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ANALYSIS OF THE 2002 DESIGN GUIDE FOR RIGID PAVEMENT

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CHAPTER 1 – INTRODUCTION

BACKGROUND

Traditionally, rigid pavement design has been accomplished through empirically based procedures. Probably the most well-known procedure for highway pavements, the AASHTO Method, was based on the AASHO Road Test held near Ottowa, Illinois between 1958 and 1960. The design procedure utilized empirical relationships developed from the AASHO Road Test and is therefore limited to the conditions of that test. All empirically-based methods share this same common disadvantage in that they are limited to the conditions and observations of the particular road sections on which the procedure was based. This can cause problems by forcing designers to extrapolate outside the original bounds of the method.

Conversely, mechanistic-empirical (M-E) design is more robust since it combines the elements of mechanical modeling and performance observations in determining the required pavement thickness for a set of design conditions. The mechanical models are based on physics, not empirical relationships, and determine pavement stresses and strains due to wheel loads. The empirical part of the design uses the calculated stresses and strains to predict the life of the pavement based on site-specific field performance. Basically, M-E design has the capability of changing and adapting to new developments in pavement design by relying primarily on the mechanics of materials

The 2002 Design Guide for flexible and rigid pavements is a mechanistic-empirical design approach that is a product of years of research undertaken by the AASHTO Joint Task Force, the National Cooperative Highway Research Program

(NCHRP), and a research team comprised of several internationally recognized pavement design experts. This state-of-the-art design approach was developed under National Cooperative Highway Research Program Project 1-37A.

The 2002 Design Guide is extensive and comprehensive. It includes procedures for the analysis and design of new, reconstructed, and rehabilitated asphalt and concrete pavements, procedures for evaluating existing pavements, procedures for subdrainage design, recommendations for rehabilitation treatments and foundation improvements, and procedures for life cycle cost analysis. The design procedure, based on a mechanistic-empirical (M-E) methodology, is integrated into a software program called "Design Guide 2002". The mechanistic-empirical design procedure to be included in the 2002 Design Guide will allow the designer to evaluate the effect of variations in materials (both inherent and due to construction procedures) on pavement performance. The 2002 Design Guide will provide a rational relationship between construction and materials specification and the design of the pavement structure.

Since M-E procedures can account for climate, aging, modern materials, and modern vehicle loadings, variation in performance with relation to design life should be reduced. That feature will allow agency managers to make better decisions based on life cycle costs and cash flow. According to a report entitled "LTPP and the 2002 Design Guide" issued by the LTPP research staff in 2001, based on probabilistic life cycle cost analysis, it is conservatively estimated that improved pavement design procedures will reduce premature failures and result in average annual savings in pavement rehabilitation costs of $1.14 billion per year over the next 50 years. This analysis was based on a design life of 20 years and the assumption that the percentage of pavement failures within the first 10 years of a pavement's life would drop from 5 percent to 0.5 percent. It was further assumed that the range of performance lives for the remaining pavements, 10 to 30 years for current practice, would increase to 15 to 30 years using the 2002 procedure.

During the development of previous pavement design guides, the AASHTO design staff recognized that future design procedures would eventually need to be based on mechanistic-empirical principles due to constantly changing load configurations, different subgrade types, and different environmental conditions than that encountered during the AASHO Road Test of 1958. However, for such an approach to be practical, it would require that pavement designers have ready access to computers capable of handling mechanistic design programs. Today, personal computers with computational capabilities greater than many mainframes of the early 1980s are on virtually every engineer's desk. Pentium® and higher-level personal computers should be adequate for the design procedures that will be developed. The following is a list of the benefits of the mechanistic design procedure. This list was taken from the NCHRP 1-37A Fall 2001 Research Summary:

1. The consequences of non-traditional loading conditions can be evaluated. For example, the damaging effects of increased loads, high tire pressures, and multiple axles can be modeled.
2. Better use of available materials can be made. For example, the use of stabilized materials in both rigid and flexible pavements can be simulated to predict future performance.

3. Improved procedures to evaluate premature distress can be developed to analyze why some pavements exceed their design expectations. In effect, better diagnostic techniques can be developed.
4. Aging effects can be included in estimates of performance. For example, PCC strength increases with time, which, in turn, affects slab cracking.
5. Seasonal effects such as thaw weakening can be included in estimates of performance.
6. Consequences of subbase erosion under rigid pavements can be evaluated.
7. Methods can be developed to better evaluate the long-term benefits of providing improved drainage in the roadway section.

Currently, the 1993 *AASHTO Guide for Design of Pavement Structures* is the primary document used by State Highway Agencies, including the Alabama Department of Transportation, to design new and rehabilitated highway pavements. According to a brief historical summary developed by the NCHRP 1-37a research team, the Federal Highway Administration's 1995-97 National Pavement Design Review found that some 80% of the States make use of either the 1972, 1986, or 1993 AASHTO Pavement Design Guides. All of these versions are empirically based on performance equations developed using 1958-1960 AASHO Road Test data. The 1986 and 1993 guides contained some refinements in materials input parameters and empirical procedures for rehabilitation design.

The AASHTO Joint Task Force on Pavements (JTFP) has responsibility for the development and implementation of pavement design technologies. This charge has been pursued by the JTFP since the AASHO Road Test and led to the development of the 1993

AASHTO Guide and prior Guides. More recently, and in recognition of the limitations of earlier Guides, the JTFP initiated an effort to develop an improved Guide by the year 2002. As part of this effort, a workshop was convened in California during 1996 to develop a framework for improving the Guide. The workshop attendees—pavement experts from public and private agencies, industry, and academia—addressed the areas of traffic loading, foundations, materials characterization, pavement performance, and environment to help determine the technologies best suited for the 2002 Guide. At the conclusion of that workshop, a major long-term goal identified by the JTFP was the development of a design guide based as fully as possible on mechanistic principles. The 2002 Design Guide is the fruition of that goal and the subsequent research efforts. The 2002 Design Guide is based on a mechanistic empirical design procedure and is depicted in Figure 1.1.

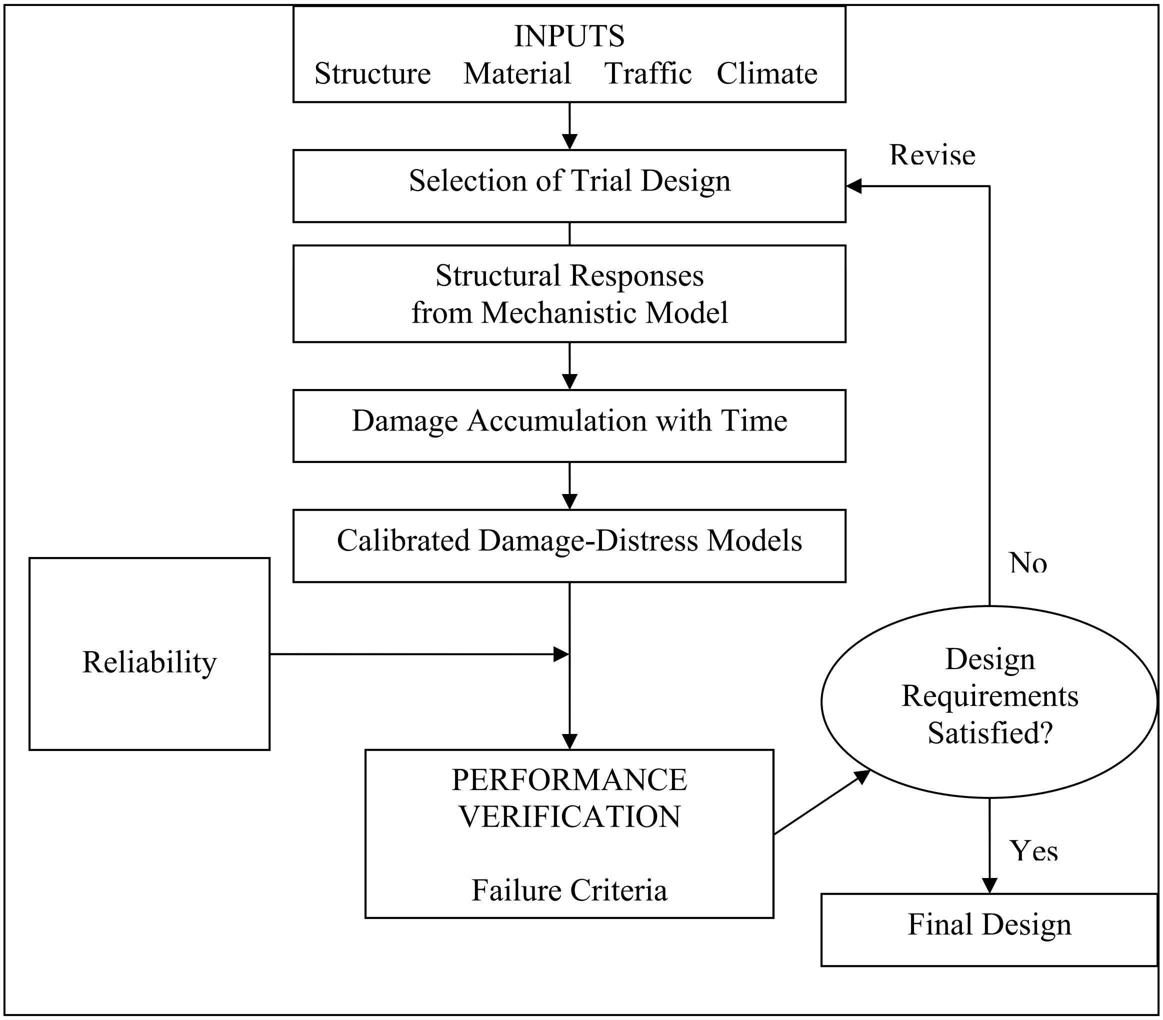

Figure 1.1 2002 Design Process (after Design Guide, 2002).

There are many new inputs and parameters that are incorporated into the 2002 Design Guide as well as new methods that greatly differ from the 1993 Design Guide in order to more effectively determine the proper pavement, base, and subbase types and thicknesses for a given traffic flow and climate. This poses a special challenge to agencies currently using the 1993 Design guide since use of the new guide may require additional training and equipment to fully implement the 2002 Design Guide.

Generally speaking, the 2002 Design Guide is an iterative process and includes the following basic steps for rigid pavement design:

1. Inputs are gathered for the trial design. These include the structure of the pavement (i.e.base type, subbase type, type of subgrade, slab specifications, etc.), the material strengths used, the expected traffic loading on the pavement structure, and the environmental conditions.
2. The designer inputs a trial design. This includes slab lengths, slab thickness, base thickness, load transfer mechanisms (i.e. dowels or no dowels).
3. Once the trial design is set by the designer, the 2002 Design Guide program is executed. The software first calculates structural responses due to traffic loading and environmental conditions.
4. The program then predicts the damage and key distresses over the design life. These key distresses include mean joint faulting, percent of slabs cracked, and a smoothness prediction, International Roughness Index (IRI).
5. The trial design is checked against the preset distress criteria through output files displayed in a spreadsheet format, and the design may be modified as needed to meet performance and reliability requirements.

This procedure is expected to be a vast improvement over the current empirical AASHTO methodology pictured schematically Figure 1.2.

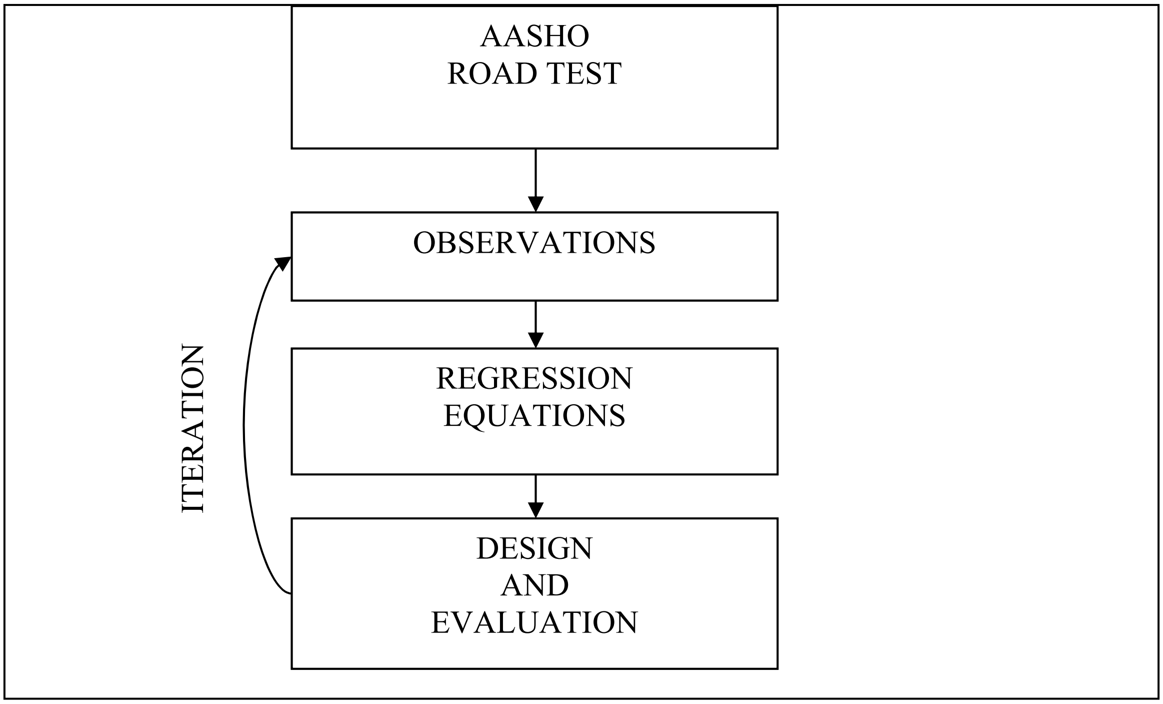

Figure 1.2 Current AASHTO Empirical Design Approach.

