with the earlier editions of the AASHTO Guide. This level would be used whenever resources or testing equipments are not available for tests required for Level 1.

Level 3 inputs provide the lowest level of accuracy. This level would typically be used for design where there are minimal consequences of early failure such as for low-volume roads. Inputs for this level would be user-selected or typical average for the region.

Performance Indicators of HMA-surfaced Pavements in MEPDG

The MEPDG defines various transfer functions and regression equations that are used to predict various performance indicators considered important in many pavement management programs. The following is a listing of the specific performance indicators calculated by the MEPDG for hot-mix asphalt-surfaced pavements (conventional and deep strength flexible pavements, full depth HMA pavements, semi-rigid pavements, and full depth reclamation) and HMA overlays:

- Total Rut Depth in HMA, Unbound Aggregate Base, and Subgrade Rutting
- Non-Load Related Transverse Cracking
- Load-Related Alligator Cracking, Bottom Initiated Cracks
- Load-Related Longitudinal Cracking, Surface Initiated Cracks
- Reflection Cracking in HMA Overlays of Cracks and Joints in Existing Flexible, Semi-Rigid, Composite and Rigid Pavements
- Smoothness (IRI)

Reference 4 lists the various distress prediction equations for the above performance indicators.

Performance Indicators of Rigid Pavements in MEPDG

The JCP design is based on transverse cracking, transverse joint faulting, and pavement smoothness (IRI). The designer may select one or all of these performance indicators and establish criteria to evaluate a design. For CRCP design, crack width, space, and LTE (Load Transfer Efficiency), punchouts, and smoothness are the key performance indicators.

Section 3
Pavement Design Categories

The structural design of pavements can be categorized as follows:

- new pavement design
- reconstruction design, and
- rehabilitation design

New pavement design is applied when a construction of a non existing route is necessary. In such cases, a combination of base and surface course is placed on existing subgrade to support the traffic load. The new construction may also include a new parallel roadway adjacent to an existing two-lane highway to convert into a four-lane highway. The surface can be an HMA in the case of flexible design or a cement concrete for rigid design. **Figure 3.3** is a flow chart of this design process for new or full reconstruction.

Reconstruction design usually involves complete removal and replacement of existing pavement structure. Complete deterioration of the roadway generally dictates this reconstruction strategy.

Pavement rehabilitation design involves resurfacing, restoration and rehabilitation work to restore serviceability and extend the service life of an existing roadway. Partial recycling of the pavement and/or addition of surface material can be part of this rehabilitation strategy, assuming that the facility possesses some degree of remaining life.

Figure 3.4 and 3.5 are flow charts for rehabilitation of flexible and rigid pavements, respectively.

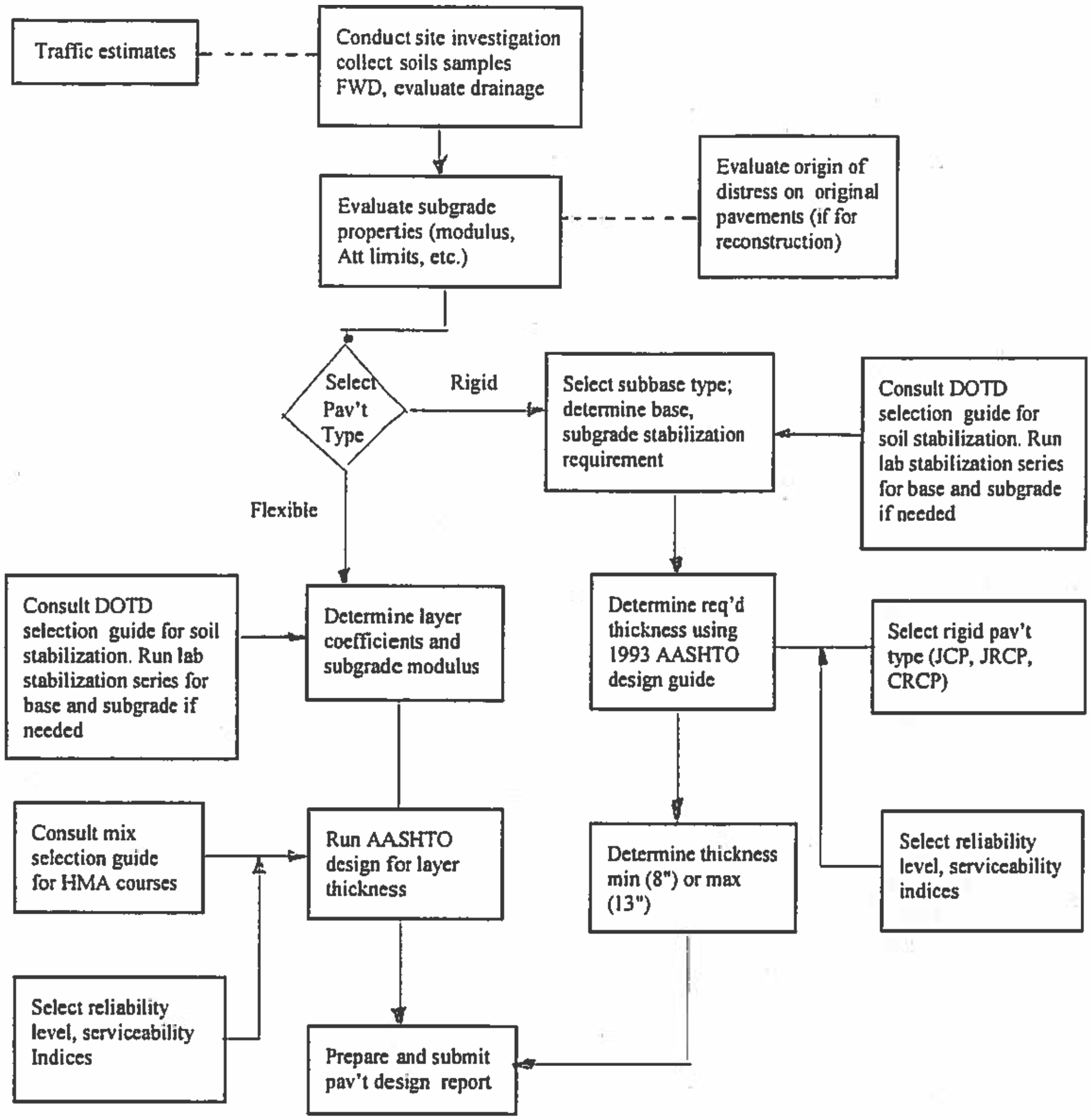

Figure 3.3: Design Process for a New/full Reconstruction Pavement (AASHTO 1993)

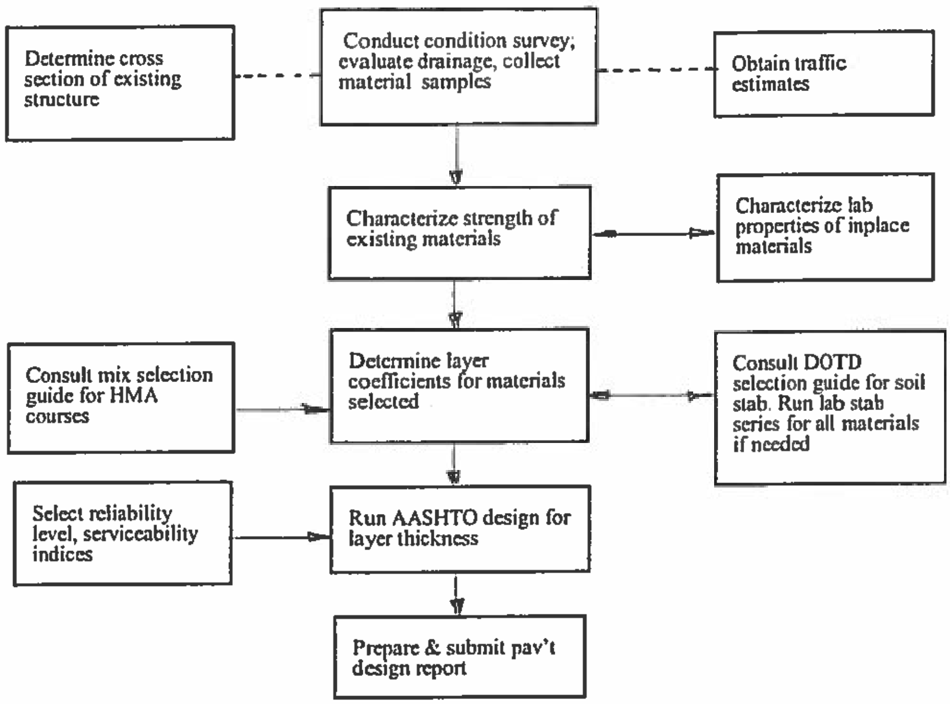

Figure 3.4: Design Process for Flexible Pavement Rehabilitation (AASHTO 1993)

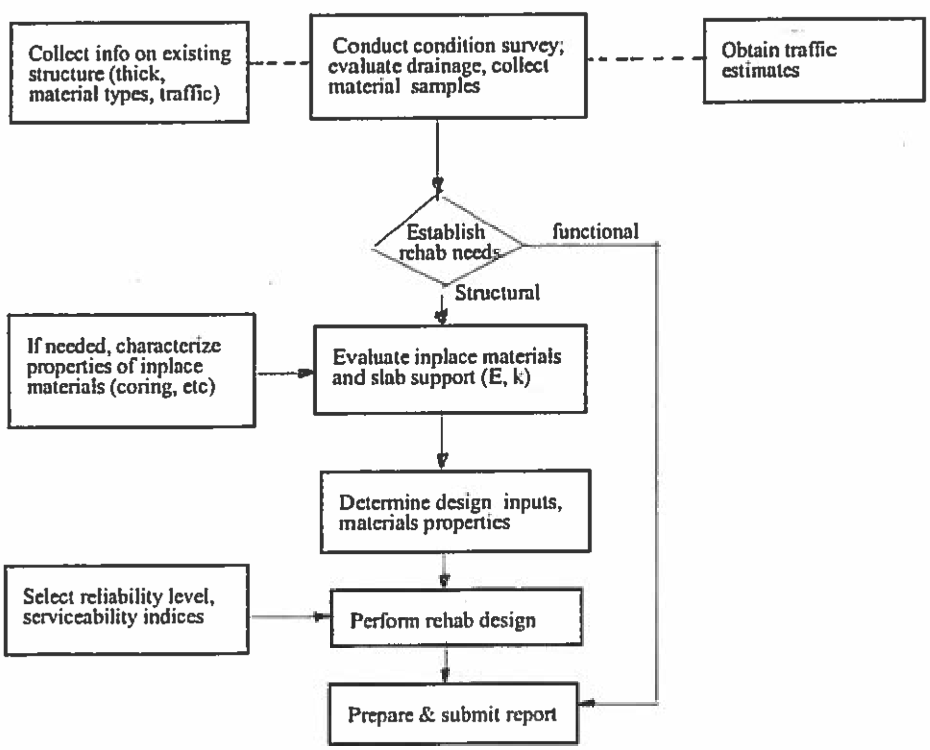

Figure 3.5: Design Process for Rigid Pavement Rehabilitation (AASHTO 1993)

Chapter 4
Information Needed for Input in Pavement Design for AASHTO 1993

Contents

Section 1
Design Period

This factor is common to both flexible and rigid pavement design. Although it is not a direct input into the design equation of pavement structure, it is a prerequisite to input other factors such as traffic and serviceability prediction at the end of the selected design period.

The design period is the time from the original construction to a terminal condition for a pavement. AASHTO essentially defines design period, design life, and performance period as being the same terms. There is also an analysis period which AASHTO defines as the time to conduct an economic analysis. Such an analysis period can include provisions for periodic surface renewal and/or rehabilitation strategies, which will extend the overall service life of a pavement before complete reconstruction is required.

Louisiana policy on design period is as follows:

New and reconstruction	20 years
Rubblize and overlay (interstate)	15 years
Structural overlay	10 years

Section 2
Traffic Loads

Another common factor to flexible and rigid pavement design is the traffic loading. Pavements are designed, regardless of the type of materials used, to withstand repeated applications of traffic loads. Such loads, along with environment, are the main contributors to pavement damage over time. This damage is cumulative over the design life of the pavement and when it reaches some maximum value, the structure is considered to be at the end of its useful service life.

Loads can be characterized by the following parameters:

- tire loads
- tire and axle configurations
- repetition of axle loads
- traffic distribution (direction and lane)
- traffic projection

Tire Loads

Tire loads are the loads imposed at the tire-pavement contact points. Such loads are generally assumed to be equal for all tires on any given axle and are uniformly applied over a circular area.

Tire and Axle Configuration

Another factor that has pronounced effect on pavement performance is the arrangement of axles and tires on the vehicle. Such tire-axle combinations are described as **single axle-single tire, single axle-dual tires, tandem axle-single tire,** and **tandem axle-dual tires**. Although other axle configuration exist (tridem and quad), their existence is rare.

Repetition of Axle Loads

The design of pavement structure requires quantification of expected loads that will be experienced by the facility over its design life. There are two basic methods for characterizing axle load repetitions:

1. **ESAL (Equivalent single axle load).** Based on the AASHTO Road Test results, this quantification is expressed in terms of ESALs, generated over the design life span of the pavement. This approach converts axle configurations and axle loads of various magnitudes and repetitions to an equivalent number of *standard* or *equivalent* loads. In Louisiana (and the U.S.), the most commonly used equivalent load is the 18-kip equivalent axle load. The AASHTO Road Test produced equivalency factors that can be used to convert the relative damage associated with a single or tandem axle of known weight, to an equivalent unit of damage associated with an 18-kip single axle load. As a result, any vehicle of known axle loading can be converted to an equivalent single axle loading (ESAL, 18-kip).

In an effort to best estimate total expected loading, pavement designers use a design factor, termed Vehicle Equivalency Factor, which is characteristic of a particular vehicle type. In the case of trucks, which are the most damaging to pavements, these factors take into consideration the fact that not all trucks in the traffic stream are always loaded. These design factors are produced by periodically conducting loadometer studies in which trucks are sampled for weighing without bias to a load or no load condition. The information published for use in pavement design is entitled the "W-4" table, which is specific to three criteria:

- Terminal PSI
- Initial assumption of thickness of pavement structure
- Number of axles for which the 18-kip equivalence factor is desired

Appendix A lists the vehicle equivalency factors used in Louisiana.

2. **Load spectra**. The proposed M-E Guide discussed in Chapter 3, Section 2 and in NCHRP 1-37A, essentially does away with the ESAL and determines loading effects directly from axle configurations and loads. Tables are available that show the relative axle load frequencies for each common axle combination (single, tandem, etc.) over a given time period.

Traffic Distribution

Besides load type and repetition, load distribution across pavement need to be considered in the design. Likewise, not all lanes on multilane highways carry the same loads within one direction. The outermost lanes generally carry the most trucks and are subjected to heaviest loading.

To account for these types of unequal load distributions, the 1993 AASHTO Guide provides the following equation for calculating directional loading:

$\mathbf{w_{18} = D_D \times D_L \times w18}$ Where:

w_{18} = traffic or load in the design lane
D_D = directional distribution factor, expressed as a ratio (generally 0.5 for most roadways).
D_L = lane distribution factor expressed as a ratio when two or more lanes are available in one direction.
w_{18} = cumulative two-directional18-kip ESAL units predicted for the section of highway

The AASHTO Design Guide (2/) provides the following values for this variable. However, the traffic and planning (T&P) division should be consulted for directional distribution to be used in the design.

