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IMPLEMENTATION OF THE 2002 AASHTO DESIGN GUIDE FOR PAVEMENT STRUCTURES IN KDOT

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16 Abstract

The AASHTO Guide for the Design of Pavement Structures is the primary document used by state highway agencies to design new and rehabilitated highway pavements. Currently the Kansas Department of Transportation (KDOT) uses the 1993 edition of the AASHTO pavement design guide, based on empirical performance equations, for the design of Jointed Plain Concrete Pavements (JPCP). However, the newly released Mechanistic-Empirical Pavement Design Guide (MEPDG) provides methodologies for mechanistic-empirical pavement design while accounting for local materials, environmental conditions, and actual highway traffic load distribution by means of axle load spectra.

The major objective of this study was to predict pavement distresses from the MEPDG design analysis for selected in-service JPCP projects in Kansas. Five roadway sections designed by KDOT and three long term pavement performance (LTPP) sections in Kansas were analyzed. Project-specific construction, materials, climatic, and traffic data were also generated in the study. Typical examples of axle load spectra calculations from the existing Weigh-in-Motion (WIM) data were provided. Vehicle class and hourly truck traffic distributions were also derived from Automatic Vehicle Classification (AVC) data provided by KDOT. The predicted output variables, IRI, percent slabs cracked, and faulting values, were compared with those obtained during an annual pavement management system (PMS) condition survey done by KDOT. A sensitivity analysis was also performed to determine the sensitivity of the output variables due to variations in the key input parameters used in the design process. Finally, the interaction of selected significant factors through statistical analysis was identified to find the effect on current KDOT specifications for rigid pavement construction.

The results showed that IRI was the most sensitive output. For most projects in this study, the predicted IRI was similar to the measured values. MEPDG analysis showed minimal or no faulting and was confirmed by visual observation. Only a few projects showed some cracking. It was also observed that the MEPDG outputs were very sensitive to some specific traffic, material, and construction input parameters such as average daily truck traffic, truck percentages, dowel diameter, tied concrete shoulder, widened lane, slab thickness, coefficient of thermal expansion, compressive strength, base type, etc.

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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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ABSTRACT

The AASHTO Guide for the Design of Pavement Structures is the primary document used by state highway agencies to design new and rehabilitated highway pavements. Currently the Kansas Department of Transportation (KDOT) uses the 1993 edition of the AASHTO pavement design guide, based on empirical performance equations, for the design of Jointed Plain Concrete Pavements (JPCP). However, the newly released Mechanistic-Empirical Pavement Design Guide (MEPDG) provides methodologies for mechanistic-empirical pavement design while accounting for local materials, environmental conditions, and actual highway traffic load distribution by means of axle load spectra.

The major objective of this study was to predict pavement distresses from the MEPDG design analysis for selected in-service JPCP projects in Kansas. Five roadway sections designed by KDOT and three long term pavement performance (LTPP) sections in Kansas were analyzed. Project-specific construction, materials, climatic, and traffic data were also generated in the study. Typical examples of axle load spectra calculations from the existing Weigh-in-Motion (WIM) data were provided. Vehicle class and hourly truck traffic distributions were also derived from Automatic Vehicle Classification (AVC) data provided by KDOT. The predicted output variables, IRI, percent slabs cracked, and faulting values, were compared with those obtained during an annual pavement management system (PMS) condition survey done by KDOT. A sensitivity analysis was also performed to determine the sensitivity of the output variables due to variations in the key input parameters used in the design process. Finally, the interaction of selected significant factors through statistical analysis was

identified to find the effect on current KDOT specifications for rigid pavement construction.

The results showed that IRI was the most sensitive output. For most projects in this study, the predicted IRI was similar to the measured values. MEPDG analysis showed minimal or no faulting and was confirmed by visual observation. Only a few projects showed some cracking. It was also observed that the MEPDG outputs were very sensitive to some specific traffic, material, and construction input parameters such as average daily truck traffic, truck percentages, dowel diameter, tied concrete shoulder, widened lane, slab thickness, coefficient of thermal expansion, compressive strength, base type, etc.

For the asphalt pavement sections, the MEPDG procedure resulted in much thinner sections when compared to the sections obtained following the 1993 AASHTO design guide. However, these results were found to be sensitive to the failure criteria chosen. Four of the PCC sections, designed using the 1993 AASHTO guide, were thicker than those analyzed following the NCHRP MEPDG. The thickness of the fifth project was the same in both cases.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

1.1 Introduction

The most widely used procedure for design of concrete pavements is specified in the *Guide for Design of Pavement Structures*, published in 1986 and 1993, by the American Association of State Highway and Transportation Officials (AASHTO 1986; AASHTO 1993). A few states use the 1972 American Association of State Highway Officials (AASHO) Interim Guide procedure, the Portland Cement Association (PCA) procedure, their own empirical or mechanistic-empirical procedure, or a design catalog (Hall 2003). The 1986 and 1993 Guides contained some state-of-practice refinements in materials input parameters and design procedures for rehabilitation design. In recognition of the limitations of earlier Guides, the AASHTO Joint Task Force on Pavements (JTFP) initiated an effort in the late nineties to develop an improved Guide by 2002. The major long-term goal identified by the JTFP was the development of a design guide based as fully as possible on mechanistic principles. The National Cooperative Highway Research Program (NCHRP) sponsored project 1-37A to develop a user-friendly procedure capable of doing mechanistic-empirical design while accounting for local environment conditions, local highway materials, and actual highway traffic distribution by means of axle load spectra. The overall objective of the NCHRP guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures (now known as MEPDG) is to provide the highway community with a state-of-the-practice tool for the design of new and rehabilitated pavement structures, based on mechanistic-empirical principles.

1.2 Problem Statement

The *AASHTO Guide for the Design of Pavement Structures* is the primary document used by the state highway agencies to design new and rehabilitated highway pavements. The National Pavement Design Review conducted by the Federal Highway Administration (FHWA) during 1995-1997 found that 80% of the states use either the 1972, 1986, or 1993 AASHTO Pavement Design Guide (KSU Proposal 2003). The Kansas Department of Transportation (KDOT) is currently using the 1993 AASHTO pavement design guide. All AASHTO Design Guide versions are based on empirical performance equations developed using the AASHO (now AASHTO) Road Test data from the late 1950's. Although various editions of the AASHTO Design Guide have served the pavement community well for several decades, many serious limitations exist for their continued use as primary pavement design procedures. The limitations are described as follows (NCHRP 2004):

(a) One of the serious limitations of the AASHO Road Test is the traffic loading deficiency. Heavy truck traffic design volume levels have increased tremendously (about 10 to 20 times) since the design of the pavements used in the Interstate system in the 1960's. The original Interstate pavements were designed for 5 to 15 million trucks over a 20 year period, whereas today these same pavements must be designed for 50 to 200 million trucks and sometimes, for even longer design life (e.g., 30-40 years).

(b) Pavement rehabilitation was not included in the Road Test experimental design. The rehabilitation design procedures described in the 1993 Guide are completely empirical and limited, especially in consideration of heavy traffic.

(c) Since the Road test was conducted at one geographic location, it is very difficult to address the effects of different climatic conditions on the pavement performance equation developed.

(d) Only one type of subgrade soil was used for all test sections at the Road Test.

(e) During the Road Test, only one type of surface material was used for each of the different pavement type, such as one hot mix asphalt (HMA) mixture for flexible and one Portland cement concrete (PCC) mixture for concrete pavements. Currently, different types of mixture, such as Superpave, Stone Mastic Asphalt (SMA), high-strength PCC, etc., are available.

(f) Only unstabilized, dense graded granular bases were included in the main pavement sections (limited use of treated base was included for flexible pavements). Currently, most pavements are constructed over stabilized base or subbase, especially for heavier traffic loading.

(g) Vehicle suspension, axle configuration, and tire types and pressures were representatives of the vehicles used in the late 1950's. Most of those are outmoded today.

(h) Pavement designs, materials and construction were representative of those at the time of the Road Test. For example, no sub-drainage was included in the Road Test sections, but positive subdrainage has become common in today's highways.

(i) The Road Test only lasted approximately two years, and has been used for the design of pavements that are supposed to last for 20 years. Therefore, significant extrapolation is required to ensure the design life reliability.

(j) Earlier AASHTO procedures relate the thickness of the pavement surface layers (asphalt layers or concrete slab) to serviceability. However, research and observations have shown that many pavements need rehabilitation for reasons that are not related directly to the pavement thickness (e.g., rutting, thermal cracking, and faulting). These failure modes are not considered directly in the previous versions of the AASHTO Guide.

(k) According to the 1986 AASHTO Guide, desired reliability level can be achieved through a large multiplier of design traffic loading and this has never been validated. The multiplier increased greatly with the design level of reliability and may result in excessive layer thickness for pavements carrying heavier traffic.

These limitations have long been recognized by the pavement design community. Beginning in 1987 with the NCHRP Project 1-26, formal steps were taken to include mechanistic principles in the AASHTO design procedures. An NCHRP report published in 1990 first recommended the inclusion of the mechanistic procedures in the AASHTO guide. This research proposed two programs, ILLI-PAVE and ILLI-SLAB for flexible and rigid pavement design, respectively. In turn, mechanistic design procedures for the rigid pavement were included as a supplement to the *1993 Guide* (NCHRP 1990).

The AASHTO Join Task Force on Pavements (JTFP) initiated an effort to develop an improved design guide in 1997. NCHRP project 1-37 was the initial step toward developing this new guide. Finally the objective was accomplished through developing MEPDG itself, which is based on the existing mechanistic-empirical technologies. User-oriented computational software and documentation based on the MEPDG procedure have also enhanced the objective. Since the resulting procedure is very sound and flexible, and considerably surpasses any currently available pavement design and analysis tools, it is expected it will be adopted by AASHTO as the new AASHTO design method for pavements structures (NCHRP 2004). It is also expected that KDOT will adopt the new AASHTO design method to replace the 1993 AASHTO design method currently in use.

1.3 Objectives

The major objective of this study was to predict distresses from the MEPDG design analysis for selected in-service Jointed Plain Concrete Pavement (JPCP) projects in Kansas. The predicted distresses were then to be compared with the available measured distresses. Sensitivity analysis was also to be performed for determining the sensitivity of the output variables due to variations in the key input parameters used in the design process. For this task, project-specific material, climatic, and traffic inputs also needed to be generated. Typical examples of axle load spectra calculation from the existing Weigh-in-Motion (WIM) data were calculated. Vehicle class and hourly truck traffic distributions needed to be derived from the KDOT-provided Automatic Vehicle Classification (AVC) data. The final objective was to identify the interactions of some significant factors through statistical analysis to find the effect on the current KDOT specifications for rigid pavement construction process.

1.4 Organization of the Report

The report is divided into six chapters. Chapter 1 is an introduction to the problem. Chapter 2 presents an overview of the Jointed Plain Concrete Pavements

(JPCP). It also describes the framework for the Mechanistic-Empirical pavement design method and the new MEPDG software. Chapter 3 describes the study sections and input data generation process. Chapter 4 presents the MEPDG design analysis of some existing JPCP projects in Kansas and the sensitivity analysis of the factors that significantly affect predicted JPCP distresses. Chapter 5 presents the pavement surface type selection. Finally Chapter 6 presents the conclusions based on this study and recommendations for future research.

CHAPTER 2 - LITERATURE REVIEW

This chapter presents an overview of the Jointed Plain Concrete Pavement, the framework for the Mechanistic-Empirical pavement design method, and an introduction to the new Mechanistic-Empirical Design Guide software developed by the National Cooperative Highway Research Program (NCHRP).

2.1 Jointed Plain Concrete Pavement (JPCP)

2.1.1 Geometric Design

The principal elements of a roadway cross section consist of the travel lanes, shoulder, and medians (for some multilane highways). Marginal elements include median and roadside barrier, curbs, gutters, guard rails, sidewalks, and side slopes. Figure 2.1 shows a typical cross section for a two-lane highway.

Travel lane widths usually vary from 9 to 12 feet, with a 12-foot lane being predominant on most high-type highways (AASHTO 2004). It is generally accepted that lane widths of 12 feet should be provided on main highways. A widened lane of 14 feet width was used in Kansas, for some experimental purposes, though this lane width is not a very common practice nationwide.

A shoulder is the portion of the roadway contiguous with the traveled way for accommodation of stopped vehicles, for emergency use, and for lateral support of sub-base, base, and surface courses (AASHTO 2004). Shoulder width varies from only 2 feet on minor rural roads to about 12 feet on major roads. Shoulder can be tied or untied. Tie bars are used to construct tied shoulder. Shoulder width in Kansas is governed by the shoulder design policy. Full-width shoulders for a 2-lane pavement are 10 feet wide. For 4-lane highways, the outside shoulders are 10 feet wide and the inside shoulders are 6 feet wide. Lower volume highways may have shoulders from 3 to 10 foot widths. Some shoulders may be composite with the 3 foot shoulder adjacent to the traveled way being paved and the shoulder outside this may be turf or aggregate surfaced.

The right lane in a four-lane divided highway section is designated as the driving lane. The left lane is the passing lane. Two lane and wider undivided pavements on tangents or on flat curves have a crown or high point in the middle, and slope downward toward both edges. This provides a cross slope, whose cross section can be either curved or plane or a combination of the two. AASHTO recommended (2004) cross slope rates are 1.5 percent to 2.0 percent for high-type pavements, 1.5 percent to 3.0 percent for intermediate-type of pavements, 2.0 to 6.0 percent for low-type pavements. In Kansas, the traveled way has a typical cross slope of 1.6 percent (3/16 inch per foot) as shown in Figure 2.2. The shoulders have a cross slope of 4.2 percent (1/2 inch per foot). The shoulder side slope is 1:6.

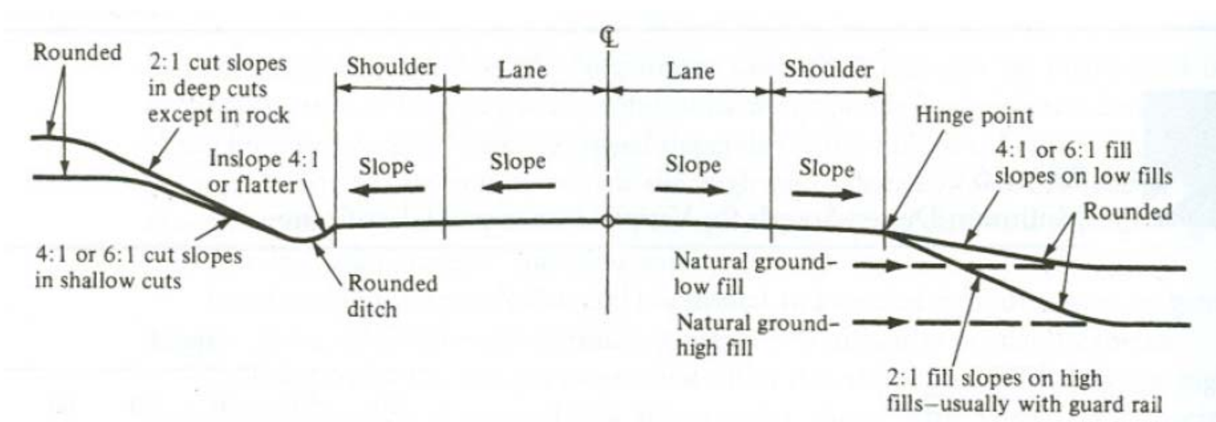


Figure 2.1: Typical cross-section of a two-lane highway (AASHTO 1984)

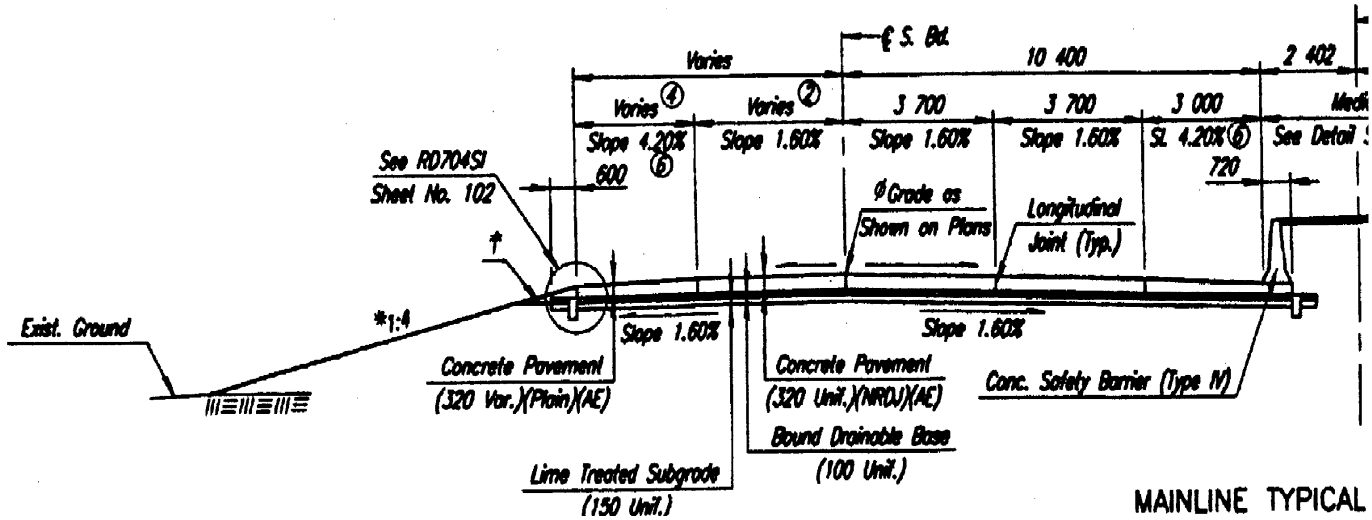


Figure 2.2: Typical cross-section of a divided highway in Kansas

2.1.2 Drainage

Pavement drainage systems may be grouped into three general classes based on their geometry: (1) longitudinal drains, (2) drainage blankets, and (3) transverse drains (NHI 1992).

*Longitudina*l drains are usually located near the pavement or shoulder edge and run parallel to the roadway centerline. *Drainage blankets* are layers of highly permeable material that normally extend across the entire pavement width for an appreciable length along the pavement centerline. *Transverse drains* are generally placed laterally under the pavement, usually at right angles to the pavement centerline.

Most sub-drainage systems consist of some combination of the drainage layers, filter layers, water collection systems, and outlet systems. *Drainage layers* are usually constructed of granular material whose gradation has been selected and controlled to ensure a high degree of permeability. Asphalt stabilized, open-graded material and porous concrete can also be used as drainage layers. The drainage layer may serve as a part of the drainage system and as the pavement base or subbase. *Filter layers* consist of either specially graded granular material or commercial filter fabric. *Collector systems* serve to gather the water from drainage layers or surrounding materials. They often consist of perforated pipes placed in the permeable granular drainage layer. *Outlet systems* convey the water from the collector system to some suitable discharge point.

The addition of drainable base layer in the design of PCC pavements has been recommended practice by FHWA since 1992. KDOT, along with many other state agencies, had been using this design practice since 1988 to help elevate moisture-related damage to the pavement structure. When the drainage layers are properly designed, constructed and maintained, this damage can be minimized. In 2002 KDOT changed their policy and only uses a drainage layer in special circumstances where an investigation shows a need for drainage.

Based on the precipitation intensity data for the United States, Kansas averages 1.4 in/hr in the West and 1.8 in/hr in the East based on a two-year, one-hour rainfall event. However, the majority of the PCC pavements in Kansas do not incorporate a drainable base because the traffic volume is low to medium, the pavement is dowelled, and shoulders are concrete, and are tied to the mainline pavement.

Until 2000, the KDOT design guidelines were as follows for the dowelled Portland Cement Concrete Pavements: less than 275 Equivalent Single Axle Loads (ESAL)/day in the design lane an unbound, dense-graded aggregate base is used. From 275 to 650 ESAL/day in the design lane, a bound dense graded base (either with Portland cement or asphalt cement) is used. For loads greater than 650 ESAL/day a bound drainable base (BDB) with edge drains, as shown in Figure 2.2, is used. BDB are designed to have a minimum permeability of 1000 feet/day. The 7-day required compressive strength for 6 inch x 6 inch cylinders of concrete mixes bound with fly ash or Portland cement shall be in the range of 595 psi to 1,200 psi with a Marshall stability of 400 psi for bases bound with asphalt cement. The water carried into the BDB layer can be removed by the edge drains and outlets, or on a case-by-case basis, BDB may be daylighted to the shoulder slope (KDOT 1990).

The contractor chooses the aggregate gradations based on the mix design required to obtain the minimum permeability. However, the contractors are allowed to crush and recycle the existing concrete pavement to be used in the drainable base layer. Typically, the contractors use 70 to 100% of the crushed concrete sweetened with sand-sand gravel, two to three percent of cement, and four to five percent of fly ash. A minimum water/cement ratio of 0.45 is recommended to increase the workability (NHI 1998).

The longitudinal grade also facilitates the surface drainage of PCC pavements. AASHTO recommends a minimum of 0.2% to 0.3% longitudinal grade for adequate drainage of most high-type pavements with recommended cross slope.

The normal cross slope used by KDOT is a crown at the center of the highway. Thus, water flows toward both sides of the pavement. The edge drains are constructed longitudinally down the highway, on both sides of the concrete shoulder. The edge drain pipe is placed in a (8 in x 8 in) trench which is wrapped with a geosynthetic to avoid contamination from the fine-grained soils. The pipe is perforated and is generally 4 inches in diameter. The material around the pipe in the trench is the same aggregate as used in BDB but without any binder.

An outlet pipe is placed every 500 feet that carries the water from the edge drain pipe to the ditch. The outlet pipe is also 4 inches in diameter and is not perforated. Since a high percentage of these pipes get crushed, the stiffness of these pipes is much greater than the stiffness of the edge drain pipe.

When a pavement is superelevated, the edge drain pipe is discontinued on the high side of the superelevation and continued again when coming out of the superelevation. Because of the potentially increased flow of water within the base due to the single cross-slope, the outlet spacing on the low side of the superelevation should be reduced to 200 feet.

2.1.3 Concrete Slab

This is the topmost layer of the PCCP system as shown in Figure 2.3. The desirable characteristics include friction, smoothness, noise control, and drainage. The slab must have a thickness that is adequate to support the loads, to be applied during its service life, and the design must be economical. Several design methods are available to determine the required thickness of the PCCP slab. The Portland Cement Association (PCA 1984) method is one such design process. According to PCA, design considerations that are vital to the satisfactory performance and long life of a PCCP are: reasonably uniform support for the pavement, elimination of pumping by using a thin treated or untreated base course, adequate joint design, and a thickness that will keep load stresses within safe limits. Thickness is determined based on two criteria: erosion analysis, and fatigue analysis. Fatigue analysis recognizes that pavements can fail by the fatigue of concrete, while in erosion analysis pavements fail by pumping, erosion of foundation, and joint faulting. Several tables and nomographs are available to determine thickness based on material properties (modulus of rupture of concrete and modulus of subgrade reaction) and traffic (in terms of actual single and tandem axle load distributions).

The AASHTO (1993) method is another widely used method. The procedure uses empirical equations obtained from the AASHO Road Test with further modifications based on theory and experience. Unlike the PCA method, the AASHTO method is based on the 18-kip equivalent single-axle load (ESAL) applications. KDOT currently uses the 1993 AASHTO Design Guide for JPCP design. The minimum PCC slab thickness on

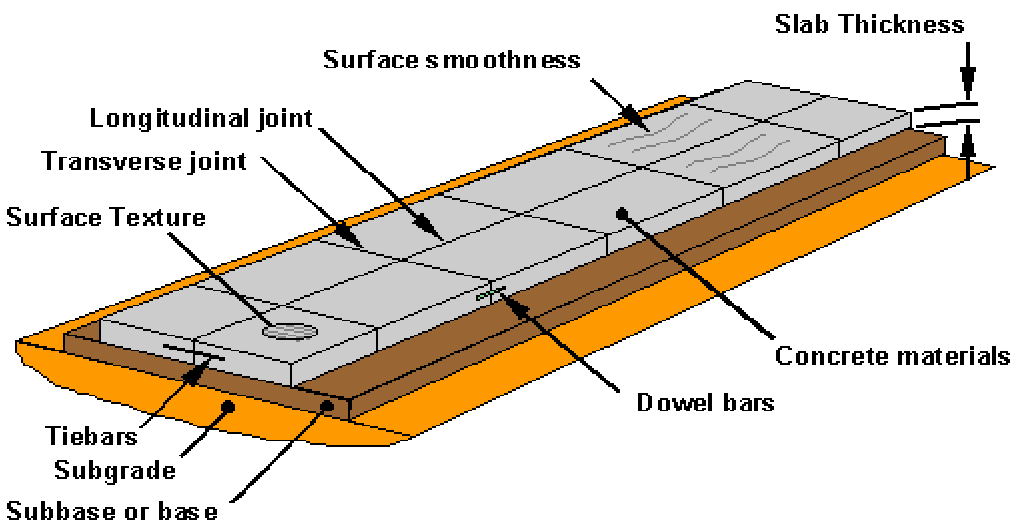


Figure 2.3: Basic component of a concrete pavement (ACPA 2005)

Kansas highways are 8 inch (200 mm). Normally, rigid pavements in Kansas have 2-stage construction since an overlay is planned after 20 years of service life for the initial design.

2.1.4 Concrete mixture

The process of determining required and specified characteristics of a concrete mixture is called mix design. Concrete is a mixture of Portland cement, water, and aggregates (coarse and fine) with or without air entraining or other admixtures. Desirable concrete characteristics include: (1) fresh/plastic concrete properties; (2) required mechanical properties of hardened concrete such as, strength and durability requirements; and (3) the inclusion, exclusion, or limits on specific ingredients. Mix design leads to the development of a concrete specification (PCA 2002).

2.1.4.1 Cement Content and Type

According to the KDOT standard specifications, either Type IP Portland–Pozzolan cement, Type I (PM) Pozzolan-Modified Portland cement or Type II Portland cement shall be used for the construction of concrete pavement. The cementitious material content is usually determined from the selected water-cementitious materials ratio and water content, although minimum cement content frequently is included in specifications in addition to maximum water-cementitious materials ratio (KDOT 1990). According to the Portland Cement Association (PCA), minimum cement content varies from 470 to 610 pound per cubic yard based on the nominal maximum aggregate size of 1 ½ inch to 3/8 inch, respectively. KDOT standard specification specifies minimum 602 pounds of cement per cubic yard of concrete, prepared with coarse and fine aggregate. Concrete with mixed aggregates (mostly siliceous) will require minimum 620 pounds of cement per cubic yard.

2.1.4.2 Coarse Aggregate

Two characteristics of aggregates, grading and nature of particles (shape, porosity, and surface texture), have an influence on proportioning concrete mixtures to affect the workability of fresh concrete. KDOT uses Siliceous Gravel, Chat or Calcite Cemented Sandstones as coarse aggregates for concrete pavement with minimum soundness of 0.90. In addition to the above stated aggregates, crushed Limestone or Dolomite is also commonly used for pavement construction. Maximum loss in the Los Angeles (L.A.) wear test for all these aggregates is 50%. Table 2.1 summarizes the typical gradation for coarse aggregates for concrete pavement in Kansas.

Table 2.1: Gradation for Coarse Aggregate for Pavement Concrete (KDOT 1990)

Sieve size	Percent Retained
1 ½”	0
1”	0-10
¾”	30-65
½”	-
3/8”	70-93
No.4	-
No.8	95-100
No.30	-

KDOT also uses mixed aggregates which are naturally occurring predominant siliceous aggregates or may be a sweetened basic aggregate with a minimum soundness of 0.90 and maximum L.A. wear of 50%. Total mixed aggregate may not be used in concrete for pavement, unless a sample of a total mixed aggregate from the source has met the desired modulus of rupture and expansion requirements. Current KDOT special provisions require that mixed aggregate should have a minimum 60-day modulus of rupture of 550 psi and expansion should not exceed more than 0.070% at 365 days (KDOT 1990).

2.1.4.3 Fine Aggregate

KDOT uses two types of fine aggregates: *FA-A* and *FA-B*. FA-A consists of natural sand resulting from the disintegration of siliceous and/or calcareous rocks or manufactured sand produced by crushing predominantly siliceous materials or a combination of natural and manufacture sand. FA-B consists of fine granular particles resulting from the crushing of zinc and lead ores. Typical grading requirements are shown in Table 2.2.

Table 2.2: Grading Requirements for Fine Aggregate (KDOT 1990)

Sieve size	Percent Retained
3/8”	0
No.4	0-5
No.8	0-24
No.16	15-50
No.30	40-75
No.50	70-90
No.100	90-100

2.1.4.4 Admixtures and Air Content

Entrained air must be used in all concrete that will be exposed to freezing and thawing and de-icing chemicals. Air entrainment also improves workability. Air entrainment is accomplished by using an air entraining Portland cement or by adding an air-entraining admixture at the mixer. KDOT specifies about 6 ± 2 percent of entrained air by volume for JPCP slab concrete.

2.1.4.5 Concrete Consistency

The consistency of the concrete when delivered to the paving train is usually designated by the Engineer. According to KDOT specification, the tolerance permitted from the designated slump shall be plus or minus ¾ of an inch. The maximum slump allowable shall be 2 1/2 inches. When the designated slump is greater than 1 ¾ inches, the upper limit will be determined by the maximum slump.

2.1.4.6 Water-Cement Ratio

In mix design, the water cement ratio is simply the mass of water divided by the mass of total cement content. The water content should be the minimum necessary to maintain the required workability. KDOT specifies a maximum water cement ratio of 0.49 for paving concrete.

2.1.4.7 Concrete Mixing and Delivery

Concrete is mixed in quantities required for immediate use. Most specifications require that concrete should not be used when it has developed initial set or is not in place ½ hour after the water has been added for non-agitated concrete. Concrete may be mixed at the work site, in a central-mix plant, or in truck mixers. Finally, concrete shall be discharged without delay. For the delivery purpose, approved covers shall be provided for protection against the weather as per requirements.

2.1.5 Shoulder and Widened Lane

Shoulder in concrete pavement is considered as a safety area for errant or wandering vehicles. It also serves as an auxiliary area for emergency stopping. Structurally, the shoulder may provide lateral support to a mainline concrete pavement as in the case of widened slabs and/or tied concrete shoulders. Two basic types of shoulder are used in concrete pavements: (1) Tied concrete shoulder and (2) Asphalt shoulder.

Tied concrete shoulder is a paved slab that is tied to a mainline concrete pavement. The concrete shoulder provides lateral support to the mainline pavement by shear load transfer (ties bars and if, paved with the mainline, aggregate interlock) as well as by increased bending resistance. A tied shoulder will have the same thickness as the mainline pavement if the two are paved together. In rural areas, a tied shoulder is sometimes constructed to a lesser thickness than the mainline pavement, with a dense granular fill material below. Tie bars can be placed on chairs or inserted during paving. Both the tie bar and the aggregate interlock at the lane/shoulder joint provide shear load transfer. The tie bar should be deformed steel, at least No. 5 [0.625 in] bars, spaced at no more than 30 in center to center. Transverse joints in the concrete shoulder should match those in the mainline pavement (NHI 2001).

Asphalt shoulders are commonly used adjacent to the concrete mainline pavements because they are less expensive than the concrete shoulder. They may be asphalt concrete or an asphaltic surface treatment. In general, they do not provide lateral support to the mainline pavement.

The 1986/1993 AASHTO Guide procedure takes a tied concrete shoulder into consideration in the assignment of a lower “J” factor, an important factor in rigid pavement design. Kansas PCC pavements, which are dowel jointed and usually have tied PCC shoulders, have a J factor of 2.8. If the pavement does not have tied shoulders or doweled joints, then the J factor is increased. The maximum value is typically 4.0 when there is very poor load transfer across joints and the shoulders are not tied PCC. In Kansas, all new rigid pavements will have tied PCC shoulders or in the case of a 3 foot paved shoulder, it may be considered as a widened lane. These pavements will have a J value of 2.8.

Widened slabs are paved slightly wider (1 to 3 feet) than the conventional slabs, but the travel lane is striped at a width of 12 feet. This keeps trucks from encroaching near the edge of the slab, thereby greatly reducing slab stresses and deflections at the edges and corners. Lane widening is typically done for the outside traffic lane, which usually carries more traffic. In field studies, excellent performance has been observed with widened slab which will reduce the faulting and cracking in JPCP, although excessive widening may lead to longitudinal cracking (NHI 2001).

2.1.6 Base/Subbase

Subbase is the layer between the concrete slab and the foundation layer or subgrade. Experimental work has indicated that subbases play a minor role in increasing the structural capacity of a concrete pavement, which obtains its load carrying capacity mainly from the structural rigidity of the slab. Experiments have also indicated that one inch of concrete is equivalent to about six inches of subbase, and unless suitable subbase material can be obtained economically it will usually be economical to increase the thickness of the slab in order to increase the load carrying capacity (Sharp 1970). In practice, a subbase is used under a concrete slab mainly for construction purposes, i.e., to protect the subgrade soil and to facilitate the paving operation. There are, however, one or two exceptions. Some clayey and silty subgrades tend to exhibit the phenomenon called “pumping.” Under repeated heavy traffic and with ingress of water, these materials readily assume the consistency of mud and are sometimes pumped out through the joints and cracks in concrete slabs. In such cases, a subbase is essential to prevent pumping unless traffic is light. It may also be necessary to provide a subbase to insulate frost-susceptible subgrade soils from frost penetration. A subbase may also be used as a drainage layer in the pavement. It also reduces the bending stress in the slab and deflections at the joints and cracks. Improved joint/crack load transfer is also obtained for dowelled JPCP with treated subbases.