As shown in Figure 1.2, the empirical approach is based on the AASHO Road Test. In essence, it is based, one type of traffic loading, one climate, one subgrade soil, and one base type. The observations made from this road test were transformed into regression equations for the design of rigid and asphalt pavements. The limited design variables used to generate the data at the AASHO Road Test cause much extrapolation from the original observations through the regression equations, and thus shows the necessity for a more mechanistic-based approach to pavement design.

While there is a national movement toward M-E based pavement design, and the reasons for the shift are well documented, there is a serious concern about adopting this method at the state agency level. The current design methods have been in place for some time and there is extensive experience regarding their use in practice. Therefore, prior to adoption of a new approach, there must be an extensive investigation of the approach to warrant its use in practice. Specifically, in regard to rigid pavement design, there are three main questions that must be answered:

1. When designing for the same design conditions, what slab thickness will the new approach yield relative to the old approach?
2. How sensitive is the new approach to the various design inputs?
3. Is there a definable relationship between the old and new methods of rigid pavement design?

OBJECTIVES

The objectives of this research serve to answer the questions above and include:

1. Establish a link between the existing methodology (1993 Design Guide) and the new methodology (2002 Design Guide).
2. Identify inputs that are not significant in the final design and analysis so that greater resources may be devoted to those that are significant.
3. Compare design thicknesses generated by the 1993 Design Guide with those obtained through the 2002 Design Guide.

SCOPE AND OVERVIEW

A literature review was conducted to explore current mechanistic-empirical rigid pavement design methods as well as the current accepted pavement design methodologies for rigid pavement. Information was obtained regarding analysis techniques within each approach.

An analysis was performed to compare the output of the 2002 Design Guide with the output of the 1993 Design Guide. Several hundred simulations were run comparing the 1993 and the 2002 Design Guide for rigid pavements and the output received was analyzed through regression analyses. The 2002 Design Guide was further analyzed through a slab thickness comparison between the 2002 Design Guide and the current 1993 AASHTO Design Guide and also through a sensitivity analysis based on the new inputs for rigid pavement design in the 2002 Design Guide.

CHAPTER 2 – LITERATURE REVIEW

PAVEMENT DESIGN BACKGROUND

Although pavement design has gradually evolved from art to science, empiricism still plays an important role. Prior to the 1920's, the thicknesses of pavements were based purely on experience. The same thickness was used for a section of highway even though widely different soils were encountered. As experience was gained throughout the years, various methods were developed by different agencies for determining the thickness of pavement required. It is neither feasible nor desirable to document all the methods that have been used so far; however the majority of pavement design has been done empirically.

Empirical design is based on a pavement's ability to withstand traffic over a given time frame known as a design period (Huang, 1993). The ability to withstand traffic is measured by an overall ride quality factor called the Present Serviceability Index (PSI), a concept introduced at the AASHO Road Test. The PSI is a rating of the performance of the pavement based on smoothness of travel over the pavement. PSI ranges from 5 (optimum performance) to 0 (worst performance) (Yoder and Witczak, 1975). Test data were obtained from the American Association of State Highway Officials (AASHO) Road Test, and for each section a PSI vs. Time plot was formed from the start of traffic operations until the PSI dropped to a minimum tolerance, typically 1.5. This is shown in Figure 2.1.

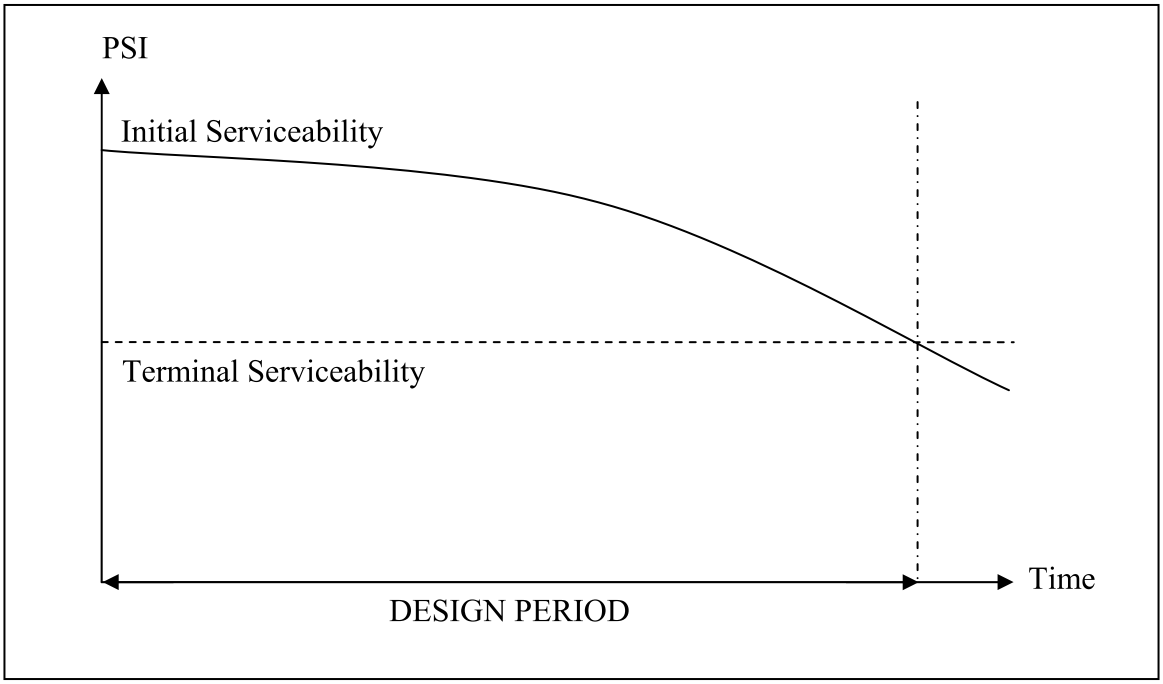

Figure 2.1 Serviceability vs. Time.

These historical data were used to develop equations for flexible and rigid pavements (McAuliffe et al., 1994). The main drawback to empirical pavement modeling is at the foundation of the AASHO Road Test. At the AASHO road test only certain subgrade types and strengths and only certain slab properties as well as a single environment type were utilized. Any differing design conditions from the original properties at the AASHO Road Test are to be considered an extrapolation. In most cases, design inputs vary geographically and are rarely unique or constant, thus available empirical procedures are, in a sense, inadequate.

Traffic weights and volume are the most variable and uncertain design inputs due to regional differences in economies. Subgrade strength is also a highly variable input to the design process. Within a particular state, there may be a wide variety of soil types that require different design considerations. Pavement layer strength is highly variable due to material type, layer location, and construction control exercised to obtain uniformity in the material. Some empirical or deterministic design methods apply a general variability term for all inputs; however this is inadequate as all inputs vary at different levels (Basma et.al., 1989).

Selecting the most effective and economical design for a given project is imperative to the pavement engineer and is at the core of all engineering practice. This demand for better design processes in pavement technology arises from limited pavement funds and materials, public need for better performance, and less traffic delay due to maintenance (Basma et al., 1989).

From this need for better design methods, Mechanistic-Empirical (M-E) design evolved. M-E design evolved in the 1970's on the basis of known relationships between material behavior under stressed conditions and corresponding empirical performance of pavement structure for all important combinations of loading and environmental conditions. The M-E approach uses the principles of engineering mechanics to calculate pavement responses and relate them to rates of deterioration as shown in Figure 2.2.

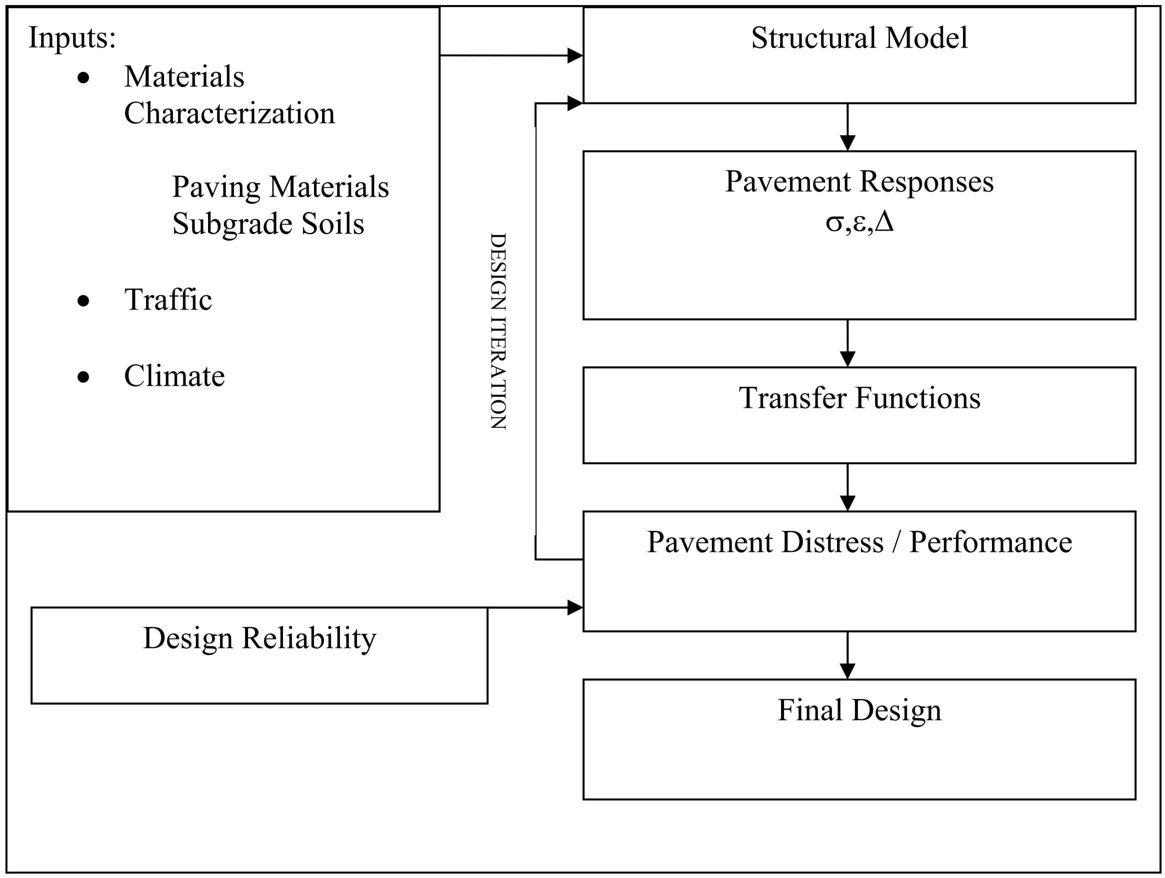

Figure 2.2 Mechanistic-Empirical Design Approach.

Miner's hypothesis is typically employed in the M-E approach to sum the damage caused by traffic and environmental loading. Miner's hypothesis is defined as the sum of applied loads on the pavement structure over the total allowable loads on the pavement structure:

$$D = \sum_{i=1}^{k} \frac{n_i}{N_i} \tag{2.1}$$

where:

D = damage factor

If D>1 then the pavement distress level has exceeded the limit

If D<1 then the pavement distress level has not exceeded the limit

n_i = expected number of load repetitions on the pavement

N_i = allowable number of load repetitions calculated by the transfer function

i = loading or seasonal condition

M-E design is more robust than simple empirical design since it combines the elements of mechanical modeling with empirical performance observations in determining required pavement thickness for a set of design conditions. The mechanical load-displacement model is based upon physics, not empirical relationships, and determines pavement stresses and strains due to wheel loads. The empirical part of the design uses calculated stresses and strains to predict the life of the pavement based upon site-specific field performance. In short, M-E design has the capability of changing and adapting to new developments in pavement design by relying primarily on the mechanics of materials (McAuliffe et al., 1994).

EXISTING RIGID PAVEMENT DESIGN METHODOLOGIES

In rigid pavement design, the most widely used procedures for design is the Guide for Design of Pavement Structures published in 1993 by the American Association of State Highway and Transportation Officials (AASHTO). A few states use the 1972 AASHTO Interim Guide procedure, the Portland Cement Association (PCA) procedure, their own empirical or mechanistic-empirical procedure, or a design catalog (Hall, 2000). Regardless of the method, the key input parameters in any concrete pavement design procedure are traffic over a design period, subgrade, environment, concrete material properties, base properties, performance criteria, and design reliability. These key input parameters are outlined in the following subsections.

Key Parameters in Rigid Pavement Design

Traffic

The number of heavy truck axle loads anticipated over the design life must be estimated from current truck traffic weights and volumes and growth projections. In the AASHTO methodology, the anticipated spectrum of truck loads over the design period is expressed in terms of an equivalent number of 18-kip single axle loads, computed using load equivalency factors that relate the damage done by a given axle type and weight to the damage done by this standard axle. The 18-kip equivalency for single axle loads, also known as an ESAL, is a widely accepted standard axle in the U.S. and around the world. A standard legal axle load limit has generally been imposed for highway travel, hence maximum gear loads in highway design have not appreciably increased with time. For this reason, the effects of other vehicles have normally been accounted for in the design phase by the use of 18-kip single axle loads (Yoder et al., 1975).

It is important to note that the 2002 Design Guide will no longer utilize ESALs and rely instead upon load spectra. Load spectra are simply the weight distributions of various axle configurations. In fact, load spectra are used in the 1993 AASHTO method to computer ESALs.