Number of Lanes in Each Direction	**Percent of 18-kip ESAL in Design Lane**
1	100
2	80 - 100
3	60 - 80
4	50 - 75

Traffic Projections

For the design of new and reconstruction, traffic projection to 20 years is used for both flexible and rigid pavement structures. Rubblize and overlay on interstate require a 15-year projection. For structural overlay, a design projection of 10 years is required.

Section 3
Serviceability Index

This is the third common input variable in the design of pavements. The concept of serviceability was derived during the AASHO Road Test. The concept is related to the primary function of the pavement to provide a smooth, comfortable, and safe ride to the traveling public. A scale of 0 (impassable road) to 5 (perfect road) is used to evaluate the present serviceability index (PSI).

Newly constructed or rehabilitated pavements start with a high level of serviceability, generally between 4.0 and 5.0. This is the initial serviceability index, $\mathbf{P_i}$. With time, traffic loading, and the environment, the serviceability index decreases and can reach a level below 2.0. This is the terminal serviceability index, $\mathbf{P_t}$. The serviceability level inputs used in Louisiana for different classes of highway systems is listed in Table 4.1. The **ΔPSI** is the design serviceability loss in both the flexible and rigid pavement design equations 3.1 and 3.2, respectively, discussed in Section 1 of Chapter 3. It is based on the expected initial serviceability index ($\mathbf{P_i}$) and the minimum desirable terminal index ($\mathbf{P_t}$).

Table 4.1
Design Loss, Initial and Terminal Serviceability Index used in Louisiana

	ΔPSI	P_i	P_t
Interstate	1.5	4.3	2.8
Primary	1.8	4.3	2.5
Collector	2.0	4.0	2.0
Local	2.0	3.5	1.5

Section 4
Reliability

This common variable in the design of flexible and rigid pavements is defined, according to the AASHTO Design Guide, as "the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design period." Although 100% reliability is desired in the pavement structure, the cost of producing such 100% reliable pavement structure is not economically feasible since any increase in reliability increases the initial cost of construction.

Reliability of pavements designed using the DOTD's design procedures is related to maintaining serviceability at or above the specified minimum (terminal) serviceability index throughout the desired service life. Further, it is recognized that the inputs in the design equation have some variability associated with them. For example, the materials used in the pavement, construction process, traffic load predictions, etc. all have variability. Generally, the higher the desired assurance that the structure will perform as designed, the thicker the pavement will be or the thickness must be offset using high quality materials. Figures 4.1 and 4.2 (5/) show the sensitivity of this factor on the thickness of rigid and flexible pavements, respectively.

Table 4.2 lists the AASHTO and Louisiana reliability levels for both the flexible and rigid pavement design

Table 4.2
AASHTO and Louisiana Suggested Reliability Levels

Location	Functional Class	AASHTO Suggested Reliability Levels	Louisiana Recommended Reliability Levels
Urban	Interstate	85 - 99.99	99
	Principal	80 - 99	97
	Collector	80 - 95	90
	Local	50 - 80	75
Rural	Interstate	80 - 99.99	97
	Principal	75 - 95	95
	Collector	75 - 95	85
	Local	50 - 80	70

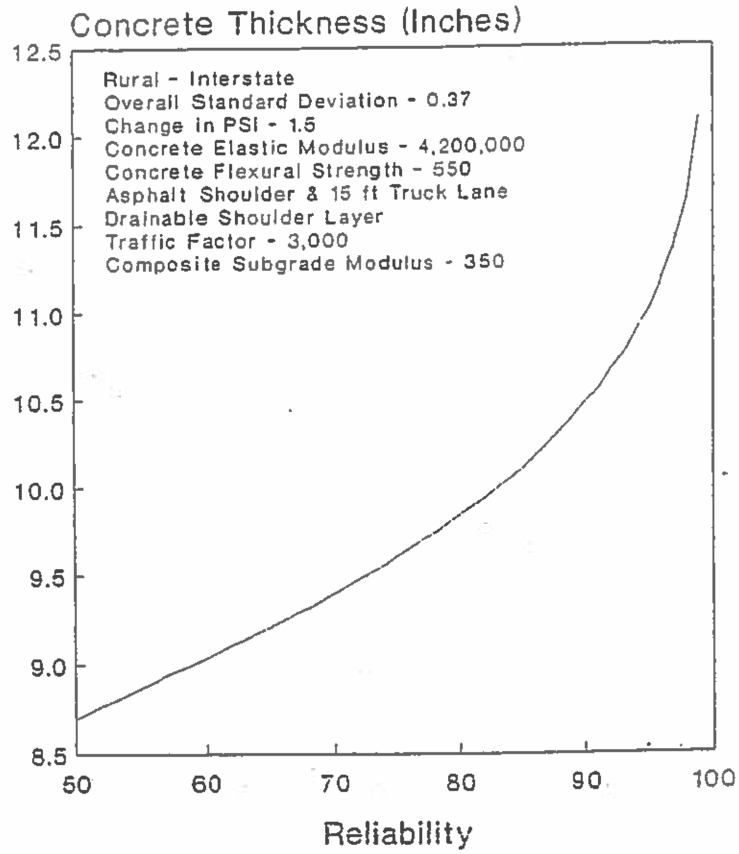

Figure 4.1: Sensitivity Analysis of Reliability Factor (Rigid Design)

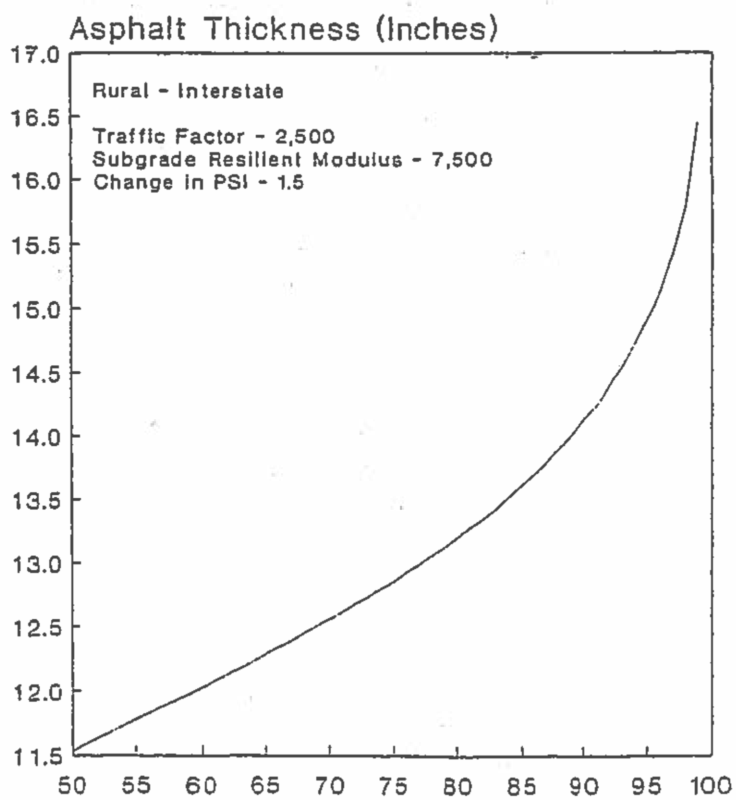

Figure 4.2: Sensitivity Analysis of Reliability Factor (Flexible Design)

Reliability Design Factor

As mentioned before, the reliability concept is intended to provide the designer with a mechanism for applying a factor of safety in design as a safeguard against incorrect traffic predictions and overloads, unfavorable environmental effects, and variations in material strengths that cannot be controlled by construction and specification requirements. The new AASHTO design procedure accomplishes this by calculating a *reliability design factor* using the reliability level shown in Table 4.2, and the expected overall standard deviation associated with type of pavement under design. This reliability design factor is defined in equations 3.1 and 3.2 of Chapter 3 by the combined term $Z_R \times S_O$.

The Z_R term is the standard normal deviate and corresponds statistically to the level of reliability selected in the design (Table 4.2). The term S_O, the overall standard deviation, is included to account for expected variation in the prediction of pavement performance for a given traffic loading. Values for this term are selected by the type of pavement structure being designed (flexible or rigid). The AASHTO Guide recommends the following range of values:

Rigid pavements	0.33 - 0.39
Flexible pavements	0.44 - 0.49

In Louisiana, the midpoint values are used for general design practice. These are:

Rigid pavements	0.37
Flexible pavements	0.47

The effect of change in overall standard deviation (S_O) on the thickness of pavements is demonstrated in Figures 4.3 and 4.4 (5/) for rigid and flexible pavements, respectively.

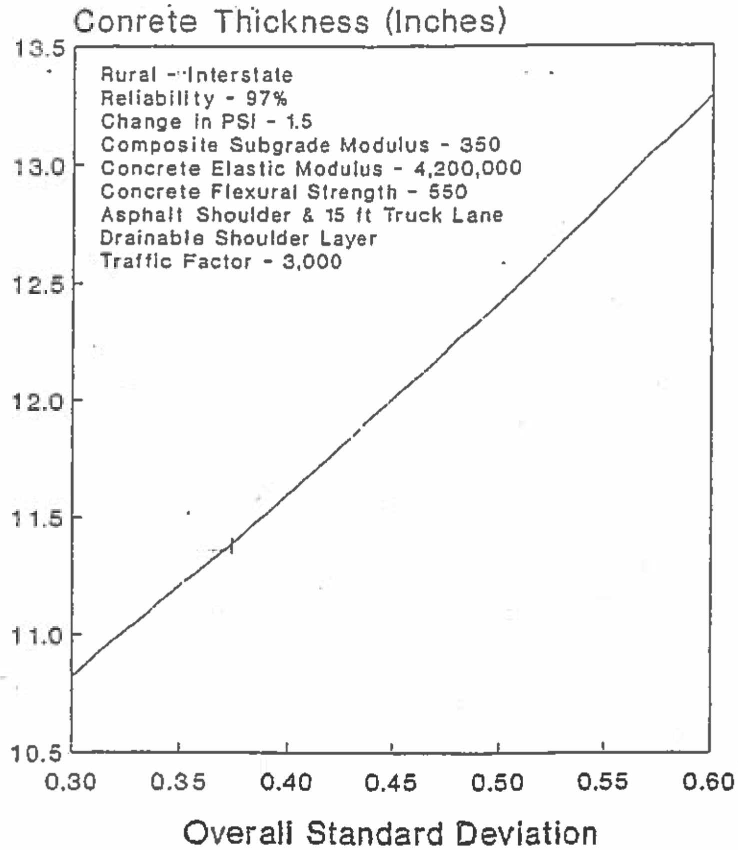

Figure 4.3: Sensitivity Analysis of Standard Deviation (Rigid Design)

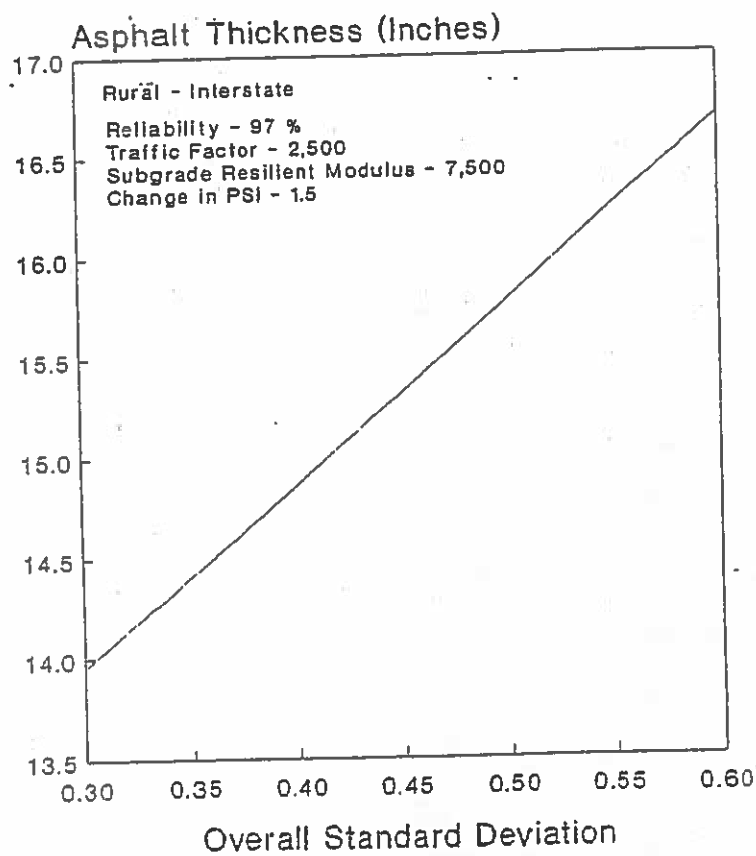

Figure 4.4: Sensitivity Analysis of Standard Deviation (Flexible Design)

Section 5
Material Characterization

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

Measurements of Moduli for Flexible Pavement Design

Laboratory Measurements

Laboratory tests can be used to determine the parameters that affect properties of materials, such as moisture susceptibility, stress level, strain amplitude and strain rate. The laboratory method of choice, especially for the flexible pavements, is the resilient modulus (M_r in equation 3.1, Chapter 3) test. The testing procedure for the resilient modulus consists of subjecting a specimen to a sequence of confining pressure and cyclic deviator stress levels. The resilient modulus is the ratio of the repeated axial deviator stress divided by the recovered axial strain.

Field-based Measurements

The most common method is the Falling Weight Deflectometer (FWD). The deflection measurements obtained from this device can be used in back-calculation methods to determine pavement structural layer stiffness, the subgrade elastic modulus, and the depth to stiff layer. The back-calculation method is a mechanistic evaluation of pavement surface deflection basins generated by various pavement deflection devices.

The next chapter, "Geotechnical Investigation of Pavement Structure," details the various laboratory and field methods for characterizing material properties for input in the design of pavements.

Measurements of Moduli for Rigid pavement Design

Laboratory Measurements

Modulus of Rupture (S'_c)
Modulus of Elasticity (E_c)

Field-based Measurements

Modulus of Subgrade Reaction (k)

Section 6
Drainage Characteristics

Drainage of Flexible Pavements

Drainage of flexible pavement sections is encouraged in the AASHTO Design Guide and a mechanism is included in the design to account for the effect of drainage. The design provides for a layer thickness adjustment for a drainable layer by increasing the drainage coefficient (m) of the material in the equation $SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$ by m_j.

Drainage of Rigid Pavements (2/, 5/)

The drainage coefficient (C_d in equation 3.2 for rigid design, Section 1, Chapter 3) is used to account for the expected level of drainage a rigid pavement is to encounter over its design life. Values of C_d are dependent on the quality of drainage and the percent of time during the year the pavement structure is normally exposed to moisture levels approaching saturation. Table 4.3 describes the quality of drainage and the time to remove water from the pavement. Table 4.4 lists the AASHTO recommended C_d values. The effect of C_d on slab thickness is shown in Figure 4.5.