The stabilization of natural soils and aggregates is now widely used in the construction for road bases. Traditional base construction methods using mechanically stable material are not suitable for present day heavy traffic intensities and loadings, but they can be readily improved by incorporating some type of binder. Soil cement has been used in the United States since 1935 (Sharp 1970). The use of lean concrete on a large scale started as a development from soil cement, being a material more suited to the pavements carrying heavy traffic. Cement bound granular material is a further development from lean concrete where some relaxation of gradation of the constituent aggregates is permitted (Croney and Croney 1991). In the United States, the use of lean concrete as a subbase layer for JPCP is a common practice.

2.1.6.1 Granular Base

Granular base consists of untreated dense-graded aggregate, such as crushed stone, crushed slag, crushed or uncrushed gravel, sand, or a mixture of any of these materials. Granular bases have historically seen the most use beneath concrete pavements, but because of their susceptibility to pumping and erosion, may not be suitable for pavements subjected to high traffic levels (NHI 2001).

2.1.6.2 Asphalt-treated Base

Asphalt treated bases use the same aggregates as in the granular bases, but mixed with an asphaltic binder. Typically three to five percent asphalt has been added, but PIARC recommends five to six percent to provide resistance to erosion (NHI 2001).

2.1.6.3 Cement-Treated Base

Cement-treated bases consist of conventional dense-graded aggregates mixed with Portland cement (typically about 3 to 6 percent). About four to five percent cement should produce a 28 day strength of about 750 psi for a cement-treated base. PIARC studies have shown that in order for the base to be erosion resistant, six to eight percent cement is required (NHI 2001). However, some cement-treated bases constructed with these cement contents have been responsible for increased slab cracking, in cases where the bases and the slab were not bonded and the slab experienced high curling stresses. In such cases, shorter joint spacing should be employed. KDOT standard specification requires that that the minimum cement content will be five percent by weight of dry aggregate and the maximum will be ten percent by weight. In Kansas, the Portland cement treated base (PCTB) is constructed two feet wider than the pavement surface. This provides the contractor with a solid surface for the paver’s track line.

2.1.6.4 Lean Concrete Base

Lean concrete is similar to paving concrete, but contains less cement (typically about 200 lbs/cubic yd). Thus, it is lower in strength than the conventional paving concrete (about 20 to 50% of strength of paving concrete). The greatest structural contribution of a lean concrete base is achieved with a high degree of friction (resistance to horizontal sliding) and bond (resistance to vertical separation) between the slab and the base (NHI 2001).

2.1.6.5 Permeable Base

Permeable bases are open graded materials, constructed using high quality crushed stone, with high permeabilities that allow rapid removal of water from the pavement structure. A collector system and a separator layer are required. The base may be treated or untreated. Treated bases are preferred for higher traffic volumes and also to facilitate construction. Stabilized permeable bases also contribute to the bending stiffness of the pavement structure. Theoretically, compared to a dense-graded asphalt-treated base, permeable asphalt-treated base should be less susceptible to stripping and debonding at the slab/base interface (NHI 2001).

In Kansas, Portland cement-treated base (PCTB), asphalt-treated base (ATB), and granular subbase are commonly used for rigid pavements. Base types are selected based on the traffic on the route. Bound drainable base is used in special situations when drainage is a concern. BDB has a very open graded gradation. The water infiltrating the base is meant to move to the edge drain system. To stabilize the base, a small percentage of binder is added. This can be either Portland cement or asphalt cement. The required permeability of the base is 1000 feet/day (KDOT 1990).

2.1.7 Subgrade

The term “subgrade” is commonly used to refer to the foundation upon which the base and concrete layers are constructed. The foundation consists of the natural soil at the site, possibly an embankment of improved material, a rigid layer of bedrock or hard clay at a sufficiently shallow depth. Although a pavement’s top layer is the most prominent, the success or failure of a pavement is more often dependent upon the underlying subgrade-the material upon which the pavement structure is built (NHI 2001). Subgrades are composed of a wide range of materials although some are better than others. The subgrade performance generally depends upon three of its basic characteristics: load bearing capacity, moisture content, and shrinkage and/or swelling (WSDOT 2003). These characteristics are interrelated to each other. The properties of soil that are important for pavement construction are: volume stability, strength, permeability, and durability (Ingles and Metcalf 1972). Subgrade needs to be characterized for concrete pavement design purposes. Both dense liquid (k) and elastic solid (E) models attempt to describe the elastic portion of soil response. However, real soil also exhibits plastic (permanent deformation), and time-dependent responses; slow dissipation of pore water pressures under static loading results in larger deflections than rapid dynamic loading (NHI 2001).

To ensure satisfactory concrete pavement performance, the subgrade must be prepared to provide the stiffness which was assumed in design, uniformity, long-term stability, and a stable platform for construction of the base and slab. Poor subgrade should be avoided if possible, but when it is necessary to build over weak soils there are several methods available to improve subgrade performance. Poor subgrade soil can simply be removed and replaced with high quality fill, although it can be expensive. Other methods are soil stabilization, mixing with coarse material, reinforcement with geosynthetics. Subgrade stabilization includes stabilizing fine-grained soils in place (subgrade) or borrow materials, which are used as subbases, such as hydraulic clay fills, or otherwise poor quality clay and silty materials obtained from cuts or borrow pits

(Little 1995). The presence of highly expansive clay soils, subject to wide fluctuations in moisture content and resulting shrink-swell phenomenon, has clearly been proven to be detrimental to the pavements. Stabilization has been found to be most beneficial for these soils. Different binders are used for stabilization such as lime, Portland cement, and emulsified asphalt. The selection of the binder depends on the subgrade soil. Lime is the most popular binder used now. By adding an appropriate quantity of lime to the subgrade soils, which are suitable for lime stabilization, the engineering properties of these soils can be improved. The stronger, stiffer, and more stable (volumetrically) lime-treated subgrade provides better protection for the pavement. KDOT uses pebble quick lime or hydrated lime for lime treatment of potentially expansive subgrade soils. Soils with more than 2% swell potential require the top six inches of the subgrade be treated with 5% hydrated lime. Soils that do not have over 2% swell potential, or are silty sized particles shall have the top 6 inches of the subgrade treated with approximately 12% fly ash by weight assuming the density of soil at 110 pcf. The top six inch of sandy soils shall be treated with 7% cement by weight. For natural subgrade, the top 18 inches are usually compacted to a density greater than or equal to 95% of the standard density.

2.1.8 Joints in JPCP

Joints are installed in concrete pavements to control the stresses induced by volume change in concrete and to allow for a break in construction at the end of the day’s work. Joint spacing varies from agency to agency and depends on the amount of reinforcement used in the pavement. Generally spacing between 12 to 20 feet is used, although thicker slabs can have longer joint spacing. Dowel bars are used under the joint as load transfer device. Construction joints may be placed at the end of the day's run or when work ceases due to some other interruption.

Expansion joints are provided to allow expansion of concrete. Dowel bars are usually used for this type of joint. A longitudinal joint in a concrete pavement is a joint running continuously the length of the pavement. The joint divides, for example, a two-lane pavement into two sections, the width of each being the width of the traffic lane. The purpose of longitudinal joints is simply to control the magnitude of temperature warping stresses in such a fashion that longitudinal cracking of the pavement will not occur. Longitudinal cracking has been almost completely eliminated in concrete pavements by the provision of adequate longitudinal joints (Wright and Paquette 1987). In two-lane pavements, the two slabs are generally tied together by means of steel tie bars extending transversely across the joint and spaced at intervals along the length of the joint.

Joints may be placed either at fixed interval or at variable spacing [12-15-13-14 feet]. Another variation in joint design is the layout of the joints. Joints can be placed perpendicular to the centerline of the pavements or at an angle to the pavement in a counterclockwise view (known as skewed joint). Skewed joints may be beneficial in reducing faulting of non-doweled pavements, although effectiveness is questionable for doweled pavements. Kansas uses 15 foot joint spacing for new Jointed Plain Concrete Pavement (JPCP).

2.1.8.1 Load Transfer Devices

Load transfer may be defined as the transfer or distribution of load across the discontinuities such as joints or cracks (AASHTO 1993). When a load is applied at a joint or crack, both loaded slab and adjacent unloaded slab deflect. The amount the unloaded slab deflects is directly related to the joint performance. If a joint performs perfectly, both loaded and unloaded slabs deflect equally. Load transfer efficiency depends on temperature, joint spacing, number and magnitude of load applications, foundation support, aggregate particle angularity, and the presence of mechanical load transfer devices, such as, dowel bars (WSDOT 2003). Load transfer is accomplished through aggregate interlock or by dowel bars. In some cases, base courses also contribute to load transfer but are not considered a formal load transfer method.

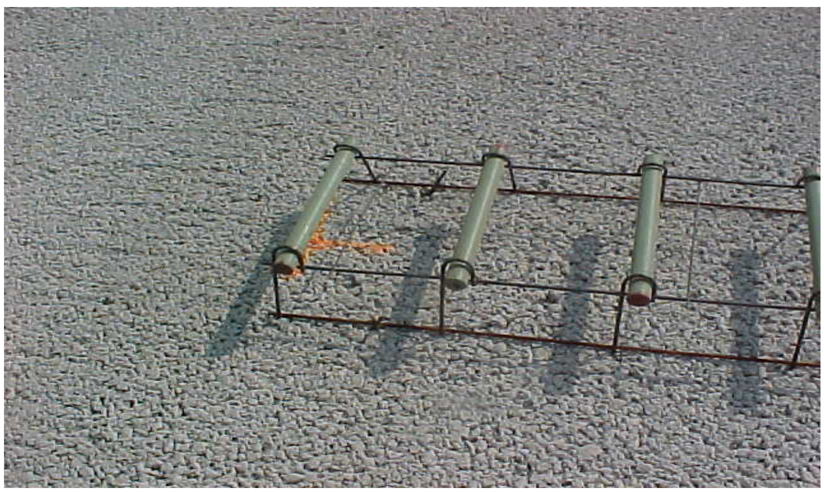


Figure 2.4: Short steel dowel bars

Dowel bars are short steel bars (as shown in Figure 2.4) that provide a mechanical connection between slabs without restricting horizontal joint movement. They increase load transfer efficiency by allowing the leave slab to assume some of the load before the load is actually over it. This reduces joint deflection and stress in the approach and leave slabs. Dowel bars are recommended for all medium and high traffic facilities (pavements that are thicker than 8 inch). Dowel bar diameter (commonly one-eighth of the slab thickness) typically varies from $\frac{5}{8}$ to $1\frac{1}{2}$ inches, with lengths varying from 10 to 20 inches. Bar spacing generally is 12 to 15 inches from center to center and are usually placed at mid-slab depth on baskets. Figure 2.5 shows the typical dowel layout.

Figure 2.5 Typical dowel layout

Dowels are not bonded to the concrete on one side, and freedom of movement is ensured by painting or lubricating one end of the dowel, by enclosing one end in a sleeve, or by other similar methods. It is essential that the freedom of movement be ensured in the design and placing of the dowel bars, since the purpose of the joint will be largely destroyed if the movement is prevented. Because the concrete is cracked at the joint, the dowels also provide vertical transfer of the load from one slab to the next. The dowels are either placed on baskets or implanted into the plastic concrete. Where a widened slab exists, some agencies may place a dowel bar in the outside widened area, depending on the width of the widening. Currently steel dowels are coated with a corrosion inhibitor to prevent corrosion and subsequent lock up of the dowels. Epoxy is the most commonly used corrosion inhibitor.

In Kansas, joints are doweled with a dowel spacing of 1 foot. Dowel bars in Kansas are smooth, rounded steel bars with a diameter one-eighth of the slab thickness in inches. The length of the dowel bar is 18 inch. The dowels are coated with a bond breaker to ensure the joint is a working joint, i.e., the dowels permit the concrete to expand and contract freely.

Tie bars (Figure 2.6) are either deformed steel bars or connectors used to hold the faces of abutting slabs in contact (AASHTO 1993). Although they may provide some minimal amount of load transfer, they are not designed to act as load transfer devices and should not be used as such (AASHTO 1993). Tie bars are typically used at longitudinal joints or between an edge joint and a curb or shoulder. Kansas uses steel tie bars that are typically #5 deformed bars placed perpendicular to the pavement's centerline. They are used to tie adjacent lanes or shoulders to the slab. Tie bars are generally spaced 2 feet apart and are 30 inches long.

Figure 2.6: Deformed tie bars

2.1.9 PCCP Construction

Pre-paving Activities

The main construction activities that precede the actual paving of the concrete slab are subgrade preparation, base preparation, and joint layout.

Subgrade Preparation includes any needed mixing of coarser material or stabilizer, grading and compaction to the required density, accurate trimming to establish grade and setting grade stakes for later base/ slab paving (NHI 2001). Both the subgrade and the granular or treated base/subbase are required to be brought to the grade lines according to the designated plan (KDOT 1990). The entire subgrade and granular or treated subbase should be thoroughly compacted. Before placing any surfacing material on any section, the ditches and drains along that section should be completed to drain the highway effectively.

The base course is placed on the finished roadbed, and is often used as a haul road to facilitate construction. For slipform slab paving, a minimum base width of 2 feet on each side beyond the traffic lane width is recommended to accommodate the slipform tracks.

Prior to paving, all transverse and longitudinal joints must be laid out in conformance with details and positions shown on the plan. Tie bars for longitudinal joints and dowel bars for transverse joints are placed on the base in chairs, if not inserted during paving. Epoxy coated dowels are placed with care and sometimes, a metal cap is fitted on one end to allow for expansion of the concrete. Dowels are placed in baskets to control their depth, spacing, and alignment. The baskets are pinned down so that they will not be shoved out of position during concrete placement. Accurate and horizontal alignments of the dowels are essential to the correct functioning of the joints (NHI 2001).

Paving Activities

Paving activities include mixing and transporting concrete to the job site, placing the concrete, and consolidating the concrete. The main goal in mixing and transporting concrete is to optimize workability and finishability while avoiding segregation. The concrete is spread, consolidated, screeded, and float finished in one pass of the paving train. Slipform pavers are used to perform the paving operation. Figure 2.7 shows a PCCP paving operation in Kansas. Since the paving concrete is stiff, it must be effectively consolidated to remove entrapped air and distribute the concrete uniformly around the dowels and reinforcement (NHI 2001). When concrete is placed in more than one layer or full depth, consolidation of each layer is done by vibrators. The concrete should be sufficiently and uniformly vibrated across the full width and depth of the pavement to ensure the density of the pavement concrete is not less than 98 percent of the rodded unit weight (KDOT 1990). This density requirement may be eliminated on such miscellaneous areas as entrance pavement, median pavement, gore areas, etc. PCC paving in Kansas is done with the slip form pavers.

Post-paving Activities

Post-paving activities include finishing, texturing, and curing the concrete, and after the concrete hardens, sawing and sealing the transverse and longitudinal joints. Figure 2.8 shows the sawing of joints. Joints are sawed immediately upon hardening of concrete.

Figure 2.7: PCCP paving operation in Kansas

Figure 2.8: Typical joint sawing operation in Kansas

Finishing consists of screeding off the concrete surface level to the desired height and machine floating the surface to fill in low spots. For thicker pavements, additional spreaders are employed. In Kansas, concrete consolidation is accomplished with the gang vibrators on the paver. Initial texturing provides microtexture, which contributes to surface friction by adhesion with the vehicle tires. Initial texturing is usually accomplished by a burlap or Astroturf drag directly behind the paver. Final texturing provides macrotexture, which contributes to the surface friction by tire deformation, and also channels surface water out from between the pavement and the tire (NHI 2001). Final texturing should be done as soon as possible as the bleed water sheen disappears.

Curing is done to enhance hydration and strength gain by retaining moisture and heat in the concrete immediately after placement and finishing (NHI 2001). Curing can be accomplished by wet burlap cover, liquid membrane-forming compounds, white polyethylene sheeting, concrete curing blanket or reinforced white polyethylene sheeting. In Kansas, liquid membrane-forming curing compound is extensively used for concrete pavements as shown in Figure 2.9.

Joint sawing (Figure 2.8) is done to establish transverse and longitudinal contraction joints and thereby control the cracking that inevitably occurs in a new concrete pavement as it dries (NHI 2001). With conventional equipment, sawcuts are made to a depth of one fourth to one third of the slab thickness, and 1/8 to 3/8 inches wide. After the sealant reservoir is sawed, it must be cleaned by abrasives (i.e., sandblasting) to remove the sawing residue so that the sealant will adhere well to the reservoir wall. After sandblasting, the reservoir is cleaned by air blowing, and the backer rod is installed. The sealant is then installed in the joint. Most of the newer JPCP's in Kansas are sealed with preformed neoprene seals.

Figure 2.9: Curing operation using liquid membrane forming curing compound

2.2 Framework for the Mechanistic-Empirical Design Method

2.2.1 Basic Design Concept

Mechanistic-empirical (M-E) design combines the elements of mechanical modeling and performance observations in determining required pavement thickness for a given set of design inputs. The mechanical model is based on elementary physics and determines pavement response to the wheel loads or environmental condition in terms of stress, strain, and displacement. The empirical part of the design uses the pavement response to predict the life of the pavement on the basis of actual field performance (Timm, Birgisson, and Newcomb 1998). Mechanistic-empirical procedures were not practical until the advent of high-speed computers. The reason is the computational demands associated with the differential equations and finite element matrix solutions employed by various analysis models. The choice of a model and how it was applied often were functions of the computational requirements and how much time was required to accomplish those computations (Proposal 2003).

2.2.2 Advantages over Empirical Design procedure

There are some specific advantages of M-E design over traditional empirical procedures. Those are outlined below:

- Consideration of changing load types;
- Better utilization and characterization of available materials;
- Improved performance predictions;
- Better definition of the role of construction by identifying parameters that influence pavement performance;
- Relationship of the material properties to actual pavement performance;
- Better definition of existing pavement layer properties; and
- Accommodation of environmental and aging effects of materials.

In essence, M-E design has the capability of changing and adapting to new developments in pavement design by relying primarily on the mechanics of materials. For example, M-E design can accurately examine the effect of new load configuration on a particular pavement. Empirical design, however, is limited to the observations on which the procedure was based (e.g., single axle load). Additionally, since the M-E design process is modular, new advances in pavement design may be incorporated without disrupting the overall procedure (Timm, Birgisson and Newcomb 1998).

2.2.3 Design Overview

The major components of the mechanistic-empirical JPCP pavement design are as follows (NHI 2002):

- Inputs—Materials, traffic, climate and structure.
- Structural response model – to compute critical responses.
- Performance models or transfer functions – to predict pavement performance over the design life.
- Performance criteria – to set objective goals by which the pavement performance will be judged.
- Design reliability and variability.

The inputs to the M-E design process include those related to the pavement structure, pavement materials, climactic conditions, season, soil conditions, and traffic loading. In an M-E design approach, the user has the complete flexibility over all design factors while designing a structure. For this reason, the M-E design approach is not just a thickness design procedure.

Structural response modeling was one of the weakest links in the M-E design process prior to the advent of modern computers and computational power. However, this situation has changed considerably due to the availability of numerous computer programs, capable of solving complex pavement problems. In the M-E design process, critical pavement responses for each distress type are estimated from the structural response models based on the loadings applied, pavement layer thicknesses and material properties. However, the accuracy of the responses will be a function of the underlying assumptions of each approach and the theoretical pavement model. Due to the finite nature of concrete pavement slabs and the presence of discontinuities in the form of transverse and longitudinal joints, elastic layer programs, which assume infinite extents in the horizontal direction, are not usually applicable for response calculation. Three methods have been traditionally used to determine stresses and deflections in concrete pavements: closed-form equations, influence charts, and finite element computer programs. Several finite element programs have also been developed over the years to perform rigid pavement analysis. They include ILLI-SLAB, JSLAB, WESLIQID, WESLAYER, RISC, and 3-D EVERFE. Today the use of finite element programs to analyze rigid pavements is fast becoming a norm due to the geometric complexities of such pavements (NHI 2002).

As pavement sections age and traffic and climatic loads act on them, they undergo functional and structural deterioration. This is manifested in terms of pavement distresses. The progression of pavement distress is directly tied to the pavement responses; therefore, in M-E design, the goal is to keep the critical stresses and strains in the pavement below acceptable limits. In this design process, the focus is on load-associated distresses because they can be controlled directly by changing the structural section to reduce critical pavement stresses and strains. Common load-related distresses for the JPC pavements are fatigue cracking, faulting, etc.

Distress transfer functions relate pavement responses determined from the structural response models to pavement performance as measured by the type and severity of distresses. Pavement responses are computed using structural response models and the pavement performance over time is predicted using transfer functions or distress models. In that sense, transfer functions are a vital component in the overall mechanistic-empirical design process. While several advancements have been made in developing accurate models to compute pavement structural responses, pavement transfer functions are a subject of continuous refinement. Transfer functions used in M-E design are developed relating a phenomenological distress progression function (i.e., a model based on a plausible theoretical correlation between a relevant mechanistic structural response and the distress parameter under question) to observed performance of actual pavement test sections through statistical calibration procedures. The calibration process introduces other relevant pavement variables of interest to the performance equation in addition to the primary mechanistic independent variable, such as, pavement structural properties and climatic variables. Not all pavement distress types can be included in the M-E design process because: (1) lack of a mechanistic basis for the distress under question (e.g., distresses caused by functional inadequacies or material-related failures that cannot be easily modeled); (2) lack of adequate observational data required to establish a clear statistical relationship between the dependent and the explanatory variables; and (3) inadequate statistical modeling. For rigid pavements, models to predict faulting, transverse cracking, and pumping exist (NHI 2002). Two types of transfer functions can be found in the literature:

- Functions that directly calculate the magnitude of the surface distress at any given time or traffic based on structural response parameters and other pertinent variables.
- Functions that first calculate a damage index based on structural responses and then use damage to distress correlations to assess the distress progression over time.

Pavement deterioration due to traffic and environmental factors (temperature cycles) is termed “damage.” Damage can be defined as an alteration of the physical properties of the pavement structure due to application of wheel loads. A refinement of the damage concept is the incremental damage accumulation. Pavements are loaded incrementally in the field, i.e., every hour a number of axle loads travel over the traffic lane and cause stress, strains, and deformations in the pavement and subgrade. Damage occurs in increments hourly, daily, monthly, and yearly. During these times, many of the key variables that affect pavement performance vary or change. The most obvious are climatic conditions (temperature and moisture) that vary daily and seasonally. Others would be differences in day and night time traffic loadings and seasonal traffic loadings, joint load transfer (for PCC pavements) varying over the day and seasonally (NHI 2002).

According to the incremental approach, damage is not accumulated equally over time. Damage accumulation is higher when critical structural and climatic factors that negatively influence pavement structural responses act in unison, and vice versa. An incremental damage accumulation process breaks up the design period into smaller time increments (e.g., months, seasons) and computes damage for each applied traffic category (truck class, traffic path, and so on) within each time increment. The number of load applications allowed within each time increment is typically referred to as N. Damage at any point in time, D, is defined as the ratio of the accumulated load applications, n, to the total allowable number based on the structural responses within that increment, N, or

$$D = n/N \qquad \text{Equation 2.1}$$

Miner's hypothesis (Miner 1945) is commonly used to sum damage over time in pavements. An example of the form of Miner's equation to compute fatigue-related damage in PCC pavements is presented below:

$$FD = \sum_{k=1}^{o} \sum_{j=1}^{m} \sum_{i=1}^{n} \frac{n_{ijk}}{N_{ijk}} \qquad \text{Equation 2.2}$$

Where,

FD= Fatigue Damage;

n = Number of applied 80-kN (18 kip) single axles;

N = Number of allowable 80-kN (18 kip) single axles; and

i, j, k = Categories over which damage will be summed.

Once damage over a given time increment is computed in this manner, pavement distress of interest can be determined using the damage-distress transfer function. An example of the general nature of the fatigue damage correlation to slab cracking is shown in the equation below:

$$PercentSlabsCracked = f(FD) \qquad \text{Equation 2.3}$$

The damage-distress correlation can then be converted to distress-time or distress-traffic correlation. Figure 2.10 illustrates the typical scheme adopted in the incremental damage approach to predict distresses over time or traffic.

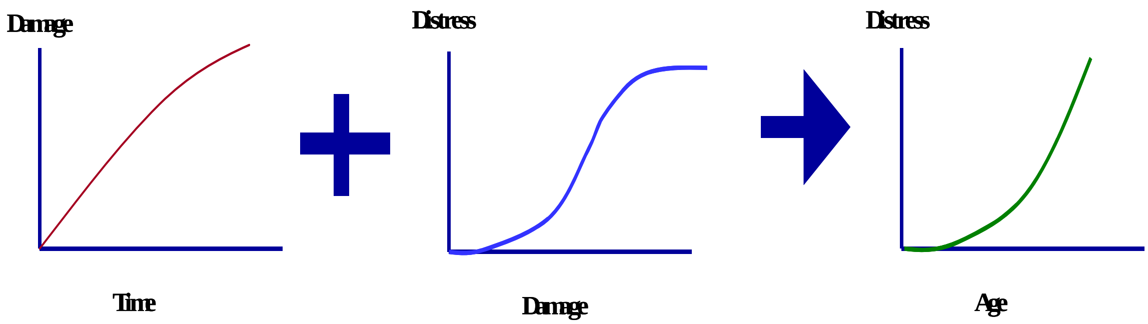


Figure 2.10: Prediction of distress over time

Using pavement structural distresses as measures of performance is unique to the M-E design. This allows performance to be expressed in objective and measurable quantities that engineers can relate to.

Although the M-E approach will require the prediction of specific distress types such as fatigue cracking and faulting, it is still highly desirable to continue with the original serviceability criterion developed at the AASHO Road Test. This will assure the traveling public of a safe and comfortable ride during the pavement’s design life. However, since Present Serviceability Rating (PSR) is not a parameter directly measured in the field, it is necessary to use another parameter that is routinely measured and that correlates well with this index value. It has been found from numerous research studies conducted since the AASHO Road Test that pavement smoothness, measured in terms of the International Roughness Index (IRI), correlates extremely well with PSR. Pavement smoothness information is routinely collected in the field and is a viable alternative for measuring the functional quality of pavements. It is an objective indicator of the ride quality of the pavement, which is the most important parameter from a road user’s perspective. However, there are no models to predict pavement profile over time based on the M-E concepts. The present technology uses correlations between the pavement distresses, smoothness immediately after construction (initial IRI), and other related parameters to predict smoothness. Since pavement distresses are predicted using mechanistic methods, and they form inputs to the IRI equation, it can be argued that the prediction of IRI is quasi-mechanistic at this point (NHI 2002).

Reliability of a given design is the probability that the performance of the pavement predicted for that design will be satisfactory over the time period under consideration. Reliability analysis is a requirement in pavement design due to the stochastic nature of the inputs to the design as well as the predicted outputs from the design (e.g., pavement distress or smoothness).

Before a given design is accepted, the probabilistically predicted pavement distresses and smoothness for that design are checked against a set of failure criteria to verify its adequacy. These criteria are preset by agencies based on their maintenance and rehabilitation policies. In M-E design, a number of failure criteria, each directed to a specific distress type, must be established. This is in contrast to the current AASHTO method where the Present Serviceability Index (PSI), which indicates the general pavement condition, is used. In addition to setting failure thresholds for each distress type, a threshold value for IRI is also important because it is quite possible that a pavement exhibiting low amounts of structural distresses can have an unacceptable ride quality. The following are the example of failure criteria:

Fatigue cracking	– Maximum percent of cracked slabs.
Faulting	– Maximum amount of mean joint faulting/km.
Smoothness, IRI	– Maximum IRI, m/km.

The designer may choose to check for all the distress types and smoothness, or any possible combinations thereof.

2.3 NCHRP Mechanistic-Empirical Pavement Design Guide (MEPDG)

Yoder and Witczak (1975) pointed out that for any pavement design procedure to be completely rational in nature, three elements must be fully considered: (i) the theory used to predict the assumed failure or distress parameter; (ii) the evaluation of the materials properties applicable to the selected theory; and (iii) the determination of the relationship between the magnitude of the parameter in question to the performance level desired. The NCHRP MEPDG considered all three elements.

2.3.1 Design Approach

The design approach followed in MEPDG is summarized in Figure 2.11. The format provides a framework for future continuous improvement to keep up with the changes in truck traffic technology, materials, construction, design concepts, computers, and so on. As shown in the figure, in this guide, the designer first considers site conditions (traffic, climate, material and existing pavement condition, in case of rehabilitation) and construction conditions in proposing a trial design for a new pavement or rehabilitation. The trial design is then evaluated for adequacy against some predetermined failure criteria. Key distresses and smoothness are predicted from the computed structural responses of stress, strain and deflection due to given traffic and environmental loads, such as temperature gradient across the PCC slab. If the design does not meet desired performance criteria at a preselected level of reliability, it is revised and the evaluation process is repeated as necessary (NCHRP 2004). This approach makes it possible to optimize the design and to more fully insure that specific distress types will not develop.

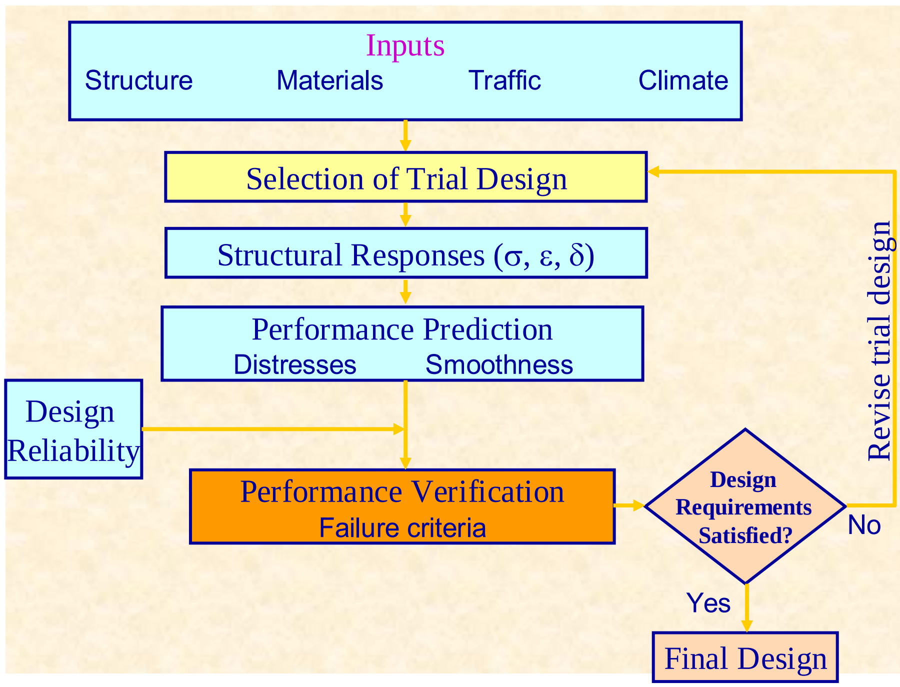


Figure 2.11: PCC mechanistic-empirical design framework (NCHRP 2004)

2.3.2 Overview of the Design Process for JPCP

The overall JPCP design process is illustrated in Figure 2.12. In the first step, a trial design is assembled for specific site conditions including traffic, climate, and foundation. Foundation includes different layer arrangement, PCC and other paving material properties and design and construction feature inputs are also needed. Then failure criteria are established based on the acceptable pavement performance at the end of the design period (i.e., acceptable levels of faulting, cracking and IRI for JPCP). Reliability levels are also selected for each of these performance indicators. Then these inputs are processed and structural responses are computed using finite element-based rapid solution models for each axle type and load and for each damage-calculation increment throughout the design period. ISLAB 2000, an enhanced 2-D finite element program, was used to make millions of calculations involving typical JPCP pavements. Then neural network technology was incorporated in this Guide for structural response calculation based on these ISLAB 2000 results. Key distresses, faulting and cracking, and smoothness are predicted month by month throughout the design period using the calibrated mechanistic-empirical performance models, provided in MEPDG. Predicted smoothness is a function of the initial IRI, distresses that occur over time, and site factors at the end of each time increment. Expected performance of the trial design is evaluated at the given reliability level. If the design does not meet the established criteria, then it needs to be modified and therefore, iteration continues.