Subgrade Support

The modulus of subgrade reaction of the foundation can be measured by plate bearing tests. Load is applied at a predetermined rate until a pressure of 10 psi is reached. The pressure is held constant until the deflection increases not more than 0.001 in. per minute for three consecutive minutes. The average of the three dial readings is averaged to determine the deflection. A more detailed description of the test is provided in (Huang, 1993). The modulus of subgrade reaction (k) is important in rigid pavement design because it defines both the subgrade and subbase support. The modulus of the subgrade reaction is given by the following equation.

$$k = \frac{p}{w} \tag{2.2}$$

where,

p = pressure on the plate (psi)

w = deflection of the plate (in)

Since the plate loading test is time-consuming and expensive, the k-value is usually estimated from correlations to simpler tests such as California Bearing Ratio (CBR) and R-value tests. Figure 2.3 shows the approximate relationship between the k-value and other soil properties.

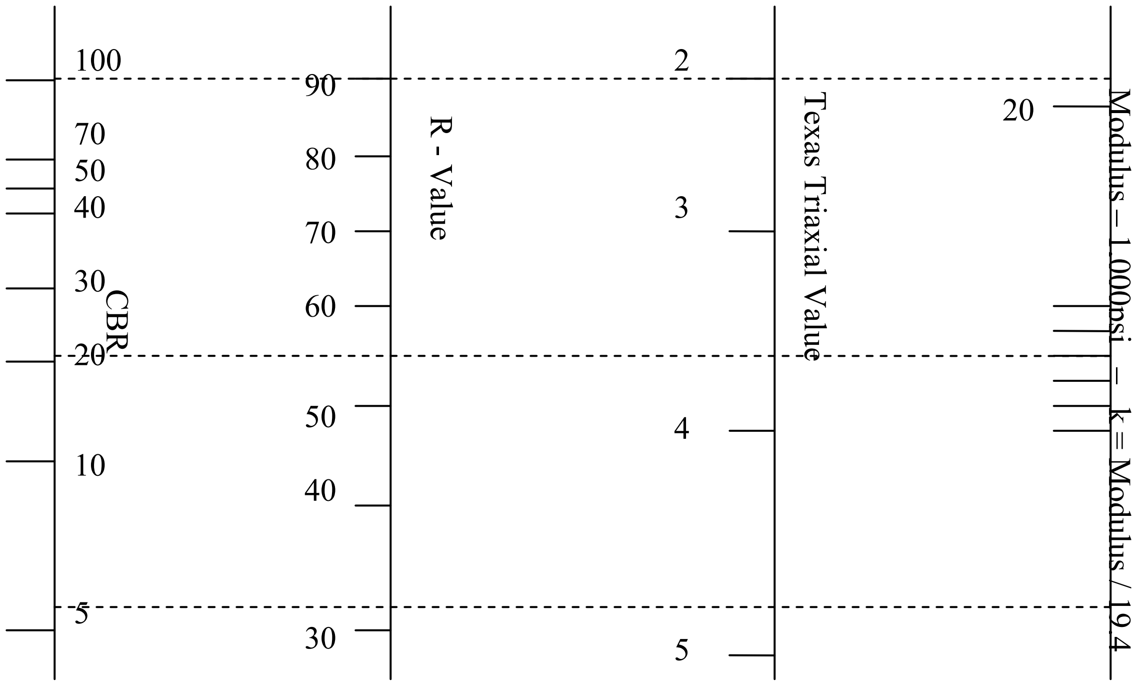

Figure 2.3 k-value Relationships with Other Soil Support Values (after 1993 AASHTO Design Guide).

The subgrade and subbase support strengths varies over the course of a year. The k-values are low during spring thaws and high during the longer freezing periods. However, as evident at the AASHO Road Test, k-values do not have a great effect on required thickness of concrete pavements. Other methods, such as the PCA method, avoid the tedious method of considering seasonal variations in k-values by using normal summer or fall k-values for design purposes (PCA, 1984).

Climate

The climatic variables, particularly temperature variation, can greatly influence concrete behavior The following list, taken from a Fall 2001 NCHRP 1-37A progress update on climatic data research for the 2002 Design Guide displays how daily and seasonal variations in temperature and moisture influence the behavior of concrete pavements in many ways, including:

1. Opening and closing of transverse joints in response to daily and seasonal variation in slab temperature, resulting in fluctuations in joint load transfer capability, which is the ability of each slab group to transfer wheel loads from one slab to the next.
2. Upward and downward curling of the slab caused by daily cycling of the temperature gradient through the slab thickness as pictured in Figure 2.4.

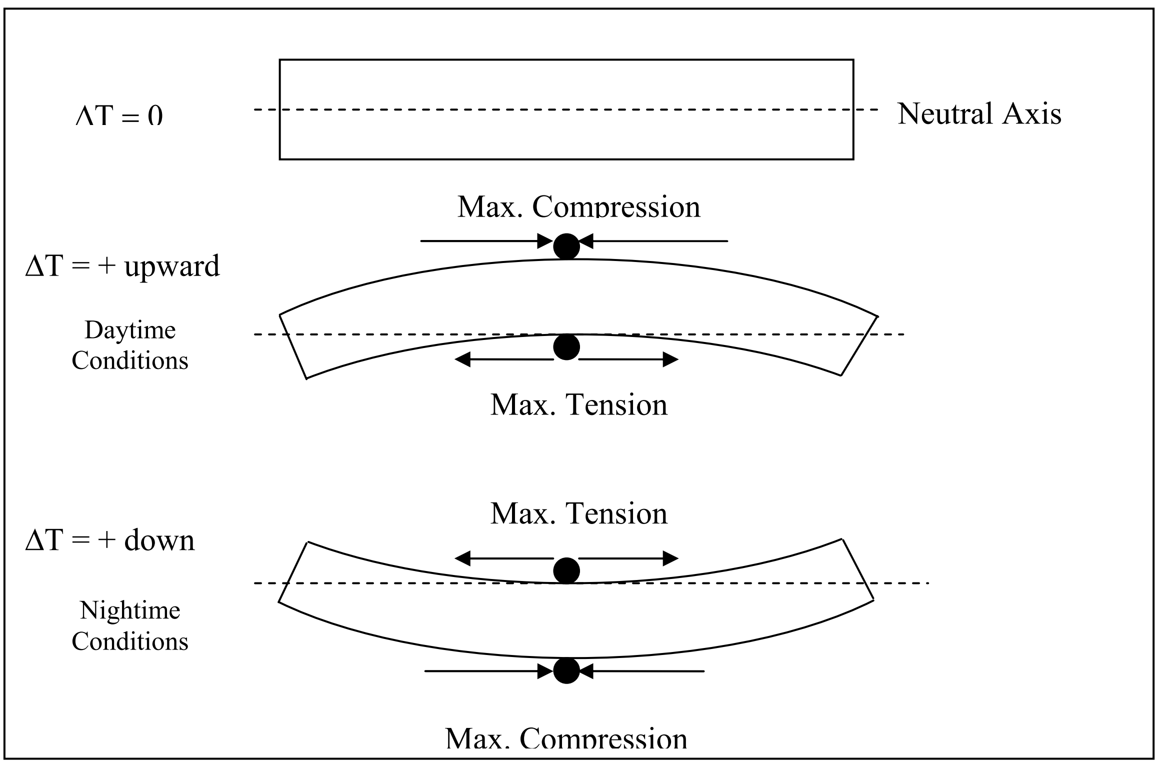

Figure 2.4 Maximum Stresses due to Curling and Warping.

3. Permanent upward curling of the slab, which in some circumstances may occur during construction, as a result of the dissipation of a large temperature gradient that existed in the concrete while it cured.
4. Upward warping of the slab caused by seasonal variation in the moisture gradient through the slab thickness.

5. Erosion of base and foundation materials caused by inadequate drainage of excess water in the pavement structure, primarily from precipitation.
6. Freeze-thaw weakening of the subgrade soil.
7. Freeze-thaw damage to certain types of coarse aggregates in the concrete mix.
8. Corrosion of dowel bars, steel reinforcement, or both, especially in coastal environments and in areas where deicing salts are used in winter.

Although the effects of the climate on concrete pavement behavior and performance have been recognized since the time of the earliest concrete pavement design experiments, concrete pavement thickness design practice traditionally has not explicitly considered most of these climatic effects. Several recent field and analytical studies have contributed greatly to better understanding and quantifying these effects so that they may be more adequately considered in thickness design (Hall, 2000). Climatic effects will be more adequately considered in thickness design in the 2002 Design Guide through a program that compiles much of the recent field and analytical studies into weather stations that allow the designer to triangulate the design location or to input its exact latitude and longitude from weather stations in major cities. The program is called the Enhanced Integrated Climate Model (EICM).

Concrete Properties

For the purpose of pavement thickness design, concrete is characterized by its flexural strength (S_c') as well as its modulus of elasticity (E). Concrete flexural strength is usually characterized by the 28-day modulus of rupture from third point loading tests of beams or it may be estimated from compressive strength as shown in Equation 2.3.

The flexural strength of concrete is a measure of the quality and durability of the concrete. A higher flexural strength of the concrete will most likely result in a lower concrete slab thickness. The modulus of elasticity can also be predicted from compressive strength as indicated in Equation 2.4 for normal weight concrete.

$$S_c = 8\sqrt{f'_c} \quad to \quad 10\sqrt{f'_c} \qquad \text{(Huang, 1993)} \qquad (2.3)$$

$$E_c = 57{,}000\sqrt{f'_c} \qquad \text{(ACI Code 318)} \qquad (2.4)$$

where:

S_c = concrete modulus of rupture, psi

E_c = concrete elastic modulus, psi

f_c' = concrete compressive strength, psi

Performance Indices

All pavement thickness design procedures incorporate performance criteria that define the end of the performance life of the pavement. In the current AASHTO methodology, the performance criterion is the loss of serviceability, which occurs as a result of accumulated damage caused by traffic load applications. The Portland Cement Association (PCA) procedure uses both fatigue cracking and erosion criteria.

Reliability

The reliability level for which a pavement is designed reflects the degree of risk of premature failure that the agency is willing to accept. Facilities of higher functional classes and higher traffic volumes warrant higher safety factors in design. In the AASHTO methodology, this margin of safety is provided by applying a reliability adjustment to the traffic ESAL input. The magnitude of the adjustment is a function of the overall standard deviation associated with the AASHTO model, which reflects error associated with the estimation of traffic and strength inputs and error associated with the quality of fit of the model to the data on which it is based (AASHTO, 1993). When reliability adjustments are made to the traffic input in this manner, AASHTO recommends that average values should be used for the material inputs.

Before the introduction of reliability concepts in pavement thickness procedures, the traditional approach to introducing a margin of safety into concrete pavement thickness design was to apply a safety factor to the concrete modulus of rupture. This approach is still used in the PCA procedure. The PCA procedure also accounts for uncertainty by the reduction of the concrete modulus of rupture by one coefficient of variation and increasing the traffic weights by a percentage depending upon the type of roadway.

CURRENT AASHTO METHODOLOGY

The AASHO Road Test data and the research on the rigid pavement inputs were compiled to develop a general regression equation for rigid pavement design for AASHTO. The empirical model for the performance of the jointed plain concrete pavement (JPCP) and jointed reinforced concrete pavement (JRCP) sections in the main loops of the AASHO Road Test predicts the log of number of axle load applications as a function of the slab thickness, axle type and weight, and terminal serviceability. The empirical model used in the 1993 AASHTO Design Guide is shown below:.

$$\log W_{18} = Z_R S_o + 7.35\log(D+1) - 0.06 + \left(\frac{\log\left[\dfrac{\Delta PSI}{4.5-1.5}\right]}{1+\dfrac{1.624\times 10^7}{(D+1)^{8.46}}} \right) + (4.22 - 0.32 p_t)\log\left\{ \frac{S_c C_d \left(D^{0.75} - 1.132\right)}{215.63\left[\dfrac{D^{0.75} - 18.42}{\left(\dfrac{E_c}{k}\right)^{0.25}}\right]} \right\} \tag{2.5}$$

where:

Z_R = z-statistic corresponding to level of design reliability

S_o = overall standard deviation of design (0.35 often assumed for rigid pavement design)

W_{18}= Number of ESALs before the terminal level of serviceability is reached

D = slab thickness (in)

ΔPSI = Change in serviceability index = $p_o - p_t$

p_t= terminal serviceability

S_c = Concrete modulus of rupture (psi)

C_d = Drainage coefficient

E_c = Concrete modulus of elasticity (psi)

k = modulus of subgrade reaction (pci)

This original model applies only to the designs, traffic conditions, climate, subgrade, and materials of the AASHO Road Test. It has been modified and extended to make possible the estimation of allowable axle load applications to a given terminal serviceability level for conditions of concrete strength, modulus of subgrade reaction, and concrete elastic modulus different than those of the AASHO Road Test. The AASHTO design methodology has also been extended to accommodate the conversion of mixed axle loads to ESALs through the use of load equivalency factors (AASHTO, 1993).

A limitation of the extended AASHO model is that the loss of serviceability that corresponds to a predicted number of axle load applications does not include any contribution of faulting to pavement roughness because, although the doweled pavements in the AASHO Road Test experienced substantial loss of support, they did not fault. The design loss of serviceability is presumed to be entirely due to slab cracking (Hall, 2000).

Furthermore, it is an extrapolation of the AASHTO model to apply it to the prediction of performance of undoweled jointed pavements, jointed pavements with stabilized bases, jointed pavements with spacings other than those used in the AASHO Road Test, continuously reinforced concrete pavement (CRCP), or concrete pavements of any type in climates that may produce significantly greater curling and warping stresses than those experienced by the AASHO Road Test sections. These limitations provide further justification for moving toward an M-E based methodology.

The 1986-1993 AASHTO Guide incorporates many modifications to the procedures for concrete pavements, although the basic design models for both remained the same as in previous versions. The principal modifications to the AASHTO concrete pavement design methodology in the 1986-1993 procedure are the following (Hall, 2000):

1. Addition of drainage adjustment factor, C_d; a multiplier of the slab thickness that presumably is less than 1.0 for drainage conditions worse than those in the AASHO Road Test and greater than 1.0 for better drainage conditions. The quality of drainage

is measured by the length of time for water to be removed from the bases and subbases and depends primarily on their permeability. The percentage of time during which the pavement structure is exposed to moisture levels approaching saturation depends on the average yearly rainfall and the prevailing drainage conditions.