Table 4.3
Quality of Drainage

Quality of Drainage	Water Removed Within
Excellent	2 Hours
Good	1 Day
Fair	1 Week
Poor	1 Month
Very Poor	No Drainage(water will not drain)

Table 4.4
AASHTO Recommended Values For Drainage Coefficient (C_d)

Quality of Drainage	Percent of Time Pavement structure is Exposed to Moisture Levels Approaching Saturation			
	<1%	1 - 5%	5 - 25%	25%
Excellent	1.25 - 1.20	1.20 - 1.15	1.15 - 1.10	1.10
Good	1.20 - 1.25	1.15 - 1.10	1.10 - 1.00	1.00
Fair	1.15 - 1.10	1.10 - 1.00	1.00 - 0.90	0.90
Poor	1.10 - 1.00	1.00 - 0.90	0.90 - 0.80	0.80
Very Poor	1.00 - 0.90	0.90 - 0.80	0.80 - 0.70	0.70

The choice of C_d in Louisiana is based on the presumption that greater than 25% of the time pavements will be exposed to moisture levels approaching saturation. The following values are recommended for each paving situation listed below:

	C_d
Drainable layer through the shoulder	1.10
Longitudinal edge drains outside shoulder	1.05
No drainage feature	0.90

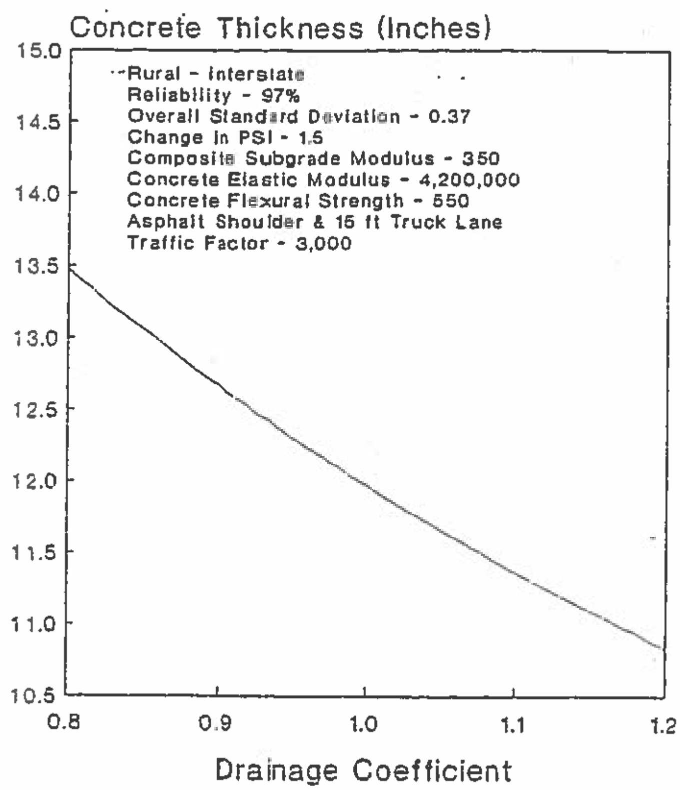

Figure 4.5: Sensitivity Analysis of Drainage Coefficient (Rigid Design)

Chapter 5
Geotechnical Investigation of Pavement Structure

Contents

Section 1
Introduction

Comprehensive geotechnical information is a prerequisite to building all transportation systems because such systems are built on, in, or with soil and products from the ground. In that respect, the characterization and evaluation of soil is critical to the performance of pavement structures.

The level of investigation needed, as well as when to perform such an investigation, depends on the nature of the project and the engineering properties desired. For example, new construction and reconstruction will require the greatest effort and time to compile roadbed information, followed by, to a lesser degree, resurfacing and/or overlay.

The primary focus of this chapter is the various tests that are needed for input in the design equation for flexible and rigid pavement structures.

Section 2
Determination of Flexible Pavement Design Input Parameters

As previously stated in Section 5 of Chapter 4, roadbed resilient modulus is one of the primary inputs in the flexible pavement design equation. Resilient Modulus (M_r) is the definitive material property used by the AASHTO Guide to characterize roadbed soils. It is a measure of the elastic property of soil that recognizes certain nonlinear characteristics. M_r properties vary within a project due to variability within soil layers, multiple soil strata, compaction, moisture content, water table, environmental conditions, and phases of construction (3/). Methods available for determination of M_r are:

1. Repeated Load Triaxial Test:

Resilient values can be determined through AASHTO T307-99(2007). However, in Louisiana, this is not a standard test for production work.

2. Soil Characteristic:

The Resilient Modulus can also be estimated from the following soil characteristic tests which can be input in a software to obtain M_r values.

1. Soil classification
2. Liquid, plastic limits
3. Percent retained on #4, 10, 40, and 200 sieves
4. Percent silt and percent clay estimation

Maps have been developed for general soil types encountered in different parishes in Louisiana with their corresponding modulii values determined from R-values - M_r correlation studies (6/).

3. Estimation (3/):

A more generalized method can be used to estimate M_r using the following relationship. DOTD has developed charts for M_r values derived from SSV (Soil Support Value) for each parish in the state. Since this method is not project specific, the method should be considered approximate.

$$\mathbf{M_r = 1500 + 450 \{(53/5)(SSV - 2)\} - 2.5 \{(53/5)(SSV - 2)\}^2}$$

where,
M_r = Resilient Modulus (psi)
SSV = Soil Support Value

4. Non-destructive Testing (NDT)

Three field methods are available for determination of resilient modulus. These are:

1. ***Falling Weight Deflectometer, FWD*** (31)

This device is essential in establishing the insitu stiffness properties of the pavement layers through analysis of deflection data. Two methods are available to determine the subgrade modulus: one is the **ELMOD** backcalculation software and the **AASHTO** method. These are briefly discussed below.

A. ELMOD: Two methods are available for input in the backcalculation software. One method requires seed value and the other where seed values are not input in the calculation.

In the **first method**, the resulting subgrade modulus derived from the software is input into the following correlation equation to obtain laboratory equivalent resilient modulus, which is used in the pavement design equation (equation 3.1).

$M_r = 0.39E_{fwd} + 0.61$ Where,

M_r = laboratory equivalent resilient modulus (ksi)
E_{fwd} = back-calculated modulus using seed values (ksi)

In the **second method,** where seed values are not used, the following correlation equation is available for calculation of M_r:

$M_r = 0.39E_{fwd} + 0.64$ Where,

M_r = laboratory equivalent resilient modulus (ksi)
E_{fwd} = back-calculated modulus without seed values (ksi)

B. AASHTO Method: The 1993 AASHTO design guide provides the following formula to determine subgrade modulus:

$M_r = 0.24P/d_r r * C$ Where,

M_r = laboratory equivalent resilient modulus (ksi)
P = applied load, kips
d_r = deflection at a distance r from the center of the load plate
r = distance from the load plate (36 in.)
C = (0.33), a correction factor to correlate to equivalent laboratory resilient modulus

2. Dynaflect

This is another field device that can be used to determine the subgrade modulus through series of charts and nomographs developed by DOTD. The following correlation equation can be used to obtain the laboratory equivalent subgrade resilient modulus.

$$M_r = 0.41E_d + 2.26$$ Where,

M_r = laboratory equivalent resilient modulus (ksi)
E_d = subgrade modulus from dynaflect (ksi)

3. Dynamic Cone Penetrometer, DCP

This portable device is a secondary tool used to verify unbound pavement layer thicknesses and stiffness. The method requires boring a small pilot hole through bound materials. The test is conducted in accordance with ***DOTD TR-645***. Two models, a ***Soil Property Model*** and a ***Direct Model***, are available to convert DCP values to moduli values.

A. Soil Property Model:

$$M_r = 165.5 \left\{\frac{1}{DCPI^{1.147}}\right\} + 0.0966 \{\gamma_d / w\}$$ Where,

M_r = Resilient modulus (ksi)
DCPI = Dynamic cone penetration index (mm/blow)
γ_d = Dry unit weight, pcf
w = Water content, %

B. Direct Model:

$$M_r = \left\{\frac{151.8}{DCPI^{1.096}}\right\}$$ Where,

DCPI = Dynamic cone penetration index (mm/blow)

Section 3
Determination of Rigid Pavement Design Input Parameters

There are five data input variables unique to the design of rigid pavements (equation 3.2). These are:

1. Modulus of Concrete Rupture
2. Modulus of Elasticity
3. Modulus of Subgrade Reaction
4. Load Transfer Coefficient
5. Drainage Coefficient

A brief discussion of how the input values are derived follows:

1. Modulus of Rupture (S'_c)

This laboratory derived value is determined through the use of mean value of third point loading failure tested according to AASHTO T97, ASTM C78. AASHTO suggests that the actual mean flexural strength test value be used for design rather than the normal construction specification values. The suggested value for Louisiana is **600 psi**. This value assumes substitution of 20% fly ash for cement and the use of gravel aggregate, and is therefore conservative when alternative materials are used.

2. Modulus of Elasticity (E_c)

This variable can be determined through the procedure described in ASTM C459. The value recommended for Louisiana is **4.2 x 10^6 psi.**

3. Effective Modulus of Subgrade Reaction (k)

The value for this variable ('k' in equation 3.2) is determined in the field and provides a measure of the support provided to the concrete slab by the underlying layers. It is termed *Effective Modulus of Subgrade Reaction* because it considers the support by every underlying layers, base, subbase and subgrade. The modulus of subgrade reaction for a roadbed soil material is different than the effective subgrade modulus and is determined through a plate load test. The k value is measured in the field by applying a load of 10 psi on the subgrade/base combination using a 30-in. diameter steel plate. The k-value is then calculated by dividing 10 psi by the measured deflection (in inches) of the layers under the plate. This value is generally used when the concrete slab is constructed directly on the subgrade.

Since k is not very sensitive to concrete design thickness, DOTD uses a single value for effective modulus of subgrade reaction that is determined from the soil support value (SSV) developed for all parishes. This SSV is selected for the parish in which the project is to be built. From this SSV value, k is selected from correlation curves for soil support. This value is then used as a single input value for k to be used in determining the required concrete thickness.

Soil classification and other physical tests can also be used to determine SSV from Texas Triaxial - R value correlation curves (6/).

4. Load Transfer Coefficient

The load transfer coefficient ("J" in equation 3.2 in Chapter 3) is used to account for the ability of a rigid pavement to transfer load across joints and/or cracks in the pavement. The use of load transfer devices and tied shoulders result in lower stresses in the concrete slab.

New rigid pavement designs may contain tied PCC shoulders rather than AC shoulders although, at times, AC shoulders are also used. The use of load transfer devices, tied concrete shoulders, and widened lanes all serve to lower the "J" values selected thereby reducing the final concrete thickness. The sensitivity of this factor on the thickness of concrete pavement is illustrated in Figure 5.1 (5/).

Louisiana's policy on rigid pavement design includes 20-ft. transverse joint spacing with steel dowels placed on 12-in. centers. Interstate design may require tied concrete shoulders and an outside lane of 15-ft. wide. The recommended values for "J" are:

Asphalt shoulder and 12-ft. truck lane	3.2
Asphalt shoulder and 15-ft. truck lane or concrete shoulder	2.5

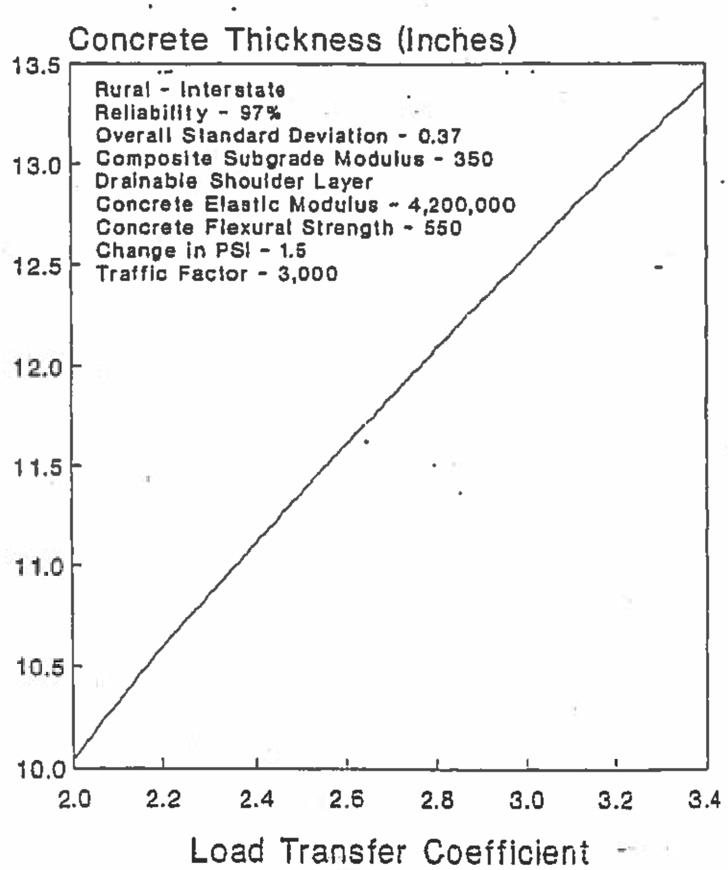

Figure 5.1: Sensitivity Analysis of Load Transfer Coefficient

5. Drainage Coefficient

See Section 6 of Chapter 4 for discussion of this input variable in the design of rigid pavements.

Section 4
Adjustment Factors on Overlays Using Rigid Pavement Design Formula (3/)

Adjustment factors are applied whenever overlays of concrete pavements are considered. The overlays could be bonded PCC, unbonded PCC, Asphaltic Concrete (AC) over unbroken PCC or AC over AC on unbroken PCC. These factors are applied to compensate for loss in Present Serviceability Index, PSI, due to existing distress in concrete pavement that is to be overlaid. The following adjustment factors are to be considered in overlay design. Details are provided in the AASHTO Design Guide 1993, Part III (2/).

Adjustment Factors for Bonded/Unbonded PCC Overlays and AC over PCC Overlays

Joints and Cracks Adjustment Factor (F_{jc})

This factor adjusts for the loss in PSI due to deteriorated reflection cracks resulting from any unrepaired deteriorated joints, cracks, and other discontinuities. A deteriorated joint or crack in the existing slab will rapidly reflect in an AC overlay and contribute to loss of serviceability. The following information is needed to determine F_{jc}.

- Number of unrepaired deteriorated joints per mile
- Number of unrepaired deteriorated cracks per mile
- Number of unrepaired punchouts per mile
- Number of expansion joints, exceptionally wide joints (> 1 "), and full-depth, full-lane width AC patches per mile

Adjustment factors are calculated when the above data is input in the Darwin 3.1 software. If the above data is not available, estimates may be used for input in the software

Fatigue Damage Adjustment Factor (F_{fat})

This factor adjusts for the past fatigue damage that may exist in the slab. It is determined by observing the extent of transverse cracking in JRCP and JPCP or punchouts in CRCP that may be caused primarily by repeated loading. Condition survey data from PMS files and the guidelines in Table 5.1 may be used to estimate this factor.

Durability Adjustment Factor (F_{dur})

This factor adjusts for an extra loss in PSI when the existing slab had durability problems such as D-Cracking due to some aggregate pore structure. Table 5.2 lists the durability adjustment factor for input.

Table 5.1: Fatigue Damage Adjustment Factor

F_{fat}	Condition
0.97-1	Few transverse cracks/punchouts exist (none caused by D-Cracking or reactive aggregate distress)
	JPCP: < 5% slabs are cracked
	JRCP: <25 working cracks/mile
	CRCP: <4 punchouts/mile
0.94 - 0.96	A significant number of transverse cracks/punchouts exist (none caused by D-Cracking or reactive aggregate distress)
	JPCP: 5 - 15% slabs are cracked
	JRCP: 25 -75 working cracks/mile
	CRCP: 4 - 12 punchouts/mile
0.90 - 0.93	A large number of transverse cracks/punchouts exist (none caused by D-Cracking or reactive aggregate distress)
	JPCP: > 15% slabs are cracked
	JRCP: > 75 working cracks/mile
	CRCP: > 12 punchouts/mile

Table 5.2: Durability Adjustment Factor

F_{dur}	Condition
1.00	No sign of PCC durability problems
0.96 - 0.99	Durability cracking, but no spalling
0.88 - 0.95	Substantial cracking and some spalling
0.80 - 0.88	Extensive cracking and severe spalling

Adjustment Factors for AC over AC/PCC

Durability Adjustment Factor (F_{dur})

For overlays with AC over existing AC/PCC, the factors listed in Table 5.3 are applied depending on the condition history of the pavement.