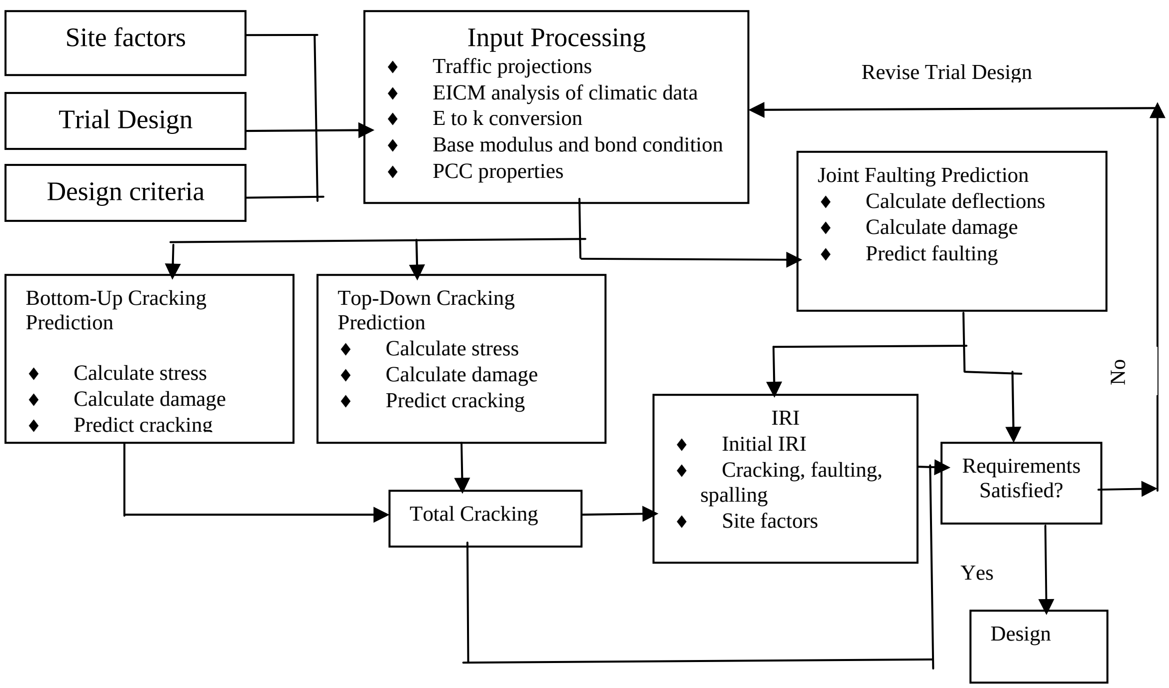


Figure 2.12: Overall design process for JPCP (NCHRP 2004)

2.3.3 Key JPCP Distresses and Critical Responses

The structural distresses considered for JPCP design are fatigue-related transverse cracking of the PCC slabs and differential deflection-related transverse joint faulting. *Transverse cracking* of PCC slabs can initiate either at the top surface of the PCC slab and propagate downward (top-down cracking) or vice versa (bottom-up cracking) depending on the loading and environmental conditions at the project site, as well as material properties, design features, and the conditions during construction (NHI 2002).

Bottom-up cracking is induced by fatigue that accumulates due to repeated loading from truck axles near the longitudinal edge of the slab midway between the transverse joints. This results in critical edge stresses at the bottom of the slab, as shown in Figure 2.13, increases greatly when there is a high positive temperature gradient across the slab. Repeated loadings from heavy axles result in fatigue damage along the edge of the slab that eventually results in micro-cracks that propagate to the slab surface and transversely across the slab.

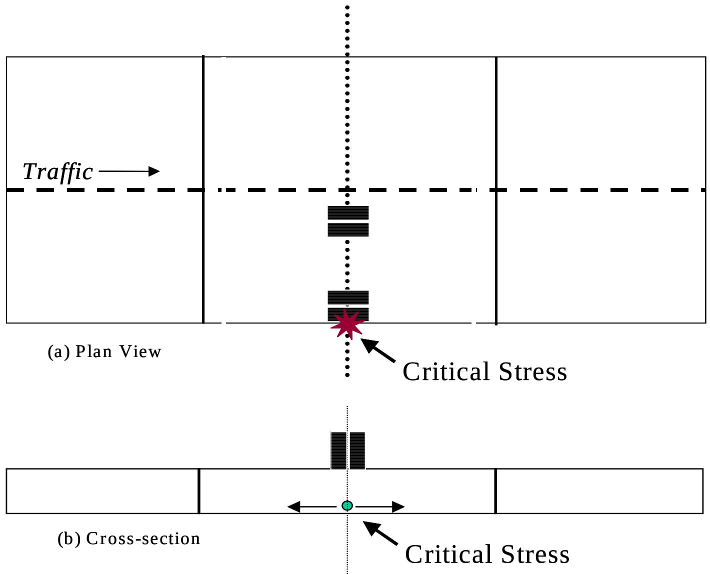


Figure 2.13: Critical loading and structural response location for JPCP bottom-up transverse cracking

When the truck steering axle is near the transverse joint and the drive axle is within 10 to 20 feet away and still on the same slab, a high tensile stress occurs at the top of the slab between axles, some distance from the joint as shown in Figure 2.14. This stress increases when there is a negative temperature gradient through the slab, a built-in negative gradient from construction, or significant drying shrinkage at the top of the slab (all of these conditions are common). Repeated loading of heavy axles results in fatigue damage at the top of the slab, which eventually results in micro-cracks that propagate downward through the slab and transversely or diagonally across the slab.

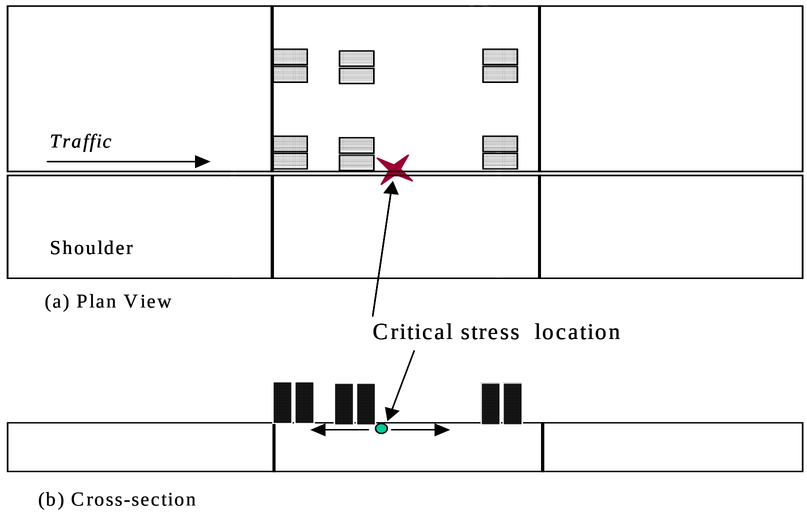


Figure 2.14: Critical loading and structural response location for JPCP top-down transverse cracking

Faulting is the difference of elevation across joints or cracks. Faulting is considered an important distress of JPCP because it affects ride quality. If significant joint faulting occurs, there will be a major impact on the life-cycle cost of the pavement in terms of rehabilitation and vehicle operating costs. Faulting is caused in part by a build-up of loose materials under the trailing slab near a joint or crack as well as the depression of the leading slab. Lack of load transfer contributes greatly to faulting (Huang 2003). Figure 2.15 shows the schematic of faulting.

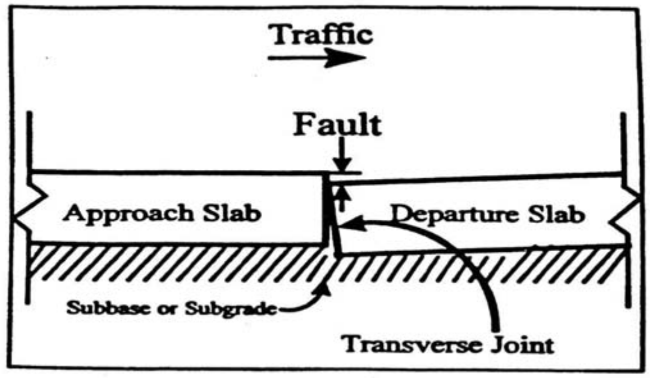


Figure 2.15: Schematic of faulting

Repeated heavy axle load crossing transverse joints create the potential for joint faulting. If there is less than 80 percent joint load transfer efficiency, an erodible base, subbase, shoulder, subgrade, or free moisture beneath the slab, then faulting can become severe and may cause loss of ride quality triggering early rehabilitation (NHI 2002). Figure 2.16 shows the critical loading and response location for faulting. The critical pavement response computed at this location is corner deflection.

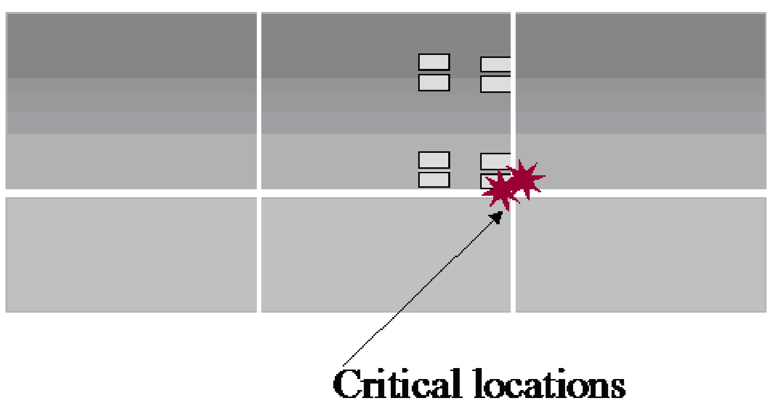


Figure 2.16: Critical loading and structural response location for JPCP faulting analysis

2.3.4 Smoothness International Roughness Index (IRI) prediction

The IRI over the design period depends upon the initial as-constructed IRI and the subsequent development of distresses over time. These distresses include transverse slab cracking, joint faulting, and joint spalling for JPCP. The calibrated model for JPCP relates IRI at any time to the as-constructed initial IRI and to the change in IRI due to occurrence of the previously described distresses. These models also include subgrade and climatic calibration factors. Finally IRI is estimated incrementally over the entire design period on a monthly basis.

2.3.5 JPCP Performance Prediction Models

2.3.5.1 Cracking Model

For JPCP transverse cracking, both bottom-up and top-down modes of cracking are considered (NCHRP 2004). The percentage of slabs with transverse cracks in a given traffic lane is used as the measure of transverse cracking and is predicted using the following model for both bottom-up and top-down cracking:

$$CRK = \frac{1}{1 + FD^{-1.68}} \qquad \text{Equation 2.4}$$

(R^2 = 0.68, N= 521 observations, and SEE=5.4 %)

Where,

CRK= Predicted amount of bottom-up or top-down cracking (fraction); and

FD= Calculated fatigue damage.

The general expression for fatigue damage is:

$$FD = \sum \frac{n_{i,j,k,l,m,n}}{N_{i,j,k,l,m,n}} \qquad \text{Equation 2.5}$$

Where, FD= Fatigue damage;

$n_{i,j,k,l...}$= Applied number of load application at condition i,j,k,l,m,n;

$N_{i,\,j,\,k.....}$= Allowable number of load applications at condition i, j, k, l, m, n;

i= age, j=month, k=axle type, l=load level, m= temperature difference, and n=traffic path.

The allowable number of load applications is determined using the following fatigue model:

$$\log(N_{i,j,k,l,m,n}) = C_1 \cdot \left(\frac{MR_i}{\sigma_{i,j,k,l,m,n}} \right) + 0.4371 \qquad \text{Equation 2.6}$$

Where, $N_{i,j,k,...}$= Allowable no. of load applications at condition i,j,k,l,m,n.

MR = PCC modulus of rupture at age *i*, psi;

$\sigma_{i,\,j,\,k....}$= Applied stress at condition *i, j, k, l ,m, n;*

C_1= Calibration constant=2.0; and

C_2=Calibration constant=1.22.

2.3.5.2 Faulting Model

The faulting at each month is determined as a sum of faulting increments from all previous months in the pavement life since the traffic opening using the following model:

$$Fault_m = \sum_{i}^{m} Fault_i \qquad \text{Equation 2.7}$$

$$\Delta Fault_i = C_{34} * (FAULTMAX_{i-1} - Fault_{i-1})^2 * DE_i \qquad \text{Equation 2.8}$$

$$FAULTMAX_i = FAULTMAX_0 + C_7 * \sum_{j=1}^{m} DE_j * \log(1 + C_5 * 5.0^{EROD})^{C_6} \qquad \text{Equation 2.9}$$

$$FAULTMAX_0 = C_{12} * \delta_{curling} * \left[\log(1 + C_5 * 5.0^{EROD}) * \log(\frac{P_{200} * WetDays}{P_s}) \right]^{C_6} \qquad \text{Equation 2.10}$$

The model statistics are: (R^2 =0.71, SEE=0.029, and N= 564)

Where,

$Fault_m$ =Mean joint faulting at the end of month m, in;

$\Delta Fault_i$ = Incremental change in mean transverse joint faulting in month i, in;

$FAULTMAX_i$ = Maximum mean transverse joint faulting for month i, in;

$FAULTMAX_0$ =Initial maximum mean transverse joint faulting, in;

EROD =Base/subbase erodibility factor;

DE_i =Differential deformation energy accumulated during month I;

$\delta_{curling}$ =Maximum mean monthly slab corner upward deflection PCC due to temperature curling and moisture warping;

P_S = Overburden on subgrade, lb;

P_{200} = Percent subgrade material passing US No.200 sieve;

WetDays = *Average* annual number of wet days (greater than 0.1 in rainfall);

FR = Base freezing index defined as the percentage of time the temperature of the base top is below freezing (32ºF) temperature

C_1 through C_8 and C_{12}, C_{34} are national calibration constants:

$C_{12} = C_1 + C_2 * FR^{0.25}$ and $C_{34} = C_3 + C_4 * FR^{0.25}$

C_1 =1.29, C_2 =1.1, C3 =0.001725, C4 = 0.0008, C5 =250, C_6 =0.4, and C_7 =1.2

2.3.5.3 Smoothness Model

Smoothness is the most important pavement characteristic valued by the highway users. In MEPDG, smoothness is defined by IRI. The IRI model was calibrated and validated using Long Term Pavement Performance (LTPP) and other field data. The following is the final calibrated model:

$$IRI = IRI_I + C1 * CRK + C2 * SPALL + C3 * TFAULT + C4 * SF \qquad \text{Equation 2.11}$$

The model statistics are: (R^2=0.60, SEE=27.3, and N= 183)

Where, *IRI*= Predicted IRI, in/mi;

IRI_I= Initial smoothness measured as IRI, in/mi;

CRK= Percent slabs with transverse cracks (all severities);

SPALL= Percentage of joints with spalling; and

TFAULT= Total joint faulting cumulated per mi, inch

SF= Site factor= $AGE\ (1+0.5556*FI)\ (1+P_{200})*10^{-6}$ Equation 2.12

Where, *AGE* = Pavement age, yr;

FI= Freezing index, ºF-days; and

P_{200}= Percent subgrade material passing No.200 sieve.

The constants evaluated in the calibration process are:

C1= 0.8203, *C2*= 0.4417, *C3*= 1.4929, and *C4*= 25.24

2.4 JPCP Evaluation and Management in Kansas

KDOT uses a comprehensive, successful pavement management system (PMS) for all pavement types in Kansas. The network level PMS of KDOT is popularly known as the Network Optimization System (NOS). In support of NOS, annual condition surveys are conducted based on the methodologies proposed by Woodward Clyde consultants (now URS Corp.) and subsequently, refined by the KDOT Pavement Management Section. Current annual condition surveys include roughness (IRI), faulting and joint distresses for rigid pavements. Different severity levels and extent are measured in the survey. While the roughness and faulting data are collected using automated methods, joint distress surveys are done manually. These survey results constitute basic inputs into the NOS system. The performance prediction methodology in the NOS system is based on the Markov process. The technique uses transition matrices to predict future condition based on current condition for multi-year programming (Kulkarni et al. 1983).

2.4.1 Profile/Roughness Data Collection

Pavement profile data consists of elevation measurements at discrete intervals along a pavement surface. Profile data is collected on both wheel paths (left and right) of driving lanes on the pavement sections using an International Cybernetics Corporation (ICC) South Dakota-type profiler (Figure 2.17) with a three-sensor configuration. The profiler is operated at highway speeds (usually 50 mph or 80 km/h). These sensors measure the vertical distance from the front bumper to the pavement surface, and the profiler is equipped with accelerometers at each of the wheel path sensors to compensate for the vertical motion of the vehicle body. The KDOT ICC profiler has three Selcom 220 laser sensors. The outer two sensors are spaced at about 66 inches apart. The third sensor is located in the middle. KDOT profiler aggregates profile elevation data at every 3 inches from the laser shots taken at the rate of 3200 per second (Miller et al. 2004).

A number of summary statistics are available to represent road roughness using road profile data. International Roughness Index (IRI) is most commonly used by many agencies because of its acceptance by FHWA.

Figure 2.17: KDOT South Dakota-type profilometer

The IRI is a profile-based statistic that was initially established in a study by the World Bank (Sayers 1985). It is used worldwide as the index for comparing pavement roughness. The IRI mathematically represents the response of a single tire on a vehicle suspension (quarter-car) to roughness in the pavement surface (Figure 2.18), traveling at 50 mph. The model, shown schematically in Figure 2.18, includes one tire, represented with a vertical spring, the mass of the axle supported by the tire, a suspension spring and a damper, and the mass of the body supported by the suspension for that tire. The quarter-car filter calculates the suspension deflection of a simulated mechanical system with response similar to a passenger car. The simulation suspension motion is accumulated and divided by the distance traveled to give an index with units of slope (Sayers 1985). IRI is expressed in m/km (inches/mile).

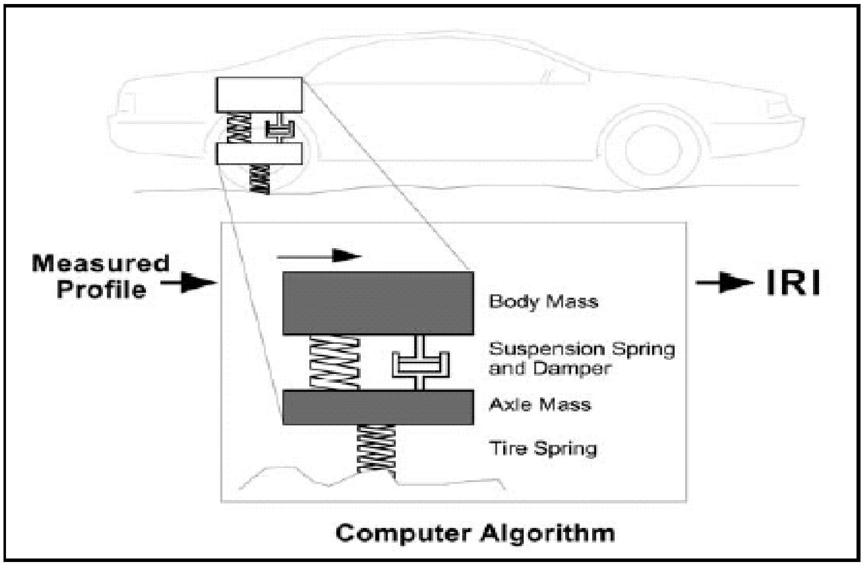


Figure 2.18: Quarter Car Simulation of road roughness (Sayers 1985)

2.4.2 Faulting

The fault values are calculated from the profile data using an algorithm developed by KDOT internally. In this process, anytime the absolute relative elevation difference between two points at 6 inch intervals for the right sensor from the output of profile elevation data processing software exceeds 0.09 inch, then the relative elevation difference (fault) values are algebraically summed until either three consecutive fault values are less than 0.09 inch or 3 feet has been traversed. The calculated fault value would be the algebraic sum of the points divided by two as illustrated in Figure 2.19. Once a fault has been detected, the next fault must be located at least 10 feet away. A 0.1-mile aggregation is used for the data analysis (Miller et al. 2004).

MP	Relative Elevation	
	Left Sensor/ Wheel Path	**Right Sensor/ Wheel Path**
1.41548	0.02	0.03
1.41543	-0.02	-0.03
1.41538	-0.03	0.04
1.41534	0.05	0.07
1.41529	0.08	0.15
1.41524	0.01	0.15
1.41519	-0.04	0.11
1.41515	-0.04	0.05
1.41510	-0.02	-0.02
1.41505	-0.02	-0.08
1.41501	-0.02	-0.10
1.41496	0.00	-0.08
1.41491	0.00	-0.06
1.41487	0.01	-0.04

$\sum = 0.36$

Fault =0.36/2=0.18"

$\sum = -0.28$

-0.28/2= -0.14"

Not 10' displacement

Figure 2.19: KDOT fault calculation algorithm

2.4.3 Joint Distress

For *Joint Distress*, three 100-foot randomly selected test sections are used to determine the expected condition for any 100-foot portion of the PMS pavement section under survey (usually one mile) (NOS 2004). The distress severity is done manually by comparing the condition observed with a set of photographs showing different severity levels. However, this distress is not compatible with the JPCP distresses predicted in the MEPDG analysis.

2.4.4 Definition of Pavement Condition

In Kansas, pavement condition is represented by the pavement performance level. This performance level is defined by Distress State and type of pavement. Distress State is the condition of the segment at the time of survey and is represented by a three-digit number. For rigid pavements, the first digit indicates roughness, the second digit indicates the joint distress and the third digit indicates faulting. Each digit thus represents a level of the pavement condition parameters, roughness, faulting, and joint distress. This level ranges from 1-3 with 1 being the best condition and 3 being the worst, resulting in a total of nine different distress states. Three performance levels (level 1, level 2 and level 3) are obtained by combining these nine distress states with the pavement type. Performance level 1 represents segments that appeared to require no corrective action at the time of the survey and is denoted "Good" or "Acceptable" condition. Performance level 2 represents segments that appeared to require at least routine maintenance at the time of survey and is denoted "Deteriorating" or "Tolerable" condition. Performance level 3 represents segments that appeared to require rehabilitative action beyond routine maintenance at the time of the survey and is denoted "Deteriorated" or "Unacceptable" condition (Vedula et al. 2004).

Roughness

The first component of the JPCP Distress State discussed earlier is roughness. Roughness is expressed in KDOT PMS in ranges of IRI as follows (KDOT 2004):

- "1" indicates an IRI value of less than 105 inches per mile.
- "2" indicates an IRI value of 105 to 164 inches per mile.
- "3" indicates an IRI value of more than 164 inches per mile.

Faulting

There are three faulting severity codes (KDOT 2004):

F1: >0.125" and <0.25"

F2: 0.25" to 0.5"

F3: >0.5"

With these codes a "Fault Score" is generated by:

Fault Score = [percentage of joints in a segment exhibiting F1 faulting] + 2* [percentage of joints in a segment exhibiting F2 faulting] + 4* [percentage of joints in a segment exhibiting F3 faulting]

Using the Fault Score, the Fault Code (F) is assigned as:

1: 4 <Fault Score <= 45 2: 45 <Fault Score <= 100 3: 100 <Fault Score

Then severity levels, F1, F2, and F3 in percentages are expressed as the weighted average percent of codes 1,2 and 3 faults per mile based on 352 joints per mile (15 foot joint spacing).

Joint Distress

The severity codes for joint distress are (KDOT 2004):

J1: Noticeable staining and/or minimal cracking at each joint.

J2: Staining and/or hairline cracking with minimum spalling.

J3: Significant cracking and spalling. Some patching done or necessary.

J4: Advanced cracking and severe spalling. Patching deteriorated, and 2 to 3 feet wide along joint.

Minimal cracking or spalling is defined as *less than* 2 *feet* along the joint length. *Significant* cracking or spalling is defined as *more than* 2 *feet* along the joint length.

More than one severity level may be coded per test section. Extent is the number of full width joints in each severity code (KDOT 2004).

2.5 Mechanistic-Empirical Pavement Design Guide (MEPDG) Software

The MEPDG software is the primary tool used for the design of new and rehabilitated pavement structures using MEPDG algorithm. The software provides an interface to the input design variables, computational engines for analysis and performance prediction, and results and outputs from the analyses in formats suitable for use in an electronic document, such as, Excel or for making hardcopies (NCHRP 2004).

MEPDG software is a user-friendly program. It has a tree-structured layout most suitable for novices as well as experienced users. The software (runs on Widows 98, 2000, NT, and XP) handles both U.S. customary and SI units. Figure 2.20 shows the opening screen for the NCHRP MEPDG software. In this study, all analysis was done from April 2005 to August 2005, using the on-line release of the MEPDG software.

2.5.1 MEPDG Software Layout

Figure 2.21 shows a typical layout of the program. To open a new project, select “New” from the “File” menu on the tool bar. The user first provides the software with the General Information of the project and inputs in three main categories, Traffic, Climate, and Structure. All inputs for the software program are color coded as shown in Figure 2.22. Input screens that have not been visited are coded “red”. Those that have default values are coded “yellow” and those that have complete inputs are coded “green”.

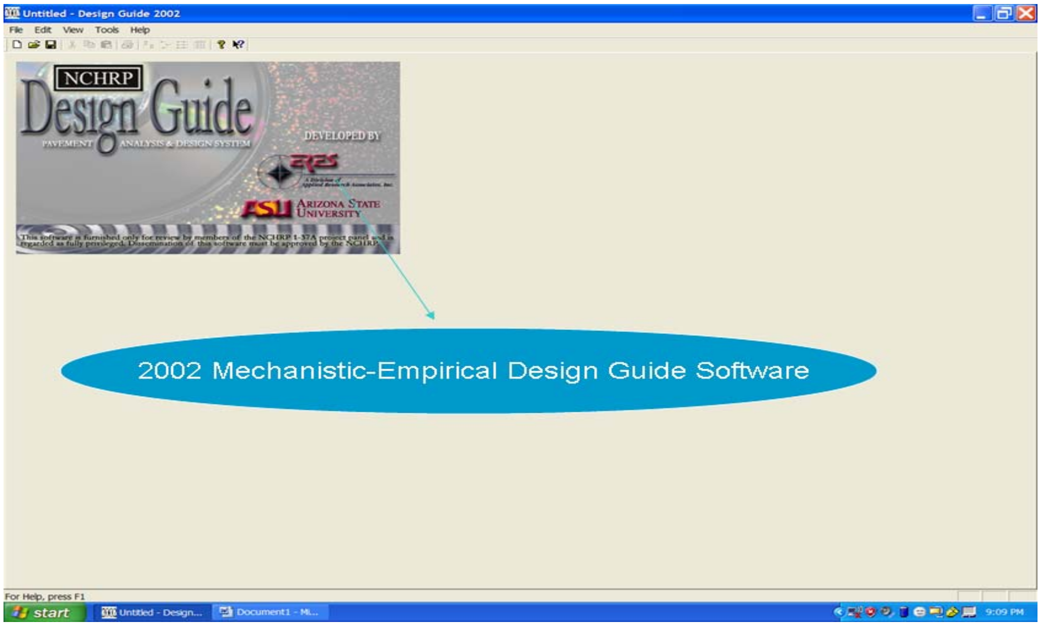


Figure 2.20: Opening screen for MEPDG software

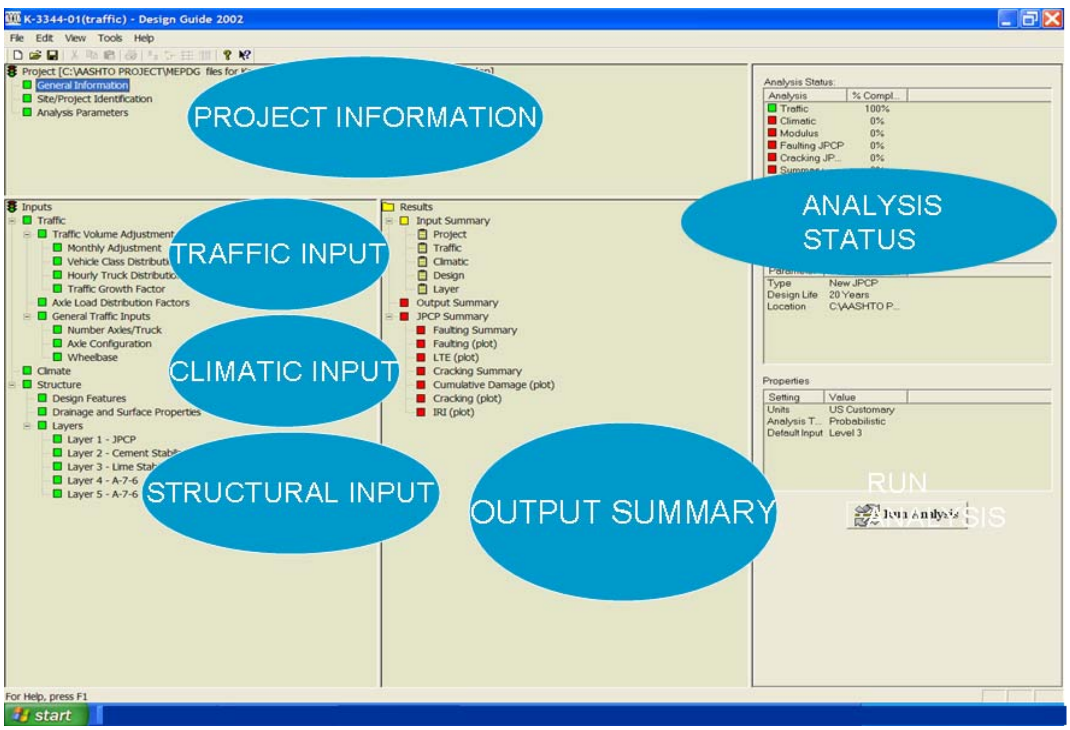


Figure 2.21: Program layout

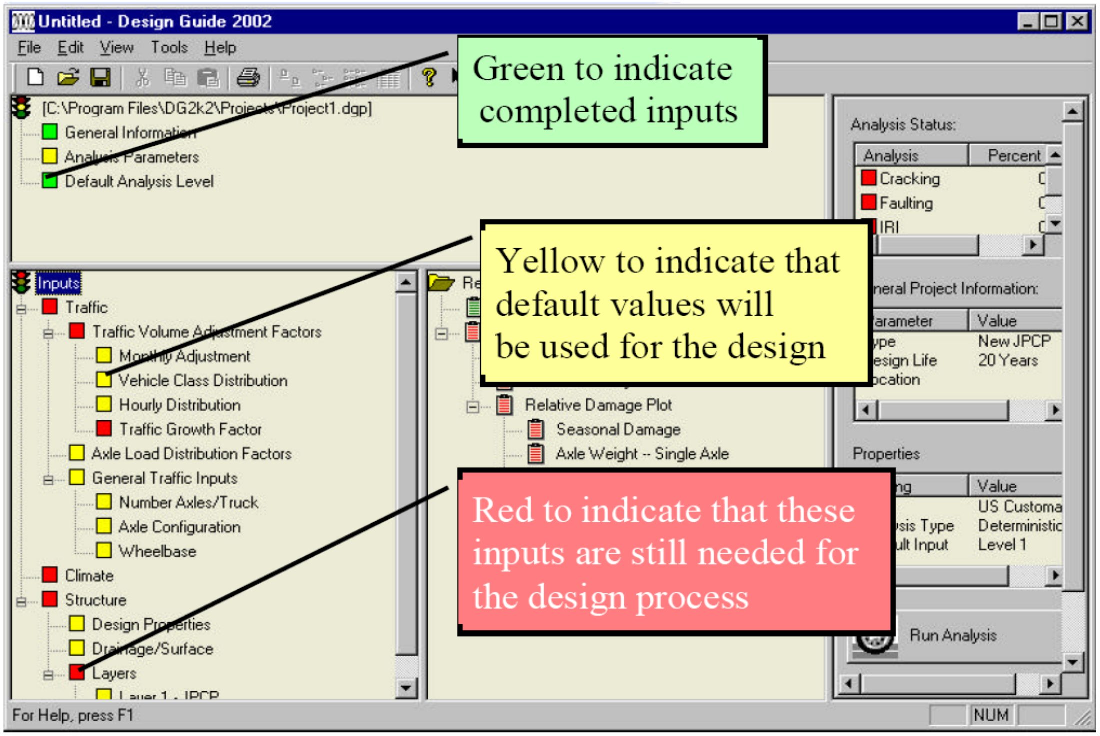


Figure 2.22: Color-coded inputs

Project related information such as, design life of the pavement, construction date, traffic opening month, site identification, mile-post limit and direction of traffic is input into general information. It also includes analysis parameters such as, initial IRI and the target distress limits with corresponding reliability levels.

Figures 2.23 through 2.27 show the screens for traffic data input. Traffic screen window allows the user to make general traffic volume inputs and provides a link to other traffic screens for Volume Adjustments, Axle Load Distribution Factors, and General Inputs. It requires some general information such as, initial two-way AADT, number of lanes in the design direction, percent of trucks in the design direction and on the design lane, operational speed and traffic growth factor, etc. Monthly adjustment factors, vehicle class distribution, hourly truck traffic distribution and load spectra are also required traffic inputs.