2. Incorporation of the design subgrade reaction modulus, or k-value, as a function of the subgrade resilient modulus, depth to rigid layer, base thickness and elastic modulus, erodability of the base material, and seasonal variation in soil support.
3. Incorporation of a load transfer adjustment (J-factor) as a function of pavement type, load transfer, and shoulder type
4. Addition of a reliability adjustment applied to the design ESAL input instead of using a factor of safety on the modulus of rupture.

The revised AASHTO design model for concrete pavements presented in the 1998 Supplement to the AASHTO Guide was developed under NCHRP Project 1-30 (Hall, 2000). The purpose of the NCHRP Project 1-30 study was to evaluate and improve the AASHTO Guide's characterization of subgrade and base support. The original AASHO empirical model was calibrated to the springtime k-value measured in plate load tests on the granular base, whereas the 1986 Guide's method for determining the design k-value was based on seasonally adjusted annual average k-value for the composite k-value. A key recommendation of the 1-30 study was that, for purposes of concrete pavement design in the existing AASHTO methodology, both the AASHO Road Test subgrade and the subgrade of the project under design should by characterized by the seasonally adjusted annual average static elastic k-value. The 1998 AASHTO Supplement presents guidelines for determination of an appropriate design k-value on the basis of the plate bearing tests, correlations with soil types and properties, CBR, or deflections measured on in-service pavements.

Using the same process by which the original AASHO Road Test empirical model was extended in 1961, a new AASHTO design model was derived to be consistent with the recommended characterization of the design k-value and consider the effects on the stress in the slab of base modulus, base thickness, slab and base friction, joint spacing, edge support, temperature and moisture gradients, and traffic loading. The stress analyses were conducted using a three-dimensional finite element model, which was validated by comparison with stresses in the AASHO Road Test, and measured slab deflections. Regression equations were then developed to relate the computed stresses to the design factors. The three-dimensional finite element model was also used to develop a design check for corner loading for undoweled jointed pavements.

As in the earlier versions of the AASHTO rigid pavement design procedure, the computed slab thickness is that which is required to support the anticipated ESALs to a selected terminal serviceability level, assuming that the serviceability level loss is due only to slab cracking. If faulting were to develop on a pavement to such a degree that it contributed significantly to loss of serviceability, the pavement would have been under designed; that is, it would have reached terminal serviceability sooner than predicted. The appropriate way to prevent this is not to increase the slab thickness, but rather to design the joint load transfer system so that faulting will not develop to the degree that it contributes significantly to loss of serviceability.

One major output of the AASHO Road Test was the load equivalency factor (LEF). LEFs were used to quantify the damage different axle loads and configurations caused to the pavement relative to an 18-kip single axle load. One shortcoming of LEFs is that they are based on the AASHO Road Test concrete pavements, which failed due to slab cracking only. There are several types of failure modes prevalent in rigid pavement structures (Hall, 2000) not represented by the AASHO LEF equations. Most rigid pavement structures fail due to faulting and fatigue cracking.

Some further limitations of the AASHTO Design Guide are that the effects of widened lanes and tied concrete shoulders cannot be analyzed in detail. The AASHTO Design Guide also does not directly consider joint spacing and curling stresses in rigid pavements (Roesler et al., 2000). The 1993 AASHTO Design Guide also is limited as to incorporating climate, predicting subgrade, base, and pavement layer strengths, and also accounting for different types of pavement loading due to traffic, particularly heavy vehicle types and different axle configurations. These shortcomings are due primarily to the limited pavement design inputs available at the AASHO Road Test.

PCA METHODOLOGY

The PCA rigid pavement design procedure evaluates a candidate pavement design with respect to two potential failure modes: fatigue and erosion. This M-E procedure was developed using the results of a finite element analyses of stresses induced in concrete pavements by joint, edge, and corner loading. The analyses took into consideration the degree of load transfer provided by dowels or aggregate interlock and the degree of edge support provided by a concrete shoulder (PCA, 1984). The PCA procedure, like the 1986-1993 AASHTO procedure, employs the composite k concept in which the design k is a function of the subgrade soil k, base thickness, and base type.

The fatigue analysis incorporates the assumption that approximately 6% of all truck loads will pass sufficiently close to the slab edge to produce a significant tensile stress. The erosion analysis quantifies the power with which a slab corner is deflected by a wheel load as a function of the slab thickness, foundation k-value, and estimated pressure at the slab-foundation interface (Hall, 2000).

For each load level considered, the expected number of load repetitions over the design life is expressed as a percentage of allowable load repetitions of that load level with respect to both fatigue and erosion. An adequate thickness is one for which the sum of the contributions of all axle load levels to fatigue and erosion levels is less than 100 percent. This evaluation method is consistent with Miner's Hypothesis described previously.

Other concrete pavement design methods range from empirical adaptations of the AASHTO method to calibration and mechanistic-empirical extension of the AASHTO method and methods that combine mechanistic stress calculation with an empirical fatigue cracking model.

The latest versions of the PCA thickness design for concrete highway and street pavements have more mechanistic features than the empirically based AASHTO guide. The PCA uses the load spectra analysis to calculate the bending stress in the concrete due to various axle loads and configurations. Load spectra analysis is more theoretically sound than ESAL analysis because fundamental stresses and strains are calculated and related to the performance of laboratory concrete fatigue beam tests. Load spectra analysis also allows for calculation of pavement stresses due to axle loads and configurations not originally considered in the AASHO Road Test. The limitations of the

PCA guide include no ability to analyze widened lanes or different joint spacings and no consideration of load transfer across the shoulder-lane joint (Packard, 1984). This is significant because widened lanes are an often used type of edge support, and also because differences in length between joints in concrete slabs are often encountered in a rigid pavement design.

OVERVIEW OF MECHANISTIC-EMPIRICAL PAVEMENT DESIGN

The development of mechanistic-empirical (M-E) design procedures was needed to account for situations where existing empirical studies could not be extrapolated to find a reasonable thickness design solution. Mechanistic-based design addresses the theoretical stresses, strains, and deflections in the pavement structure due to the environment, pavement materials, and traffic. These stresses, strains, and deflections are then related to the field performance of in-service rigid pavements through transfer functions (Roesler et al., 2000).

In an M-E design procedure, new, old, and current pavement features may be analyzed to determine their effect on pavement performance. The pavement engineer can make changes to these design features to accommodate the specific location and constraints of the proposed pavement structure (Roesler et al, 2000). For example, the behavior of a pavement in a high desert environment should not be expected to be the same as a pavement in a coastal environment, and some pavement design features may need to be adjusted to account for the different environments.

In contrast, with an empirical design guide such as the AASHTO Design Guide, changes can be made only to the pavement features that are included in the original road test. Extrapolation of designs not included in the original road test could result in unrealistic designs. In M-E procedures, analysis can be used to describe the failure of field tests in terms of stresses, strains, and deflections. Future designs can be outside the scope of any field testing because the mechanisms of pavement failure are quantified with theoretical analysis (Barenberg, 1992).

A mechanistic-based model is verified through calibration with field test results. A purely mechanistic model would not have to be calibrated with field data, but an M-E design approach still needs calibration to account for unknown slab behaviors. These unknown behaviors are also addressed in applying reliability to the design. Applying design reliability gives a factor of safety against premature failure.

For rigid pavement M-E design, typically two models are used to estimate pavement life; a model for predicting cracking in the slabs and a model for predicting faulting in the joints between each slab. For the fatigue cracking model, typical M-E procedures use critical stresses in the slab to estimate total wheel load applications before cracks begin, propagate, and ultimately fracture the pavement. For the faulting model, the example below is of a mechanistic-empirical equation where performance evaluation relates faulting rates to the maximum concrete bearing stress (COPES, 1993):

$$F = (N_{18})^{0.5377}\left[2.2073 + 0.002171(S)^{0.4918} + 0.003292(JS)^{1.0793} - 2.1397(k)^{0.01305}\right] \tag{2.6}$$

where:

F = pavement faulting (in.)

N_{18} = number of ESALs in millions

S = maximum bearing stress (psi)

JS = transverse joint spacing (ft)

k = estimated modulus of subgrade reaction on the top of the subbase (pci)

For the slab cracking model, an example of a mechanistic-empirical equation is shown in the following equation where performance evaluation relates cracking rates to the concrete modulus of rupture and the tensile stress (Kessler et al., 1953).

$$\text{Log } N_f = 17.61 - 17.61(\sigma/S_c) \tag{2.7}$$

where:

N_f = Number of ESALs to failure

σ= Tensile Stress (psi)

S_c = Concrete Modulus of Rupture (pci)

Fatigue life is the number of stress repetitions, at a magnitude less than its strength, required for structural failure of pavement. Faulting and cracking are both considered modes of structural failure (McAuliffe et al., 1994).

M-E PAVEMENT DESIGN GUIDE OVERVIEW

NCHRP 1-37A was started in 2000 in an attempt to create a user-friendly, mechanistic-empirical design program that would encompass all types of pavement design. The overall objective of this project was to develop and deliver the 2002 Guide for Design of New and Rehabilitated Pavement Structures, based on mechanistic-empirical principles, accompanied by the necessary computational software, for adoption and distribution by AASHTO. As it is now called, the Mechanistic Empirical Pavement Design Guide (MEPDG) was to use existing structural models and be a fully M-E design approach. No new models or databases were created for this design guide. The MEPDG was to provide a uniform basis for the design of flexible, rigid, and composite pavements and employ common design parameters for traffic subgrade, environment, and reliability.

The focus of this research is the MEPDG module for rigid pavement design. The rigid pavement design portion includes jointed concrete pavement, jointed reinforced concrete pavement, and continuously reinforced concrete pavement design. The MEPDG procedure verifies trial design against user input performance criteria. The distress types considered in the MEPDG for rigid pavements are joint faulting and transverse cracking in jointed plain concrete pavements and punchouts in continuously reinforced concrete pavements. The MEPDG takes directly into account the effects of temperature/environmental conditions for the rigid pavement section trail designs by use of the Enhanced Integrated Climate Model (EICM). The EICM algorithms are linked to the design guide software as an independent module through interfaces and design inputs.

This research focuses on the 2002 Design Guide for Rigid Pavements and how it compares with the 1993 Design Guide for Rigid Pavements. Since the 1993 AASHTO Design Guide is the most widely used empirical pavement guide, it was compared with the new mechanistic-empirical 2002 Design Guide for Rigid Pavements. The main difference in the 1993 Design Guide and the 2002 Design Guide is in its prediction of pavement thickness and distress. The 1993 Design Guide predicts pavement thickness based a pavements terminal serviceability index, anticipated traffic, and strength parameters of the pavement. The 2002 Design Guide takes into account strength parameters, anticipated traffic, and user input thicknesses of slab, base, and subgrade and predicts distress. If the predicted distresses are less than the user input thresholds for all distress criteria, then the pavement can be considered successful.

In the next chapter, Methodology, the process of comparing the existing empirical 1993 AASHTO Design equation for rigid pavements to the 2002 Design Guide for rigid pavements is presented.

CHAPTER 3 - METHODOLOGY

INTRODUCTION

The task of comparing two different approaches for pavement design through design simulations and statistical analysis was divided into several phases. The first phase consisted of comparing the change in present serviceability (ΔPSI) from the 1993 Design Guide to a derived ΔPSI from the pavement distresses taken from the 2002 Design Guide. The second phase consisted of a sensitivity analysis that analyzed new inputs encountered in the 2002 Design Guide with respect to their bearing on the pavement distresses in the 2002 Design Guide. The last phase was a comparison of slab design thicknesses determined from the 1993 Design Guide and the 2002 Design Guide.

PHASE 1 – ΔPSI REGRESSION ANALYSIS

The 2002 Design Guide and 1993 Design Guide offer two very distinct methods of pavement design. In the 1993 Design Guide, the input data are entered into a regression-based design equation which directly outputs the required slab thickness. The primary performance criterion in the 1993 AASHTO Design Guide is the change in serviceability over the design life (ΔPSI).

In the 2002 Design Guide, the pavement cross section, along with the proper traffic, structural, and material properties are input and predicted distresses are output, and the user determines the efficiency of the design based on the user input distress criteria. The rigid pavement distress criteria used in the 2002 Design Guide is IRI, % Slabs Cracked and Mean Joint Faulting. Mean joint faulting is defined as the average slab elevation differential at each transverse joint over the entire rigid pavement system.

Percent slabs cracked is the predicted percentage of slabs in the rigid pavement system with at least one crack, and IRI is a measure of overall roughness of the pavement system. Examples of each of these 2002 Design Guide outputs are shown in the following figures. These figures were created by the 2002 Design Guide as output files and are representative of predicted distresses over the design life of the pavement.