Table 5.3: Durability Adjustment Factors for AC over AC/PCC

F_{dur}	Condition
1.00	No evidence of history of PCC durability problems
0.96 - 0.99	Pavement is known to have durability problems but no localized failures or related distresses are visible
0.88 - 0.95	Some durability distress (localized failures)
0.80 - 0.88	Extensive durability distress (localized failures)

AC Quality Adjustment Factor

This factor adjusts the existing AC layer's contribution to D_{eff} (effective thickness of existing slab) based on the quality of the AC material. The value selected should depend only on distresses related to the AC layer (no reflection cracking) that are not eliminated by surface milling (e.g., rutting, stripping and shoving, as well as weathering and raveling if the surface is not milled). When the AC layer projects poor condition, complete removal should be considered. Table 5.4 lists the factors for this quality adjustment.

Table 5.4: AC Quality Adjustment Factors for AC over AC/PCC

F_{ac}	Condition
1.00	No AC material distress
0.96 - 0.99	Minor AC material distress such as weathering, raveling not corrected by surface milling
0.88 - 0.95	Significant AC material distress such as rutting, stripping, shoving
0.80 - 0.88	Severe AC material distress (rutting, stripping, shoving)

Chapter 6
Flexible Pavement Design

Contents

Section 1
Overview

The primary design procedure used by DOTD for flexible pavement structure is the *AASHTO Guide for Design of Pavement Structures (1993)* (2/). The basic equation for the design of flexible pavement was discussed in Chapter 3 (equations 3.1). As mentioned in that chapter, the concept for design is based on identifying a flexible pavement SN to withstand the projected level of axle load traffic. The various inputs defined in the equation for the design of pavements were discussed in the previous chapters. This chapter summarizes these inputs and provides a typical example of the completed design using two methods, a nomograph of the design equation and the AASHTO-developed software DARWin.

Section 2
Design Inputs in New and Reconstructed Flexible Pavements

Table 6.1 summarizes the flexible pavement design process and the data inputs discussed in Chapters 4 and 5. Once the input data is determined, the design thickness can be determined using either the equation or the nomograph of the equation for flexible design (Equation 3.1, Chapter 3) or the DARWin Pavement Design Software System, which is the main tool used by DOTD to design their pavements.

Table 6.1: Summary of LA Design Requirements for Flexible Pavements

Variable	Input Required ?	Input Value			
Performance/Design Period	Yes	New and reconstruction - 20 Yrs			
Traffic (expressed in 18-kip Equivalent Single Axle Loads, ESAL over design period)	Yes	Computed from Tables for Vehicle Classification Equivalency Factors (Appendix A)			
Reliability Level, Z_R	Yes		Urban	Rural	
		Interstate	99	97	
		Principal	97	95	
		Collector	90	85	
		Local	75	70	
Serviceability Index, Initial(P_i), Terminal(P_t), and Design Loss(ΔPSI)	Yes		P_i	P_t	ΔPSI
		Interstate	4.3	2.8	1.5
		Principal	4.3	2.5	1.8
		Collector	4.0	2.0	2.0
		Local	3.5	1.5	2.0
Overall Std Dev, S_o	Yes	0.47			
Effective Roadbed Soil Resilient Modulus (M_r)	Yes	Determined from laboratory tests or parish maps showing soil types and their typical M_r values (Appendix B)			
Layer Coefficients (a_i)	Yes	From updated values (Appendix C)			
Drainage Coefficient	Yes	m_i in equation $SN=a_1D_1 + a_2D_2m_2 + a_3D_3m_3$			

Section 3
Determination of Structural Number (SN) and Layer Thicknesses

3.1 Using Nomograph

The design of flexible pavement is based on identifying a flexible pavement structural number (SN) to withstand a projected level of axle load traffic. Equation 3.1 can be used to determine this SN. However, a nomograph of this equation has been developed for SN determination. Figure 6.1 represents this nomograph. Use of this figure requires information on variables listed in Table 6.1. Caution is recommended in using the SN number derived using this nomograph since it is only an approximation based on mean values. A typical example in the use of this nomograph is given below:

Inputs:

• Performance/Design Period	20 yrs
• 18-kip ESALS over design period (Calculated from equivalency tables)	1.9×10^6
• Initial Serviceability Index	4.3
• Terminal Serviceability Index	2.5
• Design Serviceability Loss, ΔPSI	1.8
• Reliability Level	95%
• Overall Standard Deviation	0.47
• Roadbed Soil Resilient Modulus	10ksi

Output:

Using the above input data, the nomograph of Figure 6.1 provides SN approximately equal to 3.7. This number can be converted to layer thicknesses using the equation for structural layer coefficients (a_i) and thickness (D_i).

Since the SN equation does not have a single unique solution, recommendations on base and surface layer type and thickness should be governed by Sections in Part III and V of *2006 Standard Specifications for Roads and Bridges* and applicable DOTD policy on Pavement Design. Appendix D defines the current policy on asphaltic concrete pavement design.

NOMOGRAPH SOLVES:

$$\log_{10} W_{18} = Z_R * S_o + 9.36*\log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2 - 1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32*\log_{10} M_R - 8.07$$

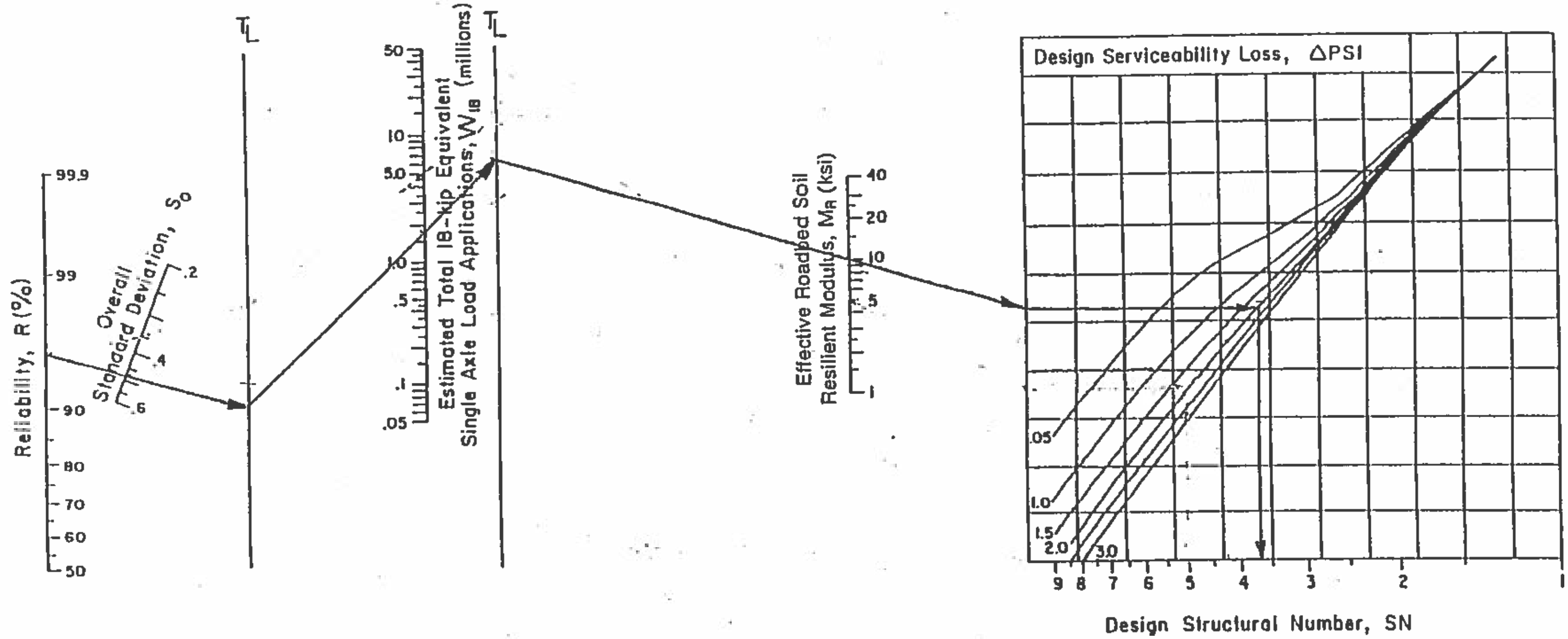

Figure 6.1: Design Chart for Flexible Pavement Based on Using Mean Values for Each Input Variable

3.2 Using DARWin Pavement Design Software System

As mentioned before, DOTD uses this proprietary software developed and distributed by AASHTO. Three separate input modules are required to derive the final thickness of the pavement structure.

One input module computes 18-kip ESALS over the initial performance period. The following information on input is required to compute this 18-kip ESALS:

- Performance period, years
- Two-way traffic, ADT
- Number of lanes in design direction
- Percentage of all trucks in design lane
- Percentage trucks in design direction
- Percentage of ADT for each FHWA vehicle classification
- Annual percentage growth
- Equivalency factors for each vehicle classification
  (based on Road System Classification and Terminal Serviceability Index tables in Appendix A)
- Annual percentage growth in truck factor

The second input module calculates the design Structural Number SN from the following information, which is available in Table 6.1.

- 18-kip ESALS over design period
  (Calculated previously above)
- Initial Serviceability Index
- Terminal Serviceability Index
- Reliability Level
- Overall Standard Deviation
- Roadbed Soil Resilient Modulus
  (Parish maps in Appendix B)

The third set of input computes the SN for each layer type recommended in the design. The SN for each layer is calculated as $a_iD_im_i$ where a, D, and m represent the layer coefficient, thickness of the layer, and layer drainage coefficient, respectively.

As mentioned before, the SN equation does not have a single unique solution, i.e., there are many combinations of layer thicknesses that are satisfactory solutions. Therefore, selection of base and surface layer type and thickness should be governed by Parts III and V of *Louisiana's Standard Specifications for Roads and Bridges, 2006 Edition,* and the most current policy on pavement design. Appendix D spells out the current policy on Asphaltic Pavement Design.

An example of the input data and the corresponding output is shown in Figure 6.2. Notice that there is not much deviation between the SN value derived from DARWin software (3.63) and the one derived using nomograph (3.7).

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Flexible Structural Design Module

EXAMPLE

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	1,860,647
Initial Serviceability	4.3
Terminal Serviceability	2.5
Reliability Level	95 %
Overall Standard Deviation	0.47
Roadbed Soil Resilient Modulus	10,278 psi
Stage Construction	1

Input 2

Calculated Design Structural Number	3.63 in

Output 2

Rigorous ESAL Calculation

Performance Period (years)	20
Two-Way Traffic (ADT)	4,559
Number of Lanes in Design Direction	1
Percent of All Trucks in Design Lane	100 %
Percent Trucks in Design Direction	50 %

Input 1

Vehicle Class	Percent of ADT	Annual % Growth	Average Initial Truck Factor (ESALs/ Truck)	Annual % Growth in Truck Factor	Accumulated 18-kip ESALs over Performance Period
1	0.46	1.3	0.0005	0	43
2	65.27	1.3	0.0005	0	6,161
3	22.08	1.3	0.0188	0	78,363
4	0.6	1.3	0.1932	0	21,883
5	2.91	1.3	0.1932	0	106,134
6	1.04	1.3	0.4095	0	80,397
7	0.07	1.3	0.4095	0	5,411
8	1.77	1.3	0.8814	0	294,510
9	5.12	1.3	1.1	0	1,063,203
10	0.43	1.3	1.45	0	117,704
11	0.1	1.3	1.84	0	34,735
12	0.03	1.3	1.84	0	10,421
13	0.12	1.3	1.84	0	41,682
Total	100	-	-	-	1,860,647

Growth	Compound

Total Calculated Cumulative ESALs	1,860,647

Output 1

Figure 6.2: Flexible Pavement Structural Design using DARWin Software

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	Superpave Asphaltic Concrete Wea...	0.44	1	2	-	0.88
2	Superpave Asphaltic Concrete Bind...	0.44	1	3	-	1.32
3	Class II Base Course	0.14	0.9	12	-	1.51
Total	-	-	-	17.00	-	3.71

Input 3 **Output 3**

Figure 6.2 (Cont'd): Flexible Pavement Structural Design using DARWin Software

Chapter 7
Rigid Pavement Design

Contents

Section 1
Overview

As with flexible pavement design, the primary design procedure used by DOTD for rigid pavement structure is the *AASHTO Guide for Design of Pavement Structures* (1993) (2/). The basic equation for the design of rigid pavement was discussed in Chapter 3 (Equations 3.2). As mentioned in that chapter, the concept for design is based on identifying concrete pavement thickness that will withstand the projected level of axle load traffic. The various input variables defined in the equation for the design of pavements were discussed in the previous chapters. This chapter summarizes these input variables and provides a typical example of the completed design using nomograph and DARWin software.

Section 2
Design Inputs in New Rigid Pavements

Table 7.1 summarizes the rigid pavement design process and the data inputs discussed in Chapters 4 and 5. Once the input data is determined, the design thickness can be determined using either the equation or the nomograph of the equation for rigid design (Equation 3.2, Chapter 3) or the DARWin Pavement Design Software System. As with flexible pavement design, this software is the main tool used by DOTD to design rigid pavements.

Table 7.1: Summary of LA Design Requirements for Rigid Pavements

Variable	Input Required ?	Input Value			
Performance/Design Period	Yes	New - 20 Yrs			
Traffic (expressed in 18-kip Equivalent Single Axle Loads, ESAL over design period)	Yes	Computed from Tables for Vehicle Classification Equivalency Factors (Appendix A)			
Reliability Level, Z_R	Yes		Urban	Rural	
		Interstate	99	97	
		Principal	97	95	
		Collector	90	85	
		Local	75	70	
Serviceability Index, Initial(P_i), Terminal(P_t), and Design Loss(ΔPSI)	Yes		P_i	P_t	ΔPSI
		Interstate	4.3	2.8	1.5
		Principal	4.3	2.5	1.8
		Collector	4.0	2.0	2.0
		Local	3.5	1.5	2.0
Overall Std Dev, S_o	Yes	0.37			
Modulus of Rupture, S'_c	Yes	600 psi or 4.1×10^3 Kpa(metric)			
Elastic Modulus of Slab, E_c	Yes	4.2×10^6 psi or 2.9×10^7 Kpa(metric)			
Drainage Coefficient, C_d	Yes	Drainable Layer Through the Shoulder = 1.1 Long Edge Drain Outside Shoulder = 1.05 No Drainage Feature= 0.90			
Load Transfer Coefficient, J	Yes	AC Shoulder & 12' Truck Lane = 3.2 AC Shoulder & 15" Truck Lane or Conc Shoulder (tied, curb)= 2.5			
Mean Effective k-Value	Yes	From Chart in Appendix E			

Section 3
Determination of Rigid Pavement Thickness

3.1 Using Nomograph

An approximate method of identifying the rigid pavement thickness that will withstand a projected level of axle load traffic is the use of nomograph based on Equation 3.2 Figure 7.1 represents this nomograph. Use of this figure requires information on variables listed in Table 7.1. Caution is recommended in using the thickness derived using this nomograph since it is only an approximation based on mean values. A typical example in the use of this nomograph is given below:

Inputs:

• Performance/Design Period	20 yrs
• 18-kip ESALS over design period (Calculated from equivalency tables in Appendix A)	3.3×10^6
• Initial Serviceability Index, P_i	4.3
• Terminal Serviceability Index, P_t	2.5
• Reliability Level, Z_R	95%
• Overall Standard Deviation, S_o	0.37
• 28-day Mean PCC Modulus of Rupture, S'_c	600psi
• 28-day Mean Elastic Modulus of Slab, E_c	4.2×10^6
• Mean Effective k-value	580 pci
• Load Transfer Coefficient, J	2.5
• Overall Drainage Coefficient, C_d	0.9

Output:

Using the above input data, the nomograph of Figure 7.1 provides approximate thickness of 8.0 in.