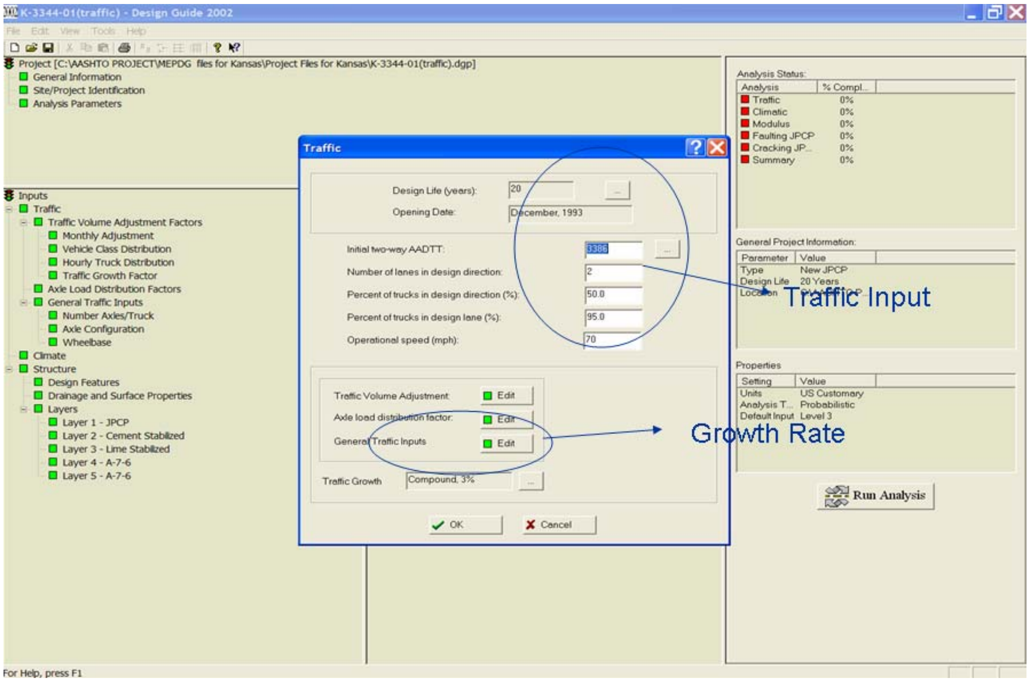


Figure 2.23: Traffic screen

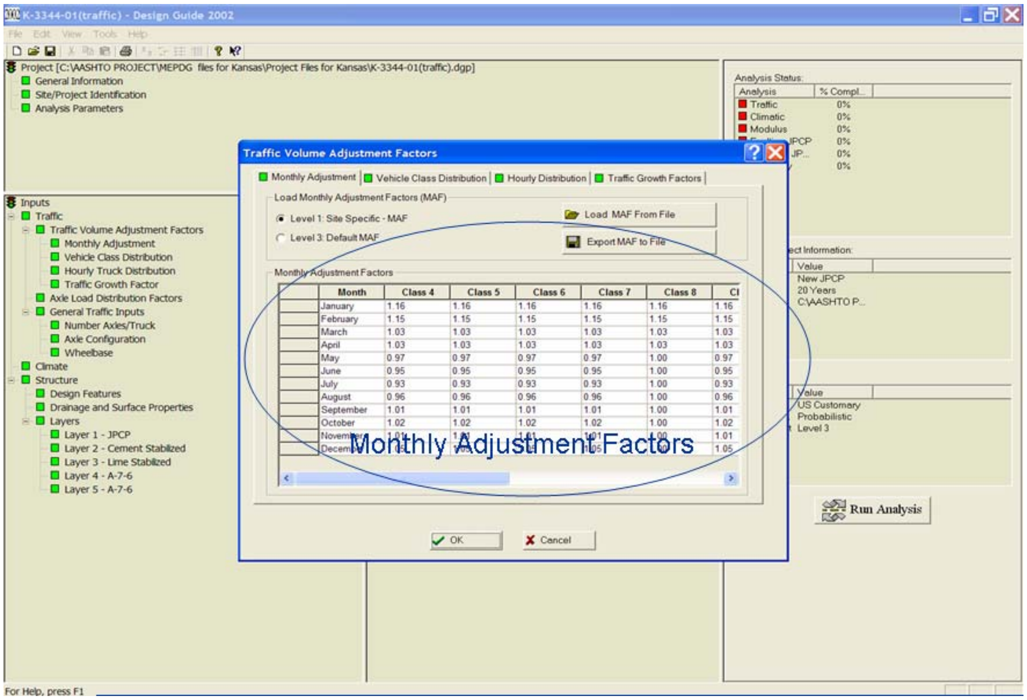


Figure 2.24: Monthly adjustment factors screen

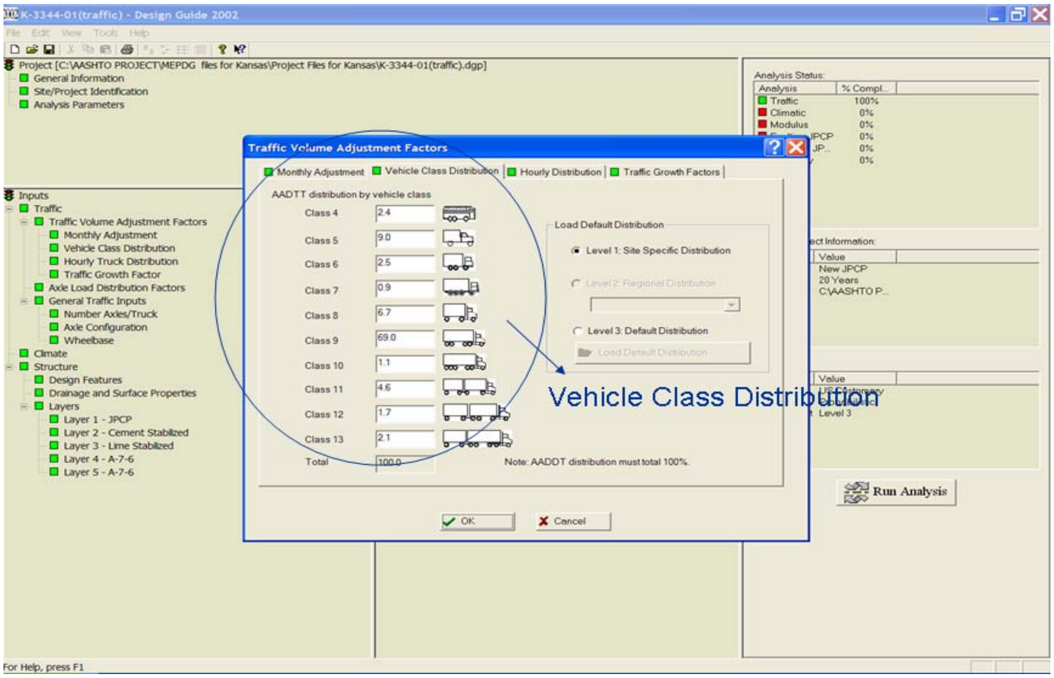


Figure 2.25: Vehicle class distribution screen

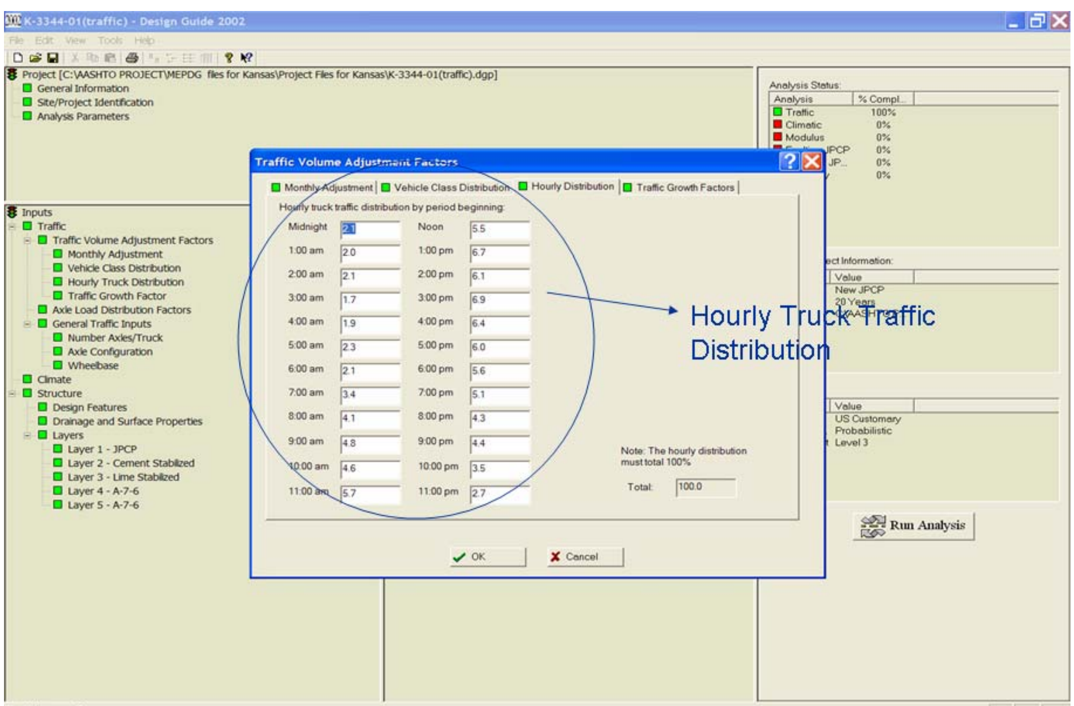


Figure 2.26: Hourly distribution screen

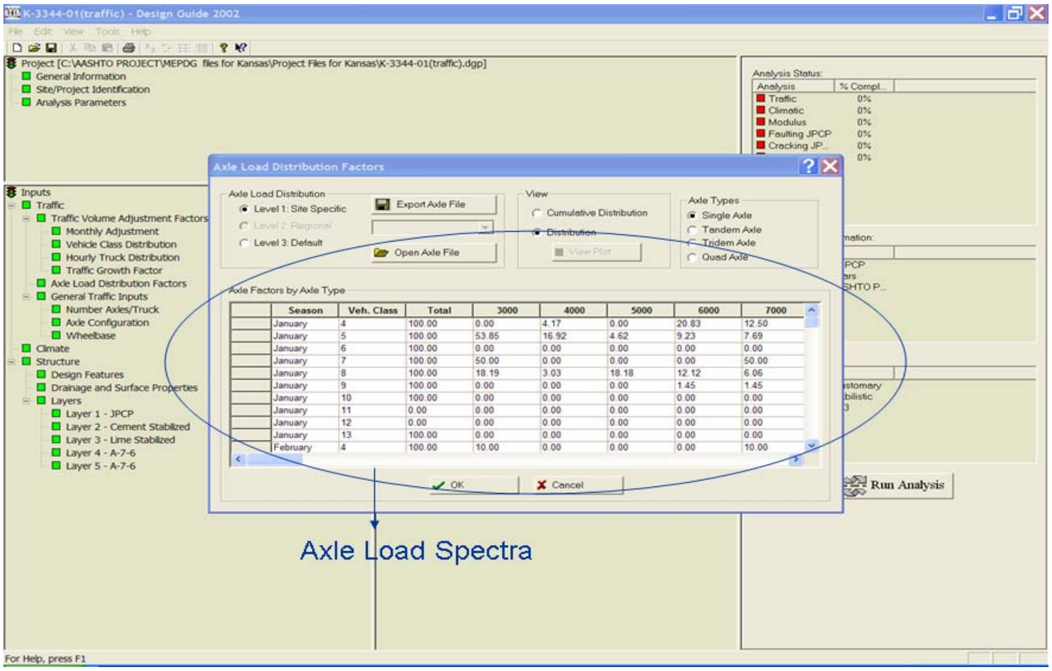


Figure 2.27: Axle load spectra screen

Figure 2.28 shows the climatic input screen. MEPDG recommends the weather inputs based on the pre-generated weather station, near the specific project site. The software includes a database of 800 weather stations throughout the United States. This database can be accessed by specifying the latitude, longitude, and elevation of the project location.

Figures 2.29 and 2.30 show the structural input screens. Figure 2.29 shows the project specific design features inputs such as, type of design, dowel/undoweled joint, joint spacing, dowel diameter, etc. Figure 2.30 shows the inputs related to the layers in the pavement structure. Inputs for the PCC layer include PCC thickness, unit weight, compressive-strength, Poisson ratio, modulus of rupture, cement content and type, etc. If the PCC structure has a base or subbase layer, layer properties such as, elastic modulus, Poisson ratio, etc are also required inputs. Subgrade layer inputs include soil gradation, resilient modulus, etc.

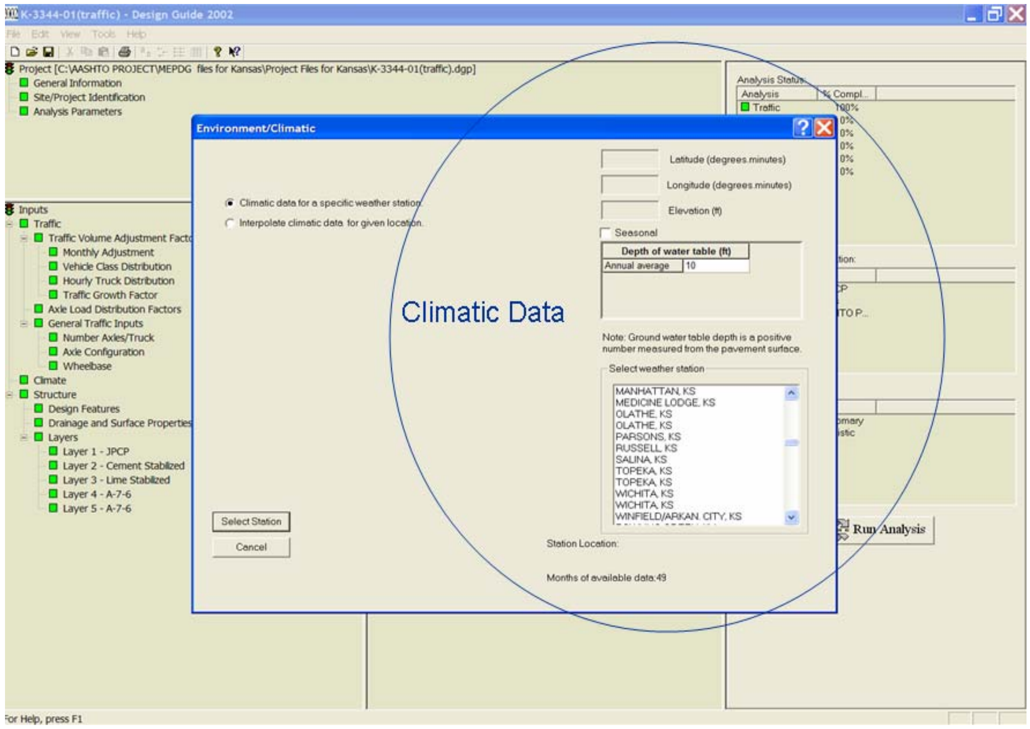


Figure 2.28: Climatic screen

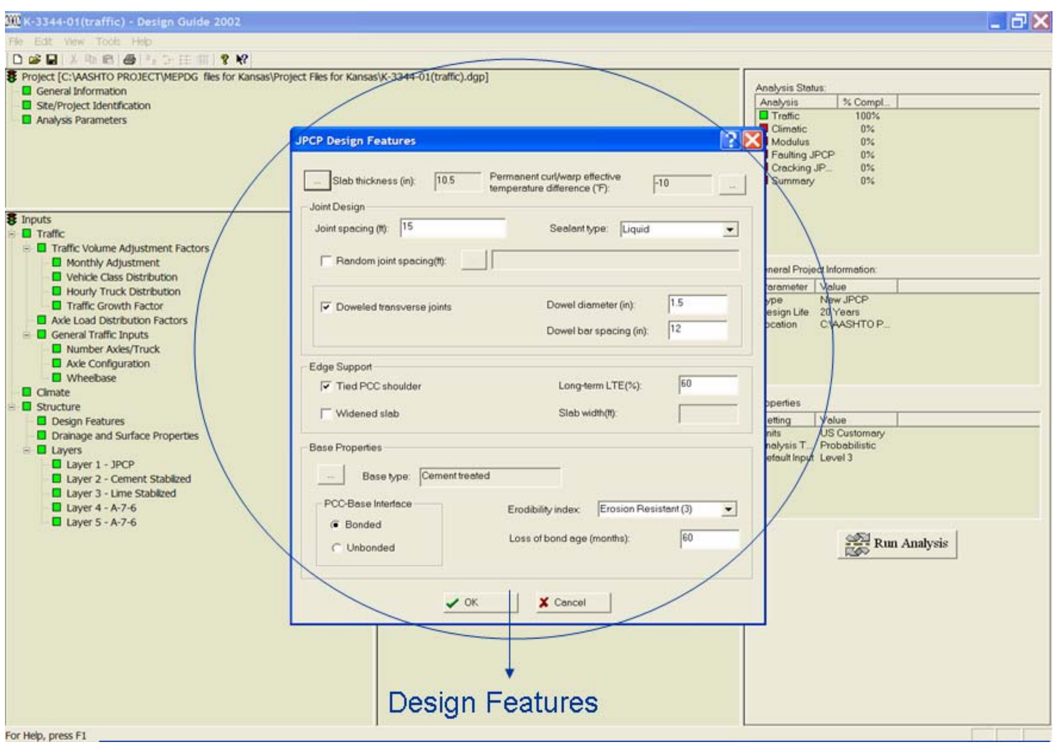


Figure 2.29: JPCP design features screen

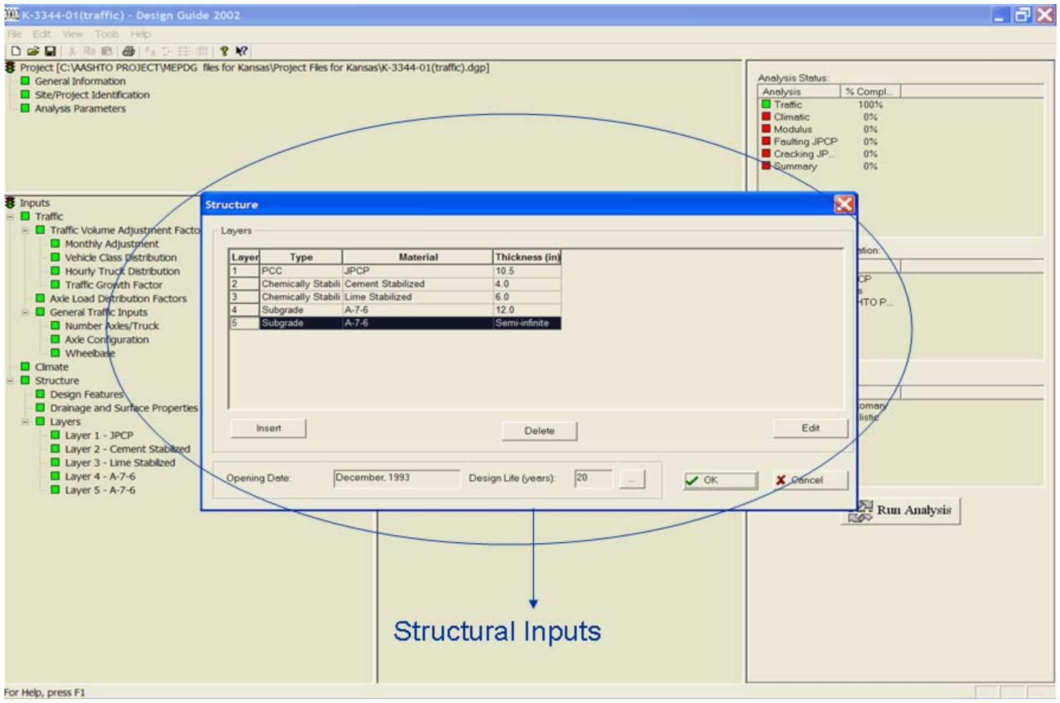


Figure 2.30: JPCP layers screen

After providing all design inputs and by clicking the *Run Analysis* button*,* the Design Guide software runs the analysis process to predict the performance of the trial design over the design life of the pavement. During running, the program reports the analysis status on the upper right hand corner of the screen. The software executes the damage analysis and the performance prediction engines for the trial design inputs. At the end of the analysis, the program creates a summary file and other output files in the project directory. The files are in the MS Excel format. The summary file contains an input summary sheet, reliability summary sheet, distress, faulting, and cracking summary sheets in a tabular format, and the predicted faulting, Load Transfer Efficiency (LTE), differential energy, cumulative damage, cracking, and IRI in a graphical format.

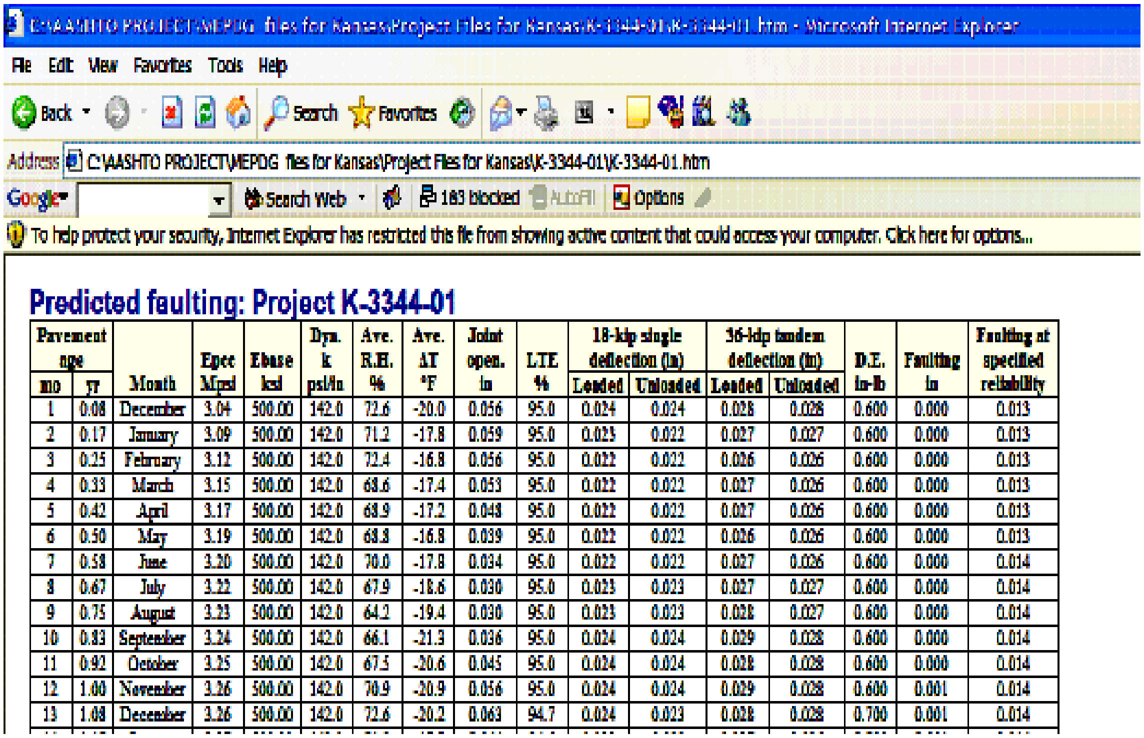
For a given trial design, IRI, transverse cracking, and faulting are predicted over the design period at a certain selected reliability level. Figures 2.31 through 2.33 show the output summary tables for the predicted distresses in JPCP. Figure 2.31 shows the distress summary table that includes the predicted IRI. It also predicts the PCC modulus, base modulus, the cumulative heavy trucks and IRI at the specified reliability level for every month of the design life.

As shown in Figure 2.32, the faulting summary table shows the k-value, relative humidity, joint opening, LTE, loaded/unloaded slab deflection, predicted faulting and faulting at specified reliability level, month-by-month and year-by-year. Figure 2.33 shows the cracking summary that includes the top-down and bottom-up cracking for different axle-type and traffic wheel load distribution and percent slabs cracked at the specified reliability for the whole design life of the pavement.

Predicted distress: Project K-3344-01

Pavement age		Month	Epcc Mpsi	Ebase ksi	Dyn. k psi/in	Faulting in	Percent slabs cracked	IRI in/mile	Heavy Trucks (cumulative)	IRI at specified reliability
mo	yr									
1	0.08	December	3.04	500.00	142.0	0.000	0.0	95.7	39759	132.0
2	0.17	January	3.08	500.00	142.0	0.000	0.0	95.9	79518	132.2
3	0.25	February	3.12	500.00	142.0	0.000	0.0	96.0	119277	132.3
4	0.33	March	3.15	500.00	142.0	0.000	0.0	96.0	159036	132.3
5	0.42	April	3.17	500.00	142.0	0.000	0.0	96.0	198795	132.4
6	0.50	May	3.19	500.00	142.0	0.000	0.0	96.1	238554	132.5
7	0.58	June	3.20	500.00	142.0	0.000	0.0	96.2	278313	132.5
8	0.67	July	3.22	500.00	142.0	0.000	0.0	96.3	318072	132.6
9	0.75	August	3.23	500.00	142.0	0.000	0.0	96.3	357831	132.6
10	0.83	September	3.24	500.00	142.0	0.000	0.0	96.3	397590	132.7
11	0.92	October	3.25	500.00	142.0	0.000	0.0	96.4	437349	132.8
12	1.00	November	3.26	500.00	142.0	0.001	0.0	96.4	477108	132.8
13	1.08	December	3.28	500.00	142.0	0.001	0.0	96.5	518060	132.9

Figure 2.31: Output summaries for IRI



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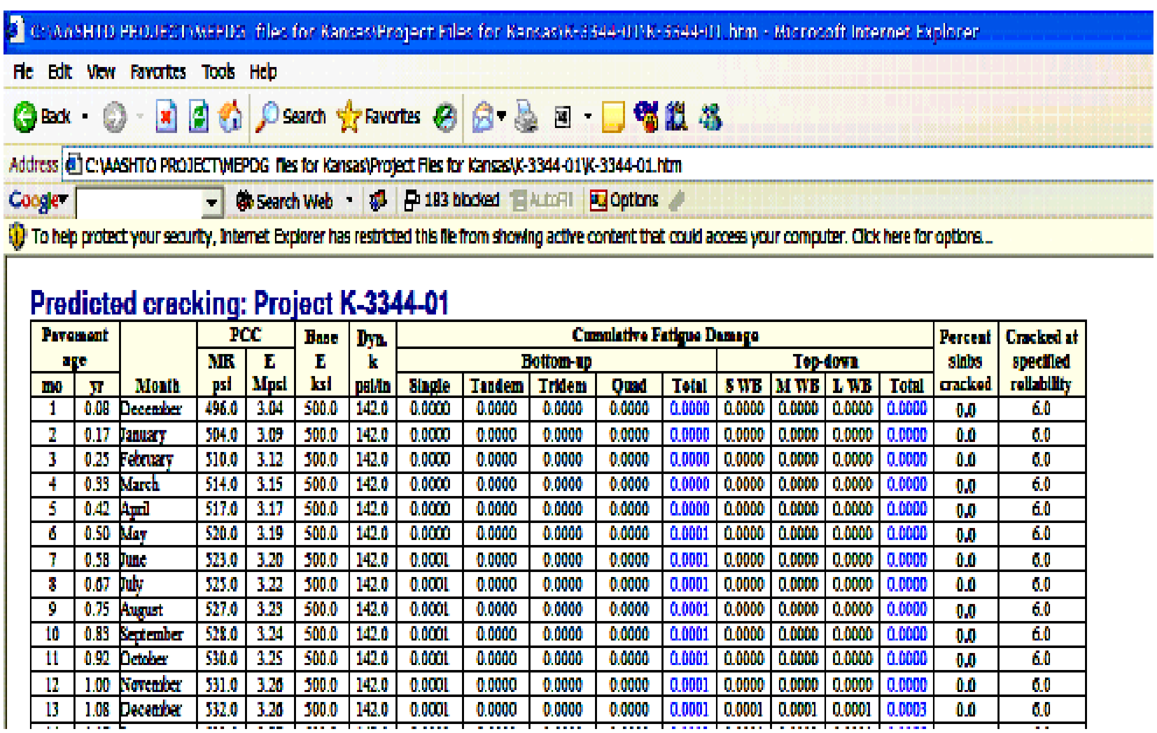
Predicted faulting: Project K-3344-01

Pavement age		Month	Epcc	Ebase	Dyn. k	Ave. R.H.	Ave. ΔT	Joint open.	LTE	18-kip single deflection (in)		36-kip tandem deflection (in)		D.E.	Faulting	Faulting at specified reliability
mo	yr		Mpsi	ksi	psi/in	%	°F	in	%	Loaded	Unloaded	Loaded	Unloaded	in-lb	in	
1	0.08	December	3.04	500.00	142.0	72.6	-20.0	0.056	95.0	0.024	0.024	0.028	0.028	0.600	0.000	0.013
2	0.17	January	3.09	500.00	142.0	71.2	-17.8	0.059	95.0	0.023	0.022	0.027	0.027	0.600	0.000	0.013
3	0.25	February	3.12	500.00	142.0	72.4	-16.8	0.056	95.0	0.022	0.022	0.026	0.026	0.600	0.000	0.013
4	0.33	March	3.15	500.00	142.0	68.6	-17.4	0.053	95.0	0.022	0.022	0.027	0.026	0.600	0.000	0.013
5	0.42	April	3.17	500.00	142.0	68.9	-17.2	0.048	95.0	0.022	0.022	0.027	0.026	0.600	0.000	0.013
6	0.50	May	3.19	500.00	142.0	68.8	-16.8	0.039	95.0	0.022	0.022	0.026	0.026	0.600	0.000	0.013
7	0.58	June	3.20	500.00	142.0	70.0	-17.8	0.034	95.0	0.022	0.022	0.027	0.026	0.600	0.000	0.014
8	0.67	July	3.22	500.00	142.0	67.9	-18.6	0.030	95.0	0.023	0.023	0.027	0.027	0.600	0.000	0.014
9	0.75	August	3.23	500.00	142.0	64.2	-19.4	0.030	95.0	0.023	0.023	0.028	0.027	0.600	0.000	0.014
10	0.83	September	3.24	500.00	142.0	66.1	-21.3	0.036	95.0	0.024	0.024	0.029	0.028	0.600	0.000	0.014
11	0.92	October	3.25	500.00	142.0	67.5	-20.6	0.045	95.0	0.024	0.024	0.028	0.028	0.600	0.000	0.014
12	1.00	November	3.26	500.00	142.0	70.9	-20.9	0.056	95.0	0.024	0.024	0.029	0.028	0.600	0.001	0.014
13	1.08	December	3.26	500.00	142.0	72.6	-20.2	0.063	94.7	0.024	0.023	0.028	0.028	0.700	0.001	0.014

Figure 2.32: Output summaries for faulting



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Predicted cracking: Project K-3344-01

Pavement age		Month	PCC		Base	Dyn.	Cumulative Fatigue Damage									Percent	Cracked at
			MR	E	E	k	Bottom-up					Top-down				slabs	specified
mo	yr		psi	Mpsi	ksi	psi/in	Single	Tandem	Tridem	Quad	Total	S WB	M WB	L WB	Total	cracked	reliability
1	0.08	December	496.0	3.04	500.0	142.0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0	6.0
2	0.17	January	504.0	3.09	500.0	142.0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0	6.0
3	0.25	February	510.0	3.12	500.0	142.0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0	6.0
4	0.33	March	514.0	3.15	500.0	142.0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0	6.0
5	0.42	April	517.0	3.17	500.0	142.0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0	6.0
6	0.50	May	520.0	3.19	500.0	142.0	0.0000	0.0000	0.0000	0.0000	0.0001	0.0000	0.0000	0.0000	0.0000	0.0	6.0
7	0.58	June	523.0	3.20	500.0	142.0	0.0001	0.0000	0.0000	0.0000	0.0001	0.0000	0.0000	0.0000	0.0000	0.0	6.0
8	0.67	July	525.0	3.22	500.0	142.0	0.0001	0.0000	0.0000	0.0000	0.0001	0.0000	0.0000	0.0000	0.0000	0.0	6.0
9	0.75	August	527.0	3.23	500.0	142.0	0.0001	0.0000	0.0000	0.0000	0.0001	0.0000	0.0000	0.0000	0.0000	0.0	6.0
10	0.83	September	528.0	3.24	500.0	142.0	0.0001	0.0000	0.0000	0.0000	0.0001	0.0000	0.0000	0.0000	0.0000	0.0	6.0
11	0.92	October	530.0	3.25	500.0	142.0	0.0001	0.0000	0.0000	0.0000	0.0001	0.0000	0.0000	0.0000	0.0000	0.0	6.0
12	1.00	November	531.0	3.26	500.0	142.0	0.0001	0.0000	0.0000	0.0000	0.0001	0.0000	0.0000	0.0000	0.0000	0.0	6.0
13	1.08	December	532.0	3.26	500.0	142.0	0.0001	0.0000	0.0000	0.0000	0.0001	0.0001	0.0001	0.0001	0.0003	0.0	6.0

Figure 2.33: Output summaries for cracking

2.6 Summary

The new Mechanistic Empirical Design Guide has significant influence on the performance of the JPCP. Deficiency in different design and construction features of JPCP contributes to the pavement smoothness (IRI), faulting and cracking. Identification of these distresses can help to figure out different alternative design strategies to compensate the distresses. Current AASHTO Design Guide (1993) is based on empirical performance equations for the design of Jointed Plain Concrete Pavements (JPCP). The Guide ignores a number of important design features. However, the newly released Mechanistic-Empirical Pavement Design Guide (MEPDG) provides methodologies for mechanistic-empirical pavement design while accounting for local materials, environmental conditions, and actual highway traffic load distribution by means of axle load spectra.

CHAPTER 3 - TEST SECTIONS AND DESIGN ANALYSIS INPUTS

This chapter describes the test sections selected and the material characteristics required for the mechanistic-empirical (M-E) design analysis described in MEPDG. It also outlines the Kansas-specific input data generation process.

3.1 Test Sections

Eight in-service JPCP projects were selected for the MEPDG design analysis. Three of these projects were the experimental sections chosen from the Kansas Specific Pavement Studies -2 (SPS-2) projects located on Interstate route 70. Two other projects are located on I-70, two on US-50 and one on route K-7. Table 3.1 tabulates the general features of these sections. The SPS-2 sections were each 500 feet long, and the rest are one to several miles long.