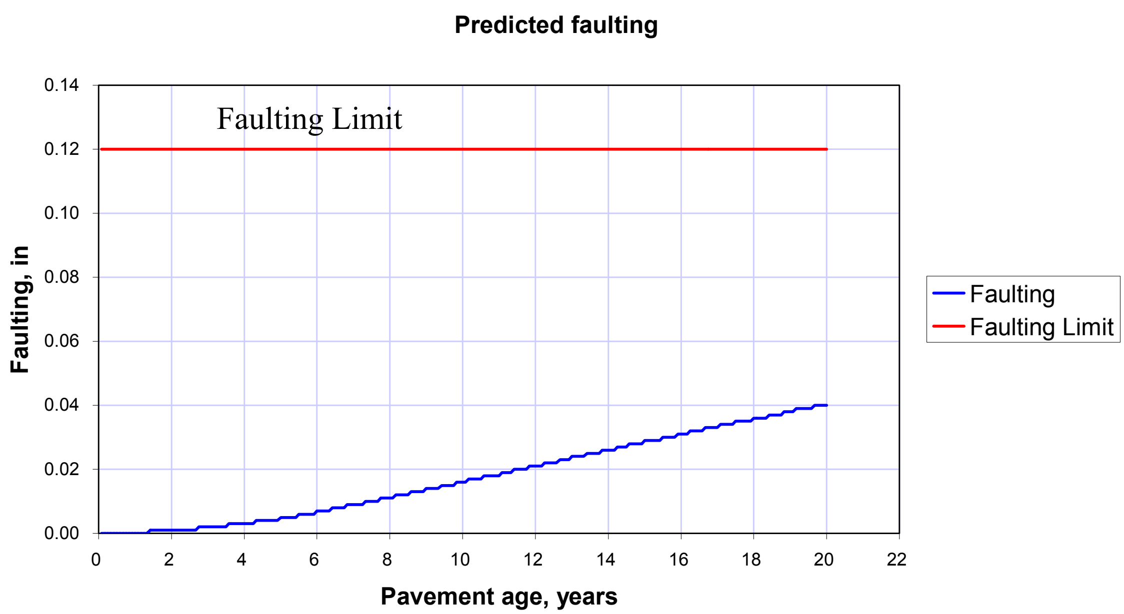

Figure 3.1 Faulting Output From 2002 Design Guide.

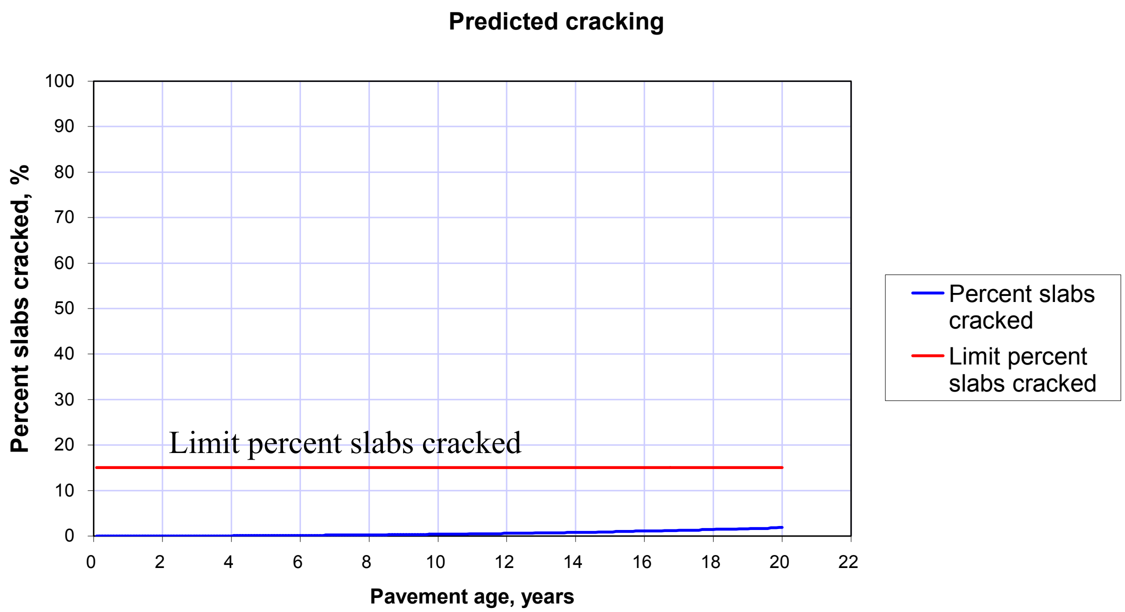

Figure 3.2 Slabs cracked Output From 2002 Design Guide.

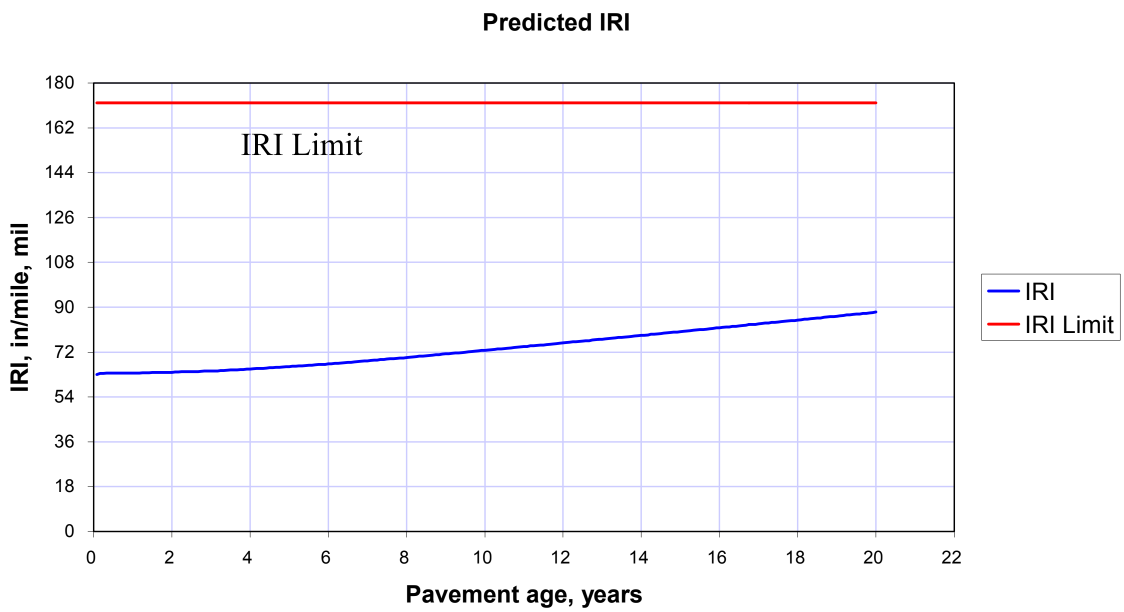

Figure 3.3 IRI Output From 2002 Design Guide.

These output distresses from the 2002 Design Guide were correlated to a predicted ΔPSI through regression analysis. Figure 3.4 shows the method by which the 2002 Design Guide output distresses were correlated to ΔPSI.

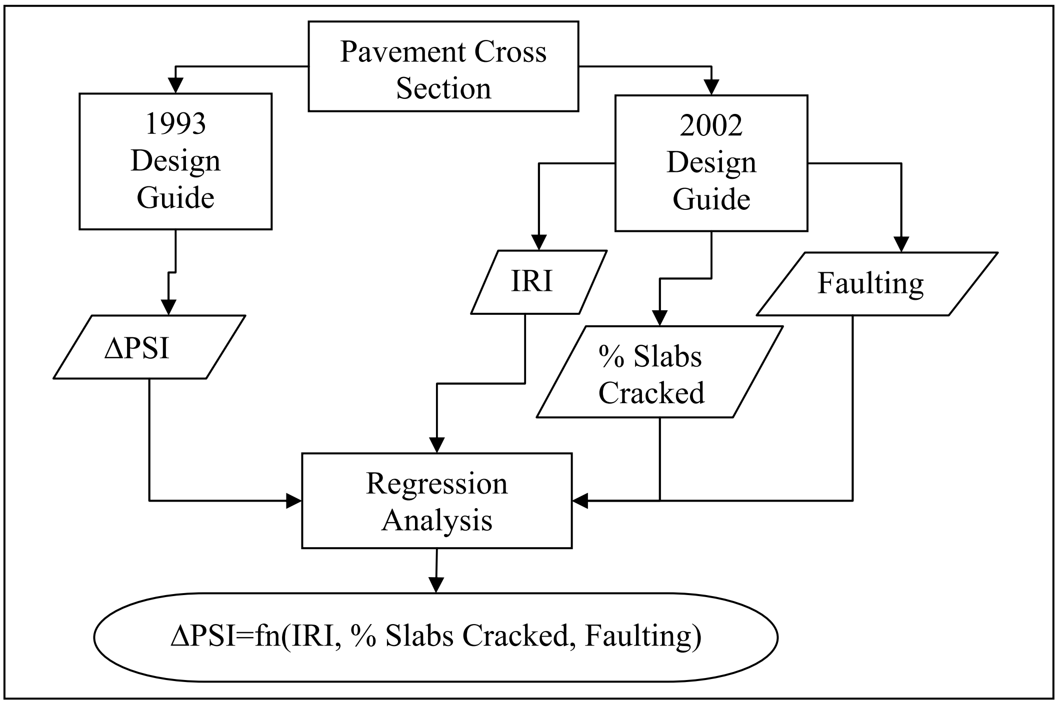

Figure 3.4 1993 ΔPSI vs. 2002 Distress Criteria Analysis Schematic.

From Figure 3.4, it can be seen that this analysis started with a rigid pavement section. This pavement section was input into both the 2002 and 1993 Design Guides, respectively. Each design approach was then executed and output performance predictions, as shown in Figure 3.4 were obtained. Once all design simulations were run in both design guides, a regression analysis was performed relating the 1993 output to the 2002 output. The information from the regression analysis was then used to determine a ΔPSI from the 2002 distress output. The relation of the 1993 ΔPSI to the 2002 ΔPSI is the first step in analyzing the 2002 Design Guide as it relates to current empirical design approaches.

There are three main differences between the 1993 Design Guide and the 2002 Design Guide inputs. They are traffic, subgrade support, and load transfer coefficients. As different as these two approaches may seem, a procedure to bridge the gap between the two methodologies had to be derived through manipulation of the required inputs for design in the 1993 and 2002 Design Guides.

Traffic

Traffic input for the 2002 Guide is radically different from the 1993 Guide. Where the 1993 Guide uses ESALs to characterize traffic, the 2002 Design Guide uses axle load spectra to characterize the traffic. The ESAL number used in the 1993 Guide is a representative number of the axle load spectra. The method of obtaining design ESALs from load spectra followed procedures outlined in the 1993 Design Guide. Specifically, the default load spectra in the 2002 Design Guide were converted using the load equivalency factor equations to determine the total design ESAL. Table 3.1 lists the relevant traffic parameters used in this study while Figures 3.5 through 3.7 illustrate the default load spectra for the single, tandem and tridem axles, respectively.

Table 3.1 Traffic Parameters.

Parameter	Value
Average Annual Daily Traffic (AADT)	10,000
Average Annual Daily Truck Traffic (AADTT)	1,500 (15% of AADT)
Lane Distribution Factor	90%
Directional Distribution Factor	50%
Design Period	10 and 20 years
Growth Rate	4%

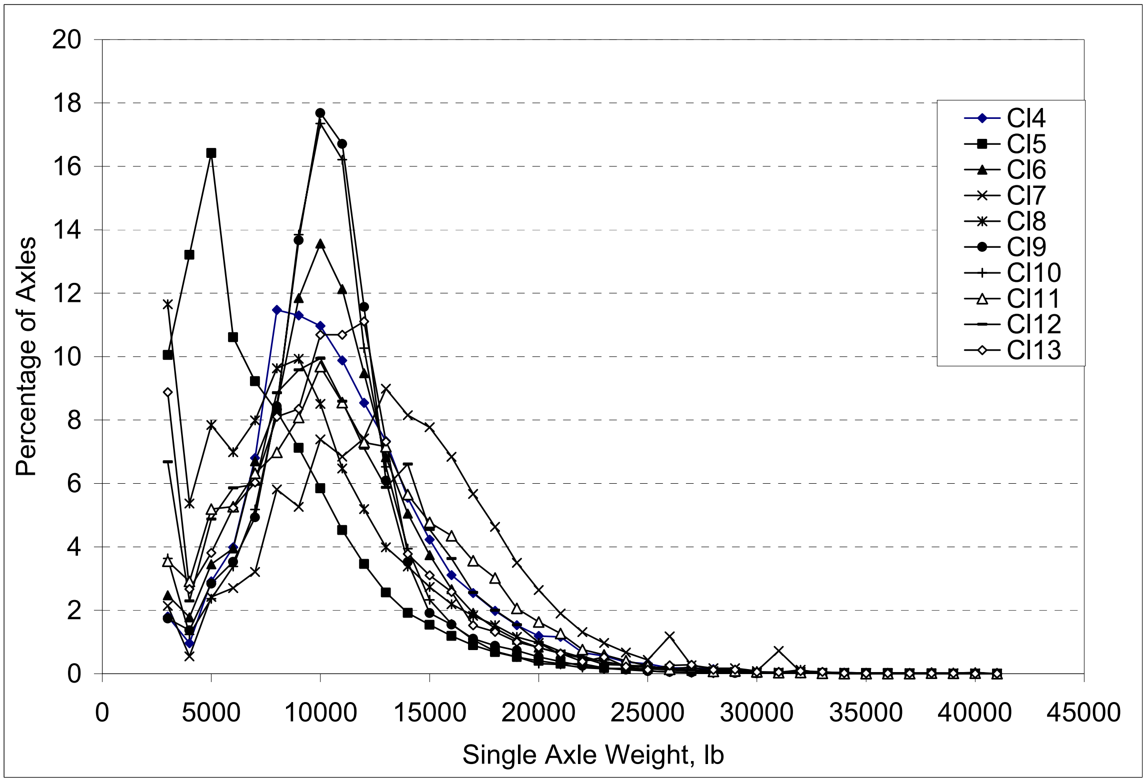

Figure 3.5 Single Axle Weight Distribution – By Vehicle Class.