NOMOGRAPH SOLVES:

$$\log_{10} W_{18} = Z_R * S_o + 7.35 * \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5 - 1.5}\right]}{1 + \frac{1.624*10^7}{(D+1)^{8.46}}} + (4.22 - 0.32p_t) * \log_{10}\left[\frac{S_c' * C_d \left[D^{0.75} - 1.132\right]}{215.63*J\left[D^{0.75} - \frac{18.42}{(E_c/k)^{0.25}}\right]}\right]$$

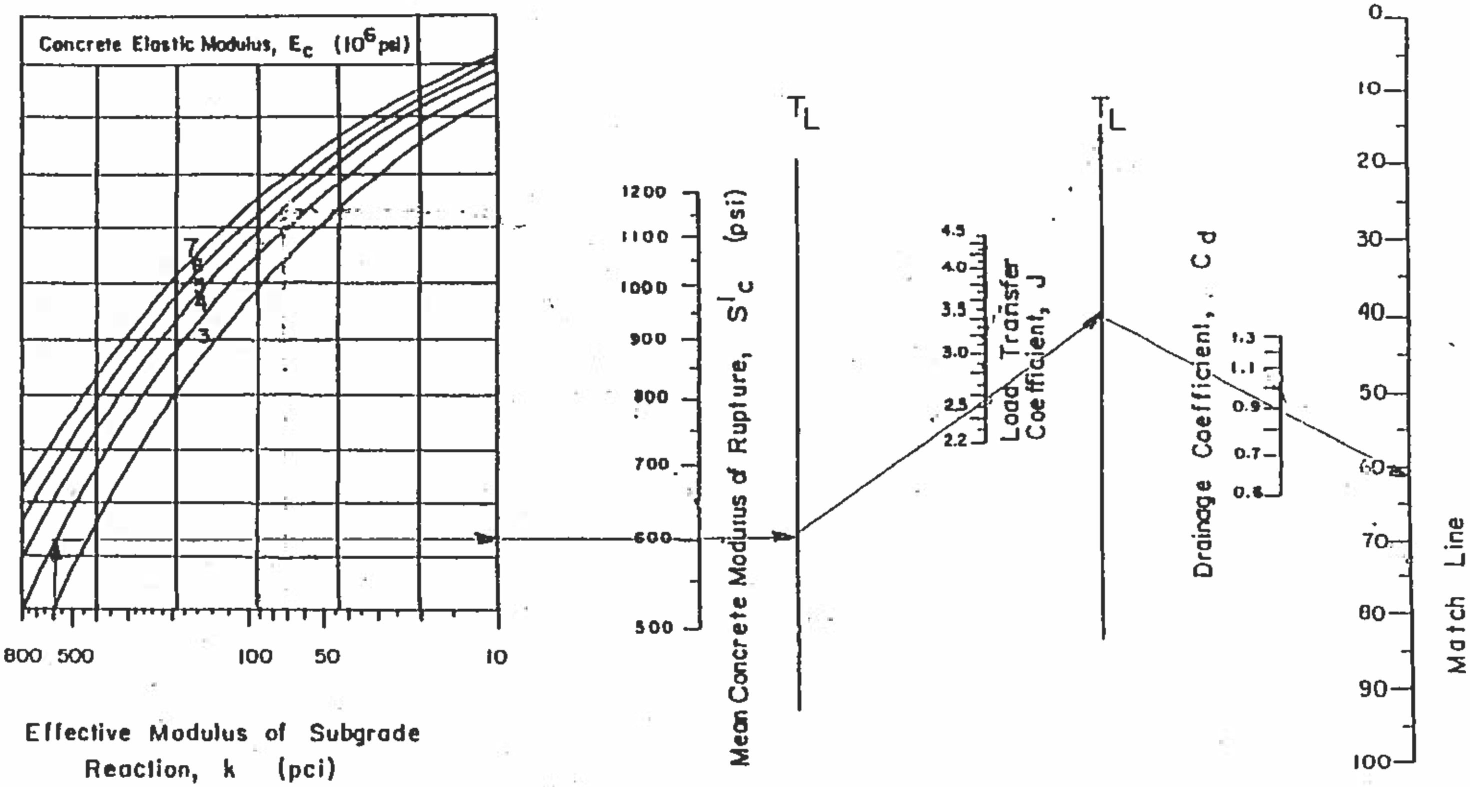

Figure 7.1: Design Chart for Rigid Pavement Based on Using Mean Values for Each Input Variable (Segment 1

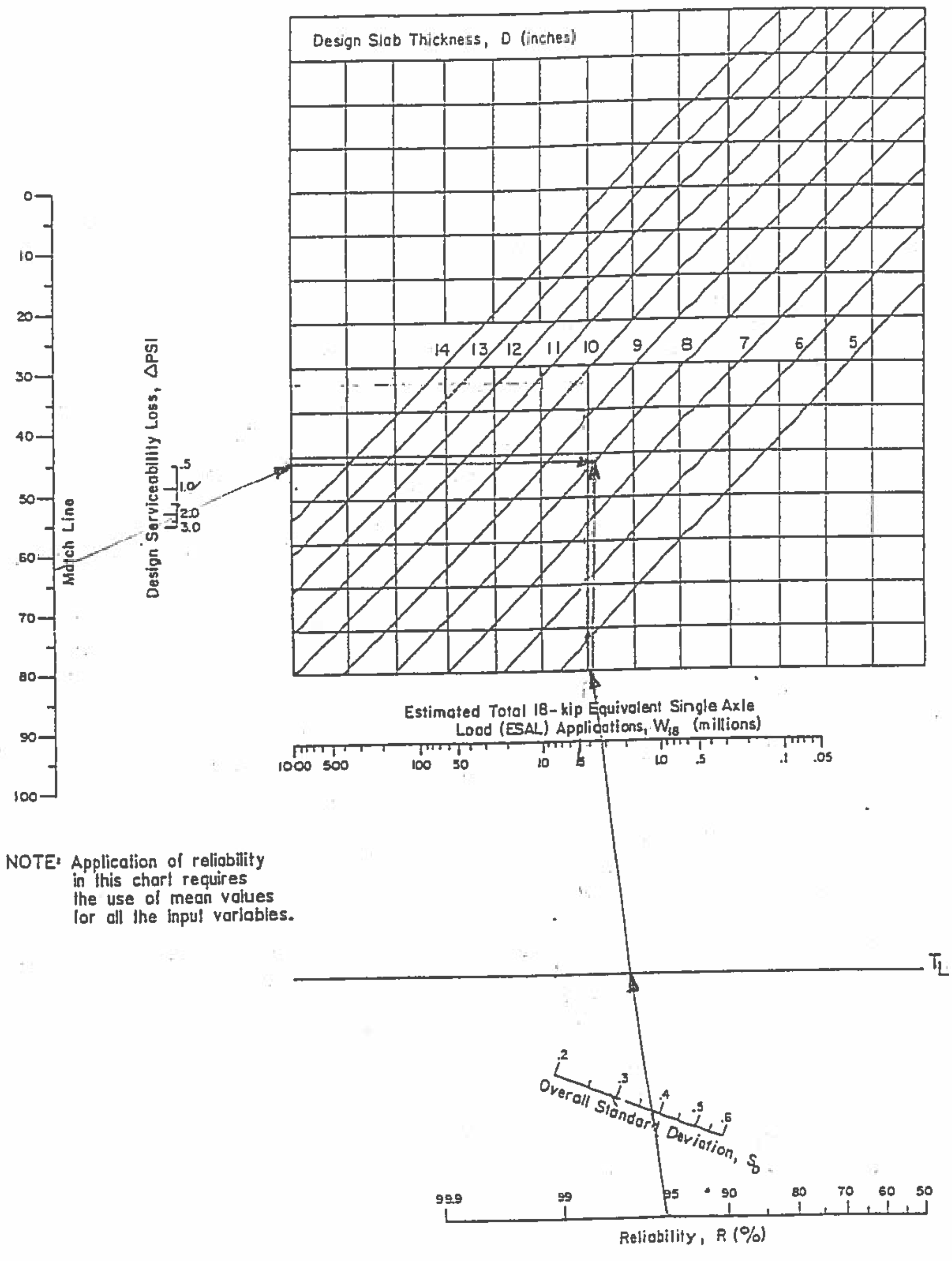

Figure 7.1: Design Chart for Rigid Pavement Based on Using Mean Values for Each Input Variable (Segment 2)

3.2 Using DARWin Pavement Design Software System

As mentioned before, DOTD uses this proprietary software developed and distributed by AASHTO. Two separate input modules are used to arrive at the final thickness of pavement structure.

One set of input data computes 18-kip ESALS over initial performance period. The following information on input is required to compute this 18-kip ESALS.

- Performance period, years
- Two-way traffic, ADT
- Number of lanes in design direction
- Percentage of all trucks in design lane
- Percentage trucks in design direction
- Percentage of ADT for each FHWA vehicle classification
- Annual percent growth
- Equivalency factors for each vehicle classification
  (based on Road System Classification and Terminal Serviceability Index tables in Appendix A)
- Annual percentage growth in truck factor

The second set of input data calculates design thickness from the following information which is obtained from Table 7.1.

- 18-kip ESALS over design period
  (Previously calculated above)
- Initial Serviceability Index
- Terminal Serviceability Index
- 28-day Mean Modulus of Rupture, psi
- 28-day Mean Elastic Modulus of Slab, psi
- Mean Effective k-value, pci
- Reliability Level
- Overall Standard Deviation
- Load Transfer Coefficient, J
- Overall Drainage Coefficient, C_d

An example of the input data and the corresponding output is shown as Figure 7.2. The calculated design thickness comes out to be 7.86 in. or 8.0 in. This compares closely with 8.0 inches derived using the nomograph.

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Rigid Structural Design Module

EXAMPLE

Rigid Structural Design

Pavement Type	JPCP
18-kip ESALs Over Initial Performance Period	3,298,558
Initial Serviceability	4.3
Terminal Serviceability	2.5
28-day Mean PCC Modulus of Rupture	600 psi
28-day Mean Elastic Modulus of Slab	4,200,000 psi
Mean Effective k-value	580 psi/in
Reliability Level	95 %
Overall Standard Deviation	0.37
Load Transfer Coefficient, J	2.5
Overall Drainage Coefficient, Cd	0.9

Input 2

Calculated Design Thickness	7.86 in

Output 2

Rigorous ESAL Calculation

Performance Period (years)	20
Two-Way Traffic (ADT)	2,500
Number of Lanes in Design Direction	2
Percent of All Trucks in Design Lane	80 %
Percent Trucks in Design Direction	100 %

Input 1

Vehicle Class	Percent of ADT	Annual % Growth	Average Initial Truck Factor (ESALs/ Truck)	Annual % Growth in Truck Factor	Accumulated 18-kip ESALs over Performance Period
1	0.2	1.6	0.0004	0	14
2	55.4	1.6	0.0004	0	3,780
3	27.4	1.6	0.0142	0	66,374
4	0.5	1.6	0.1677	0	14,304
5	4.6	1.6	0.1677	0	131,598
6	1.3	1.6	0.5715	0	126,741
7	0.1	1.6	0.5715	0	9,749
8	2.4	1.6	0.9875	0	404,303
9	7.3	1.6	1.7686	0	2,202,473
10	0.5	1.6	2.873	0	245,055
11	0.1	1.6	1.84	0	31,389
12	0.1	1.6	1.84	0	31,389
13	0.1	1.6	1.84	0	31,389
Total	100	-	-	-	3,298,558

Total Calculated Cumulative ESALs	3,298,558

Output 1

Figure 7.2: Rigid Pavement Structural Design using DARWin Software

Chapter 8
Asphaltic Concrete Overlays of Existing Pavements

Contents

Section 1
Overview

Asphaltic Concrete (AC) overlays on flexible and rigid PCC pavements, including fractured PCC pavements, can be categorized as structural and non-structural. Structural overlays are designed using the 1993 AASHTO Pavement Design Guide and DARWin software. The design requires reasonable investigation of the condition of the existing substructure and hot mix on the project to ensure the desired performance of the structural overlay.

Non structural overlays are designed using a combination of experience and established DOTD guidelines. Such overlays are used to improve ride, texture, and cross-slope drainage, and are often categorized as "preventive maintenance."

AC overlays of PCC pavements with existing AC overlay also require extensive condition survey data for input into the DARWin Design software. Thin AC overlays can dramatically improve the skid resistance of CRCP pavements that are still in very good structural condition, with few or no punchouts. AC overlays have also been used on jointed concrete pavements.

This chapter summarizes the input variables and provides a typical example of the completed overlay design using DARWin software.

Section 2
Design Inputs for Overlays on Existing Pavements

2.1 AC Overlays on Flexible Pavement

Table 8.1 summarizes the inputs required for the structural design of asphaltic concrete overlay on flexible pavements. The input is basically the same as in Table 6.1 of Chapter 6. Once the input data is determined, the design thickness can be determined using the DARWin Pavement Design Software System for overlays, which is further discussed in Section 3.

Table 8.1: Summary of Design Inputs for AC Overlays on Flexible Pavements

<table>
<tr><th>Variable</th><th>Input Required ?</th><th colspan="4">Input Value</th></tr>
<tr><td>Performance/Design Period</td><td>Yes</td><td colspan="4">10 Yrs</td></tr>
<tr><td>Traffic (expressed in 18-kip Equivalent Single Axle Loads, ESAL over design period)</td><td>Yes</td><td colspan="4">Computed from Tables for Vehicle Classification Equivalency Factors (Appendix A)</td></tr>
<tr><td rowspan="5">Reliability Level, Z_R</td><td rowspan="5">Yes</td><td></td><td colspan="2">Urban</td><td>Rural</td></tr>
<tr><td>Interstate</td><td colspan="2">99</td><td>97</td></tr>
<tr><td>Principal</td><td colspan="2">97</td><td>95</td></tr>
<tr><td>Collector</td><td colspan="2">90</td><td>85</td></tr>
<tr><td>Local</td><td colspan="2">75</td><td>70</td></tr>
<tr><td rowspan="5">Serviceability Index, Initial(P_i), Terminal(P_t), and Design Loss(ΔPSI)</td><td rowspan="5">Yes</td><td></td><td>P_i</td><td>P_t</td><td>ΔPSI</td></tr>
<tr><td>Interstate</td><td>4.3</td><td>2.8</td><td>1.5</td></tr>
<tr><td>Principal</td><td>4.3</td><td>2.5</td><td>1.8</td></tr>
<tr><td>Collector</td><td>4.0</td><td>2.0</td><td>2.0</td></tr>
<tr><td>Local</td><td>3.5</td><td>1.5</td><td>2.0</td></tr>
<tr><td>Overall Std Dev, S_o</td><td>Yes</td><td colspan="4">0.47</td></tr>
<tr><td>Effective Roadbed Soil Resilient Modulus (M_r)</td><td>Yes</td><td colspan="4">Determined from laboratory tests or Parish maps showing soil types and their typical M_r values (Appendix B)</td></tr>
<tr><td>Layer Coefficients (a_j)</td><td>Yes</td><td colspan="4">From updated values (Appendix C)</td></tr>
<tr><td>Drainage Coefficient</td><td>Yes</td><td colspan="4">m_i in equation $SN=a_1D_1 + a_2D_2m_2 + a_3D_3m_3$</td></tr>
<tr><td>Existing Pavement Condition</td><td>Yes</td><td colspan="4">From PMS data files</td></tr>
</table>

2.2 AC Overlays on AC/PCC Pavements

Table 8.2 summarizes the inputs required for the structural design of asphaltic concrete overlay on existing AC over PCC pavements. The design process also requires condition survey data of existing pavement, which can be obtained from DOTD's Pavement Management System (PMS) files. From this input data, the design thickness can be determined using the overlay design module in DARWin Pavement Design Software System for overlays discussed in Section 3.