All sections have 15 foot joint spacing with dowelled joints. Three of the experimental sections have 1.5-in diameter steel dowels and other sections have dowel diameters varying from 1.125 in to 1.375 inches. All sections have 12 foot lanes with tied concrete shoulders except the SPS-2 Section 6. That section has a widened lane of 14 feet with tied PCC shoulders. The sections were constructed on a stabilized base and a treated subgrade. Base stabilization was done with Portland cement. Depending upon the cement content and gradation, the bases were designated as Portland cement-treated base (PCTB), bound drainable base (BDB) or lean concrete base (LCB). Base thickness ranged from 4 to 6 inches as shown in Table 3.1. The projects have primarily silty clay as subgrade. The top six inches of the natural subgrade were treated with lime or fly ash to reduce the plasticity and/or control the moisture during construction. PCC slab thickness, designed according to the 1986 or 1993 AASHTO Design Guide, ranged from 9 to 12 inches. The strength (modulus of rupture or compressive strength) values shown in Table 3.1 are the actual average values obtained during construction. The as-constructed International Roughness Index (IRI) values on these projects varied from 59 inches/mile to 122 inches/mile.

The annual average daily traffic (AADT) on these sections ranged from 2,080 for a US-50 Chase County project to 36,000 for the I-70 Shawnee County project. Very high percentage (45.5%) of truck traffic was observed on the US-50 projects and the lowest percentage (5%) was on the I-70 Shawnee County project.

Table 3.1: Project Features of the Study Sections

Project ID	Route	County	Year Built	Mile post Limit	Traffic Direc-tion	PCC Thickness (in)	PCC 28-day Strength (psi)	Initial IRI (in/mi)	Subgrade Soil Type
K-2611-01*	I-70	Geary	Nov 1990	0 – 7	WB	11	690	60	A-6
K-3344-01**	I-70	Shawnee	Oct 1993	9 – 10	WB	10.5	473	96	A-7-6
SPS-2 (Sec-5)†	I-70	Dickinson	July 1992	20 – 22.61	WB	11	945	122	A-6
SPS-2 (Sec-6)†	I-70	Dickinson	July 1992	20 –22.61	WB	11	617	98	A-6
SPS-2 (Control)*	I-70	Dickinson	July 1992	20 – 22.61	WB	12	647	95	A-6
K-3216-02***	US-50	Chase	Dec 1997	0 – 9	WB	10	5,569	59	A-7-6
K-3217-02***	US-50	Chase	Dec1997	9 – 19	WB	10	4,362	68	A-7-6
K-3382-01**	K-7	Johnson	Sep 1995	12 – 15	SB	9	537	81	A-7-6

*6” Portland Cement-Treated Base (PCTB)

** 4” Portland Cement-Treated Base (PCTB)

*** 4” Bound Drainable Base (BDB)

† 6” Lean Concrete Base (LCB)

3.2 Inputs

One of the most important aspects of mechanistic-empirical pavement design is the set of inputs required to perform the analysis and design. These inputs define the conditions under which a particular pavement structure is designed to perform.

As the desired project-specific information is not generally available at the design stage and mostly estimated several years in advance of construction, difficulty arises in obtaining adequate design inputs. Therefore, the designer needs to obtain as much data as possible on material properties, traffic, and other inputs for use in design to ensure as realistic data as possible.

The primary design inputs required for MEPDG are listed below:

Designer/user selected inputs for pavement structures, such as layer thickness, material types, joint spacing, etc.

Design inputs under which a pavement structure is designed to perform, such as, traffic, subgrade/foundation, and environment/climate.

Input for the material / mix design properties of the layers.

3.3 Hierarchical Design Inputs

The hierarchical approach is used for the design inputs in MEPDG. This approach provides the designer with several levels of "design efficacy" that can be related to the class of highway under consideration or to the level of reliability of design desired. The hierarchical approach is primarily employed for traffic, materials, and environmental inputs (NCHRP 2004). In general, three levels of inputs are provided:

Level 1 - Level 1 is a "first class" or advanced design procedure and provides for the highest achievable level of reliability and recommended for design in the heaviest traffic corridors or wherever there are dire safety or economic consequences of early failure. The design inputs also are of the highest achievable level and generally require site-specific data collection and/or testing. Example is the site-specific axle load spectra for traffic input.

Level 2 - Level 2 is the input level expected to be used in routine design. Level 2 inputs are typically user selected, possibly from an agency database. The data can be derived from a less than optimum testing program or can be estimated empirically. Estimated Portland cement concrete elastic modulus from the compressive strength test results is an example of Level 2 input in the material input data category.

Level 3 - Level 3 typically is the lowest class of design and should be used where there are minimal consequences of early failure. Inputs typically are user-selected default values or typical averages for the region. An example would be the default value for the Portland cement concrete coefficient of thermal expansion for a given mix class and aggregates used by an agency.

For a given design, it is permissible to mix different levels of input.

3.4 Development of Kansas-Specific MEPDG Inputs

The inputs required for the MEPDG software can be classified into four categories:

- General
- Traffic
- Climate
- Structural Inputs.

The following sections will summarize all inputs required for the design analysis of Jointed Plain Concrete Pavements (JPCP) using MEPDG and their relationships to the design process.

3.4.1 General

The general inputs consist of information required by the MEPDG software that describes the nature of the project, the timeline, the design criteria that the agency specifies, and miscellaneous information that can serve to identify the project files. The General project information can be entered in to the MEPDG software from three individual screens of General Information, Site/Project Identification, and Analysis Parameters.

3.4.1.1 General Information

This part allows the user to make broad choices about the design options. The MEPDG software considers two pavement types, “flexible” and “rigid. Rigid pavement design offers two alternatives for the surface layer, JPCP and CRCP. All pavement design projects can be classified under three main categories as New Design, Restoration, and Rehabilitation or Overlay.

The following inputs define the analysis period and type of design analysis for JPCP:

- Design Life
- Pavement Construction Month
- Traffic Opening Month
- Pavement Type

The *Design life* is the expected service life of the pavement in years. Pavement performance is predicted over the design life from the month the pavement is opened to traffic to the last month in design life.

Pavement construction month is the month when the surface (PCC) layer is placed. Due to time and environmental conditions, changes to the surface layer material properties are considered from the pavement construction month. This input is required to estimate the "zero-stress" temperature in the PCC slab at construction which affects the faulting in JPCP.

Traffic opening month and year are also required inputs for MEPDG to estimate the pavement opening date to traffic after construction. This parameter defines the climatic condition at that time that relates to the temperature gradients and the layer moduli, including that of the subgrade. This input also determines the PCC strength at which traffic load is applied to the pavement for damage calculation purposes.

Pavement type can be JPCP or CRCP. This input determines the method of design evaluations and the applicable performance models.

Project-specific "General" details are described in Table 3.1. All test sections selected in this study are JPCP with a design life of 20 years. All SPS-2 sections were constructed in July 1992 and opened to traffic in August. The I-70 Geary County project had the earliest construction year of 1990 and the Chase County projects were the latest, constructed in 1997.

3.4.1.2 Site Identification

This screen in the MEPDG software allows the user to provide information that is typically useful for identification and documentation purposes only. This information will not affect the analysis or design process but will help identify the location and stationing of the project. Typical inputs include Project location, Project ID, Section ID, begin and end mile posts, and traffic direction (EB, WB, SB, NB).

In this study, typical site identification information for the test sections is shown in Table 3.1. Five of the projects are Rural Principal Arterial Interstate, one is Principal Arterial others (Urban), and two are Rural Principal Arterial (others).

3.4.1.3 Analysis Parameters

This part includes the analysis type and the basic criteria for performance prediction. The project specific inputs for this section includes Initial IRI and performance criteria or failure criteria for verification of the trial design.

The Initial IRI defines the expected level of smoothness in the pavement soon after the completion of construction, expressed in terms of International Roughness Index (IRI). Typical value, suggested by MEPDG, ranges from 50 to 100 inches/mile. In this study, initial IRI for the test sections varied from 59 inches/mile to 122 inches/mile and are presented in Table 3.1.

Depending on the pavement type, the appropriate performance criteria need to be specified in the design analysis. Performance criteria form the basis for acceptance or rejection of a trial design being evaluated using MEPDG. In the MEPDG analysis, the key outputs are the individual distress quantities. For JPCP, MEPDG analysis predicts faulting, transverse cracking, and smoothness or IRI. Failure criteria are associated with certain design reliability for each distress type. Table 3.2 summarizes the MEPDG suggested reliability levels, based on the functional class of the roadway.

In this study, the projects were functionally classified as Rural Interstate or Principal Arterials (Urban and Rural). Therefore, a design reliability of 90% was used for all projects based on the MEPDG recommendations. The corresponding failure criteria chosen for IRI was 164 inches/mile, 0.12 inch for faulting, and 15% for slab cracking. The IRI level was based on the roughness level 2 of the KDOT pavement Management System, NOS.

Table 3.2: Recommended Design Reliability for MEPDG (NCHRP 2004)

Functional Classification	Recommended Reliability	
	Urban	Rural
Interstate/ Freeways	85-97	80-95
Principal Arterials	80-95	75-90
Collectors	75-85	70-80
Local	50-75	50-75

3.4.2 Traffic

Traffic data is a key element in the design/analysis of pavement structures. Traffic data is expressed in terms of equivalent single axle loads (ESALs) in the AASHTO Design Guides since 1972 and also in most other pavement design procedures. As MEPDG does design and performance analysis based on the principles of engineering mechanics, therefore it requires the estimation of axle loads that a pavement is expected to serve. (Milestones 2002)

3.4.2.1 MEPDG Hierarchical Traffic Inputs

MEPDG defines three broad levels of traffic data input. Regardless of the level, the same pavement analysis procedure is followed. As mentioned before, the full axle-load spectrum data are needed for MEPDG for new pavement and rehabilitation design analyses. MEPDG recognizes the fact that detailed traffic data over the years to accurately characterize future traffic for design may not be available (NCHRP 2004). Thus, to facilitate the use of MEPDG regardless of the level of detail of available traffic data, a hierarchical approach has been adopted for developing required traffic inputs. MEPDG outlines three broad levels of traffic data input (Levels 1 through 3) based on the amount of traffic data available:

Level 1 – There is a *very good* knowledge of past and future traffic characteristics. At this level, it is assumed that the past traffic volume and weight data have been collected along or near the roadway segment to be designed. Thus, the designer will have a high level of confidence in the accuracy of the truck traffic used in design. Thus, Level 1 requires the gathering and analysis of historical site-specific traffic volume and load data. Level 1 is considered the most accurate because it uses the actual axle weights and truck traffic volume distributions measured over or near the project site.

Level 2 – There is a *modest* knowledge of past and future traffic characteristics. At this level, only regional/state-wide truck volume and weights data may be available for the roadway in question. In this case, the designer will have the ability to predict with reasonable certainty the basic truck load pattern. Level 2 requires the designer to collect enough truck volume information at a site to measure truck volumes accurately. The data collection should take into account any weekday/weekend volume variation, and any significant seasonal trends in truck loads.

Level 3 – There is a *poor* knowledge of past and future traffic characteristics. At this level, the designer will have little truck volume information (for example, Average Annual Daily Traffic [AADT] and a truck percentage). In this case, a regional or state-wide or some other default load distribution must be used (NCHRP 2004).

State highway agencies often collect two major types of traffic data:

- ♦ Weigh-in-motion (WIM) data, which provide information about the number and configuration of axles observed within a series of load groups.
- ♦ Automatic vehicle classification (AVC) data for information about the number and types of vehicles, as shown in Figure 3.1, that uses a given section of roadway.

WIM data are a tabulation of the vehicle type and number, spacing and weight of axles for each vehicle weighed over a period of time. This information is used to determine normalized axle load distribution or spectra for each axle type within each truck class. This needs to be done external to the MEPDG software. AVC data are used to determine the normalized truck class distribution. This also needs to be done external to the MEPDG software. Classification is based on the specific location at which data are collected such as, site-specific, regional/statewide, or national. Another commonly collected traffic data item is vehicle count. It consists of counting the total number of vehicles over a period of time. Counts can be continuous, seasonal, or short duration.

MEPDG makes two major assumptions with regard to the truck and axle load and vehicle class distribution (NCHRP 2004):

1. The axle load distribution by axle type and vehicle class remains constant from year to year, whereas the vehicle class distribution can change from year to year.
2. The axle load distribution does not change throughout the day or over the week (weekday versus weekends, and night versus day). However, the vehicle class or truck distributions can change over the time of day or day of the week.

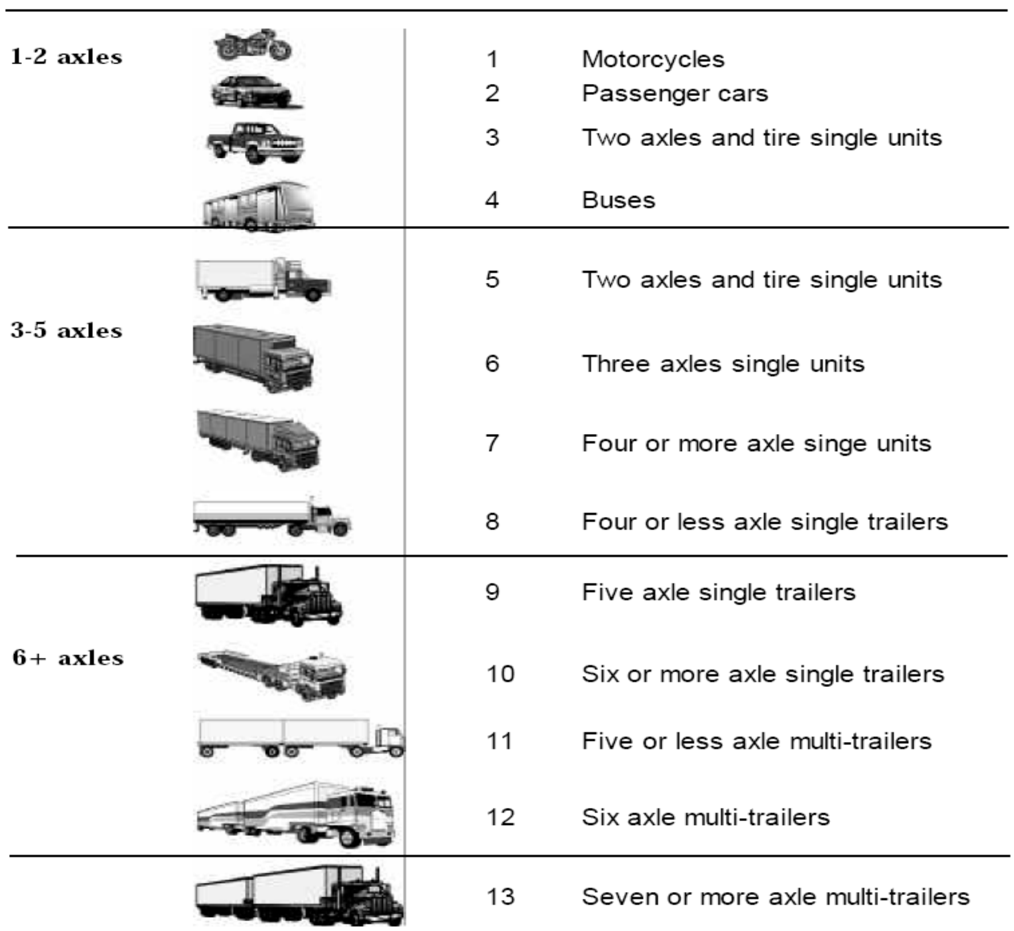


Figure 3.1: Illustration of the FHWA vehicle classes for MEPDG (Milestones 2002)

3.4.2.2 Kansas Traffic Monitoring System for Highways (TMS/H)

Kansas TMS/H provides traffic data for the Kansas Department of Transportation (KDOT) for project selection, traffic modeling, traffic forecast, transportation studies, pavement design, and air quality analysis (KDOT 2003). The TMS/H also provides traffic data to Federal Highway Administration (FHWA) for their reporting requirements. The TMS/H was developed following the concepts in FHWA's Traffic Monitoring Guide (TMG) published in February 1995 and May 2001. The 1995 TMG recommended a sample size of 300 vehicle classification sites and 90 truck weight sites which KDOT adopted. Currently, KDOT is adopting 2001 TMG recommendations of increasing number of short-term and continuous classification locations (KDOT 2003). The components of TMS/H that are important for developing traffic inputs for MEPDG are: (1) Continuous traffic counting, (2) Vehicle classification, and (3) Truck weight.

Continuous Traffic Counting

In Kansas, there are about 102 sites throughout the state on most roadway functional classes to do hourly traffic counts using permanent, roadway mounted equipment. About 12,000 24 or 48-hour counts (approximately 5,500 on the State Highway System and 6,500 off the State system) are collected each year for the short-term traffic count. Traffic counts on the State System are collected every two years except on the Interstates, ramps and freeways where data is collected annually. The permanent site data is used to develop temporal adjustment factors for the short-term traffic counts.

3.4.2.2.1 Vehicle Classification

Kansas uses FHWA's 13-category, Scheme F for vehicle classification. Classification surveys are done at 300 locations on a three-year cycle or 100 locations per year. Duration of regular survey is 48 hours. Most data are collected by machine, though some is done visually. Some locations cannot be accurately classified using portable classification equipment. Therefore, visual or manual counts are the only possible way to count vehicle types, though counts are limited to 16 hours (6 A.M. to 10 P.M.). At some locations, machine counts are satisfactorily implemented to supplement this limitation and obtain a full 24-hour count. Kansas is in the process of preparing continuous classification data and as of December 2003, there are five locations for collection of continuous classification data (KDOT 2003).

As of now, at very limited sites, Kansas collects information on the speed, class, time, axle weights and axle spacings for each commercial vehicle that crosses the equipment. Around eleven permanent and 73 portable sites provide truck weight data in the State.

A summary of the traffic data required for JPCP design is presented below along with the basic definitions of the variables and references to the default values.

3.4.2.3 Base Year Input

The base year for the traffic inputs is defined as the first year that the roadway segment under design is opened to traffic. The following pieces of base year information are required:

- Annual Average Daily Truck Traffic (AADTT) for the base year, which includes the total number of heavy vehicles (FHWA classes 4 to 13 as shown in Figure 3.1) in the traffic stream.
- Number of lanes in the design direction.
- Percent trucks in the design direction (directional distribution factor).
- Percent trucks in the design lane (lane distribution factor).
- Operational speed of the vehicles.

In this study, AADT ranged from 2,080 to 36,000 for the test sections. Truck percentages in the design direction varied from 47% (provided by LTPP for the three SPS-2 sections) to 50%. Ninety-five percent trucks were assigned in the design lane for the 4-lane divided highways based on the default level 3 inputs. Table 3.3 summarizes the base inputs.

Table 3.3: Base Year Input Summary

Project ID	Initial Two-way AADT	Truck Traffic (%)	No. of lane in Design Direction	% of Truck in Design Direction	Operational Speed (mph)	Linear Growth Rate (%)
K-2611-01	9,200	18	2	50	70	1.2
K-3344-01	36,000	5	2	50	70	3
SPS-2 (Sec-5)	11,970	22.3	2	47	70	3.5
SPS-2 (Sec-6)	11,970	22.3	2	47	70	3.5
SPS-2 (Control)	11,970	22.3	2	47	70	3.5
K-3216-02	2,080	45.5	1	50	70	2.0
K-3217-02	3,480	40.5	1	50	70	2.0
K-3382-01	13,825	7	2	50	60	6.7

3.4.2.4 Traffic Volume Adjustments

In order to characterize traffic, the following truck-traffic volume adjustment factors are required:

- Monthly adjustment factors
- Vehicle class distribution factors
- Hourly truck distribution factors
- traffic growth factors

Monthly adjustment factors

Truck traffic monthly adjustment factors (MAF) simply represent the proportion of the annual truck for a given truck class that occurs in a specific month. These values are the ratio of the monthly truck traffic to the AADTT.

$$MAF_i = \frac{AMDTT_I}{\sum_{i=1}^{12} AMDTT_i} * 12 \qquad \text{Equation 3.1}$$

Where, MAF_i = monthly adjustment factor for month i; and

$AMDTT_i$= average monthly daily traffic for month i.

The sum of MAFi for all months in a year must equal 12.

The truck monthly distribution factors are used to determine the monthly variation in truck traffic within the base year. Several other factors such as, adjacent land use, location of industries and roadway location (urban or rural) have the influence on MAFi.

In this study, monthly adjustment factors were extracted from the Traffic Monitoring Guide guidelines followed by KDOT (KDOT 2003). Weekly and monthly adjustment factors were developed based on the functional class of the roadways. Tables 3.4, 3.5 and 3.6 show KDOT-reported monthly and weekly adjustment factors for different functional classifications.

Table 3.4: Adjustment Factors Report for Rural Interstate Highways (KDOT 2003)

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.164	1.164	1.164	1.164	1.164	1.164	1.164	1.164	1.164	1.164
February	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152	1.152
March	1.034	1.034	1.034	1.034	1.034	1.034	1.034	1.034	1.034	1.034
April	1.034	1.034	1.034	1.034	1.034	1.034	1.034	1.034	1.034	1.034
May	0.973	0.973	0.973	0.973	0.973	0.973	0.973	0.973	0.973	0.973
June	0.948	0.948	0.948	0.948	0.948	0.948	0.948	0.948	0.948	0.948
July	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
August	0.956	0.956	0.956	0.956	0.956	0.956	0.956	0.956	0.956	0.956
September	1.012	1.012	1.012	1.012	1.012	1.012	1.012	1.012	1.012	1.012
October	1.022	1.022	1.022	1.022	1.022	1.022	1.022	1.022	1.022	1.022
November	1.011	1.011	1.011	1.011	1.011	1.011	1.011	1.011	1.011	1.011
December	1.048	1.048	1.048	1.048	1.048	1.048	1.048	1.048	1.048	1.048

Table 3.5: Adjustment Factors Report for Other Urban Roadways (KDOT 2003)

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.012	1.012	1.012	1.012	1.012	1.012	1.012	1.012	1.012	1.012
February	0.981	0.981	0.981	0.981	0.981	0.981	0.981	0.981	0.981	0.981
March	0.957	0.957	0.957	0.957	0.957	0.957	0.957	0.957	0.957	0.957
April	0.904	0.904	0.904	0.904	0.904	0.904	0.904	0.904	0.904	0.904
May	0.901	0.901	0.901	0.901	0.901	0.901	0.901	0.901	0.901	0.901
June	0.889	0.889	0.889	0.889	0.889	0.889	0.889	0.889	0.889	0.889
July	0.907	0.907	0.907	0.907	0.907	0.907	0.907	0.907	0.907	0.907
August	0.904	0.904	0.904	0.904	0.904	0.904	0.904	0.904	0.904	0.904
September	0.938	0.938	0.938	0.938	0.938	0.938	0.938	0.938	0.938	0.938
October	0.923	0.923	0.923	0.923	0.923	0.923	0.923	0.923	0.923	0.923
November	0.943	0.943	0.943	0.943	0.943	0.943	0.943	0.943	0.943	0.943
December	0.981	0.981	0.981	0.981	0.981	0.981	0.981	0.981	0.981	0.981

Table 3.6: Adjustment Factors Report for Other Rural Roadways (KDOT 2003)

Month	Class 4	Class 5	Class 6	Class 7	Class 8	Class 9	Class 10	Class 11	Class 12	Class 13
January	1.075	1.075	1.075	1.075	1.075	1.075	1.075	1.075	1.075	1.075
February	1.047	1.047	1.047	1.047	1.047	1.047	1.047	1.047	1.047	1.047
March	1.009	1.009	1.009	1.009	1.009	1.009	1.009	1.009	1.009	1.009
April	0.942	0.942	0.942	0.942	0.942	0.942	0.942	0.942	0.942	0.942
May	0.921	0.921	0.921	0.921	0.921	0.921	0.921	0.921	0.921	0.921
June	0.907	0.907	0.907	0.907	0.907	0.907	0.907	0.907	0.907	0.907
July	0.916	0.916	0.916	0.916	0.916	0.916	0.916	0.916	0.916	0.916
August	0.928	0.928	0.928	0.928	0.928	0.928	0.928	0.928	0.928	0.928
September	0.947	0.947	0.947	0.947	0.947	0.947	0.947	0.947	0.947	0.947
October	0.961	0.961	0.961	0.961	0.961	0.961	0.961	0.961	0.961	0.961
November	0.972	0.972	0.972	0.972	0.972	0.972	0.972	0.972	0.972	0.972
December	1.028	1.028	1.028	1.028	1.028	1.028	1.028	1.028	1.028	1.028

MEPDG default distribution assumes even distribution, i.e. an MAF of 1.0 for all months for all vehicle classes. Figure 3.2 presents a comparison of MEPDG default monthly adjustment factor distribution for all functional classes of highways with that for the Kansas Rural Interstate highways. It is evident from the figure and the above tables that although MEPDG default input shows no monthly variation of truck traffic within the year, Kansas input shows significant variation within the year. Tables 3.4, 3.5 and 3.6

illustrate that the distribution ratio is greater than 1.0 for the winter months. In general, traffic is heavier in Kansas during the winter and spring months (December through April) partly due to hauling of grains and crops from the elevators to different destinations. The same monthly distribution was used for all classes of vehicles for a particular functional class of roadways.

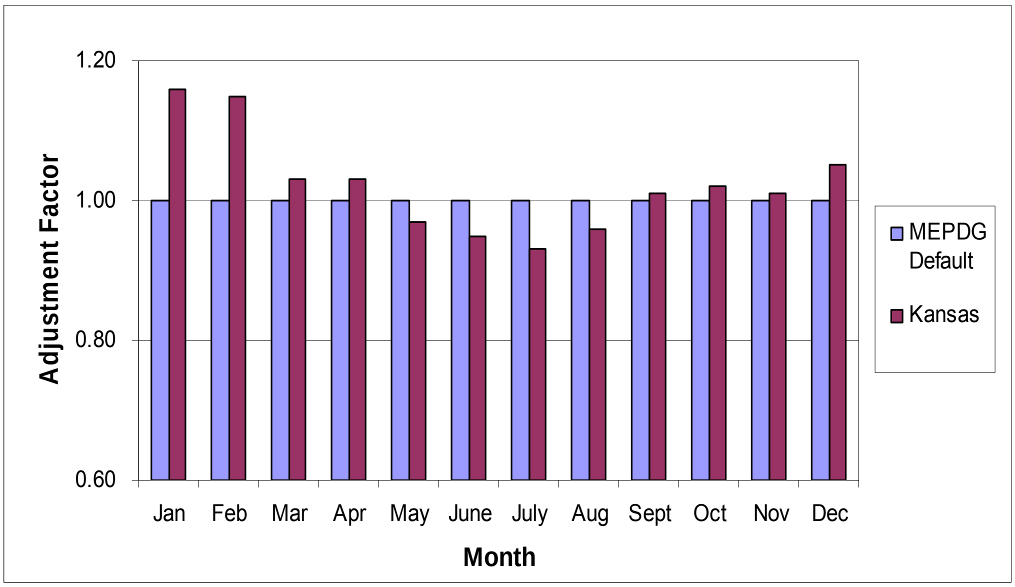


Figure 3.2: Comparison of monthly adjustment factor distribution for Kansas input and MEPDG default

3.4.2.4.1 Vehicle Class Distribution

Vehicle class distribution represents the percentage of each truck class (Classes 4 through 13) within the Annual Average Daily Truck Traffic (AADTT) for the base year. The base year for the traffic inputs is defined by MEPDG as the first year that the roadway segment under design is opened to traffic. The sum of the percent AADTT of all truck classes should be equal to 100. Usually WIM, AVC and vehicle count programs are used for the computation of vehicle class distribution. If the site-specific (Level 1) or regional data (Level 2) data are not available, truck traffic classification (TTC) can be used in conjunction with the functional class of the roadway to estimate the vehicle class distribution. MEPDG also has the option for default class distribution based on the roadway functional class and the best combination of Truck Traffic Classification (TTC) groups that describe the traffic stream expected on a given roadway. A typical comparison of the default vehicle class distribution for the Principal Arterials (urban) with the Kansas-generated input based on TMG is shown in Figure 3.3.

In this study, vehicle class distribution was generated based on the 2000-2003 Traffic Monitoring Guide (TMG) vehicle classification for regular sites as shown in Table 3.7. Project specific distribution was developed based on the functional class. Total counted vehicles from Classes 4 to 13 were added and a frequency distribution was developed so that total AADTT distribution by vehicle classes would be equal to 100%.

Table 3.7: Vehicle Class Distribution based on Functional Classification of Roadways

VEHICLE CLASS	Rural Interstate Highways	Principal Arterials Others (Urban)	Principal Arterials Others (Rural)	MEPDG Default(Interstate/ Principal Arterials)
4	2.4	6.6	2.6	1.3
5	9	42.7	17.6	8.5
6	2.5	9.8	5.6	2.8
7	0.9	2.3	1.6	0.3
8	6.7	9.4	7.9	7.6
9	69	25.2	58.2	74
10	1.1	1.4	2.1	1.2
11	4.6	1.1	2.7	3.4
12	1.7	0.4	0.6	0.6
13	2.1	1.1	1.1	1.3

The vehicle class distribution for the Rural Interstate Highway is similar to the MEPDG default distribution for that particular functional class. But the urban arterial is different than the default. The urban arterial shows a lower percentage of Class 9 trucks but a higher percentage of Class 5 trucks (delivery trucks). This trend seems to be very consistent with the location of this particular site (K-7, Johnson County).

For Rural Principal Arterial roadways, MEPDG default distribution for Class 9 vehicle is higher compared to the Kansas generated input for that particular class. Figure 3.3 shows the typical differences between MEPDG default inputs compared to the Kansas inputs for a particular functional class.

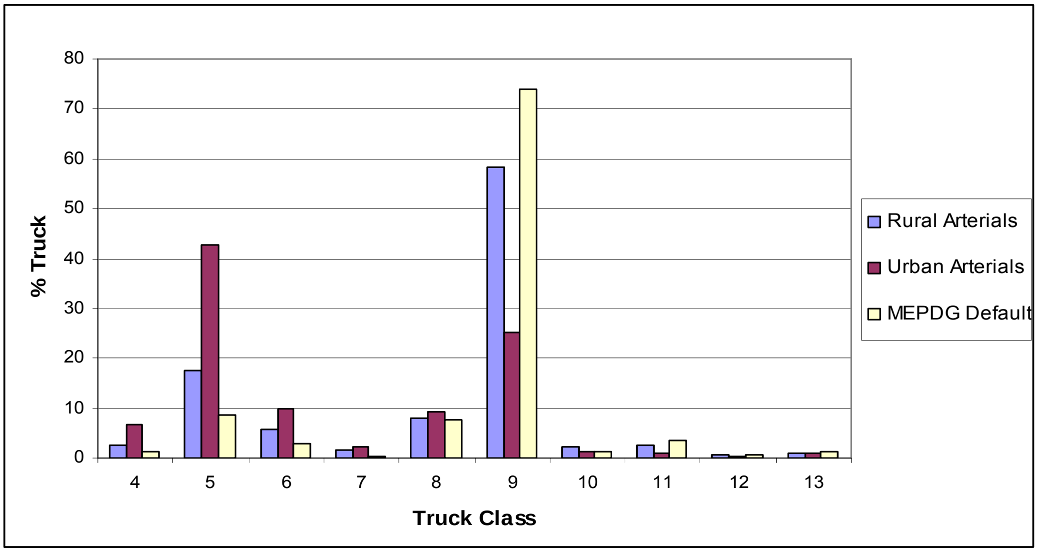


Figure 3.3: Comparison of MEPDG default truck class distribution with Kansas Inputs for Principal Arterials (urban)

3.4.2.4.2 Hourly Truck Traffic Distribution

The hourly distribution factors (HDF) represent the percentage of the AADTT within each hour of the day (NCHRP 2004). Hourly distribution factors are used to distribute the monthly average daily truck traffic (MADTT) volumes by the hour of the day. The average hourly distribution of traffic is needed for the incremental damage computations for different thermal gradients during the day (NCHRP 2004). For all level of input, HDF was computed in this study based on the truck traffic data collected continuously over a 24-hour period.

Kansas functional class-specific hourly traffic distributions were generated from the KDOT provided AVC data or C- card files for 2002 in the following manner:

- For a particular site the C-card, files were first processed according to the file coding shown in Table 3.8.
- The total numbers of trucks (Classes 4 to 13) were added for each hour of the day and the sum total for a 24-hour period was derived.
- Hourly data was divided by the sum total from the previous step. For that particular site, some other sets of hourly classification data were also generated in the same manner for a different day within the same month or for different months within the same year.
- Finally all hourly distribution values were averaged.

Table 3.8: Vehicle Classification Record ("C"- Card) File Code

Field	Columns	Length	Description
1	1	1	Record Type
2	2-3	2	FIPS State Code
3	4-9	6	Station ID
4	10	1	Direction of Travel Code
5	11	1	Lane of Travel
6	12-13	2	Year of Data
7	14-15	2	Month of Data
8	16-17	2	Day of Data
9	18-19	2	Hour of Data
0	20-24	5	Total Volume
11	25-29	5	Class 1 Count
12	30-34	5	Class 2 Count
13	35-39	5	Class 3 Count
14	40-44	5	Class 4 Count
15	45-49	5	Class 5 Count
16	50-54	5	Class 6 Count
17	55-59	5	Class 7 Count
18	60-64	5	Class 8 Count
19	65-69	5	Class 9 Count
20	70-74	5	Class 10 Count
21	75-79	5	Class 11 Count
22	80-84	5	Class 12 Count
23	85-89	5	Class 13 Count
End the record here if the FHWA 13 class system is being used.			