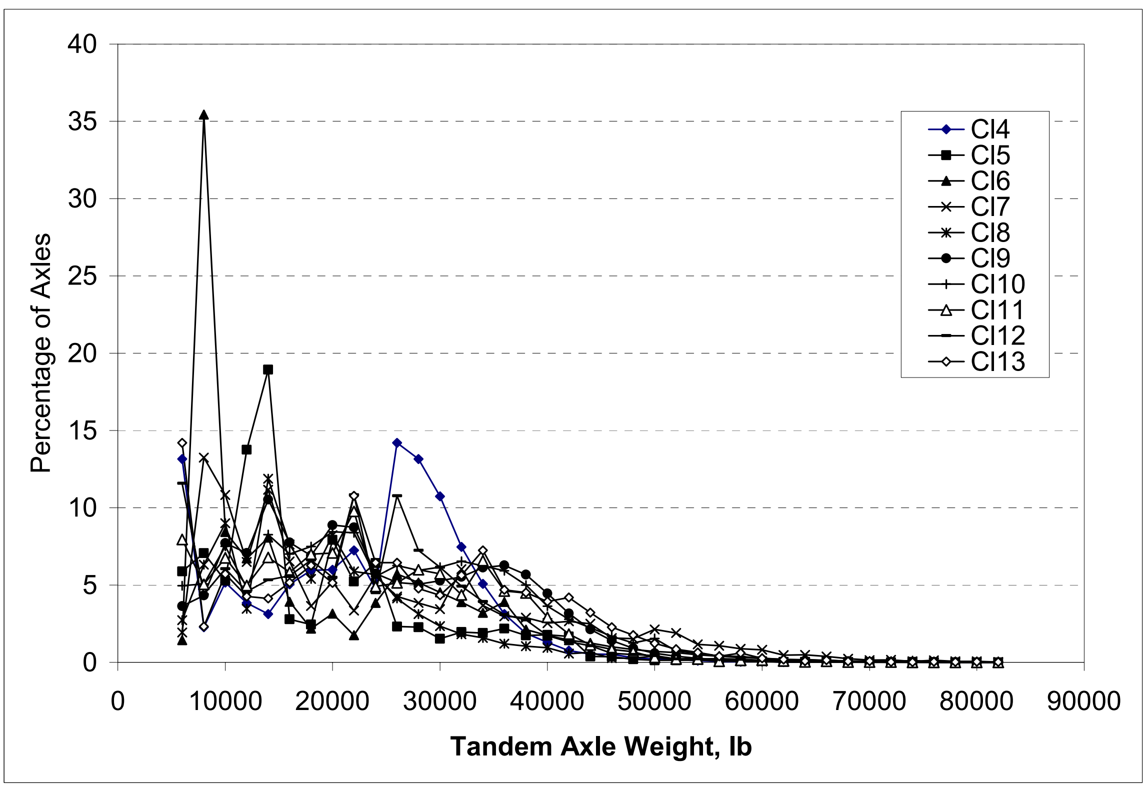

Figure 3.6 Tandem Axle Weight Distribution – By Vehicle Class.

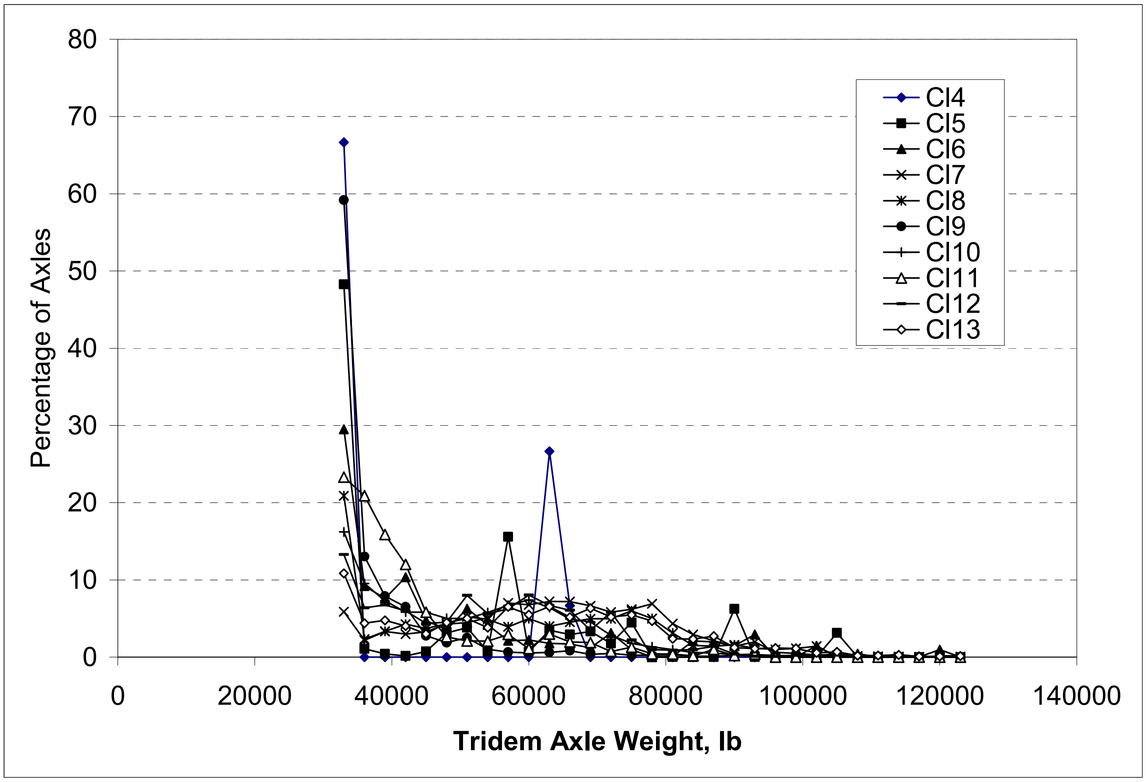

Figure 3.7 Tridem Axle Weight Distribution – By Vehicle Class.

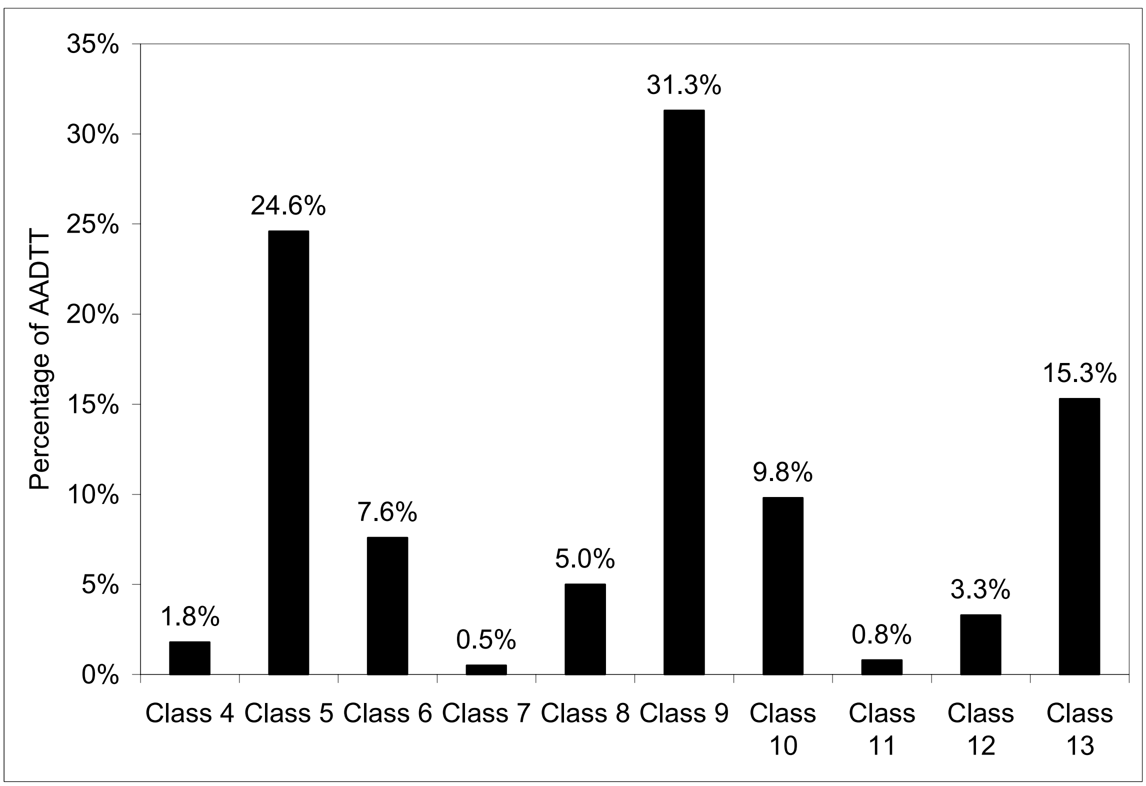

Figure 3.8 Vehicle Type Distribution.

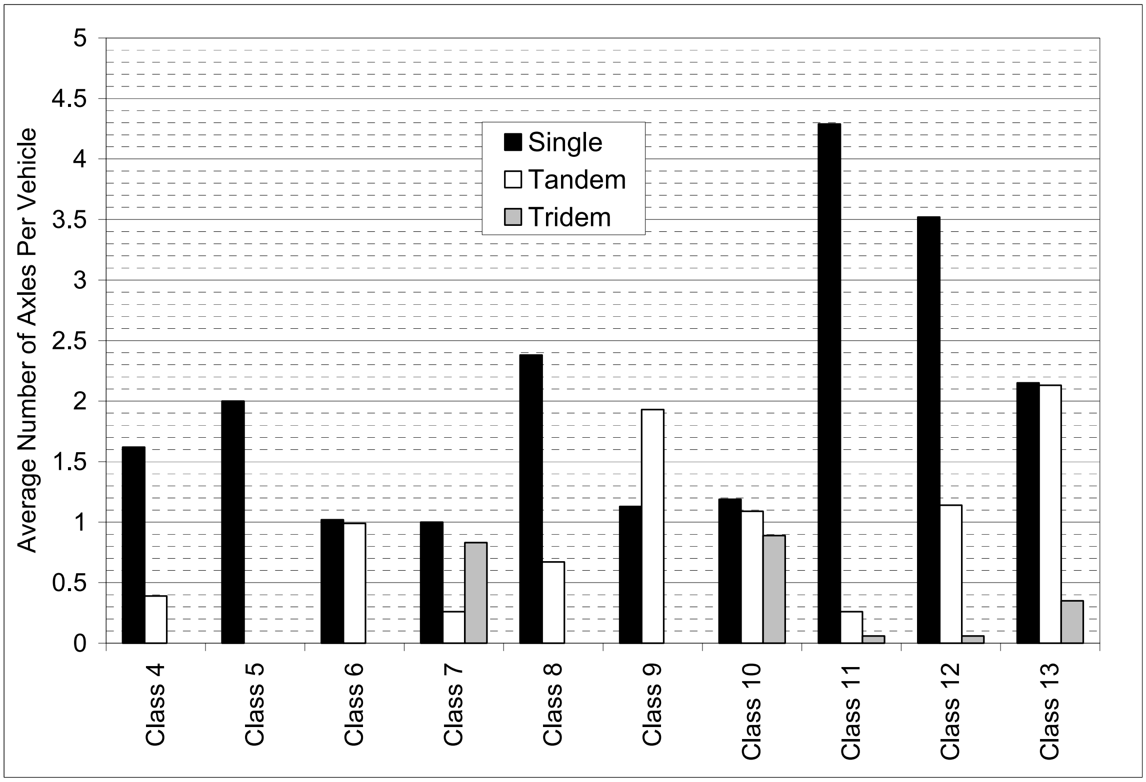

Figure 3.9 Average Number of Axles by Vehicle Class.

Subgrade

The subgrade support is not as complicated of a conversion technique as was the traffic conversion. The 2002 Guide takes into account the subgrade strength by the subgrade resilient modulus, M_R (psi). The 1993 Guide accounts for subgrade strength by the modulus of subgrade reaction, k_{eff} (pci). The following conversion was used (AASHTO, 1993):

$$k_{eff} = M_R / 19.4 \tag{3.1}$$

Load Transfer Between Slabs

The load transfer conversion is necessary because the 1993 Guide uses a J-factor that takes into account the use of dowels for load transfer, edge support, slab length and slab thickness. The 2002 Design Guide uses each of these inputs separately instead of grouping them into one representative number. Table 3.2 is commonly used to convert support conditions into a J-factor for the 1993 Design Guide. This table was used to select the proper load transfer factor (J-factor) for use in the 1993 Design Guide for each simulation. A predetermined slab thickness as well as predetermined subgrade resilient modulus, which had to be converted into a subgrade reaction modulus was factored into each simulation.

Table 3.2 J-FACTORS (After AASHTO, 1993)

Shoulder	Asphalt Shoulder		Tied PCC Shoulder	
Dowels Between Slabs	YES	NO	YES	NO
JCP and JPCP	A. 3.2	A. 4.5	A. 2.8	A. 3.7
	B. 3.2	B. 4.1	B. 2.7	B. 3.6
	C. 3.2	C. 3.9	C. 2.7	C. 3.6
	D. 3.2	D. 5.2	D. 3.0	D. 4.0
	E. 3.2	E. 4.8	E. 2.9	E. 3.8
	F. 3.2	F. 4.5	F. 2.8	F. 3.7
CRCP	A. 2.9	N/A	A. 2.5	N/A
	B. 3.0		B. 2.6	
	C. 3.1		C. 2.6	
	D. 2.6		D. 2.3	
	E. 2.8		E. 2.4	
	F. 2.9		F. 2.5	

Case A: H = 7in., k = 100pci
Case B: H = 10in., k = 100pci
Case C: H = 13 in., k = 100pci
Case D: H = 7in., k = 600pci
Case E: H = 10in., k = 600pci
Case F: H = 13in., k = 600pci

Design Simulations

Table 3.3 lists the inputs used in this investigation. The simulation matrix consisted of seven thicknesses, eleven soil support values, two dowel conditions and two design lives.