Table 8.2: Summary of Design Inputs for AC Overlays on AC/PCC Pavements

Variable	Input Required ?	Input Value			
Performance/Design Period	Yes	10 Yrs			
Traffic (expressed in 18-kip Equivalent Single Axle Loads, ESAL over design period)	Yes	Computed from Tables for Vehicle Classification Equivalency Factors (Appendix A)			
Reliability Level, Z_R	Yes		Urban	Rural	
		Interstate	99	97	
		Principal	97	95	
		Collector	90	85	
		Local	75	70	
Serviceability Index, Initial(P_i), Terminal(P_t), and Design Loss(ΔPSI)	Yes		P_i	P_t	ΔPSI
		Interstate	4.3	2.8	1.5
		Principal	4.3	2.5	1.8
		Collector	4.0	2.0	2.0
		Local	3.5	1.5	2.0
Overall Std Dev, S_o	Yes	0.37			
Modulus of Rupture, S'_c	Yes	600 psi or 4.1×10^3 Kpa(metric)			
Elastic Modulus of Slab, E_c	Yes	4.2×10^6 psi or 2.9×10^7 Kpa(metric)			
Drainage Coefficient, C_d	Yes	Drainable Layer Through the Shoulder = 1.1 Long Edge Drain Outside Shoulder = 1.05 No Drainage Feature= 0.90			
Load Transfer Coefficient, J	Yes	AC Shoulder & 12' Truck Lane = 3.2 AC Shoulder & 15" Truck Lane or Conc Shoulder (tied, curb)= 2.5			
Mean Effective k-Value	Yes	From Chart in Appendix E			
Existing Pavement Condition	Yes	From PMS data files			

Section 3
Determination of Overlay Thickness Using DARWin Software

3.1 AC Overlays on Flexible Pavement

The proprietary software DARWin, discussed in Chapters 6 and 7 for the design of new flexible and rigid pavements, also has a module for structural design of overlays on existing pavements. The software calculates the SN necessary to satisfy the desired performance when subjected to the projected future traffic. Figure 8.1 is an example of the inputs and the corresponding outputs using the overlay design module in DARWin software.

Input 1 consists of traffic data and equivalency factors for calculation of future 18-kip ESALS. This becomes input 2 to calculate the SN for future traffic.

Inputs 3 and 4 calculates the effective pavement thickness for each component (output 3) and the final layer thickness for the overlay (output 4), respectively. The SN from output 4 should be equal to or greater than output 2.

3.2 AC Overlays on AC/PCC Pavement

Figure 8.2 is an example of the input/output of overlay design module in DARWin software for AC overlay of existing AC/PCC pavements.

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

Overlay Design Module

EXAMPLE

AC Overlay of AC Pavement

Structural Number for Future Traffic 1.84 in

Output 2

Design Method	Effective Existing Structural Number (in)	Overlay Structural Number (in)
Component Analysis	0.99	0.85
Remaining Life	-	-
Non-Destructive Testing	-	-

Structural Number for Future Traffic

Future 18-kip ESALs Over Design Period	58,919
Initial Serviceability	4
Terminal Serviceability	2
Reliability Level	85 %
Overall Standard Deviation	0.47
Subgrade Resilient Modulus	10,278 psi

Input 2

Calculated Structural Number for Future Traffic	1.84 in

Output 2

Effective Pavement Thickness - Component Analysis Method

Layer	Material Description	Structural Coefficient	Drainage Coefficient	Thickness (in)
1	Existing AC	0.225	1	5
2	Exisiting Base	0.07	0.9	5

Milling Thickness 2 in

Input 3

Calculated Results

Calculated Pavement Structural Number Before Milling	1.44 in
Calculated Effective Pavement Structural Number	0.99 in

Output 3

Future Rigorous ESAL Calculation

Performance Period (years)	10
Two-Way Traffic (ADT)	496
Number of Lanes in Design Direction	1
Percent of All Trucks in Design Lane	100 %
Percent Trucks in Design Direction	50 %

Input 1

Figure 8.1: Input/Output of AC Overlay Design of Flexible Pavement Using DARWin Software

Vehicle Class	Percent of ADT	Annual % Growth	Average Initial Truck Factor (ESALs/ Truck)	Annual % Growth in Truck Factor
1	2.24	1.3	0.0004	0
2	63.94	1.3	0.0004	0
3	24.4	1.3	0.0143	0
4	0.75	1.3	0.1694	0
5	2.7	1.3	0.1694	0
6	1.55	1.3	0.3836	0
7	0.11	1.3	0.3836	0
8	0.92	1.3	0.8523	0
9	2.93	1.3	1.045	0
10	0.4	1.3	1.45	0
11	0	1.3	1.84	0
12	0	1.3	1.84	0
13	0.06	1.3	1.84	0
Total	100	-	-	-

Input 1

Accumulated 18-kip ESALs over Performance Period
9
246
3,352
1,221
4,394
5,712
405
7,533
29,415
5,572
0
0
1,061
58,919

Growth Compound

Total Calculated Cumulative ESALs	58,919

Output 1

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)
1	Superpave Asphaltic Concrete Wea...	0.44	1	2
Total	-	-	-	2.00

Width (ft)
-
-

Calculated SN (in)
0.88
0.88

Input 4

Output 4

Figure 8.1 (Cont'd): Input/Output of AC Overlay Design of Flexible Pavement Using DARWin Software

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product

Overlay Design Module

EXAMPLE

AC Overlay of AC/PCC Pavement

Pavement Thickness for Future Traffic	7.34 in	
Design Method	Effective Existing Overlay Thickness (in)	Overlay Thickness (in)
Condition Survey	9.35	0.00

Pavement Thickness for Future Traffic

Input 2

Future 18-kip ESALs Over Design Period	1,424,311
Initial Serviceability	4
Terminal Serviceability	2
PCC Modulus of Rupture	600 psi
PCC Elastic Modulus	4,200,000 psi
Static k-value	380 psi/in
Reliability Level	85 %
Overall Standard Deviation	0.37
Load Transfer Coefficient, J	3.2
Overall Drainage Coefficient, Cd	0.9

Output 2

Calculated Thickness for Future Traffic	7.34 in

Effective Pavement Thickness - Condition Survey Method

Input 3

Existing PCC Thickness	8 in
Existing AC Thickness	6 in
AC Milling Thickness	2 in
Rut Depth	- in
Durability Adjustment Factor	1
Fatigue Damage Adjustment Factor	-
AC Quality Adjustment Factor	0.96
Number of Deteriorated Joints	- per mi
Number of Deteriorated Cracks	15 per mi
Number of Unrepaired Punchouts	5 per mi
Number of Expansion Joints,	-
Exceptionally Wide Joints, or AC Full Depth Patches	5 per mi

Calculated Results

Output 3

Calculated Joints and Cracks Adjustment Factor	0.93
Calculated Effective Pavement Thickness	9.35 in

Figure 8.2: Input/Output of AC Overlay Design of AC/PCC Pavement Using DARWin Software

Future Rigorous ESAL Calculation

Performance Period (years)	10
Two-Way Traffic (ADT)	5,900
Number of Lanes in Design Direction	1
Percent of All Trucks in Design Lane	100 %
Percent Trucks in Design Direction	50 %

Input 1

Vehicle Class	Percent of ADT	Annual % Growth	Average Initial Truck Factor (ESALs/ Truck)	Annual % Growth in Truck Factor	Accumulated 18-kip ESALs over Performance Period
1	0.2	1.4	0.0004	0	9
2	59.1	1.4	0.0004	0	2,714
3	27.5	1.4	0.0173	0	54,615
4	0.5	1.4	0.1904	0	10,929
5	5	1.4	0.1904	0	109,286
6	1.6	1.4	0.5934	0	108,993
7	0.1	1.4	0.5934	0	6,812
8	1.6	1.4	1.0222	0	187,752
9	3.8	1.4	1.7901	0	780,891
10	0.3	1.4	2.873	0	98,943
11	0.1	1.4	1.84	0	21,123
12	0.1	1.4	1.84	0	21,123
13	0.1	1.4	1.84	0	21,123
Total	100	-	-	-	1,424,311

Growth Compound

Total Calculated Cumulative ESALs	1,424,311

Output 1

Figure 8.2 (Cont'd): Input/Output of AC Overlay Design of AC/PCC Pavement Using DARWin Software

References Cited

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2. *AASHTO Guide for Design of Pavement Structures 1993*, American Association of State Highway and Transportation Officials, Washington, D.C.

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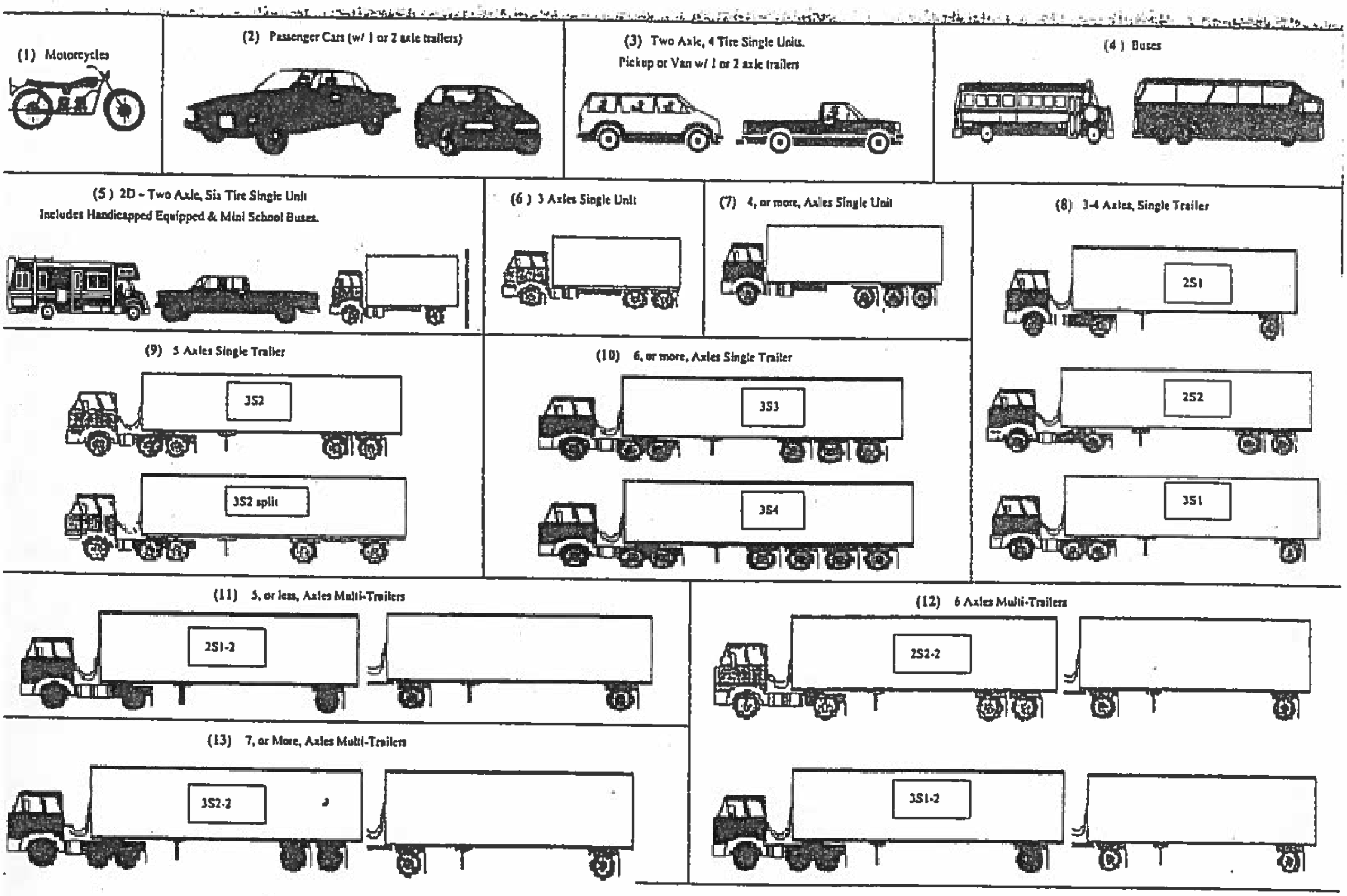
5. Temple, W. H., Carpenter, W., *Implementation of the New AASHTO Pavement Design Procedure in Louisiana,* Research Report No. FHWA/LA-90/218, Louisiana Transportation Research Center, June 1990.