The process was repeated for other sites with the same functional classification. Average distribution factors were then computed for that particular functional class. The sum of the distribution must add up to 100 percent.

Table 3.9 tabulates the project specific HDF values used in the MEPDG analysis in this study. Figure 3.4 shows the hourly truck traffic distribution for a rural Interstate (I-70) in Kansas and compares with the MEPDG default distribution. It appears that more truck travel happens in Kansas during the afternoon, evening, and night hours than during the early morning and morning hours. It is anticipated that this will have a significant impact on the calculated slab stresses.

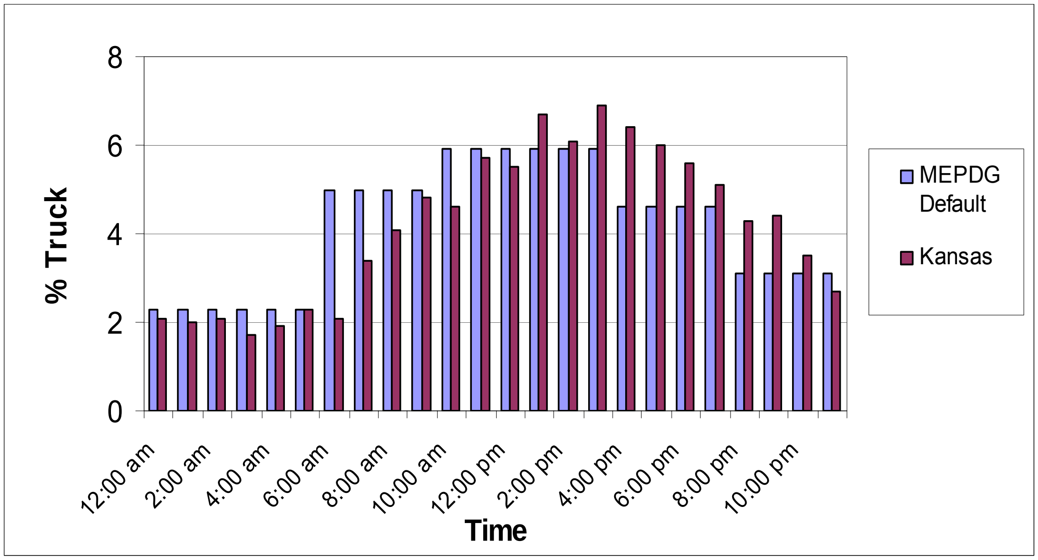


Figure 3.4: Typical hourly distribution for Rural Interstate compare to MEPDG default input

Table 3.9 shows the Kansas generated hourly distribution for different functional classes of roadways. For MEPDG default distribution, the distribution remains constant for a certain hour of the day, whereas in Kansas input, distribution varies for every hour of the day for each functional class.

Table 3.9: Hourly Truck Traffic Distribution Values Based on the Functional Classification of the Roadways

Hours	Rural Interstate Highways	Principal Arterials Others (Urban)	Principal Arterials Others (Rural)	MEPDG Default (Interstate/ Principal Arterials)
Midnight	2.1	0.5	1.4	2.3
1.00 am	2	0.4	1.1	2.3
2.00 am	2.1	0.3	1.4	2.3
3.00 am	1.7	0.3	1.6	2.3
4.00 am	1.9	0.6	2	2.3
5.00 am	2.3	1.6	2.8	2.3
6.00 am	2.1	3.9	4.1	5
7.00 am	3.4	6.2	5.1	5
8.00 am	4.1	7.5	6.2	5
9.00 am	4.8	8.3	6.5	5
10.00 am	4.6	8.1	6.7	5.9
11.00 am	5.7	8.1	6.8	5.9
Noon	5.5	7.8	6.3	5.9
1.00 pm	6.7	7.8	6.4	5.9
2.00 pm	6.1	8.3	6.2	5.9
3.00 pm	6.9	9	6	5.9
4.00 pm	6.4	8	6.1	4.6
5.00 pm	6	4.8	5.3	4.6
6.00 pm	5.6	3.6	4.1	4.6
7.00 pm	5.1	1.8	3.6	4.6
8.00 pm	4.3	1.1	3.3	3.1
9.00 pm	4.4	0.8	2.8	3.1
10.00 pm	3.5	0.8	2.5	3.1
11.00 pm	2.7	0.4	1.7	3.1

3.4.2.4.3 Traffic Growth Factors

All traffic input levels require an estimate of future traffic growth, which allows for the growth or decay in traffic over time. MEPDG allows users to use three different traffic growth functions to compute the growth rate over time as shown in Table 3.10. Different growth functions may be used for different functional classes based on several other factors such as, opening date of the roadway to traffic, pavement design life, etc. In this study, traffic growth rate was assumed to be linear based on the project-specific AADT forecast. Project-specific linear traffic growth rates varying from two to about seven percent were used in this study and summarized in Table 3.3.

Table 3.10: Function Used in Computing/ Forecasting Truck Traffic Over Time (NCHRP 2004)

Function Description	Model
No growth	$AADTT_X$ = 1.0 * $AADTT_{BY}$
Linear growth	$AADTT_X$ = GR * AGE + $AADTT_{BY}$
Compound growth	$AADTT_X = AADTT_{BY} * (GR)^{AGE}$

Where $AADTT_X$ is the annual average daily truck traffic at age X, GR is the traffic growth rate and $AADTT_{BY}$ is the base year annual average daily truck traffic.

Axle Load Distribution Factors

In order to use MEPDG effectively, the percentage of the total axle load applications within each load interval (normalized axle load distribution) for a specific axle type (single, tandem, tridem, and quad) and vehicle class (Classes 4 through 13) for each month of the year, must be computed externally. MEPDG software allows input for axle load distribution for each axle type at certain load intervals. For single axles, load distribution is from 3,000 lb to 40,000 lb at 1,000-lb intervals. For tandem axle, distribution ranged from 6,000 lb to 80,000 lb at 2,000 lb intervals, and for tridem axles, distribution interval varies from 12,000 lb to 102,000 lb at 3,000 lb intervals.

The axle load distributions were obtained in this study by analyzing the “W” card files from the WIM data provided by KDOT. The present Kansas truck weight program was established in 1990 with the acquisition of Weigh-In-Motion (WIM) equipment.

KDOT has 90 sites for monitoring purposes. Thirty sites are being monitored each year on a 3-year rotation. Nine of those sites are Long Term Pavement Performance (LTPP) monitoring sites. The Portable Weigh-In-Motion equipment, incorporating a capacitance mat weight sensor, is currently used for classification and weighing purposes in Kansas. This system is attached directly to the pavement surface and positioned perpendicular to the normal traffic flow and extends across the traveled lane in order to have only one path contact of the weight sensor. Forty-eight hours of truck weight data are collected at each site once every three years. In order to set up this portable weighing equipment at a particular site, traffic lane closure is essential. After complete setup of the roadway components, the truck weight data processor connected to the weight sensor collects the data. At the end of one session, the other lane is closed to remove the sensor. Collected data are processed for three types of correction: one for weight, one for speed and one for magnetic length. The equipment is also required to be calibrated for good results. Original data are stored in specific formats and edited according to the machine-specific criteria and processed as “W” card files. Individual vehicle records are also reviewed manually to correct any errors. Typically software is used for this review process. Finally with the data file adjusted and edited the “W” card data formats are created for yearend data submittal to the Federal Highway Administration (FHWA). This is usually done using in-house and Vehicle TRavel Information System (VTRIS) software packages (KDOT 2003).

In Kansas, permanent scales have been installed at eleven LTPP sites and at least in one traveled lane. Nine of them are equipped with piezo-electric (PE) weighing systems. Among two other sites, one is equipped with a high speed load cell system from Toledo Scale and the other site is equipped with the IRD 1060 bending plate system. To comply with the LTPP requirements, data are collected at many of these 11 sites continuously. These sites are usually calibrated semi-annually (KDOT 2003).

In this study, the following steps were followed for deriving axle load spectra manually using KDOT-provided WIM data.

1. To get the axle loads, first the KDOT- provided “W” card file was assembled and processed according to the codes of the vehicles provided by TMG as shown in Table 3.11. Nine years of portable WIM data (48-hour counts, 1995-2003) was provided by KDOT. Because of lack of continuous data NCHRP 1-39 traffic analysis software TrafLoad could not be used. TrafLoad requires uninterrupted hourly data for 365 days to generate all volume adjustment factors and axle load spectra needed by MEPDG. For the portable WIM data, truck weights were not available at any particular site for all months of a year. NCHRP MEPDG research has shown using data from a GPS-5 section in Marion County, Indiana that the variations in axle load spectra across the months within a year and along the years are insignificant (Tam and Von Quintus 2004). Thus it was decided to use different monthly data for different years from different sites and to develop a state-wide axle load distribution. MEPDG also recommends the sample size to estimate the normalized axle load distribution from the WIM data for a given level of confidence and percentage of expected error. The following months were used in this study: 1995 (February), 1996 (January), 1998 (March), 1999 (April), 2000 (May), 2002 (November), 2003 (June, July, August, September, October, and December). At least 24 hours of data were analyzed

for each month of the year. Thus the expected error for the generated axle load spectra in this study would be $\pm$ 10% error at close to 90% confidence interval.

Table 3.11: Truck Weight Record (“W”-Card) for Axle Load Distribution Factor

Field	Columns	Length	Description
1	1	1	Record Type
2	2-3	2	FIPS State Code
3	4-9	6	Station ID
4	10	1	Direction of Travel Code
5	11	1	Lane of Travel
6	12-13	2	Year of Data
7	14-15	2	Month of Data
8	16-17	2	Day of Data
9	18-19	2	Hour of Data
0	20-21	2	Vehicle Class
11	22-24	3	Open
12	25-28	4	Total Weight of Vehicle
13	29-30	2	Number of Axles
14	31-33	3	A-axle Weight
15	34-36	3	A-B Axle Spacing
16	37-39	3	B-axle Weight
17	40-42	3	B-C Axle Spacing
18	43-45	3	C-axle Weight
19	46-48	3	C-D Axle Spacing
20	49-51	3	D-axle Weight
21	52-54	3	D-E Axle Spacing
22	55-57	3	E-axle Weight
23	58-60	3	E-F Axle Spacing
24	61-63	3	F-axle Weight
25	64-66	3	F-G Axle Spacing
26	67-69	3	G-axle Weight
27	70-72	3	G-H Axle Spacing
28	73-75	3	H-axle Weight
29	76-78	3	H-I Axle Spacing
30	79-81	3	I-axle Weight
31	82-84	3	I-J Axle Spacing
32	85-87	3	J-axle Weight
33	88-90	3	J-K Axle Spacing
34	91-93	3	K-axle Weight
35	94-96	3	K-L Axle Spacing
36	97-99	3	L-axle Weight
37	100-102	3	L-M Axle Spacing
38	103-105	3	M-axle Weight

2. After processing one file, for a particular vehicle class, the axles were identified according to their default spacing provided by KDOT. It is also to be noted that "W" Card file also identifies the vehicle class and the order in which the axles were weighted. If the spacing between two axles is low then the axles were grouped together to form a tandem axle, otherwise they were treated as single axles. According to the KDOT practice, two axles 4.6 feet apart are in a tandem. In this study, in some cases, if the spacing between two axles is less than 2.5 feet, they were treated as tandem, when the spacing is greater than 30 inches they were treated as singles. Although the spacing is very short for a typical tandem axle, it is possible with low-profile tires. "W" Card file also identifies the vehicle class and the order in which the axles were weighted. For example, in a Class 9 vehicle with five axles, the front axle is leveled as "A", and the first tandem axle is leveled as "B" and so on, as shown in Figure 3.5. Tridem and Quad axle distributions were generated based on the same algorithms. A typical example of axle load spectra calculation for a particular vehicle Class 9 is presented in Table 3.12. This vehicle class is the predominant truck type in Kansas and commonly termed as 18-wheeler.
3. In this step, the total number of axles for each load interval was calculated and the frequency distribution was generated. For a specific axle-type and truck class, the summation of calculated percentages of the total number of axle applications within each load range, should be equal to 100%. A typical frequency distribution for a particular axle type (Tandem axle) and for the month of January is shown in Table 3.13. The above process was repeated for each

class of vehicle for the year and for all axle types.

Figure 3.6 illustrates the distribution of Class 9 vehicles in Kansas and compares it with the MEPDG default input. MEPDG default input shows more truck distribution in the high axle load categories. Therefore it is expected that damage would be higher for MEPDG default traffic input compared to the Kansas traffic input.

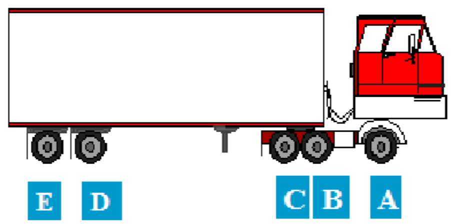


Figure 3.5: Typical axle configurations for vehicle Class 9

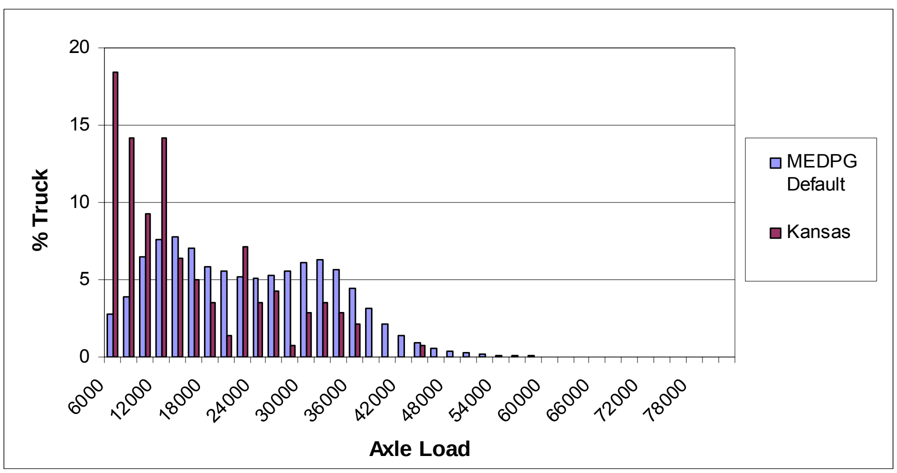


Figure 3.6: Comparison of axle load distribution for Class 9 vehicle with MEPDG default distribution

Table 3.12 shows the typical axle load spectra calculation for Class 9 vehicle. Class 9 vehicle is a five-axle truck, however the axle load distribution will depend on the axle spacing. Whenever the spacing is less than 30 inches, especially for low profile tires, they were added together for tandem or tridem axle distribution. In this table, it is observed that most of the axle spacing between two axles is around 12 inches, therefore those axles were added together for tandem or tridem axle distribution.

Table 3.12: Typical Example of Axle Load Spectra Calculation for Vehicle Class 9

Vehicle Class	A-axle weight (lb)	(A-B) axle spacing	B-axle weight (lb)	(B-C) axle spacing	C-axle weight (lb)	(C-D) axle spacing	D-axle weight (lb)	(D-E) Axle Spacing	E-axle weight (lb)
9	9460	50	9680	12	9460	93	8360	12	7480
9	12100	43	16500	13	14960	63	19800	30	19360
9	10120	50	11660	11	11000	93	13860	11	11220
9	11660	35	18260	12	19140	77	18700	12	20900
9	9900	43	8800	12	9900	86	7480	12	8140
9	4180	38	9900	74	5280	8	5280	8	6380
9	11440	49	17160	13	16500	94	17600	12	18040
9	8800	47	18700	13	19140	96	20240	12	18480
9	11000	42	16500	13	15400	62	19360	30	19580
9	11220	36	19800	12	18920	79	16720	12	20460
9	14960	48	14520	13	15620	60	13860	30	15400
9	10120	50	13860	13	12980	73	18040	12	15620

Table 3.13: Frequency Distribution of Tandem Axle for Month of January

Mean Axle Load (lbs)	Vehicle/Truck Class									
	4	5	6	7	8	9	10	11	12	13
6000	0.00	0.00	0.00	0.00	60.00	18.44	0.00	0.00	0.00	0.00
8000	0.00	0.00	12.50	0.00	0.00	14.18	0.00	0.00	0.00	0.00
10000	0.00	0.00	12.50	0.00	0.00	9.22	20.00	0.00	0.00	0.00
12000	0.00	0.00	12.50	0.00	20.00	14.18	0.00	0.00	0.00	0.00
14000	0.00	0.00	37.50	0.00	0.00	6.38	0.00	0.00	0.00	0.00
16000	100.00	0.00	0.00	0.00	20.00	4.96	0.00	0.00	0.00	0.00
18000	0.00	0.00	0.00	0.00	0.00	3.55	40.00	0.00	0.00	0.00
20000	0.00	0.00	0.00	0.00	0.00	1.42	20.00	0.00	0.00	0.00
22000	0.00	0.00	0.00	0.00	0.00	7.09	0.00	0.00	0.00	0.00
24000	0.00	0.00	0.00	0.00	0.00	3.55	0.00	0.00	0.00	0.00
26000	0.00	0.00	0.00	0.00	0.00	4.26	0.00	0.00	0.00	0.00
28000	0.00	0.00	0.00	0.00	0.00	0.71	20.00	0.00	0.00	0.00
30000	0.00	0.00	12.50	0.00	0.00	2.84	0.00	0.00	0.00	0.00
32000	0.00	0.00	0.00	0.00	0.00	3.55	0.00	0.00	0.00	0.00
34000	0.00	0.00	12.50	0.00	0.00	2.84	0.00	0.00	0.00	0.00
36000	0.00	0.00	0.00	0.00	0.00	2.13	0.00	0.00	0.00	0.00
38000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
40000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
42000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
44000	0.00	0.00	0.00	0.00	0.00	0.71	0.00	0.00	0.00	0.00
46000 & High	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

It can be observed from Table 3.13 that most of the distributions for the tandem axles exist for Vehicle Class 6, 8, 9 and 10, whereas the default distribution accounts for tandem axle distribution for FHWA Class 5, 7, 11, which should not have any tandem axle.

3.4.2.5 General Traffic Input

Most of the inputs under this category define the axle load configuration and loading details for calculating pavement responses. The “Number of Axle Types per Truck Class” and “Wheelbase” inputs are used in the traffic volume calculations.

Mean wheel location is the distance from the outer edge of the wheel to the pavement marking. For this study there was no state-specific information available, thus a default mean wheel location of 18 inches provided in the MEPDG Software was used.

Traffic Wander Standard Deviation is the standard deviation of the lateral traffic wander used to estimate the number of axle load repetitions over a single point in a probabilistic manner for predicting distresses and performance. A default traffic wander standard deviation of 10 inches was used in this study.

Design lane width, which is not the slab width, is the actual width of the lane as defined by the distance between the lane markings on either side of the design lane. The default value for the standard-width lanes is 12 feet, and this value was used in this study for all projects.

Number of Axles/Truck

These inputs specify the average number of axles for each truck class (Classes 4 to 13) for each axle type (single, tandem, tridem, and quad). It is usually calculated from the WIM data and generated over time by dividing the total number of a specific axle type measures for a truck class by the total number of trucks in that class. The traffic module assumes that the number of axles for each axle-type is constant with time. The MEPDG Software contains a default set of values derived from the LTPP data. The default data used in this study are presented in Table 3.14.

Table 3.14: Default Distribution for Number of Axles/ Truck (NCHRP 2004)

Vehicle Class	Single	Tandem	Tridem	Quad
4	1.62	0.39	0	0
5	2.0	0	0	0
6	1.02	0.99	0	0
7	1.0	0.26	0.83	0
8	2.38	0.67	0	0
9	1.13	1.93	0	0
10	1.19	1.09	0.89	0
11	4.29	0.26	0.06	0
12	3.52	1.14	0.06	0
13	2.15	2.13	0.35	0

3.4.2.5.1 Axle Configuration

These inputs allow the user to make broad inputs regarding the configuration of the typical axle and tire. A series of data are usually required to elaborate the typical tire axle and axle load configuration in order to compute the pavement responses. These data can be obtained from the manufacturer's database or measured directly in the field. The following elements are in this category:

Average axle-width - Distance between the two outside edges of an axle. A default width of 8.5 feet was used in this study.

Dual Tire Spacing - This is the center-to-center transverse spacing between dual tires of an axle. A default spacing of 12 inches was used in this study.

Axle Spacing - Axle spacing is the distance between the two consecutive axles of a tandem, tridem, or quad. MEPDG recommended spacings of 51.6, 49.2 and 49.2 inches were used for tandem, tridem and quad axles, respectively.

Tire Pressure - This is the hot inflation pressure of the tire. The tire pressure needs to be input for both single and dual tires. MEPDG recommended input of 120 psi was used for both types of tire.

Wheelbase - This information is important for determining the JPCP top-down cracking. The top-down cracking is associated with the critical loading by a particular combination of axles and the steering and drive axles of trucks. The user has to specify the percentage of trucks that have short, medium, and long spacings, and these information are used by the MEPDG software for computing pavement responses.

Average axle spacing - This is the average longitudinal distance between two consecutive axles that fall under the short, medium, and long axle spacing category. Axle spacing is applicable to only trucks in Class 8 and above. MEPDG default inputs for average axle spacing were 12 feet for short axle, 15 feet for medium axle and 18 feet for long axle.

Percentage of Trucks - This is the percentage of trucks that have short, medium, and long axle spacings specified above. A default input of 33% was used for the short and long axle spacing trucks whereas 34% was used for the medium spacing ones.

3.4.3 Climate

This is one of the four required, major categories of inputs. Environmental conditions have significant effects on the performance of rigid pavements. Factors such as, precipitation, temperature, freeze-thaw cycles, and depth to water table affect temperature and moisture contents of unbound materials which, in turn, affect the load carrying capacity of the pavement. Further, the temperature gradients induce stresses and deformations in the concrete slab. The seasonal damage and distress accumulation algorithms in the MEPDG design methodology require hourly data for five weather parameters: air temperature, precipitation, wind speed, percentage sunshine, and relative humidity (NCHRP 2004). Temperature and moisture profiles in the pavement and subgrade are modeled using the Enhanced Integrated Climatic Model (EICM) software, which is integrated into the MEPDG Software. The EICM software is linked to the MEPDG software as an independent module through interfaces and design inputs. EICM is a one-dimensional coupled heat and moisture flow program that simulates changes in the behavior and characteristics of pavement and subgrade materials in conjunction with the climatic conditions over several years of operation.

The temperature and moisture effects that are directly considered in the design of JPCP are as follows:

- The permanent built-in curling that occurs during construction is combined with the permanent warping due to differential shrinkage, and expressed as "permanent curl/warp." This parameter is a direct and influential input in the design analysis of JPCP.
- Transient hourly negative and positive non-linear temperature differences caused by solar radiation are computed using EICM.
- Transient hourly negative moisture shrinkage at the top of the slab caused by changes in relative humidity during each month of the year is converted to an equivalent temperature difference for every month.

All three above stated effects on the PCC slab are predicted and are combined with the axle loads to compute critical slab stresses in order to accumulate damage through monthly increments.

MEPDG recommends that the weather inputs be obtained from weather stations located near the project site. At least 24 months of actual weather station data are required for computation (Barry and Schwartz 2005). The MEPDG software includes a database of appropriate weather histories from nearly 800 weather stations throughout the United States. This database can be accessed by specifying the latitude, longitude, elevation and depth of water table of the project location. Depth of water table in this context is the depth of the ground water table from the top of the subgrade. Specification of the weather inputs is identical at all three hierarchical input levels in MEPDG.

3.4.3.1 Climatic file generation

The MEPDG software offers two options to specify the climate file for individual project. This file can be imported or generated. Import option is for the previously-generated climat.icm-file. This is achieved by clicking “Import” option and pointing to the file location. For “Generate” option, weather data can be updated from a single weather station or virtual weather station can be created by interpolating climatic data for that specific location based on available data from the six closest weather stations.

In this study, project-specific virtual weather stations were created by interpolation of climatic data from the selected physical weather stations. For this purpose, project specific latitude, longitude, elevation and water table depth at the given location were provided. Then the MEPDG software listed the six closest weather stations in the vicinity of the project, and the weather stations were interpolated for generating those climatic files. EICM interpolates the weather data from the selected locations inversely weighted by the distance from the location. The depth of the water table was found from the soil survey report for each county. For all projects, water table depth was greater than 10 feet or close to 10 feet. Table 3.15 summarizes the basic inputs for the climatic data generated for all projects in this study.

3.4.4 Structure

This is the fourth set of inputs required by the MEPDG software. This category allows the user to specify the structural, design, and material aspects of the trial design chosen for performance evaluation. The category allows specifying the "Design Features of JPCP," "Drainage" and "Surface" properties for that particular pavement. Furthermore it also provides an access to the input screens for different layers chosen in the structure.

Table 3.15: Summarization of the Project Specific Latitude, Longitude, Elevation and Water Table Depth

Project ID	Latitude(Deg.min)	Longitude(Deg.min)	Elevation (ft)
K-2611-01	39.38	-97.16	1070
K-3344-01	39.04	-95.38	880
SPS-2 (Sec-5)	38.97	-97.09	1194
SPS-2 (Sec-6)	38.97	-97.09	1194
SPS-2 (Control)	38.97	-97.09	1194
K-3216-02	38.15	-95.93	1200
K-3217-02	38.15	-95.93	1200
K-3382-01	38.78	-94.99	1000

3.4.4.1 Design Features of JPCP

Design features have significant effects on the mechanistic response calculated for JPCP as well as on its performance (NCHRP 2004). The inputs for JPCP design features can be broadly classified as: 1) Effective Equivalent Built-in Temperature and Moisture Difference, which directly affects the resulting critical stresses in the slab; 2) Joint Design, which directly affects the corner deflections, slab length, and resulting stresses; 3) Edge Support, which affects the magnitude of stresses depending on the location of the wheel load from the slab edge; and 4) Base Properties, which affects faulting as a result of base erosion and the levels of stresses due to bonded/unbonded condition. These features are described as follows:

3.4.4.1.1 Permanent Curl/Warp Effective Temperature Difference

This is the equivalent temperature differential between the top and bottom layers of the concrete slab that can quantitatively describe the locked in stresses in the slab due to construction temperatures, shrinkage, creep and curing conditions. This temperature difference is typically a negative number, i.e. effectively represents a case, when the top of the slab is cooler than the bottom of the slab. The magnitude of permanent curl/warp is a sensitive factor that affects JPCP performance. MEPDG recommended value of -10°F was used in this study for permanent curl/warp. This value was obtained through optimization and applicable to all new and reconstructed rigid pavements in all climatic regions.

3.4.4.1.2 Joint Design

Joint Spacing - This is the distance between two adjacent joints in the longitudinal direction and is equal to the length of the slab. The joint spacing is a critical JPCP design factor that affects structural and functional performance of JPCP, as well as construction and maintenance cost. The stresses in JPCP increase rapidly with increasing joint spacing. To a lesser degree, joint faulting also increases with increasing joint spacing. In this study, a joint spacing of 15 feet was used for all projects.

Sealant Type - The sealant type is to be chosen from the options offered in the drop-down menu. The sealant options are liquid, silicone, and preformed. Sealant type is an input to the empirical model used to predict spalling. Spalling is used in smoothness predictions, but it is not considered directly as a measure of performance in MEPDG. In this study, all projects were assumed to have liquid sealant.

Random Joint Spacing - MEPDG offers the designer the option of using random joint spacings, and up to four different values. In this case, the user needs to click on the radio button referring to random joint spacing and enter four different values. If random joint spacing is used, the MEPDG software uses the average joint spacing for faulting analysis and the maximum joint spacing for cracking analysis. In this study, the joint spacing was ordered, not random.

Doweled Transverse Joint - MEPDG has the capability to evaluate the effects of dowels across transverse joints, especially in reducing faulting. If dowels are used to achieve positive load transfer, the user needs to click the button corresponding to the doweled transverse joints and to make further inputs about the size and spacing of the dowels.

Dowel Diameter - This is the diameter of the round dowel bars used for load transfer across the transverse joints. The larger the dowel diameter, the lower the concrete bearing stress and joint faulting. The MEPDG software accepts any dowel diameter from 1 inch to 1.75 inches. Project specific inputs are provided in Table 3.16.

Dowel Bar Spacing - This is the center-to-center distance between the dowels used for load transfer across the transverse joints. The allowable spacings are from 10 to 14 inches.

MEPDG suggests that with increasing slab thickness (in order to reduce slab cracking for heavier traffic), dowel diameter be increased to control joint faulting. This may result in a small increase in predicted joint faulting due to a reduction in effective area of the bar relative to the slab thickness (NCHRP 2004). In this study, a 12-inch dowel spacing was used for all projects.

3.4 4.1.3 Edge Support

Tied PCC Shoulder - Tied PCC shoulders can significantly improve JPCP performance by reducing critical deflections and stresses. The shoulder type also affects the amount of moisture infiltration into the pavement structure. The effects of moisture infiltration are considered in the determination of seasonal moduli values of the unbound layers. The structural effects of the edge support features are directly considered in the design process. For tied concrete shoulders, the long-term Load Transfer Efficiency (LTE) between the lane and the tied shoulder must be provided.

Long-term LTE - LTE is defined as the ratio of the deflection of the unloaded to that of the loaded slab. The higher the LTE, the greater the support provided by the tied shoulder to reduce critical responses of the mainline slabs. Typical long-term deflection LTE values are:

- 50 to 70 percent for monolithically constructed tied PCC shoulder.
- 30 to 50 percent for separately constructed tied PCC shoulder.

In this study, 60 percent LTE was considered for the projects with monolithically constructed tied PCC shoulder.

Widened Slab - The JPCP slab can be widened to accommodate the outer wheel path further away from the longitudinal edge. For widened slab cases, the width of the slab has to be specified. In this study, only one project, SPS-2 Section 6, had a widened lane of 14 feet.

Slab Width - This is the selected width of the slab and this is not same as the lane width. All projects in this study, except SPS-2 Section 6, had 12 foot lanes.

3.4.4.1.4 Base Properties

PCC-Base Interface - This allows the user to specify the interface type and the quality of bond between the slab and the base. Structural contribution of a bonded stabilized base is significant compared to an unstabilized base. The interface between a stabilized base and the PCC slab is modeled either completely bonded or unbonded for the JPCP design. However, the bond tends to weaken over time around the edges. For an unbonded base layer, the layer is treated as a separate layer in the analysis. In this study, since all projects have stabilized bases, the PCC-base interface was considered as bonded.

Erodibility Index - This is an index, on a scale of 1 to 5, to rate the potential for erosion of base material. The potential for base or subbase erosion (layer directly beneath the PCC layer) has a significant impact on the initiation and propagation of pavement distresses. Different base types are classified based on this long-term erodibility behavior as follows:

- Class 1 – Extremely erosion-resistant materials.
- Class 2 – Very erosion-resistant materials.
- Class 3 – Erosion resistant materials.
- Class 4 – Fairly erodible materials.
- Class 5 – Very erodible materials.

In this study, Class 3 option was chosen for all projects since all had stabilized bases.

Loss of Bond Age - The JPCP design procedure includes the modeling of changes in the interface bond condition over time. This is accomplished by specifying pavement age at which the debonding occurs. Up to the debonding age, the slab-base interface is assumed fully bonded. After the debonding age, the interface is assumed fully unbonded. The design input is the pavement age at debonding, in months. In general, specifying debonding age greater than 5 years (60 months) is not recommended and was not used in calibration. Therefore, this default value was considered for all projects in this study.

3.4.4.2 Drainage and Surface Properties

This feature allows the user to make inputs for the drainage characteristics of the pavement. Information required under this category are:

- Pavement surface layer (PCC) shortwave absorptivity
- Potential for infiltration
- Pavement cross slope
- Drainage path length

PCC pavement Shortwave Absorptivity is a measure of the amount of available solar energy that is absorbed by the pavement surface. The lighter and more reflective the surface, the lower the surface shortwave absorptivity will be. The suggested range for the PCC layer is 0.70 to 0.90. A Shortwave Absorptivity value of 0.85 (default) was used in this study.

Infiltration defines the net infiltration potential of the pavement over its design life. In the MEPDG approach, infiltration can assume four values – none, minor (10 percent of the precipitation enters the pavement), moderate (50 percent of the precipitation enters the pavement), and extreme (100 percent of the precipitation enters the pavement). Based on this input, the EICM determines the amount of water available at the top of the first unbound layer beneath the PCC slab. A moderate infiltration (50%) value was chosen for all the projects.

Drainage Path Length is the resultant length of the drainage path, i.e., the distance measured along the resultant of the cross and longitudinal slopes of the pavement. It is measured from highest point in the pavement cross-section to the point where drainage occurs. A default drainage path length of 12 feet was used in this project.

Pavement Cross Slope is the percentage vertical drop in the pavement for unit width, measured perpendicular to the direction of traffic. This input is used in computing the time required to drain a pavement base or subbase layer from an initially wet condition. Site-specific cross slope was chosen based on the individual project plan. In this study, most projects had a cross slope of 1.6%.