Table 3.3 Design Simulation Input Parameters

Parameter	Values
Pavement Depth (in)	8, 8.5, 9, 9.5, 10, 10.5, 11
Subgrade Modulus (psi)	970, 1940, 2910, 3880, 4850, 5820, 6970, 7760, 9700, 10670, 11640
Dowels/No Dowels 1=Dowels, 0=No Dowels	0, 1
Design Life (years)	10, 20

Regression Analysis

Once the simulations had been executed in both the 1993 and 2002 Desing Guides, a regression analysis was performed on the pavement distresses received from both systems. The distress criteria set forth in the 2002 Design Guide were IRI, percent of slabs cracked, and mean joint faulting. These three distresses were related to the change in present serviceability index through a regression analysis by the following equation.

$$\Delta PSI = \beta_0 + \beta_1(IRI) + \beta_2(FAULTING) + \beta_3(\%SLABS) \quad (3.2)$$

where:

ΔPSI = change in serviceability calculated from 1993 Design Guide

$\beta_{0,1,2,3}$ = regression coefficients

IRI = 2002 Design Guide predicted International Roughness Index, in/mile

FAULTING = 2002 Design Guide predicted distress of mean joint faulting, in

%SLABS = 2002 Design Guide predicted distress of percent slabs cracked

PHASE 2 – SENSITIVITY ANALYSIS

Phase 2 of this study was a sensitivity analysis involving only the 2002 Design Guide. Within the 2002 Design Guide there are many new inputs for pavement design not previously used in the design of rigid pavements by the 1993 Design Guide. These new inputs were evaluated to determine their effect on the rigid pavement distresses in the 2002 Design Guide. When using the new software, default values are given for each of these new inputs and so each input was varied around the default value given in the 2002 Design Guide. If the distresses encountered by the pavement structure were found to be unchanged, then it was concluded that the input being varied had no effect on the structural capacity of the pavement structure and thus, default values would suffice for pavement design. Table 3.5 shows the criteria used for the new input values incorporated into this sensitivity analysis.

Table 3.4. Sensitivity Analysis Parameters

Design Input	Input Values		
	Low	Default	High
Operational Speed (mph)	40	60	80
Mean Wheel Location (in. from marking)	12	18	20
Traffic Wander SD(in)	10	14	20
Design Lane Width (ft)	12	14	20
Avg. Axle Width (ft)	6	8.5	10
Dual Tire Spacing (in)	12	20	24
Single Tire Pressure (psi)	80	120	160
Dual Tire Pressure (psi)	80	120	160
Tandem Axle Spacing (in)	40	51.6	60
Tridem Axle Spacing (in)	40	49.2	60
Quad Axle Spacing (in)	40	49.2	60
Short Wheelbase (ft)	12	14	20
Medium Wheelbase (ft)	12	15	18
Long Wheelbase (ft)	15	18	21
Percent Short Wheelbase	20	33	60
Percent Medium Wheelbase	20	33	60
Percent Long Wheelbase	20	33	60
Climate	ANC	ATL	MIA
Joint Sealant Type	None	Liquid	Silicone
Edge Support Selection	None	Wide Slab	Tied PCC
PCC-Base Interface	Unbonded	Unbonded	Bonded
Erodobility Index	Erodible	Resistant	Very Resistant
Loss of Bond Age (months)	40	60	80
Surface Shortwave Absorptivity	0.6	0.85	1
Infiltration	Low	Medium	High
Drainage Path Length (ft)	12	14	20
Pavement Cross Slope	0.1	0.2	0.5
Coeff. Of Thermal Expansion (/Fx10^{-4})	6	9	12
Thermal Conductivity (BTU/hr-ft-F)	1	1.25	1.5
Heat Capacity (BTU/hr-ft-F)	0.1	0.28	0.5
PCC-Zero Stress Temp. (^{0}F)	100	120	140
Ult. Shrinkage (microstrain)	500	670	750
Reversible Shrinkage (%)	25	50	75
Time to Develop 50% Shrinkage (days)	15	35	60
20yr/28day Compressive Strength Ratio	1	1.2	1.5
20yr/28day Elastic Modulus Ratio	1	1.2	1.5
20yr/28day Resilient Modulus Ratio	1	1.2	1.5
20yr/28day Tensile Strength Ratio	1	1.2	1.5

PHASE 3 – DESIGN SLAB THICKNESS COMPARISON

The third and final part of this investigation was a slab thickness comparison comparing the 2002 Design Guide slab thickness and the 1993 AASHTO Design Guide slab thickness. A total of 125 simulations were executed and evaluated through both the 1993 Design Guide and the 2002 Design Guide. Creating a slab thickness comparison required several more steps than Phase 1 of this investigation in which the two design guides were compared based on predicted pavement performance. In this comparison, only IRI and ΔPSI were evaluated. This was justified by the fact that IRI is not a singular pavement distress. Rather, IRI is much like the present serviceability index which is a measure of ride quality. IRI encompasses all distresses incurred by the pavement structure and denotes them as one factor called roughness. IRI can be empirically related to the present serviceability index PSI index by the following equation (Darter et al., 1992):

$$PSI = 5e^{-0.0041IRI} \tag{3.3}$$

where:

PSI = Present Serviceability Index

IRI = International Roughness Index (in/mile)

While Part 1 of this analysis was to show that the two pavement design methodologies (1993 and 2002) could be linked together, Part 3 was to expand this link into a more practical comparison between slab thicknesses between the two methodologies. In other words, would the new design guide result in thicker, thinner, or about the same designed slab thicknesses?

Traffic, soil stiffness, and concrete modulus of rupture were main variables examined in this study and are summarized in 3.5. Traffic was varied by altering the design life of the pavement. Half of the simulations were executed based on a 20-year design life and the other half was run based on a 10 year design life. An AADTT of 1,500 vehicles was selected and applied to the default axle load distribution factors, traffic growth factors, and vehicle class distributions described in Phase 1, above.

Soil stiffness was varied based on the resilient modulus of the subgrade according to Equation 3.1. The PCC modulus of rupture, S_c', was varied from 400 to 900 psi and was used to determine the PCC compressive strength, f_c', as well as the PCC modulus of elasticity, E, to be used in the 1993 Design Guide for this slab thickness comparison. The modulus of rupture was calculated from (Huang, 1993):

$$S_c' = 9\sqrt{f_c'} \tag{3.4}$$

The PCC modulus of elasticity, E, was also determined from the PCC compressive strength according to (Huang, 1993):

$$E = 57{,}000\sqrt{f_c'} \tag{3.5}$$

Finally, the 2002 Design Guide distress criterion of IRI was varied based on terminal serviceability in the 1993 Design Guide. The terminal PSI levels used for this analysis were 4.0, 3.5, 3.0, and 2.5. Each design simulation was run using 50% and 90% reliability, R. Table 3.5 summarizes the input data used for this investigation.

Table 3.5 Test Matrix for Slab Thickness Investigation

Parameter	Values
ΔPSI	0.2, 0.7, 1.2
p_t	3, 3.5, 4
IRI (in/mile)	54, 87, 125
M_R (psi)	1000, 3000, 7000
k_{eff} (pci)	51.55, 154.64, 360.82, 515.46
S'c (pci)	400, 600, 900
E (psi)	3E+06, 4E+06, 6E+06
f'c (psi)	1975.3, 4444.4, 10000
Reliability, %	50, 90
Traffic (ESALs)	5E+06, 10E+06
Design Life (years)	10, 20

Determining optimum slab thickness in the 2002 Design Guide was an iterative process since the 2002 Design Guide's output consisted only of predicted distresses and distress criteria set by the designer. The slab thickness was considered to be at an optimum level when the predicted distresses were as close to the preset distress criteria as possible without exceeding the preset distress criteria. Optimum slab thicknesses were rounded to the nearest half-inch that would not exceed the distress criteria.

SUMMARY

This chapter detailed the three phases of this investigation that included:

- A comparison between predicted pavement performance from the 1993 and 2002 Design Guides.
- A senstivity analysis to investigate the relative importance of the numerous input parameters in the 2002 Design Guide.
- A slab thickness comparison between the 1993 and 2002 Design Guides.

The results of the three phases are presented and discussed in the next chapter.

CHAPTER 4 – RESULTS AND DISCUSSION

INTRODUCTION

The results of the regression analysis relating the 1993 ΔPSI to the 2002 performance parameters, the sensitivity analysis of the new inputs in the 2002 Design Guide, and a slab thickness comparison comparing slab thicknesses between design methods are presented, respectively.

PHASE 1 – ΔPSI REGRESSION ANALYSIS

The data obtained from this analysis showed that there is a definite relationship between the change in serviceability predicted in the 1993 Design Guide and the pavement distresses output by the 2002 Design Guide. There were 154 design simulations executed in the two Guides, respectively. The best fit equation was, also shown in Table 4.1, was:

$$\Delta PSI = 129.78(Faulting) + 0.105(\%\ Slabs\ Cracked) - 0.078(IRI) + 5.16 \qquad (4.1)$$

$R^2 = 0.71$

Figure 4.1 illustrates the regression equation against the actual ΔPSI values obtained from the 1993 Design Guide. These results suggest a strong correlation between ΔPSI and the predicted levels of pavement distress from the 2002 Design Guide. Further, Equation 4.1 can be used to convert results obtained from the 2002 Design Guide into ΔPSI levels in the 1993 Design Guide. Further discussion of Equation 4.1 is provided below.

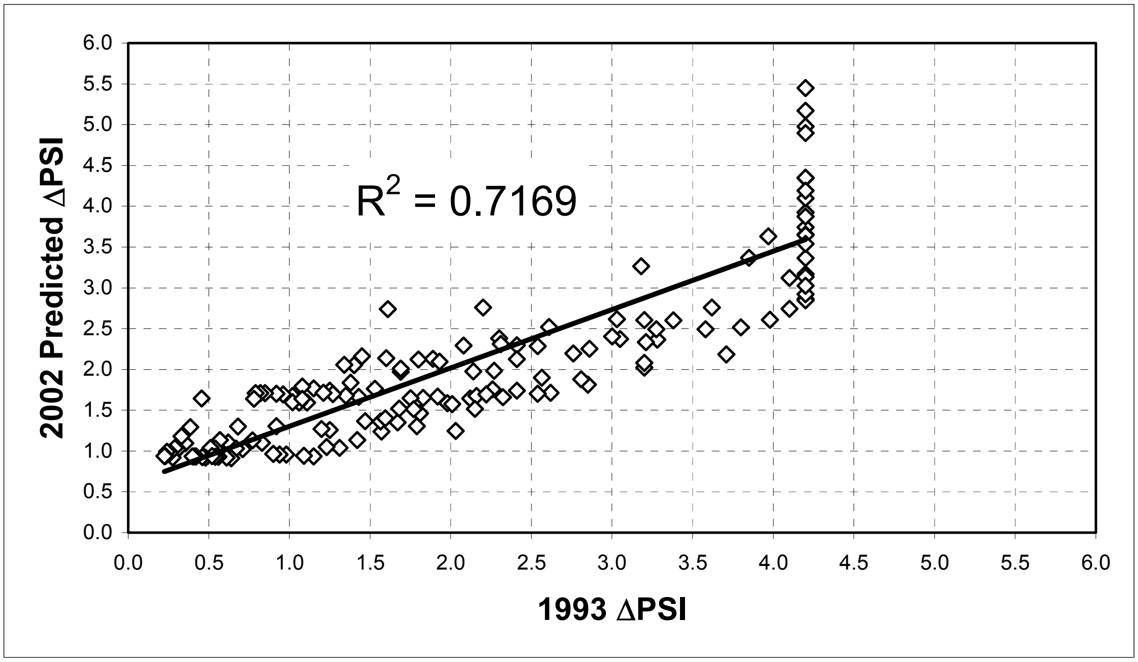

Figure 4.1. 2002 ΔPSI vs. 1993 Predicted ΔPSI.

Table 4.1 ΔPSI Regression Statistics

Regression Statistics	
Multiple R	0.845311507
R Square	0.714551544
Adjusted R Square	0.708842575
Standard Error	0.712281399
Observations	154

	Coefficients	*Standard Error*	*t Stat*	P-value
Intercept	5.161042354	4.27817998	1.20636401	0.229576392
Faulting, in	129.7880078	38.78455712	3.346383649	0.00103479
Percent slabs cracked	0.105779541	0.056079961	1.886227075	0.061197312
IRI, in/mile	-0.078695094	0.067246351	-1.170250765	0.243756006

Table 4.1, shows that the IRI distress criterion carries a negative regression coefficient relating it to ΔPSI. This is due to the fact that IRI is actually a combination of mean joint faulting and % slabs cracked that is used to express overall roughness of a

rigid pavement surface and can be directly correlated to PSI. It can be said that the IRI coefficient and the intercept are simply correction factors for the mean joint faulting and % slabs cracked values, which influence the ΔPSI.

The smaller p-value suggests a greater statistical importance in the regression analysis. In this instance, the IRI and the intercept have high p-values when compared to the faulting and percent slabs cracked p-values. While it would have been quite possible to disregard the IRI distress in this regression analysis altogether, it was included simply because it is a predicted distress used in the 2002 Design Guide, and therefore it is of relevance in this situation. The correlation between ΔPSI and IRI will be shown in the latter sections of this chapter.

PHASE 2 – SENSITIVITY ANALYSIS

Table 4.2 summarizes the sensitivity analysis. An input was considered to have a notable influence on the rigid pavement distresses in the 2002 Design Guide output if it affected one or more of the distresses. A considerable change in faulting was set at 0.1 inches. A considerable change in Percent Slabs Cracked was 10%, and a considerable change in IRI was set at 10 in/mile. If any of the three distresses measured encountered a change at or above these thresholds, then the input was listed as influential in the 2002 Design Guide. As an example, Figure 4.2 illustrates the difference between parameters that have a large versus small impact on pavement distress. The changes in the distresses shown in the Table 4.2 are absolute values of differences when changing from the low to high input value.