6. Roland, H. L., *Texas Triaxle R-Value Correlation*, Louisiana Department of Highways, Research Section, March 1963.

Appendix A

FHWA Vehicle Classifications
Equivalency Facors for Flexible Pavement Design (P_t=2.0)
Equivalency Facors for Flexible Pavement Design (P_t=2.5)
Equivalency Facors for Rigid Pavement Design (P_t=2.0)
Equivalency Facors for Rigid Pavement Design (P_t=2.5)

FHWA Vehicle Classifications



FHWA VEHICLE CLAASIFICATIONS

FHWA VEHICLE CLASSIFICATION	DESCRIPTION
1	MOTORCYCLES
2	PASSENGER CARS
3	OTHER TWO-AXLE, FOUR-TIRE SINGLE-UNIT VEHICLES
4	BUSES
5	TWO-AXLE, SIX-TIRE SINGLE UNIT TRUCKS
6	THREE-AXLE SINGLE-UNIT TRUCKS
7	FOUR-OR-MORE-AXLE SINGLE-UNIT TRUCKS
8	FOUR-OR-LESS-AXLE SINGLE-TRAILER TRUCKS
9	FIVE-AXLE SINGLE-TRAILER TRUCKS
10	SIX-OR-MORE-AXLE SINGLE-TRAILER TRUCKS
11	FIVE-OR-LESS-AXLE MULTI-TRAILER TRUCKS
12	SIX-AXLE MULTI-TRAILER TRUCKS
13	SEVEN-OR-MORE-AXLE MULTI-TRAILER TRUCKS

FLEXIBLE PAVEMENT

Terminal PSI = 2.0

FHWA VEHICLE CLASSIFICATION	DESCRIPTION	EQUIVALENCY FACTORS
1	MOTORCYCLES	0.0004
2	PASSENGER CARS	0.0004
3	OTHER TWO-AXLE, FOUR-TIRE SINGLE-UNIT VEHICLES	0.0143
4	BUSES	0.1694
5	TWO-AXLE, SIX-TIRE SINGLE UNIT TRUCKS	0.1694
6	THREE-AXLE SINGLE-UNIT TRUCKS	0.3836
7	FOUR-OR-MORE-AXLE SINGLE-UNIT TRUCKS	0.3836
8	FOUR-OR-LESS-AXLE SINGLE TRAILER TRUCKS	0.8523
9	FIVE-AXLE SINGLE-TRAILER TRUCKS	1.0450
10	SIX-OR-MORE-AXLE SINGLE-TRAILER TRUCKS	1.4500
11	FIVE-OR-LESS-AXLE MULTI-TRAILER TRUCKS	1.8400
12	SIX-AXLE MULTI-TRAILER TRUCKS	1.8400
13	SEVEN-OR-MORE-AXLE MULTI TRAILER TRUCKS	1.8400

FLEXIBLE PAVEMENT

Terminal PSI = 2.5

FHWA VEHICLE CLASSIFICATION	DESCRIPTION	EQUIVALENCY FACTORS
1	MOTORCYCLES	0.0005
2	PASSENGER CARS	0.0005
3	OTHER TWO-AXLE, FOUR-TIRE SINGLE-UNIT VEHICLES	0.0188
4	BUSES	0.1932
5	TWO-AXLE, SIX-TIRE SINGLE UNIT TRUCKS	0.1932
6	THREE-AXLE SINGLE-UNIT TRUCKS	0.4095
7	FOUR-OR-MORE-AXLE SINGLE-UNIT TRUCKS	0.4095
8	FOUR-OR-LESS-AXLE SINGLE TRAILER TRUCKS	0.8814
9	FIVE-AXLE SINGLE-TRAILER TRUCKS	1.1000
10	SIX-OR-MORE-AXLE SINGLE-TRAILER TRUCKS	1.4500
11	FIVE-OR-LESS-AXLE MULTI-TRAILER TRUCKS	1.8400
12	SIX-AXLE MULTI-TRAILER TRUCKS	1.8400
13	SEVEN-OR-MORE-AXLE MULTI TRAILER TRUCKS	1.8400

RIGID PAVEMENT

Terminal PSI = 2.0

FHWA VEHICLE CLASSIFICATION	DESCRIPTION	EQUIVALENCY FACTORS
1	MOTORCYCLES	0.0004
2	PASSENGER CARS	0.0004
3	OTHER TWO-AXLE, FOUR-TIRE SINGLE-UNIT VEHICLES	0.0173
4	BUSES	0.1904
5	TWO-AXLE, SIX-TIRE SINGLE UNIT TRUCKS	0.1904
6	THREE-AXLE SINGLE-UNIT TRUCKS	0.5934
7	FOUR-OR-MORE-AXLE SINGLE-UNIT TRUCKS	0.5934
8	FOUR-OR-LESS-AXLE SINGLE TRAILER TRUCKS	1.0222
9	FIVE-AXLE SINGLE-TRAILER TRUCKS	1.7901
10	SIX-OR-MORE-AXLE SINGLE-TRAILER TRUCKS	2.8730
11	FIVE-OR-LESS-AXLE MULTI-TRAILER TRUCKS	1.8400
12	SIX-AXLE MULTI-TRAILER TRUCKS	1.8400
13	SEVEN-OR-MORE-AXLE MULTI TRAILER TRUCKS	1.8400

RIGID PAVEMENT

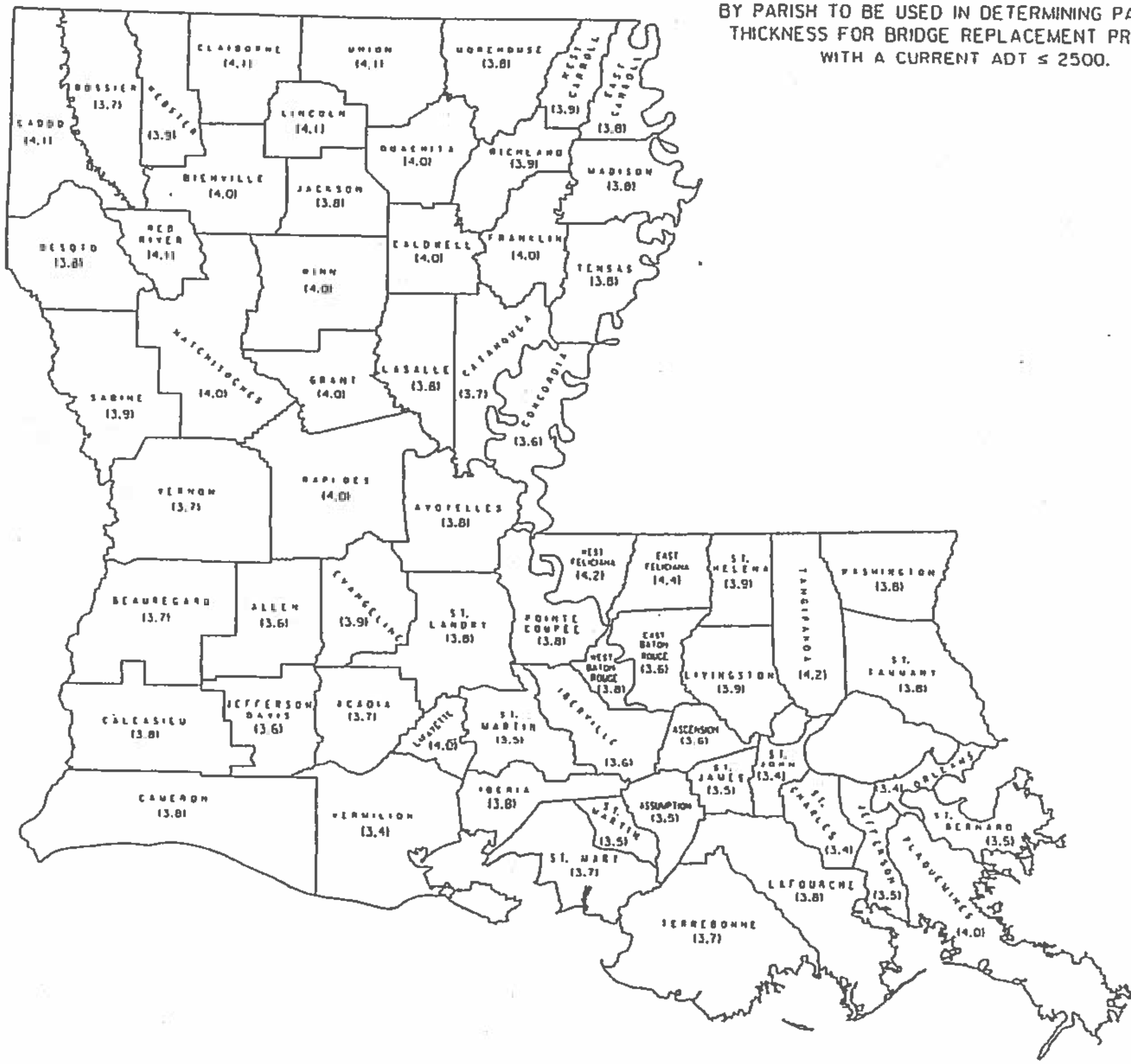
Terminal PSI = 2.5

FHWA VEHICLE CLASSIFICATION	DESCRIPTION	EQUIVALENCY FACTORS
1	MOTORCYCLES	0.0004
2	PASSENGER CARS	0.0004
3	OTHER TWO-AXLE, FOUR-TIRE SINGLE-UNIT VEHICLES	0.0142
4	BUSES	0.1677
5	TWO-AXLE, SIX-TIRE SINGLE UNIT TRUCKS	0.1677
6	THREE-AXLE SINGLE-UNIT TRUCKS	0.5715
7	FOUR-OR-MORE-AXLE SINGLE-UNIT TRUCKS	0.5715
8	FOUR-OR-LESS-AXLE SINGLE TRAILER TRUCKS	0.9875
9	FIVE-AXLE SINGLE-TRAILER TRUCKS	1.7686
10	SIX-OR-MORE-AXLE SINGLE-TRAILER TRUCKS	2.8730
11	FIVE-OR-LESS-AXLE MULTI-TRAILER TRUCKS	1.8400
12	SIX-AXLE MULTI-TRAILER TRUCKS	1.8400
13	SEVEN-OR-MORE-AXLE MULTI TRAILER TRUCKS	1.8400

Appendix B

Soil Support Value (SSV) and Resilient Modulus (M_R)

SOIL SUPPORT VALUE (SSV) & RESILIENT MODULUS (M_R) BY PARISH TO BE USED IN DETERMINING PAVEMENT THICKNESS FOR BRIDGE REPLACEMENT PROJECTS WITH A CURRENT ADT ≤ 2500.



PARISH	SOIL SUPPORT VALUE	RESILIENT MODULUS
ACADIA	3.7	8797
ALLEN	3.6	8413
ASCENSION	3.6	8413
ASSUMPTION	3.5	8023
AVOYELLES	3.8	9176
BEAUREGARD	3.7	8797
BIENVILLE	4.0	9916
BOSSIER	3.7	8797
CADDO	4.1	10278
CALCASIEU	3.8	9176
CALDWELL	4.0	9916
CAMERON	3.8	9176
CATAHOULA	3.7	8797
CLAIBORNE	4.1	10278
CONCORDIA	3.6	8413
DESOTO	3.8	9176
EAST BATON ROUGE	3.6	8413
EAST CARROLL	3.8	9176
EAST FELICIANA	4.4	11330
EVANGELINE	3.9	9549
FRANKLIN	4.0	9916
GRANT	4.0	9916
IBERIA	3.8	9176
IBERVILLE	3.6	8413
JACKSON	3.8	9176
JEFFERSON	3.5	8023
JEFFERSON DAVIS	3.6	8413
LAFAYETTE	4.0	9916
LAFOURCHE	3.8	9176
LASALLE	3.8	9176
LINCOLN	4.1	10278
LIVINGSTON	3.9	9549
MADISON	3.8	9176
MOREHOUSE	3.8	9176
NATCHITOCHES	4.0	9916
ORLEANS	3.4	7627
OUACHITA	4.0	9916
PLAQUEMINES	4.0	9916
POINT COUPEE	3.8	9176
RAPIDES	4.0	9916
RED RIVER	4.1	10278
RICHLAND	3.9	9549
SABINE	3.9	9549
ST. BERNARD	3.5	8023
ST. CHARLES	3.4	7627
ST. HELENA	3.9	9549
ST. JAMES	3.5	8023
ST. JOHN	3.4	7627
ST. LANDRY	3.8	9176
ST. MARTIN	3.5	8023
ST. MARY	3.7	8797
ST. TAMMANY	3.8	9176
TANGIPAHOA	4.2	10634
TENSAS	3.8	9176
TERREBONNE	3.7	8797
UNION	4.1	10278
VERMILLION	3.4	7627
VERNON	3.7	8797
WASHINGTON	3.8	9176
WEBSTER	3.9	9549
WEST BATON ROUGE	3.8	9176
WEST CARROLL	3.9	9549
WEST FELICIANA	4.2	10634
WINN	4.0	9916

Appendix C

Structural Coefficients for Flexible Section Design

LOUISIANA DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT
STRUCTURAL COEFFICIENTS FOR FLEXIBLE SECTION DESIGN
(SEPTEMBER 7, 2005)

		DESIGN COEFFICIENTS	
		U.S. UNITS (SN/IN)	METRIC UNITS (SN/mm)
SURFACE COURSE			
Asphaltic	Superpave AC (WC, BC)	0.44	0.01732
Concrete	SMA (WC)	0.44	0.01732
BASE COURSE			
	Crushed Stone or Crushed Slag	0.14	0.00551
	Recycled Portland Cement Concrete	0.14	0.00551
Cement Stabilized	Soil Cement (Stabilized)	0.14	0.00551
or Treated	Soil Cement (Treated)	0.10	0.00394
Asphalt	Superpave AC (Base Course)	0.33	0.01299
SUBBASE COURSE			
	Soil Cement	0.14	0.00551
	Crushed Stone	0.14	0.00551
COEFFICIENTS FOR BITUMINOUS CONCRETE OVERLAY			
	Superpave AC overlay of AC Pavement	0.225	0.00890
	Break & Seat	0.25	0.00984
	Rubblized	0.25	0.00984

Appendix D

DOTD Asphaltic Pavement Design Policy

DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT

INTRADEPARTMENTAL CORRESPONDENCE

REFERRED TO

___ REFERRED FOR ACTION
___ ANSWER FOR MY SIGNATURE
___ FOR FILE
___ FOR YOUR INFORMATION
___ FOR SIGNATURE
___ RETURN TO ME
___ PLEASE SEE ME
___ PLEASE TELEPHONE ME
___ FOR APPROVAL
___ PLEASE ADVISE ME

BY ___ DATE ___
BY ___ DATE ___
BY ___ DATE ___

IN REPLY REFER TO
FILE NO.

MEMORANDUM

TO: Each District Administrator

Each Assistant District Administrator
Mr. Guy Leonard
Road Design Administrator

Ms. Simone Ardoin
Systems Preservation Engineer Administrator

Mr. Ed Wedge
Contract Services Administrator

Mr. Randal Sanders
Contracts and Specifications Engineer

Mr. Jeff Lambert
Pavement Engineer Manager

FROM: Mr. Richard Savoie
Chief Engineer

SUBJECT: ASPHALTIC PAVEMENT DESIGN POLICY
2006 Standard Specifications - Part V (and Associated Supplemental Specifications)

DATE: September 3rd 2010

This memorandum sets forth current policy and design criteria for asphaltic pavements in accordance with Part V of the 2006 Louisiana Standard Specifications for Roads and Bridges, as amended and the 8/2010 Special Provision for Thin Lift Asphalt. This memorandum supersedes the current design policy memorandum, dated January 5, 2007. This policy covers the Special provision for Section 501 (Thin Asphaltic Concrete Applications), 502 (Superpave Asphaltic Concrete Mixtures), 507 (Asphaltic Surface Treatment), and 508 (Stone Matrix Asphalt). Exceptions to these requirements will require prior approval of the DOTD Chief Engineer.

RECOMMENDED FOR APPROVAL — DATE 3 SEP 10

RECOMMENDED FOR APPROVAL — DATE

RECOMMENDED FOR APPROVAL — DATE

APPROVED — DATE 9.24.10

I. ASPHALTIC CONCRETE PAVEMENT

A. Pavement Design

Submit the proposed typical section(s) and/or design to the Pavement Engineer Manager, along with any additional pavement design information available such as traffic data from Transportation Planning, and the subgrade soil survey.

The following describes the pavement design life required in designing the pavement thickness:

New structure – 20 years
Rubblize and overlay – 15 years
Structural overlay – 10 years

Note: The actual service life may be longer than the structural design life and is to be used for general information only when comparing surface types prior to selection.