3.4.4.3 Pavement Layers

PCC Layer - MEPDG requires input values for the following four groups of PCC material properties for JPCP design analysis. Material property data inputs and material test requirement for these properties are briefly discussed below:

1. General Properties:

   -Layer Thickness

   -Unit Weight

   -Poisson's Ratio

2. Strength Properties:

-Flexural Strength (Modulus of Rupture)

-Modulus of Elasticity

-Compressive Strength

3. Thermal Properties:

   -Coefficient of Thermal Expansion

   -Thermal Conductivity

   -Heat Capacity

   -Surface Short Wave Absorptivity

4. Mixture and Shrinkage Properties:

   -Cement type, content, w/c ratio, Aggregate Type

   -Ultimate Shrinkage

   -Reversible Shrinkage

   -Time to Develop 50% of Ultimate Shrinkage

General Properties - Surface layer thickness is the thickness of the PCC slab, which is one of the important parameters needed for concrete pavement performance. In this study, this information was available from each project plan sheet.

Unit weight is the weight of the concrete mix design per unit volume of the mix. This information was obtained from the project mix design.

Poisson's ratio is defined as the ratio of the lateral strain to the longitudinal strain for an elastic material. It is a required input for the structural response models, although its effect on computed pavement responses is not great. Its value for normal concrete typically ranges between 0.11 and 0.21. In this study, the Poisson's ratio was chosen as 0.20 for all projects. Project-specific input values are summarized in Table 3.16.

Table 3.16: Structural Input Parameters for MEPDG Rigid Pavement Design Analysis

INPUT PARAMETERS	Input Value							
	(I-70 GE)	(I-70 SN)	SPS (Sec 5)	SPS (Sec 6)	SPS (Control)	(US-50-CS1)	(US-50-CS2)	(K-7 JO)
Design								
Dowel Diameter (in)	1.375	1.375	1.5	1.5	1.5	1.25	1.25	1.125
Design Lane Width (ft)	12	12	12	14	12	12	12	12
PCC Layer								
PCC Layer thickness (in)	11	10.5	11	11	12	10	10	9
Material Unit Weight (pcf)	140	142	143	139.2	146	136.9	138.5	142
Cement Type	II	I	II	II	II	II	II	II
Cement Content (Lb/yd^3)	653.4	630	862	532	600	622.1	626.3	622.9
Poisson's ratio	0.20	0.20	0.20	0.20	0.20	0.20	0.20	0.20
Coeff of thermal. Expansion (X 10^{-6}/°F)	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5
Water-cement ratio (w/c)	0.44	0.411	0.35	0.35	0.42	0.42	0.43	0.46
Aggregate Type	Limestone	Limestone	Quartzite	Quartzite	Quartzite	Limestone	Limestone	Limestone
PCC 28-day Strength (psi)	690	473	945	617	647	5,569	4,362	537
Derived ultimate shrinkage (μm)	621	727	596	423	456	554	593	507

Table 3.16: Structural Input Parameters for MEPDG Rigid Pavement Design Analysis (Continued)

INPUT PARAMETERS	Input Value							
	(I-70 GE)	(I-70 SN)	SPS (Sec 5)	SPS (Sec 6)	SPS (Control)	(US-50-CS1)	(US-50-CS2)	(K-7 JO)
Computed Ultimate Shrinkage (μm)	300	600	200	350	386	450	555	507
Derived PCC Zero Stress temperatures (ºF)	63	86	130	111	115	48	48	102
Base Material								
Base Type	PCTB	PCTB	LCB	LCB	PCTB	BDB	BDB	PCTB
Base Thickness (in)	6	4	6	6	6	4	4	4
Poisson's ratio	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
Base material unit wt. (pcf)	135	135	135.4	135.4	135	135	135	135
Base Modulus (psi)	500,000	500,000	2,000,000	2,000,000	500,000	500,000	500,000	500,000
Treated Subgrade								
Subgrade type	N/A	LTSG	FASG	FASG	FASG	LTSG	LTSG	LTSG
Subgrade modulus (psi)	N/A	50,000	50,000	50,000	50,000	50,000	50,000	50,000
Poisson's ratio	N/A	0.20	0.15	0.15	0.15	0.20	0.20	0.20
Unit weight (pcf)	N/A	125	125	125	125	125	125	125
Poisson's ratio	N/A	0.20	0.15	0.15	0.15	0.20	0.20	0.20
Compacted Subgrade								
Subgrade soil type	A-6	A-7-6	A-6	A-6	A-6	A-7-6	A-7-6	A-7-6
*Subgrade Modulus (psi)	9746	6268	6928	6928	7523	6300	6098	7262
Plasticity index, PI	15.8	25.7	26	26	23	25.7	27.3	19.9
Percent passing # 200 sieve	71.8	93.3	78.1	78.1	76.9	92.5	91.9	94.3
% passing # 4 sieve	100	100	100	100	98	100	100	100
D_{60} (mm)	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
Poisson's ratio	0.35	0.35	0.35	0.35	0.35	0.35	0.35	0.35
Derived Parameters Physicals properties								
MDD (pcf)	107.8	88.3	90.5	90.5	92.2	88.4	87.7	91.5
Gs	2.73	2.75	2.75	2.75	2.74	2.75	2.75	2.75
Hydraulic conductivity (ft/hr)	3.25e-005	3.25e-005	3.25e-005	3.25e-005	3.25e-005	3.25e-005	3.25e-005	3.25e-005
OMC	18.7	24.2	22.7	22.7	21.6	24.1	24.7	22.1
Calculated degree of Sat.(%)	87.6	88.8	88.5	88.5	88.3	88.8	88.8	88.4

* computed by MEPDG

Strength Properties - PCC strength properties that are input in to the MEPDG software are based on the hierarchical level of input selected. The strength parameters considered in the structural and material models for JPCP are the modulus of elasticity, flexural strength, and compressive strength. Table 3.17 provides the required inputs at different hierarchical levels.

Table 3.17: Strength and Modulus of Elasticity Inputs for JPCP and CRCP Design

Input Level	JPCP
Level 1	▪ Modulus of Elasticity (Ec) and Flexural Strength (Modulus of Rupture, MR) at 7, 14, 28, 90 days ▪ Estimated ratios for 20-year to 28-day values of Ec and MR
Level 2	▪ Compressive Strength (fc) at 7, 14, 28, 90 days ▪ 20-year to 28-day fc ratio estimate
Level 3	▪ MR or fc at 28 days ▪ Ec at 28 days (optional)

According to the level 1 input, MEPDG recommends the maximum values for the ratios of 20-year and 28-day values of the strength (MR) and modulus (E). Recommended maximum ratio of the 20-year to the 28-day Ec is 1.20. The same maximum value of 1.20 is also recommended for the ratios of the 20-year to the 28-day MR and E values.

For level 2 input, the compressive strength data at various ages are first converted to the modulus of elasticity and flexural strength at those ages using correlation models for the properties shown in Equations 3.2 and 3.3.

$$Ec = 33\rho^{3/2}(fc')^{1/2} \qquad \text{Equation 3.2}$$

Where,

Ec = concrete modulus of elasticity, psi;

ρ = unit weight of concrete, pcf; and fc' = compressive strength of concrete, psi.

$$MR = 9.5\,(fc')^{1/2} \qquad \text{Equation 3.3}$$

Where, MR = concrete modulus of rupture, psi; and

fc' = compressive strength of concrete, psi.

Level 3 input requires only the 28-day strength either determined for the specific mix or an agency default. It offers the user the choice of either specifying the 28-day modulus of rupture or the 28-day compressive strength. If 28-day MR is estimated, its value at any given time, t, is determined using:

$$MR(t) = 1.0 + 0.12 \log_{10} (AGE/0.0767) - 0.01566\,[\log_{10} (AGE/0.0767)]^2$$

Where,

MR(t) = modulus of rupture at a given age *t*;

MR(28) = modulus of rupture at 28 days; and

AGE = concrete age of interest in years.

In this study, level 3 input was used for all projects. For Shawnee, Johnson, and Geary county projects, including all SPS-2 projects, 28-day MR value was used whereas for the two Chase county projects, 28-day compressive strength values were used. Shawnee county project has the lowest modulus of rupture of 473 psi, whereas SPS-2 Section 5 has the highest value of 945 psi. Project-specific values are presented in Table 3.16.

Thermal Properties - Coefficient of Thermal Expansion, Thermal Conductivity, and Heat Capacity are the required inputs for the JPCP design analysis, using the MEPDG software.

Coefficient of Thermal Expansion (CTE) is a measure of the expansion or contraction that a material undergoes with the change in temperature. CTE is considered to be a very critical parameter in the calculation of curling stresses developed and therefore, in the prediction of all distresses. Aggregates have CTE values in the range of 2.2 to 7.2 x 10^{-6} in/in/deg F and the resulting CTE of concrete can range between 4.1 and 7.3 x 10^{-6} in/in/deg F. This parameter can be determined at different hierarchical levels of input. In this study, the CTE value was determined from the TP-60 tests on the Kansas cores and the average value was 5.5 x 10^{-6} in/in/deg F. This value was used for all projects. This value also was a recommended MEPDG default for CTE.

Thermal conductivity is a measure of the ability of the material to uniformly conduct heat through its mass when two faces of the material are under a temperature differential. This value typically ranges from 1.0 to 1.5 BTU/(ft)(hr)(°F).

Heat capacity is defined as the amount of heat required to raise a unit mass of material by a unit temperature and usually ranges from 0.2 to 0.28 BTU/(lb)(°F). In this study, recommended calibrated values of 1.25 BTU/hr-feet-°F and 0.28 BTU/lb-°F were used for thermal conductivity and heat capacity, respectively.

Mixture and Shrinkage Properties - This option allows the user to provide the MEPDG software with mix design parameters and inputs for computing concrete shrinkage. Mix design related inputs are Cement type, Cement Content, Water/cement ratio, Aggregate type, PCC Zero-Stress Temperature, etc.

Three types of cement are identified in the MEPDG software. Cement content is the weight of cement per unit volume of concrete as per the mix design. The water/cement ratio (w/c) is the ratio of the weight of water to the weight of cement used in the mix design. Aggregate type is also a required input for the shrinkage strain calculation. Project specific mix design properties are used in this study and summarized in Table 3.16.

PCC Zero-stress temperature, T_z, is defined as the temperature (after placement and during the curing process) at which PCC becomes sufficiently stiff that it develops stress if restrained (NCHRP 2004). This is the temperature at which the slab would de-stress itself from all the built-in stresses, i.e. no thermal stresses are present. If the PCC temperature is less than T_z, tensile stresses occur at the top of the slab and vice-versa. Again the Tz is not actually a single temperature but varies throughout the depth of the slab (termed a zero-stress gradient). This value is computed based on the cement content and the mean monthly ambient temperature during construction. The user can, however, choose to provide this input directly into the MEPDG Software. This will require selecting the corresponding box and entering the PCC zero-stress temperature.

As mentioned earlier, *PCC zero-stress temperature*, T_z, is an important parameter that affects the stress buildup in the PCC slab immediately after construction. This parameter is also related to the time of construction since it is computed based on the cement content and the mean monthly temperature (MMT) during construction as shown below (NCHRP 2004):

$$T_z = (CC*0.59328*H*0.5*1000*1.8/ (1.1*2400) + MMT) \qquad \text{Equation 3.4}$$

where,

T_z = Temperature at which the PCC layer exhibits zero thermal stress;

CC = Cementitous material content, lb/yd^3;

H = -0.0787+0.007*MMT-0.00003*MMT^2; and

MMT = Mean monthly temperature for the month of construction, º F.

Two types of curing method are specified in this software: (1) Curing compound and (2) Wet curing. Curing compound was used as a curing method in this study.

Drying shrinkage of hardened concrete is an important factor affecting the performance of PCC pavements. For JPCP, the principal effect of drying shrinkage is slab warping caused by the differential shrinkage due to the through-thickness variation in moisture conditions leading to increased cracking susceptibility. For JPCP faulting performance, both slab warping and the magnitude of shrinkage strains are important. The magnitude of drying shrinkage depends on numerous factors, including water per unit volume, aggregate type and content, cement type, ambient relative humidity and temperature, curing, and PCC slab thickness. Drying shrinkage develops over time when PCC is subjected to drying.

Ultimate Shrinkage at 40% relative humidity (RH) is the shrinkage strain that the PCC material undergoes under prolonged exposure to drying conditions and is defined at 40 percent humidity. This input can be site specific, based on some correlation, or typical recommended value. The correlation to estimate the ultimate shrinkage is:

$$\varepsilon_{su} = C1 \cdot C2 \cdot [\ 26\ w^{2.1}\ fc^{-0.28} + 270\ ] \qquad \text{Equation 3.5}$$

Where,

ε_{su} = ultimate shrinkage strain ($\times 10^{-6}$);

C1 = cement type factor (1.0 for Type I cement, 0.85 and 1.1 for Type II and Type III cement, respectively);

C2 = curing condition factor (0.75 if steam cured, 1.0 if cured in water or 100% RH or wet burlap and 1.2 if cured by curing compound);

w = water content (lb/ft^3); and

fc' = 28-day compressive strength.

Typical value for the Ultimate Shrinkage can be used based on experience or the following equation can be used to estimate the ultimate shrinkage:

$$\varepsilon_{su} = C1 \cdot C2 \cdot \varepsilon_{ts} \qquad \text{Equation 3.6}$$

Where,

ε_{su} = ultimate shrinkage strain;

ε_{ts} = typical shrinkage strain;

600×10^{-6} for conventional PCC with fc' < 4,000 psi

650×10^{-6} for high-strength PCC with fc' > 4,000 psi

C1 = cement type factor; and C2 = curing condition factor.

In this study, correlated ultimate shrinkage strain was used based on the mixture properties such as, chosen cement type, water content, w/c ratio, aggregate type, curing method, etc. and the derived values are tabulated in Table 3.16.

Reversible shrinkage is the percentage of ultimate shrinkage that is reversible in the concrete upon rewetting. For reversible shrinkage, a recommended default value of 50% was used in this study.

Time to develop 50 percent of the ultimate shrinkage refers to the time taken in days to attain 50 percent of the ultimate shrinkage at the standard relative humidity conditions. The ACI-suggested default value of 35 days was used in this study.

In the MEPDG software, shrinkage strain ranges from 300 to 1000 micro-strains. In this study, software-predicted ultimate shrinkage was compared with these extreme values. The strain was also computed based on the tensile strength of concrete. The indirect tensile strength of concrete is determined based on AASHTO T198 or ASTM C496 protocol. Shrinkage strain is strongly related to the strength, which is a function of the water-cement ratio. Therefore, the computed shrinkage strain in this study was based on the relationship with the PCC tensile strength as shown in Table 3.18 (AASHTO 1993):

Table 3.18: Approximate Relationship between Shrinkage and Indirect Tensile Strength of PCC

Indirect tensile strength (psi)	***Shrinkage (in/in.)***
300 or less	0.0008
400	0.0006
500	0.00045
600	0.0003
700 or greater	0.0002

The project-specific tensile strength was computed based on the following equations:

$$f_t = 0.86 * S_c$$ Equation 3.7

$$f_t = 6.5\sqrt{fc'}$$ Equation 3.8

Where,

f_t = Tensile Strength (psi);

S_c = Modulus of rupture (psi); and

f_c' = 28-day PCC compressive strength (psi).

Equations 3.7 and 3.8 were based on the AASHTO and ACI recommendations, respectively. Project-specific computed values are summarized in Table 3.16.

In this study, derived (correlated) ultimate shrinkage strain, based on the mixture properties such as, chosen cement type, water content, w/c ratio, aggregate type and curing method etc., was also used. These values are also tabulated in Table 3.16.

Stabilized Base/Subbase Material - Chemically stabilized materials are used in the pavement base or subbase to achieve design properties. Stabilizing agents are either cementitious or lime. The stabilized materials group consists of lean concrete, cement stabilized, open graded cement stabilized, soil cement, lime-cement-flyash, and lime treated materials. Required design inputs for all these materials are the same for this design procedure. Layer properties can be further classified as layer material properties, strength properties, and thermal properties. For strength properties, the rigid pavement analysis requires the elastic or resilient modulus and Poisson’s ratio (NCHRP 2004).

Unit weight is the weight per unit volume of the stabilized base material. Poisson's ratio is the ratio of the lateral to longitudinal strain of the material and is an important required input for the structural analysis. Values between 0.15 and 0.2 are typical for the chemically stabilized materials.

The required modulus (elastic modulus [E] for lean concrete, cement stabilized, and open graded cement stabilized materials and resilient modulus [Mr] for soil-cement, lime-cement-flyash, and lime stabilized soils) is the 28-day modulus value and is a measure of the deformational characteristics of the material with applied load. This value can be determined either in the laboratory testing, correlations, or based on defaults. All projects in this study have stabilized bases. The inputs required for these bases were layer thickness, mean modulus of elasticity, unit weight of the material, Poisson's ratio, etc. Layer thickness ranged from 4 to 6 inches. The modulus of elasticity for the cement-treated bases (PCTB) and Bound Drainable bases (BDB) were 500,000 psi. The lean concrete base modulus was estimated as 2 million psi. All projects have 6-inch lime or fly-ash treated subgrade (LTSG/FATSG) with an input modulus of 50,000 psi. Project specific details are presented in Table 3.16.

The permeable and semi-permeable bases are used on top of the lime-treated subgrade. Permeable bases consist of open graded materials, and are constructed with high quality crushed stone. The gradation for the KDOT permeable base material CA-5 is shown in Figure 3.7. The semi-permeable base is also a granular base, similar to the permeable one. The gradation for such a base used by the Missouri Department of Transportation (MODOT) is also shown in Figure 3.7. For semi-permeable base, the percent materials passing No. 4 and 200 sieves are higher compared to the Kansas CA-

5 permeable base (Melhem et.al. 2003). Table 3.19 shows the required MEPDG inputs for different base types studied.

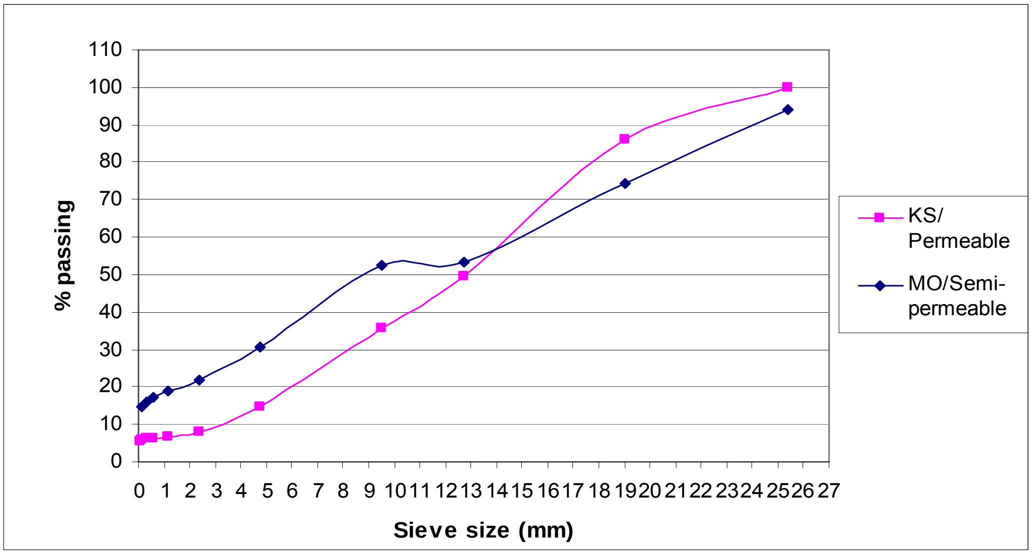


Figure 3.7: Gradation for permeable and semi-permeable base materials

Table 3.19: Inputs for Different Base Type

Input Parameters	Input Value
DGAB	
Type of materials	Crushed Stone
Derived Modulus (psi)	31,114
Plasticity index, PI	5
Percent passing # 200 sieve	14
% passing # 4 sieve	53
D_{60} (mm)	8
PERMEABLE BASE	
Type of materials	Crushed Stone
Derived Modulus (psi)	39,543
Plasticity index, PI	0
Percent passing # 200 sieve	5.6
% passing # 4 sieve	15
D_{60} (mm)	14
SEMI-PERMEABLE BASE	
D_{60} (mm)	14
Type of materials	A-1-b
Derived Modulus (psi)	37,417
Plasticity index, PI	0
Percent passing # 200 sieve	14.9
% passing # 4 sieve	52.4
D_{60} (mm)	11
BDB	
Base Thickness (in)	4
Base material unit wt. (pcf)	145
Base Modulus (psi)-Low	656,000
Base Modulus (psi)-High	1460,000
ATB	
Superpave Binder Grade	PG 64-22
Aggregate Gradation	
Cumulative % retained 3/4” Sieve	33
Cumulative % retained 3/8” Sieve	96
Cumulative % retained #4 Sieve	96
% Passing # 200 Sieve	1.8
Asphalt General	
Reference Temperature (ºF)	68
Effective Binder content (%)	2.0%
Air Void (%)	15%
Total Unit weight (pcf)	136
Poisson’s ratio	0.35

Subgrade - Subgrade materials are commonly termed as unbound materials. The input information is common for all unbound layers, regardless of whether it functions as a base or a subgrade layer in the pavement structure. For this design procedure, unbound granular materials are defined using the AASHTO classification system or Unified Soil Classification (USC) system. In addition to that, unbound base can also be categorized as crushed stone, crushed gravel, river gravel, permeable aggregate, and cold recycled asphalt. Subgrade materials are defined using both the AASHTO and USC classifications and cover the entire range of soil classifications available under both systems.

The material parameters required for the unbound materials (both granular and subgrade) may be classified into three major groups:

- Pavement response model material inputs.
- EICM material inputs.
- Other material properties

These inputs are, however, grouped into two property pages of the unbound layers screen and are identified as "Strength properties" and "ICM" properties. The inputs provided on the strength properties page, and appropriate inputs on the ICM property page would be essential to make seasonal adjustments to the strength values for seasonal changes. The user also has the option of disregarding the ICM page and making "user-input" seasonal strength values, or specifying that the program disregard seasonal changes and use only the representative values provided on the strength screen.

The strength inputs for the unbound layers can be made in three hierarchical levels. At Level 1, resilient modulus values for the unbound granular materials, subgrade, and bedrock are determined from cyclic triaxial tests on prepared representative samples.

Level 2 analyses requires the use of resilient modulus, M_r. Level 2 inputs in MEPDG use general correlations between the soil index and the strength properties and the resilient modulus to estimate M_r. The relationships could be direct or indirect. For the indirect relationships, the material property is first related to CBR and then CBR is related to M_r. MEPDG allows the user to use either of the following soil indices to estimate M_r from the aforementioned correlation:

- CBR
- R-value
- Layer coefficient
- Penetration from DCP
- Based up on Plasticity Index and Gradation

For level 2, the MEPDG software allows users the following two options:

- Input a representative value of M_r or other soil indices, and use EICM to adjust it for seasonal climate effects (i.e., the effect of freezing, thawing, and so on).
- Input M_r or other soil indices for each month (season) of the year (total of 12 months).

Level 3 inputs simply require a default value for the resilient modulus of the unbound material. For this level, only a typical representative M_r value is required at optimum moisture content. EICM is used to modify the representative M_r for the climatic effect.

For ICM properties, inputs provided by the unbound layers are used by the EICM model of the MEPDG software in predicting the temperature and moisture profile throughout the pavement structure. Key inputs include gradation, Atterberg limits, and hydraulic conductivity. Regardless of the input level chosen for the unbound layer, the input parameters required are the same.

The plasticity index, PI, of a soil is the numerical difference between the liquid limit and the plastic limit of the soil, and indicates the magnitude of the range of the moisture contents over which the soil is in a plastic condition. The AASHTO test method used for determining PI is AASHTO T-90.

The sieve analysis is performed to determine the particle size distribution of unbound granular and subgrade materials and is conducted following AASHTO T27. The required distribution includes percentage of particles passing US No.200 and No.4 sieves, and the diameter of the sieve in mm at which 60 percent of the soil material passes (D60).

In this study, natural subgrade modulus was calculated by the MEPDG software from a correlation equation involving the project-specific plasticity index and soil gradation. The correlations are shown in Equation 3.9 and 3.10. Project-specific inputs are shown in Table 3.16 (NCHRP 2004).

$$CBR = \frac{75}{1 + 0.728(wPI)}$$ Equation 3.9

$$M_r = 2555(CBR)^{0.64}$$ Equation 3.10

Where,

wPI= P200* PI;

P200 = Percent passing No. 200 sieve size;

PI = Plasticity index, percent;

CBR = California Bearing Ratio, percent; and

M_r = Resilient Modulus (psi).

The parameters maximum dry unit weight (MDD), specific gravity of solids (Gs), saturated hydraulic conductivity, optimum gravimetric moisture content (OMC), and calculated degree of saturation can be either input by the user or calculated internally by the MEPDG software. These parameters are used by the EICM model in predicting the moisture profile through the pavement structure. In this study, these parameters were derived from the MEPDG level 3 default values or determined based on correlations. Table 3.16 lists these values.

MEPDG also has the option for indicating type of compaction achieved for the unbound layer during the construction process. MEPDG internally makes adjustments to the coefficient of lateral pressure to account for the level of compaction provided to the layer and this, in turn, influences the deformational characteristics undergone by the layer for the same level of applied loads. In this study, compacted phase was indicated for the top 12 inches of the natural subgrade material and uncompacted phase was chosen for the rest of the depth of subgrade soil.

CHAPTER 4 - DESIGN AND SENSITIVITY ANALYSIS

This chapter describes the Mechanistic-Empirical Pavement Design Guide (MEPDG) design analysis of eight in-service JPCP in Kansas, and sensitivity analysis of the factors that significantly affect the pavement distresses.

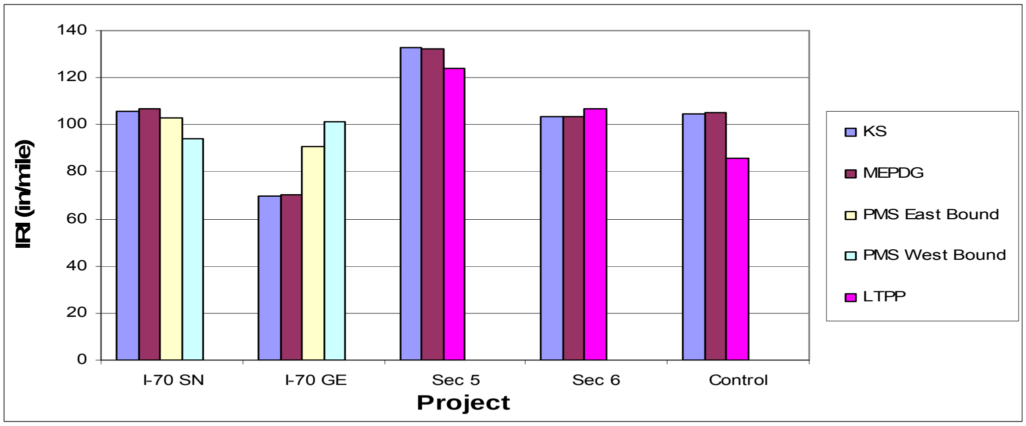
4.1 Prediction and Comparison of Distresses from Design Analysis

As mentioned earlier, key rigid pavement distresses predicted for JPCP from the MEPDG analysis are IRI, faulting, and percent slabs cracked.

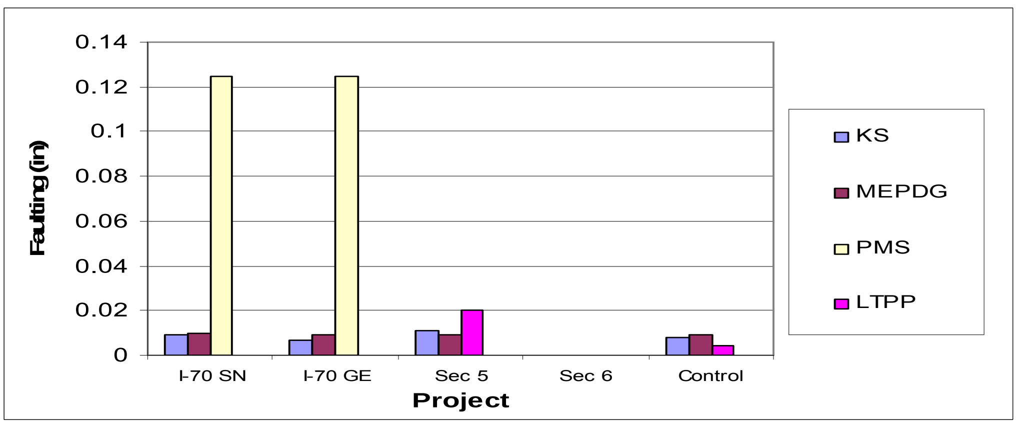
4.1.1 Smoothness or IRI

In this study, MEPDG predicted IRI for the JPCP sections were compared with the KDOT-measured and LTPP DataPave (online database) values for 2003. MEPDG prediction was done with both default and Kansas-specific traffic inputs (truck traffic distribution and axle load spectra). The average IRI, obtained from the left and right wheel path measurements on the driving lane, was used in the comparison. The profile survey for the KDOT Pavement Management System was done on both eastbound and westbound directions for the I-70 Geary and I-70 Shawnee county projects, and on northbound and southbound directions for the K-7 Johnson County project. For the SPS-2 sections, measured values were obtained from the LTPP database. Figure 4.1(a) and 4.2(a) show the comparison between the predicted and the measured IRI values for 2003 for all projects. The values are also summarized in Table 4.1. MEPDG-predicted IRI's with default and Kansas-specific traffic inputs are similar on all projects except on the K-7 Johnson County project. It has been previously shown that truck traffic distribution on this section (urban arterial and others functional class) is completely different.

(a) IRI



(b) Faulting



(c) % Slabs Cracked

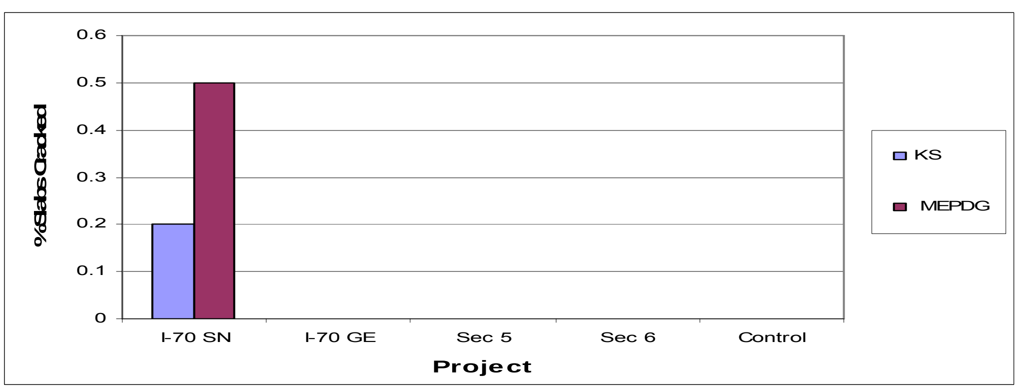
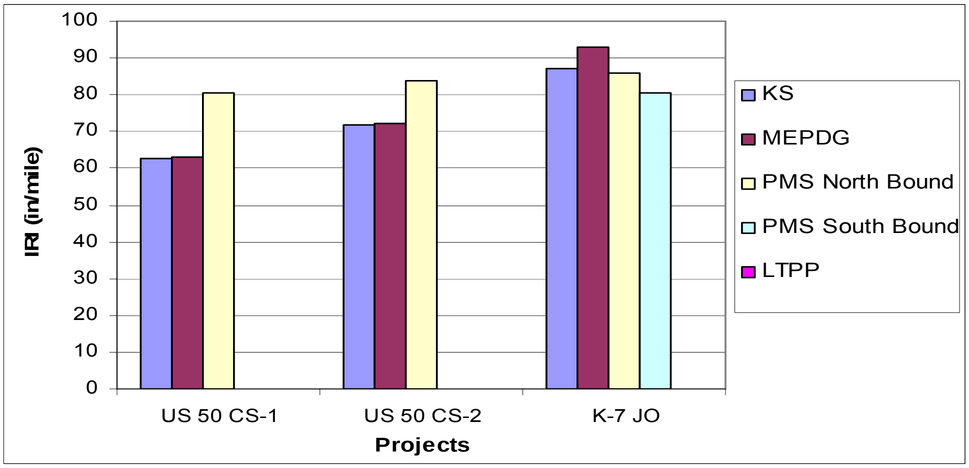
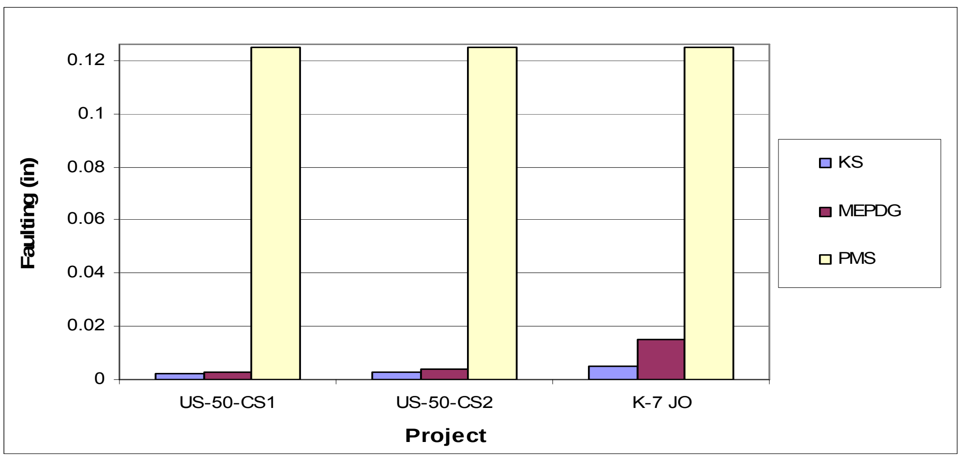


Figure 4.1: Predicted and measured JPCP distresses on Interstate sections

(a) IRI



(b) Faulting



(c) % Slabs Cracked

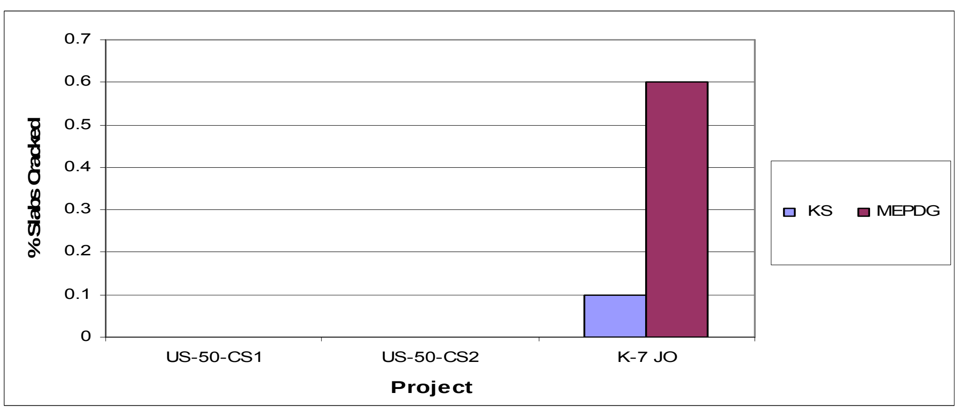


Figure 4.2: Predicted and measured JPCP distresses on Non-Interstate sections

The predicted IRI values for all Interstate sections are fairly similar to the measured values summarized in Table 4.2, except for the I-70 Geary County and SPS-2 Control Section projects. With Kansas- generated traffic input, SPS Section 5 has the highest predicted IRI of 133 in/mi and the I-70 Geary county project has the lowest. The predicted IRI for the Geary county project is 70 in/mi compared to the measured values of 91 in/mi and 102 in/mi for the eastbound and westbound directions, respectively. For the non-interstate sections, shown in Figure 4.2(a), NOS-measured IRI values are similar to the predicted values for the K-7 Johnson County project. For K-7 Johnson County project, the MEPDG predicted IRI is 87 inches/mile and 93 in/mi for the Kansas-specific traffic input and MEPDG default traffic input, respectively. These values are similar to the measured values of 86 inches/mile and 81 inches/mile for the northbound and southbound directions, respectively. For the non-interstate sections, NOS-measured IRI values are higher than the predicted values for both Chase county projects.