Table 4.2 Sensitivity Analysis Results.

Design Input	Input Values			2002 Design Guide Distresses			
	Low	Default	High	Δ Faulting	Δ % Slabs Cracked	Δ IRI	Influence
Operational Speed (mph)	40	60	80	0	0	0	NO
Mean Wheel Location (in. from marking)	12	18	20	0.014	26.2	29	YES
Traffic Wander SD(in)	10	14	20	0.009	23.8	4.9	YES
Design Lane Width (ft)	12	14	20	0	0	0	NO
Avg. Axle Width (ft)	6	8.5	10	0	0.5	0.4	NO
Dual Tire Spacing (in)	12	20	24	0	23.5	19	YES
Single Tire Pressure (psi)	80	120	160	0	0.4	0.3	NO
Dual Tire Pressure (psi)	80	120	160	0	9	7.4	NO
Tandem Axle Spacing (in)	40	51.6	60	0	9	7.4	NO
Tridem Axle Spacing (in)	40	49.2	60	0	1.1	0.7	NO
Quad Axle Spacing (in)	40	49.2	60	0	0	0	NO
Short Wheelbase (ft)	12	14	20	0	0	0	NO
Medium Wheelbase (ft)	12	15	18	0	0	0	NO
Long Wheelbase (ft)	15	18	21	0	0	0	NO
Percent Short Wheelbase	20	33	60	0	0	0	NO
Percent Medium Wheelbase	20	33	60	0	0	0	NO
Percent Long Wheelbase	20	33	60	0	0	0	NO
Climate	ANC	ATL	MIA	0.25	12.4	22	YES
Joint Sealant Type	None	Liquid	Silicone	0	0	0	NO
Edge Support Selection	None	Wide Slab	Tied PCC	0.005	4.2	4.5	NO
PCC-Base Interface	Unbonded	Unbonded	Bonded	0	0.5	0.6	NO
Erodibility Index	Erodible	Resistant	Very Resistant	0.17	0.7	8.1	Yes
Loss of Bond Age (months)	40	60	80	0	0	0	NO
Surface Shortwave Absorptivity	0.6	0.85	1	0.019	33.6	35	YES
Infiltration	Low	Medium	High	0	0	0	NO
Drainage Path Length (ft)	12	14	20	0	0	0	NO
Pavement Cross Slope	0.1	0.2	0.5	0	0	0	NO
Coeff. Of Thermal Expansion ($/^0Fx10^{-4}$)	6	9	12	0.097	95.9	130	YES
Thermal Conductivity (BTU/hr-ft-F)	1	1.25	1.5	0.11	28	29	YES
Heat Capacity (BTU/hr-ft-F)	0.1	0.28	0.5	0.01	39.7	38	YES
PCC-Zero Stress Temp. (^0F)	100	120	140	0.017	0	8.4	NO
Ult. Shrinkage (microstrain)	500	670	750	0.007	0.2	3.5	NO
Reversible Shrinkage (%)	25	50	75	0	0	0	NO
Time to Develop 50% Shrinkage (days)	15	35	60	0	0	0	NO
20yr/28day Compressive Strength Ratio	1	1.2	1.5	0	2.3	2.4	NO
20yr/28day Elastic Modulus Ratio	1	1.2	1.5	0	2.3	2.4	NO
20yr/28day Resilient Modulus Ratio	1	1.2	1.5	0	2.3	2.4	NO
20yr/28day Tensile Strength Ratio	1	1.2	1.5	0	2.3	2.4	NO

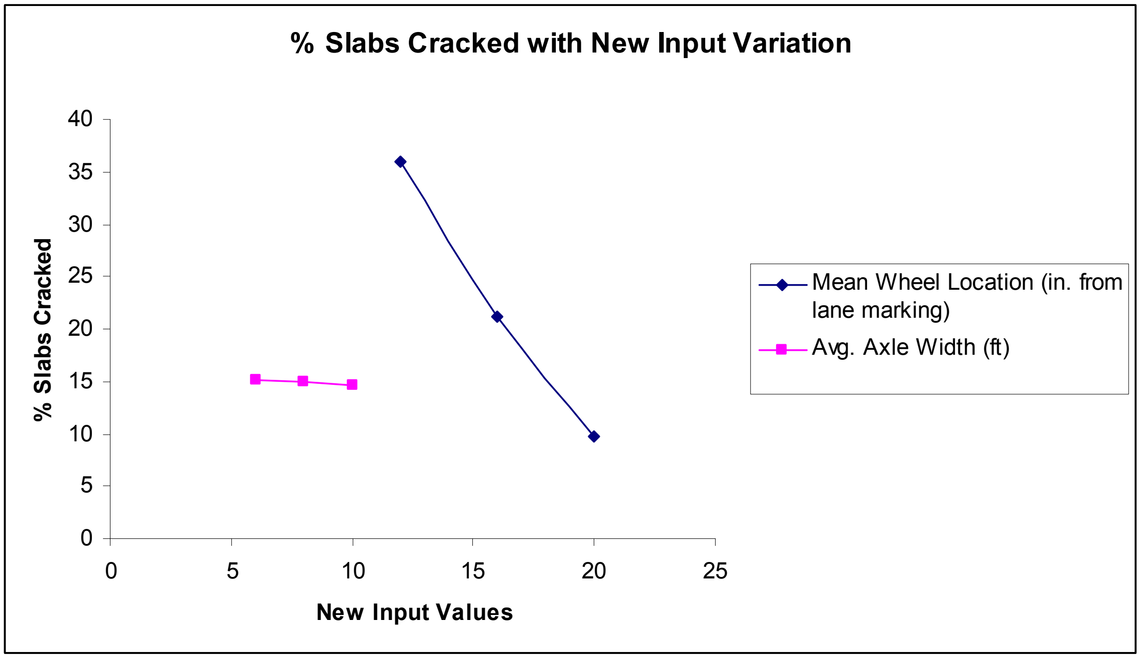

Figure 4.2 Example of Influence of Input Parameters on Predicted Distress.

The group of new inputs that produced the greatest influence on predicted pavement distresses included:

- Coefficient of thermal expansion
- Thermal conductivity
- Heat capacity
- Shortwave surface absorptivity
- Mean wheel location
- Wheel wander standard deviation
- Dual tire spacing
- Climate
- Erodibility index

It makes sense that the thermal properties of the slab would influence rigid pavement distresses, particularly the cracking of the slab. The higher coefficient of thermal expansion, causing greater expansion in the slab, surface shortwave absorptivity, causing a steeper thermal gradient in the slab which creates a higher degree of curling and warping stresses in the slab, and thermal conductivity, which increases the heat absorption in the slab, would theoretically lead to higher degrees of cracking in the slab due to the increased expansion and contraction of the slab.

It is also logical that some of the traffic inputs, aside from amount of traffic, would also affect the predicted pavement responses. The wheel location, wheel wander and dual tire spacing all affect where loads are placed on the slab and the concentration of stresses. These, in turn, affect overall performance of the pavement.

It should be noted that this sensitivity analysis was only the groundwork for a more extensive sensitivity analysis that should be performed in the future to further explore these new inputs. Further analyses should examine interaction of the various input parameters.

PHASE 3 – DESIGN SLAB THICKNESS COMPARISON

The final, and perhaps most important, phase of this research was to investigate slab thickness comparisons between the 1993 and 2002 Design Guides. Figure 4.3 summarizes the slab thickness comparisons using the approach detailed in Chapter 3. From a comparison standpoint, 78% of the designs differed by less than 1 in. between the two approaches. 93% of the designs were within 2 in. between design approaches. However, as seen in the figure, it did appear that the 2002 Design Guide arrived at slightly thinner pavement designs. On average, thicknesses from the 2002 Design Guide were 0.55 in. less than that from the 1993 Design Guide. It was clear from this figure that the 2002 Design Guide, although through a very different method of pavement design than the 1993 Design Guide, calculates optimum slab thicknesses very comparable to the 1993 AASHTO Design Guide for Rigid Pavements.

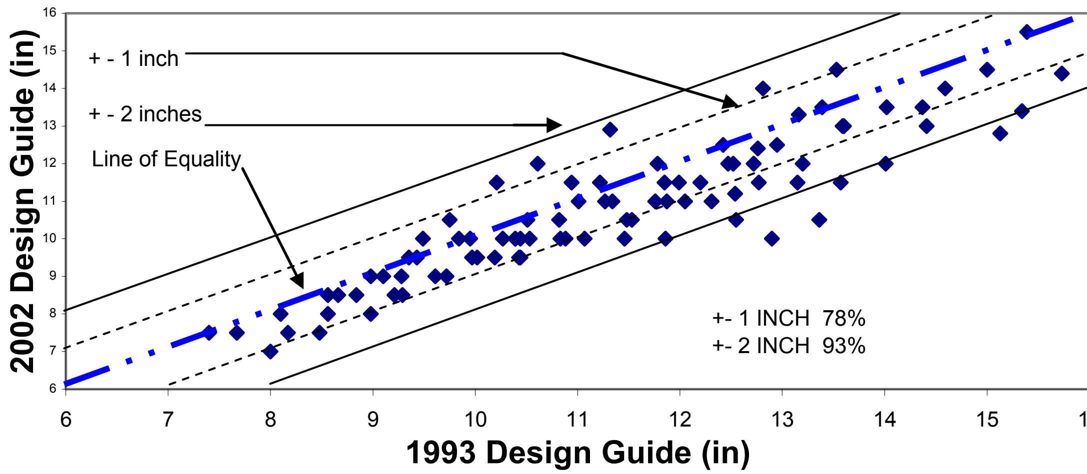

Figure 4.3. Slab Thickness Comparison Results.

SUMMARY

This chapter detailed the results of the three phases of the investigation. A regression equation was developed to determine ΔPSI, in the context of the 1993 Design Guide, from predicted distresses from the 2002 Design Guide. The sensitivity analysis showed that many parameters did not influence the predicted distresses, while some slab and traffic characteristics proved to be more influential. Finally, through slab thickness comparisons, it was found that the two approaches yielded very similar slab thicknesses with the 2002 Design Guide thickness slightly thinner.

CHAPTER 5 – CONCLUSIONS AND RECOMMENDATIONS

NCHRP 1-37A has released the beta version of the new M-E Pavement Design Guide, also called the 2002 Design Guide. Before it gains widespread acceptance and use, there is a need for studies into how the new guide compares to the existing design methodology currently used by many state agencies. This report documented a study comparing the 2002 versus the 1993 Pavement Design Guides. Specifically, the investigation was divided into three phases. Phase 1 compared performance predictions made by the 1993 and 2002 Design Guides. Phase 2 determined the sensitivity of the 2002 Design Guide to its many inputs. Finally, Phase 3 compared slab thicknesses between the two design approaches. Based upon the findings presented in this report, the following conclusions and recommendations can be made:

1. A strong relationship exists between performance predictions made by the 1993 and 2002 Design Guides. The developed regression equation, presented in Chapter 4, can translate results obtained from the 2002 Design Guide (i.e., IRI, faulting and cracking) into performance predicted by the 1993 Design Guide (ΔPSI).
2. The sensitivity analysis identified slab thermal properties and traffic characteristics as critical design inputs. Emphasis should be placed on obtaining these inputs to increase accuracy of performance predictions. However, further studies are warranted to examine interactions between the various input parameters.
3. The two design approaches produce very similar design thicknesses, though the 2002 Design Guide typically was somewhat thinner; 0.55 in. on average. Therefore, it is *not* expected that future designs using the 2002 Design Guide will dramatically

change PCC designs in terms of slab thickness. However, the 2002 Design Guide will give the designer a more complete understanding of what kinds of distress may appear over the design life of the pavement, in addition to overall ride quality.

4. To more fully understand the 2002 Design Guide, further comparison designs are warranted. New designs being developed by ALDOT for rigid pavements, using the 1993 Design Guide, could also be run through the 2002 Design Guide software to build a more extensive basis of comparison.

REFERENCES

1. Basma, A. and Adli Al-Balbissi, "Probabilistic Design of Flexible and Rigid Pavements Using AASHTO Equations," Transportation Research Record No. 1227, 1989, pp. 34-43.
2. Hall, K.T., "State of the Art and Practice in Rigid Pavement Design," AB202: Committee on Rigid Pavement Design, Transportation Research Board, Washington D.C., 2000.
3. Huang, Y.H., Pavement Analysis and Design, Prentice Hall, New Jersey, 1993.
4. Lee, Y-H, "TKUPAV: Stress Analysis and Thickness Design Program for Rigid Pavements," Journal of Transportation Engineering, July/August 1999, pp. 338-398.
5. McAuliffe, D., L. J. Bendana, and Wie-Shih Yang, "Mechanistic-Empirical Rigid Pavement Design for New York State," Transportation Research Record No. 1449, 1994, pp. 141-147.
6. Packard, R.G., Thickness Design for Concrete Highway and Street Pavements, Portland Cement Association, 1984.
7. Portland Cement Association, Thickness Design for Concrete Highway and Street Pavements, Skokie, IL, 1984.
8. Roesler, J.R., J.T. Harvey, J.Farver and F. Long, "Investigation of Design and Construction Issues for Long Life Concrete Pavement Strategies," California Department of Transportation, February, 2000.

9. Wu, Chung-Lung and Mang Tia, "Development of a Rational Thickness Design Method for Rigid Pavements," Transportation Research Record No. 1227, 1989, pp. 205-218.

10. Yoder, E.J. and M.W. Witczak, Principles of Pavement Design, Second Edition, Wiley-Interscience Publications, New York, 1975.

11. *Calibrated Mechanistic Structural Analysis Procedures for Pavements*; NCHRP Project 1-26; TRB National Research Council, Washington D.C.; 1992.

12. *Guide for Design of Pavement Structures*; American Association of State and Highway Transportation Officials; Washington D.C.; 1993.