B. Mixture Selection

Mixture selection depends on the pavement thickness, structural design, condition of the existing road, current ADT (must be listed on title sheet of plans), and required grade adjustments. The following is a brief description of mixture types and uses included in the Standard Specifications:

Section	Mixture Selection	Description	Estimated Service Life (years)
501	Thin Asphaltic Concrete Applications	A wearing course which could be specified on new or existing roadways. It is intended to be placed on a roadway which is structurally sound, without base failures. This mixture will seal cracks, slightly improve smoothness, and improve skid resistance. This mix is not conducive to handwork. This should only be specified when the lift thickness is < 1 ½ inches.	10 – 15
502	Superpave Asphaltic Concrete Mixtures	The dense-graded asphalt mixture, appropriate for a wide variety of applications. More costly than chip seal, but less costly than stone matrix asphalt, it improves smoothness, increases structure, seals, evens out surface defects such as cracking and rutting, and provides adequate skid resistance.	12 – 17

507	Asphaltic Surface Treatment (AST) or "chip seal"	Should be placed on a roadway which is structurally sound, with minimal cracking, rutting and raveling. This material will seal the smaller cracks and improve skid resistance at lowest initial cost. This does not significantly increase grade and will not improve smoothness of the road.	5 – 7
508	Stone Matrix Asphalt (SMA)	A wearing course which provides excellent rut resistance and structure. Because of the high asphalt content, SMA improves crack resistance. The large amount of coarse aggregate gives this mixture stone on stone contact for superior structural support. The higher asphalt cement content ensures durability and resistance to oxidation. This is the most expensive mix. It improves smoothness. It is not conducive to handwork.	15 – 20

Note: Refer to DOTD approved Specifications and Special Provisions when designing any surface application not listed in this memo such as Microsurfacing and Slurry Seals. If the desired application cannot be found, contact the DOTD Contracts and Specifications Section.

1. Mainline Mixes

Mainline mixes include travel lane wearing, binder, and base courses; ramps; acceleration/deceleration lanes; turnlanes;
and the two center lanes for airports. The mainline mix selection guide is as follows:

<table>
<tr><th>Total Plan Thickness (inches)</th><th>Wearing Course Lift Thickness[1] (inches)</th><th>Current ADT</th><th>Location</th><th>Section</th></tr>
<tr><td colspan="2">< 1.5</td><td>< 7,000</td><td>Wearing Course</td><td>501 or 507</td></tr>
<tr><td colspan="2">< 1.5</td><td>≥ 7,000 and < 35,000</td><td>Wearing Course</td><td>501</td></tr>
<tr><td rowspan="6">≥ 1.5</td><td rowspan="3">< 1.5</td><td rowspan="3">< 35,000</td><td>Wearing Course</td><td>501</td></tr>
<tr><td>Binder Course</td><td rowspan="5">502</td></tr>
<tr><td>Base Course</td></tr>
<tr><td rowspan="3">≥ 1.5</td><td rowspan="3">≥ 7,000 and < 35,000</td><td>Wearing Course</td></tr>
<tr><td>Binder Course</td></tr>
<tr><td>Base Course</td></tr>
<tr><td rowspan="4">Any</td><td>< 1.5</td><td>≥ 35,000</td><td>Wearing Course</td><td>501 Coarse Graded or OGFC</td></tr>
<tr><td rowspan="3">≥ 1.5</td><td>≥ 35,000</td><td>Wearing Course</td><td>508</td></tr>
<tr><td rowspan="2">≥ 35,000</td><td>Binder Course</td><td rowspan="2">502</td></tr>
<tr><td>Base Course</td></tr>
<tr><td colspan="2">< 1.5</td><td>Safety Applications[2]</td><td>Wearing Course</td><td>501 "Thin Asphaltic Concrete – OGFC"</td></tr>
</table>

[1]Consider adding a cold planing item or a surface preparation item when designing overlays with IRI >170 inches/mile or rutting > 0.5 inch as measured on most recent PMS survey.
[2]To reduce wet weather accidents and/or overspray.

a. Section 501 (Thin Asphaltic Concrete Applications)

When a thin (< 1.5 inches) lift of asphaltic concrete is desired, select Section 501 (Thin Asphaltic Concrete Applications) and use pay item No. 501-01-00001, which is paid by the ton. The pavement minimum design thickness is 0.75 inch. The design engineer must be cognizant of the existing grade and cross slope when selecting the average thickness design for asphaltic pavement applications.

For interstate surfaces and all safety applications to reduce wet weather accidents and overspray, the designer shall require on plan typical section "Thin Asphalt Concrete Applications, OGFC". For all other Thin Asphalt Concrete Applications, specify "Thin Asphalt Concrete."

The Standard Specifications describe the following thin lift mixture types available as limited by the current ADT listed on plans:

Thin Lift Mixture Type	Uses
Fine Mix	intended for low volume traffic (< 3000 ADT), < 50 mph (posted), and thin leveling
Dense Graded Wearing Course	intended for low and medium volume traffic (≤ 7,000 ADT)
Coarse Graded Wearing Course	low, medium and high volume traffic
Open Graded Friction Course (OGFC)	allowed for all levels of traffic volume, and required when thin lift is used on Interstate Highway System , and safety applications

b. Section 502 (Superpave Asphaltic Concrete Mixtures)

For Section 502, the design load level is determined by 20-year equivalent single axle loads (ESALs). The 20-year ESALs shall be shown on the applicable typical section sheet(s) of the plans. Designate mixture on the plans as Superpave Asphaltic Concrete Wearing, Binder or Base Course (Level __).

For projects with borderline ESALs, greater than 15 % truck traffic, or posted speed less than 45 mph with frequent stops (urban areas), the designer may propose an increase in design load level or the use of SMA wearing course.

Use minor superpave asphaltic pavement mixes as noted in table below.

For mainline superpave asphalt mixes, determine and designate the design level and pay item in accordance with ESALs, ADT, and allowable lift thickness given in the following table:

<table>
<tr><th colspan="2">Location</th><th>20 Year ESALs</th><th>Current ADT</th><th>Design Load Level</th><th>Lift Thickness (inches)</th><th>Pay Item No.</th></tr>
<tr><td rowspan="8">Travel Lane</td><td rowspan="4">Wearing Course except for OGFC</td><td rowspan="2">≤ 3 million</td><td>< 7,000</td><td>1</td><td>1.5 – 3</td><td rowspan="4">502-01-00100</td></tr>
<tr><td>7,000 – 35,000</td><td>1F</td><td rowspan="3">2 – 3</td></tr>
<tr><td rowspan="2">> 3 million</td><td>< 7,000</td><td>2</td></tr>
<tr><td>7,000 – 35,000</td><td>2 F</td></tr>
<tr><td>Wearing Course OGFC</td><td>All</td><td>All</td><td>N/A</td><td>0.75 – 1.5</td><td>501-01-00001 (Designate as "Thin Asphaltic Concrete, OGFC")</td></tr>
<tr><td rowspan="2">Binder Course</td><td>≤ 3 million</td><td rowspan="3">All</td><td>1</td><td rowspan="2">2 – 4</td><td rowspan="5">502-01-00100</td></tr>
<tr><td>> 3 million</td><td>2</td></tr>
<tr><td>Base Course</td><td>N/A</td><td>1</td><td>≥ 2.5</td></tr>
<tr><td rowspan="2">Airport</td><td>Wearing and Binder Courses</td><td colspan="2" rowspan="2">N/A</td><td>A</td><td>1.5 – 2</td></tr>
<tr><td>Airport Base</td><td>1</td><td>> 2.5</td></tr>
</table>

c. Section 507 (Asphaltic Surface Treatment)

When using asphaltic surface treatment, select the type and number of applications in accordance with Section 507 and the following table:

Traffic Count or Usage	Surface Treatment Type	No. Applications Allowed[1]
3,000 – 7,000 ADT	A	1 or 2
100 – 2,999 ADT	B	1 or 2
Shoulders and 0 – 1,500 ADT[2]	C	1
Less than 100 ADT, other uses (parking areas, etc)	D	1, 2 or 3
Crack Mitigation Interlayer [3]	E	2

[1]Multiple application ASTs have less noise from traffic, provide additional waterproofing, and are more durable than single application ASTs, but cost more.

[2]When compared to a 1-application Type B AST, Type C is beneficial on oxidized lower volume roads with larger crack patterns by providing larger aggregate and additional asphalt for sealing.
[3]Over existing asphalt, milled, or reconstructed surfaces, or new soil cement bases.

The quantities of liquid asphaltic material and aggregate incorporated in the completed and accepted asphaltic surface treatment will be measured separately:

Item	Unit of Measurement	Pay Item No.
Asphaltic Material	Gallon	507-01-00100 (CRS-2P Emulsion)
		507-01-00200 (PAC-15 Hot Application)
Aggregate	Square Yard	507-02-00100 (S1)
		507-02-00200 (S2)
		507-02-00300 (S3)

Design quantities are based on horizontal dimensions. Multiple applications of AST shall be based on the same quantity of surface area as a single application.

d. Section 508 (Stone Matrix Asphalt)

For Stone Matrix Asphalt wearing course, adhere to the following:

Location	Current ADT	Lift Thickness (inches)	Pay Item No.
Travel Lane Wearing Course	> 35,000	1.5 – 2	508-01-00100

2. Minor Mixes

Minor mixes include mixtures used for bike paths, crossovers, curbs, diversion roads, driveways, guard rail widening, islands, joint repair, leveling, medians, parking lanes, parking lots, patching, shoulders, turnouts, widening, miscellaneous handwork, and any other mixture that is not mainline.

For minor superpave asphalt mixes, determine the level and pay item according to the following table:

Location	Design Level	Pay Item No.
Superpave Asphaltic Concrete (Base, Binder, and Wearing)	"Superpave Asphaltic Concrete" (do not specify level)	502-01-00100
Shoulder		
Leveling[1]		
Bike Paths, Parking Lanes and Parking Lots	A	
Curb	A	
Diversion Roads[2]	A	
Joint Repair	A	510-03-00100
Crossovers, Driveways, Guard Rail Widening, Medians, Islands, Turnouts	A	502-01-00200
Patching	"Superpave Asphaltic Concrete" (do not specify level)	510-01-00100
Widening[3]		510-02-00100

[1] Mix used for leveling or slope correction shall have no minimum lift thickness. Leveling thickness shall not exceed 2". Use mainline mix types when leveling exceeds 2".
[2] For temporary diversion roads that have ADT > 35,000, specify as Superpave Asphaltic Concrete (Level 1F) when the detour is expected to be in place for at least one year or if the detour is over 1 mile long. Otherwise, specify Superpave Asphaltic Concrete (Level A).
[3] Does not include widening to add a lane.

II. GENERAL REQUIREMENTS

A. Weight measurement (0.01)Tons, of asphaltic concrete mixtures is used for wearing, binder, base and incidental mixtures. Estimated quantities are base upon 110 lbs/SY/in. Estimated quantities for OGFC are based upon 105 lbs/SY/in.

B. Volume measurement (1) cu.yd. of asphaltic concrete mixtures may be used for Class II Base course mixtures on new construction.

C. Use of area measurement (1) Sq. yd. requires design exception except for minor mixes.

CDA/

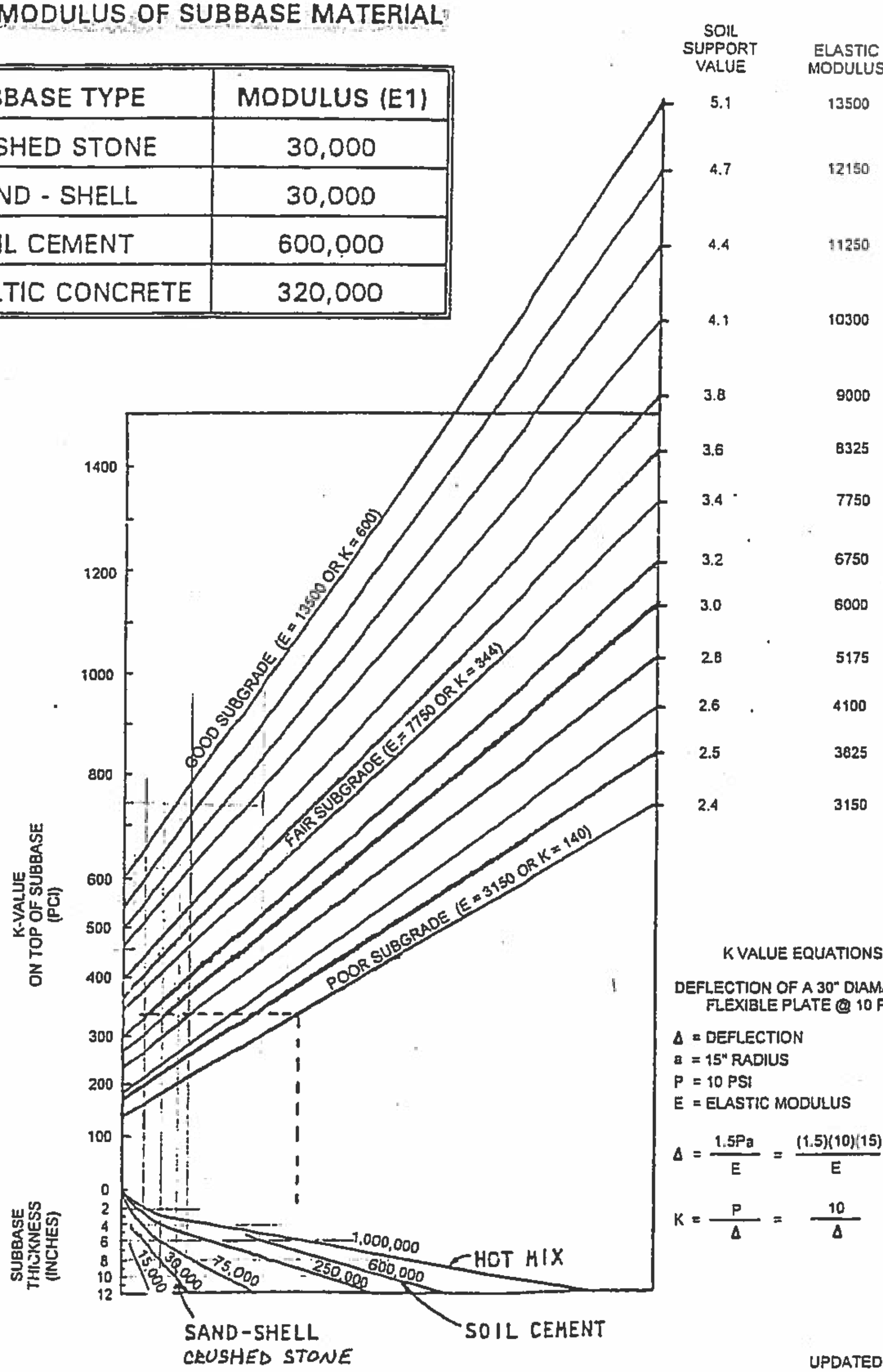
pc: Mr. Vince Russo
Ms. Janice Williams
Mr. Brian Buckel
Mr. Barry Lacy
Mr. Danny Smith
Ms. Valerie Horton
Mr. Rhett Desselle
District Area Engineers
District Laboratory Engineers
Mr. Skip Paul
Ms. Luanna Cambas
Mr. Chris Abadie
Mr. David Miller
Mr. Tom Atkinson
Mr. Mark Chenevert
Mr. Ken Naquin, AGC Chief Executive Officer
Mr. Don Weathers, LAPA Executive Director
Mr. Phil Arena, FHWA

Appendix E

Chart for calculating Mean Effective k-value

TABLE 3
ELASTIC MODULUS OF SUBBASE MATERIAL

SUBBASE TYPE	MODULUS (E1)
CRUSHED STONE	30,000
SAND - SHELL	30,000
SOIL CEMENT	600,000
ASPHALTIC CONCRETE	320,000



K VALUE EQUATIONS

DEFLECTION OF A 30" DIAMAETER FLEXIBLE PLATE @ 10 PSI

Δ = DEFLECTION
a = 15" RADIUS
P = 10 PSI
E = ELASTIC MODULUS

$$\Delta = \frac{1.5Pa}{E} = \frac{(1.5)(10)(15)}{E} = \frac{225}{E}$$

$$K = \frac{P}{\Delta} = \frac{10}{\Delta}$$

UPDATED 2-1-00