4.1.2 Faulting

Figures 4.1(b) and 4.2(b) show the comparison between the predicted faulting and the NOS or LTPP-measured values. Tables 4.1 and 4.2 tabulate the values. As shown in Figure 4.1(b), no faulting was observed for the SPS-2 Section 6 that has a widened lane of 14 foot width. Good agreement was also observed for the other SPS-2 section. SPS-2 Section 5 has higher measured faulting of 0.02 inches compared to the Kansas-specific and default traffic values. For the KDOT projects, some discrepancies were observed between the predicted and the measured faulting. However, both measured and predicted values in 2003 were negligible for all practical purposes. For example, with the Kansas-specific traffic input, the K-7 Johnson County project was projected to show faulting of 0.005 inch in 2003 as shown in Figure 4.2(b). The discrepancies between the NOS-measured and MEPDG-predicted faulting at a few locations were partly due to the way faulting is interpreted in the NOS survey. During NOS reporting, measured faulting is coded as F1, F2 or F3 depending upon the severity of faulting. F1 describes the faulting of greater than 0.125 inches but less than 0.25 inches and this is the only severity observed at a few locations on some projects. Also, in NOS faulting is rated on a per mile basis and computed from the profile elevation data. No numerical value of faulting is reported by NOS. Thus the MEPDG analysis showed minimal faulting and it was confirmed by actual observation.

Table 4.1: Comparison of Predicted Response

Project	IRI (in/mi)		Faulting (in)		% Slabs Cracked	
	MEPDG Default Traffic	Kansas Traffic	MEPDG Default Traffic	Kansas Traffic	MEPDG Default Traffic	Kansas Traffic
K-2611-01, I-70 Geary	71	70	0.009	0.007	0	0
K-3344-01, I-70	107	106	0.01	0.009	0.5	0.2
20-0208, SPS-2 (Sec 5)	132	133	0.009	0.011	0	0
20-0207, SPS-2 (Sec 6)	104	104	0	0	0	0
20-0259, SPS-2 Control	105	105	0.009	0.008	0	0
K-3382-01, K-7, Johnson	93	87	0.015	0.005	0.6	0.1
K-3216-01, US-50	623	63	0.003	0.002	0	0
K-3217-01, US-50	72	72	0.004	0.003	0	0

Table 4.2: Measured Responses

Project	IRI (in/mi)		Faulting (in)
K-2611-01, I-70 Geary	91 (EB)	101 (WB)	F1 (0.125 in)
K-3344-01, I-70 Shawnee	103 (EB)	94 (WB)	F1 (0.125 in)
20-0208, SPS-2 (Sec 5)	124.2 (LTPP)	-	0.02
20-0207, SPS-2 (Sec 6)	107.0 (LTPP)	-	0
20-0259, SPS-2 Control	86.0 (LTPP)	-	0.004
K-3216-01, US-50,Chase	81 (NB)	-	F1 (0.125 in)
K-3217-01, US-50 Chase	84 (NB)	-	F1 (0.125 in)
K-3382-01, K-7 Johnson	86 (NB)	81 (SB)	F1 (0.125 in)

4.1.3 Percent Slabs Cracked

One of the structural distresses considered for JPCP design in MEPDG is fatigue-related transverse cracking of the PCC slabs. Transverse cracking can initiate either at the top surface of the PCC slab and propagate downward (top-down cracking) or vice versa (bottom-up cracking) depending on the loading and environmental conditions at the project site, material properties, design features, and conditions during construction. This parameter indicates the percentage of total slabs that showed transverse cracking. Figures 4.1I and 4.2I show the MEPDG-predicted percent slabs cracked values that are tabulated in Table 4.1. With the Kansas-specific traffic input, only I-70 Shawnee and K-7 Johnson county sections showed some insignificant amount of cracking. The percent slabs cracked values for these projects are 0.2% and 0.1%, respectively. The predictions are insignificantly higher for the Shawnee County (0.5%) and K-7 Johnson County (0.6%) projects with the MEPDG default traffic input. It has been previously observed that the MEPDG-default traffic input has higher percentage of trucks distributed in the higher axle load categories compared to the Kansas input.

No measured cracking values were available from the Kansas NOS condition survey report and LTPP database for comparison with the MEPDG-predicted cracking. In NOS, no cracking survey is done on rigid pavements. In the LTPP survey, cracking is measured in terms of longitudinal and transverse crack lengths that cannot be interpreted as percent slabs cracked. Of course, an average value can be computed. It is to be noted that none of the SPS-2 sections in this study showed any cracking up to 2003 in the MEPDG analysis.

4.2 Sensitivity Analysis

Previous research has shown that the design features that influence JPCP performance include layer thicknesses, joint spacing, joint and load transfer design (Owusu-Antwi et al.1998; Khazanovich et al. 1998). The strength of Portland cement concrete (PCC) mix is also a basic design factor that is often controlled by the designer and is interrelated with the PCC slab thickness. Nantung et al. (2005) and Coree et al. (2005) have shown that PCC compressive strength and slab thickness have significant effects on the predicted distresses. In this part of the study, the sensitivity of the predicted performance parameters in the MEPDG analysis toward the material input (design and construction) parameters has been done for all projects except the projects in Chase County (these projects have shown localized failures, some premature distresses due to erosion of inadequately lime-treated subgrade). The following input parameters were varied:

1. Traffic (AADT, % Truck and Truck Type)

2. Material (PCC Compressive Strength, Coeff. Of Thermal Expansion, Shrinkage Strain, PCC-Zero Stress Temperature, and Soil Class)

3. Design and Construction Features (PCC Thickness, Dowel Diameter, Dowel Spacing, Tied/Untied Shoulder, Widen Lane, Curing Type, Granular and Stabilized Base type, etc.)

4. Alternative Design

The JPCP distresses were predicted by the NCHRP MEPDG analysis for a 20-year design period. All other input parameters were project specific as shown in Table 3.16. Typical JPCP distresses-IRI, faulting, and percent slabs cracked, were calculated and compared at various levels of the input parameter chosen.

4.2.1 Traffic Input

In this part of the study, the sensitivity of the predicted performance parameters in the MEPDG analysis toward selected traffic input parameters was studied. The following input parameters were varied at the levels shown and the predicted IRI, faulting, and percent slabs cracked were calculated.

1. AADT [2,080(Low); 12,562 (Medium); 36,000 (High)]

2. Truck (%) [5 (Low); 23.2 (Medium); 47 (High)]

3. Class 9 Truck Type (MEPDG default [74%]; 40%; 50%; Kansas [variable])

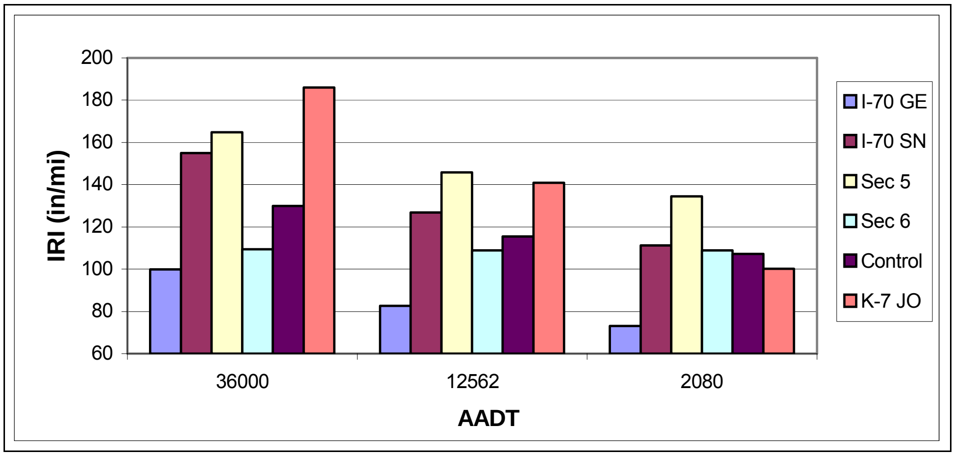
A previous study has shown that the IRI, faulting and percent slabs cracked (Coree et al. 2005) are sensitive to the variation in AADTT.

AADT - The Annual Average Daily Traffic (AADT) was varied at three levels with a constant truck percentage (23.2%), based on the range of AADT of the projects in this study. Table 4.3 lists the predicted IRI values. Figure 4.3(a) shows that with increasing AADT, IRI increases significantly. The increase is most significant for the I-70 Shawnee County and K-7 Johnson County projects. For the three levels of AADT chosen, the predicted IRI for the I-70 Shawnee County project increased from 111 inches/mile to 155 inches/mile. The effect on the K-7 Johnson County project is even more pronounced. The IRI increased from 100 inches/mile to 186 inches/mile. As mentioned earlier, these projects are different from other projects because the I-70 Shawnee County project has the lowest 28-day modulus of rupture (473 psi) and the K-7 Johnson County project has the lowest PCC slab thickness (9 inches). The effect of higher AADT is more pronounced on the PCC pavements with thinner slabs or lower strength.

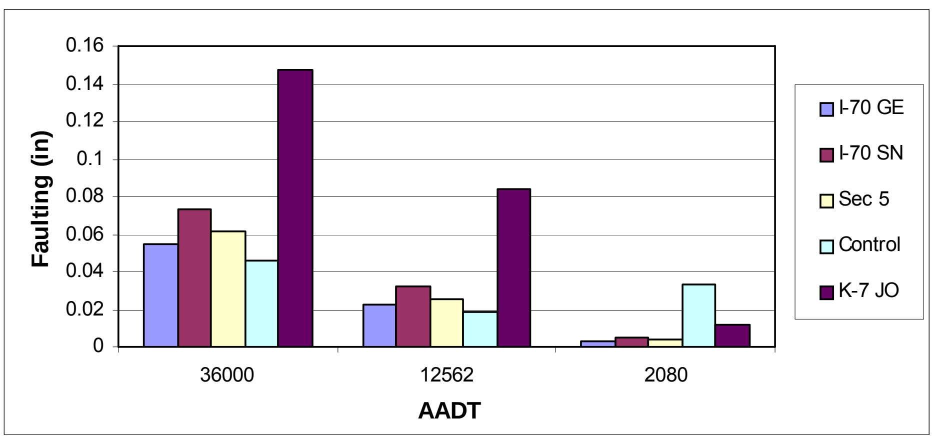
Figure 4.3(b) shows that the faulting on the K-7 Johnson County project is markedly affected by the increase in AADT. Faulting on the other sections is tolerable. Faulting on the I-70 Shawnee County project is slightly higher, though the values are negligible for all practical purposes. It appears that at higher AADT, faulting becomes a function of PCC slab thickness and strength.

Figure 4.3(c) shows that only two projects, I-70 Shawnee County and K-7 Johnson County, have predicted cracking. Cracking increases dramatically on both projects when AADT increases. In fact, the K-7 Johnson County project fails at the highest level of AADT. It appears that cracking is most sensitive to the increase in traffic level.

(a) IRI



(b) Faulting



(c) % Slabs Cracked

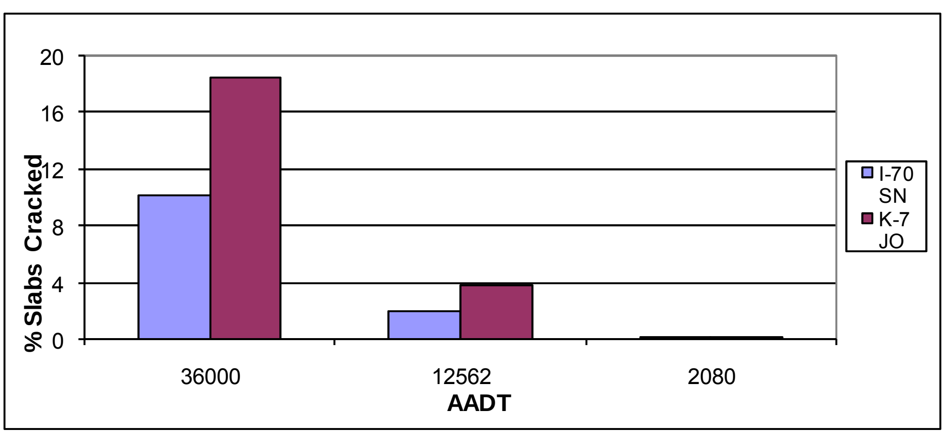


Figure 4.3: Predicted JPCP distresses at varying levels of AADT

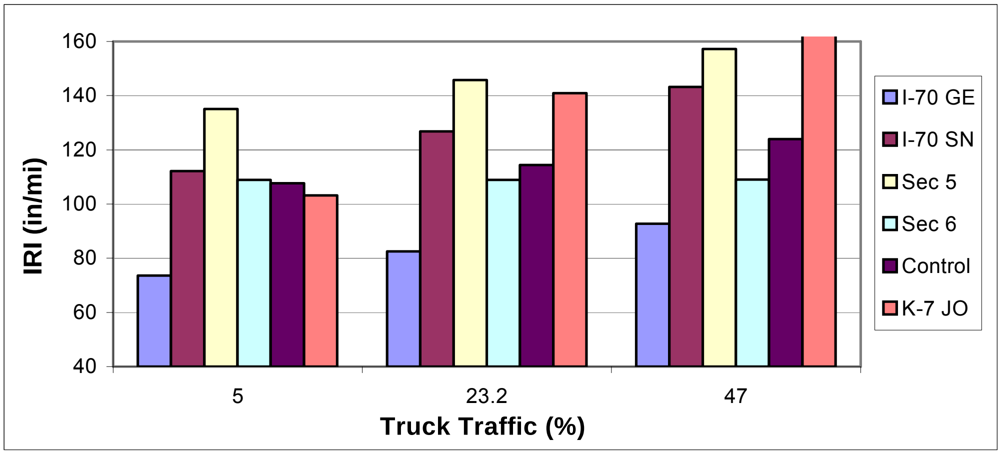
Table 4.3: Effect of Traffic (AADT) on Predicted Responses

Project	IRI (in/mi)			Faulting (in)			% Slabs Cracked		
AADT	36000	12562	2080	36000	12562	2080	36000	12562	2080
I-70 GE	100	83	73	0.055	0.022	0.003	0	0	0
I-70 SN	155	127	111.2	0.073	0.032	0.005	10.1	1.9	0.1
Sec 5	165	146	134.4	0.061	0.025	0.004	0	0	0
Sec 6	109	109	108.9	0.001	0	0	0	0	0
Control	130	116	107.2	0.046	0.019	0.033	0	0	0
K-7 JO	186	141	100.1	0.147	0.084	0.012	18.5	3.7	0.1

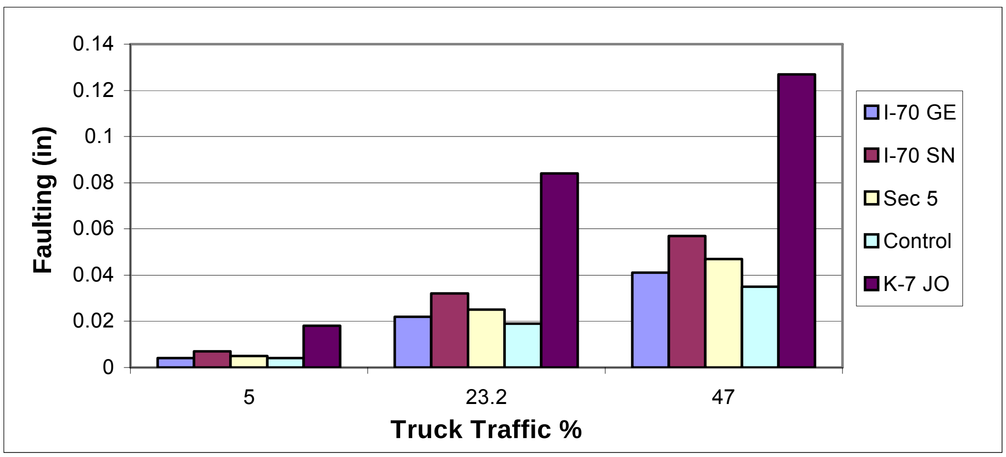
Truck Traffic - Figure 4.4 shows the variation of predicted distresses with different truck percentages at a constant AADT of 12,562, the average AADT level in this study. Figure 4.4(a) shows that with increasing percentage of trucks, IRI increases significantly for the SPS-2 Section 5, I-70 Shawnee County and K-7 Johnson County projects. SPS-2 Section 5 has a very high cement factor (862 lbs/cubic yd) and high modulus of rupture. Trend is similar for faulting as shown in Figures 4.4(b). Figure 4.4(c) shows that the I-70 Shawnee County and K-7 Johnson County projects show significant increase in cracking with higher truck percentages. No other section showed any cracking.

Truck Type - Since FHWA Class 9 trucks are the predominant truck type in Kansas, a sensitivity analysis was done with respect to varying percentages of this truck type. The MEPDG default percentage of Class 9 truck (74%), 50%, 40% and various percentages corresponding to different functional classes of routes in this study were used. The percentages varied from 75% for the I-70 projects (Interstate, Rural) to 25% for the K-7.

(a) IRI



(b) Faulting



(c) % Slabs cracked

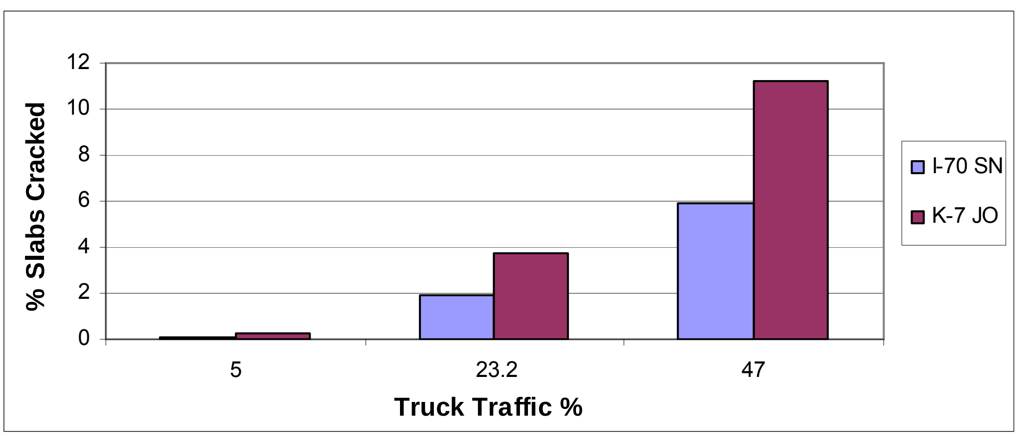
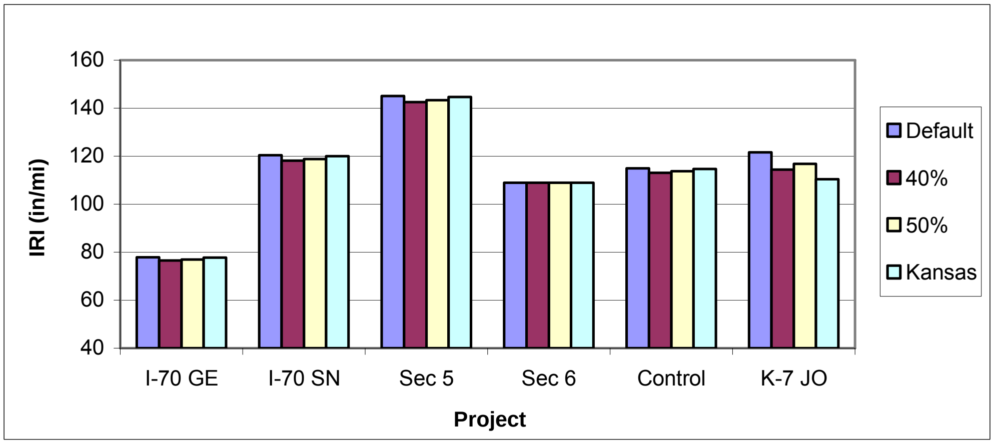
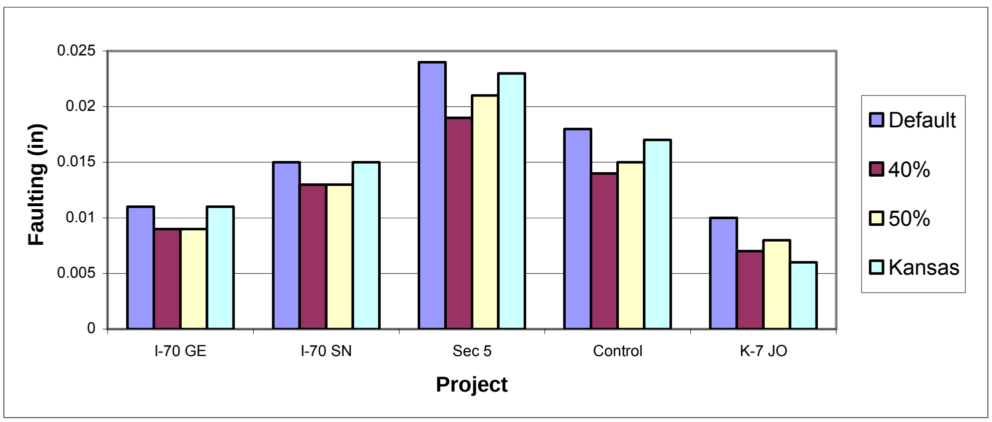


Figure 4.4: Predicted JPCP distresses at varying levels of truck traffic

(a) IRI



(b) Faulting



(c) % Slabs Cracked

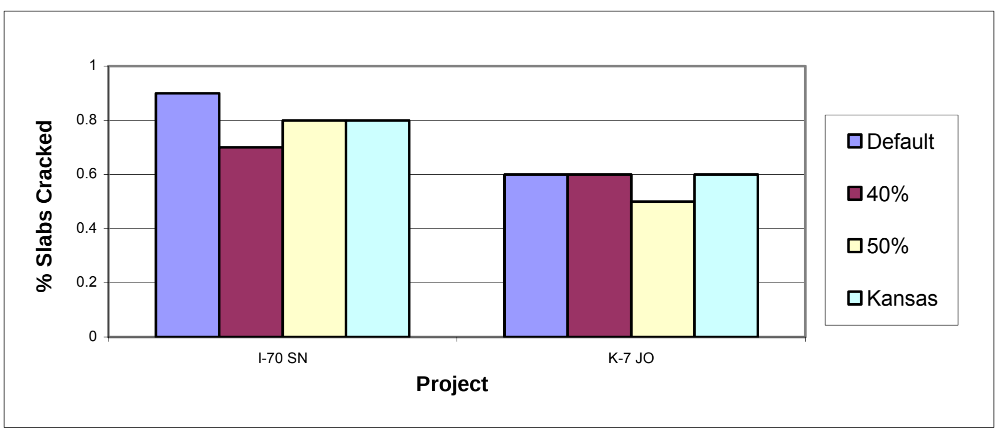


Figure 4.5: Predicted JPCP distresses at varying levels of class 9 truck type

Johnson County project (Urban Arterials, Others). The results show that the IRI values are fairly insensitive to the type of trucks. The faulting values and cracking values are relatively insensitive too.

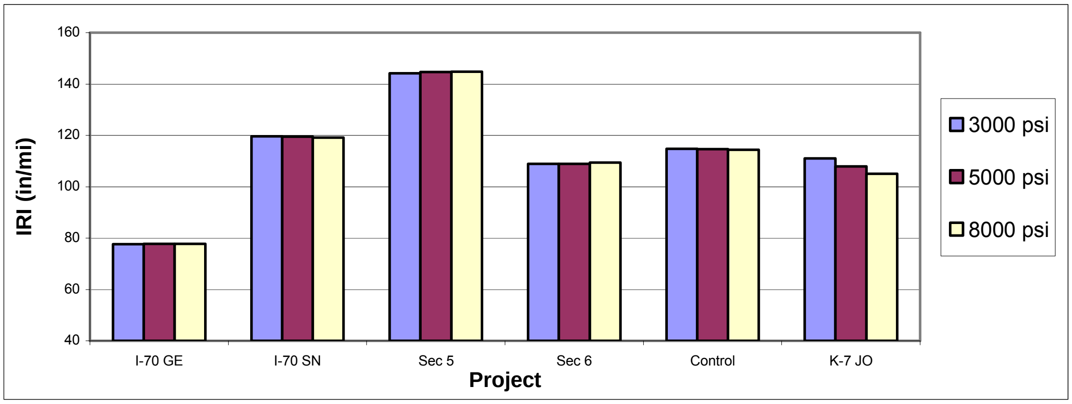
4.2.2 Material

Compressive Strength - The MEPDG-predicted IRI values for the sections were compared at three levels of compressive strength- low (3,000 psi), average (5,000 psi), and high (8,000 psi). Figure 4.6 (a) shows the results. In general, the compressive strength does not affect predicted IRI. There is a slight effect for the K-7 Johnson County project. That section has the thinnest PCC slab among all sections. When the compressive strength was increased from 3,000 psi to 5,000 psi, IRI decreased from 111 inches/mile to 108 inches/mile. When the strength was increased to 8,000 psi, IRI was 105 inches/mile. These decreases are negligible for all practical purposes.

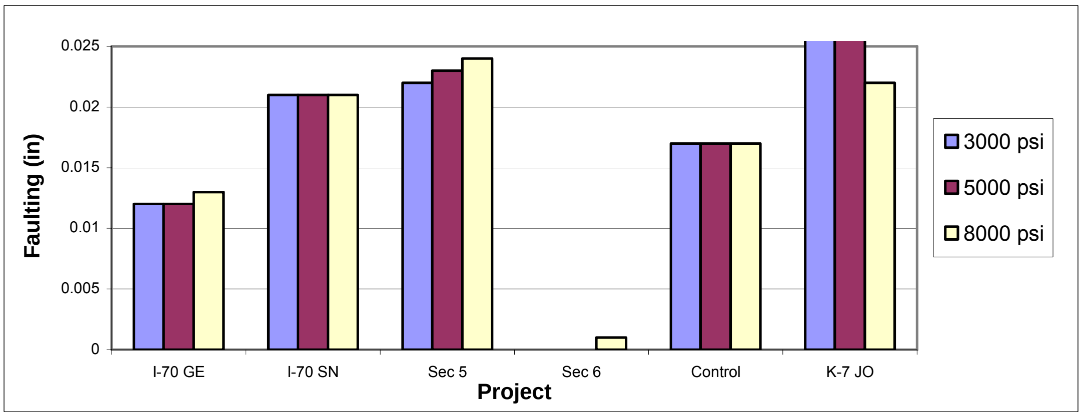
Figure 4.6(b) shows the predicted faulting on all sections corresponding to three levels of compressive strength. Almost no changes in faulting values were observed for all projects. The predicted faulting values were also negligible for all practical purposes. It appears that faulting is fairly insensitive to strength.

Figure 4.6(c) illustrates the effect of compressive strength on predicted percent slabs cracked. Although very small amounts of cracking were observed on almost all projects at the 3,000 psi level, no cracking was observed when the strength was increased to 5,000 psi. The biggest change was observed for K-7, Johnson County- the project that had the lowest PCC slab thickness.

(a) IRI



(b) Faulting



(c)% Slabs Cracked

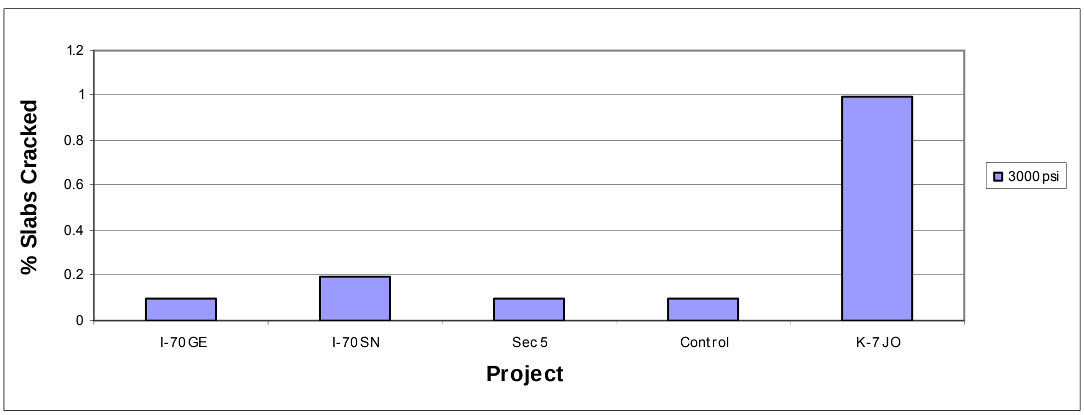


Figure 4.6: Predicted JPCP distresses for varying PCC strength

Coefficient of Thermal Expansion (CTE) - The MEPDG-predicted IRI for the sections were compared at three levels of PCC CTE input based on the TP-60 test results on the cores taken from the Kansas PCC pavements in the Long Term Pavement Performance (LTPP) program. Three levels of the PCC CTE input were, $4.3\text{x}10^{-6}$/ºF (average of the LTPP TP-60 highest 10% test results), $5.5\text{x}10^{-6}$/ºF (TP-60 test results from a recently built project), and $6.5\text{x}10^{-6}$/ºF (average of the LTPP TP-60 highest 10% test results).

Table 4.4 summarizes the results. Figure 4.7(a) shows the results. In general, higher PCC CTE would result in higher IRI. The effect is most pronounced on the I-70 Shawnee County and K-7 Johnson County projects. When the PCC CTE value increased from $4.33\text{x}10^{-6}$/ºF to 6.5 $\text{x}10^{-6}$/ºF, the predicted IRI increased from 114 inches/mile to 135 inches/mile for the I-70 Shawnee County project. For the K-7 Johnson County project, the predicted IRI increase was similar (22 inches/mile- from 101 to 123 inches/mile). It is to be noted that these projects are different from others because the I-70 Shawnee County project has the lowest 28-day modulus of rupture (473 psi) and the K-7 Johnson County project has the lowest PCC slab thickness (9 inches). It appears the effect of PCC CTE input is more pronounced on JPCP with thinner slab or lower strength. It also is to be noted that PCC CTE value variation does not have any effect on the predicted IRI for the SPS-2 Section 6. That section has a widened lane of 14 feet with tied PCC shoulder. Thus, variation in the PCC CTE values studied in this project does not affect the predicted IRI for a JPCP with widened lane and tied PCC shoulder.

Figure 4.7(b) shows the predicted faulting on all sections corresponding to three levels of PCC CTE input value. No faulting was observed for the SPS-2 Section-6 which has a widened lane of 14 feet. The effect of varying PCC CTE is most significant for the I-70 Shawnee County project, SPS-2 Section 5, and the K-7 Johnson County project. Section 5 has a very high cement factor (862 lbs/yd^3). It appears that the combination of high cement factor and higher PCC CTE would result in higher faulting. However, the predicted faulting values are negligible for all practical purposes.

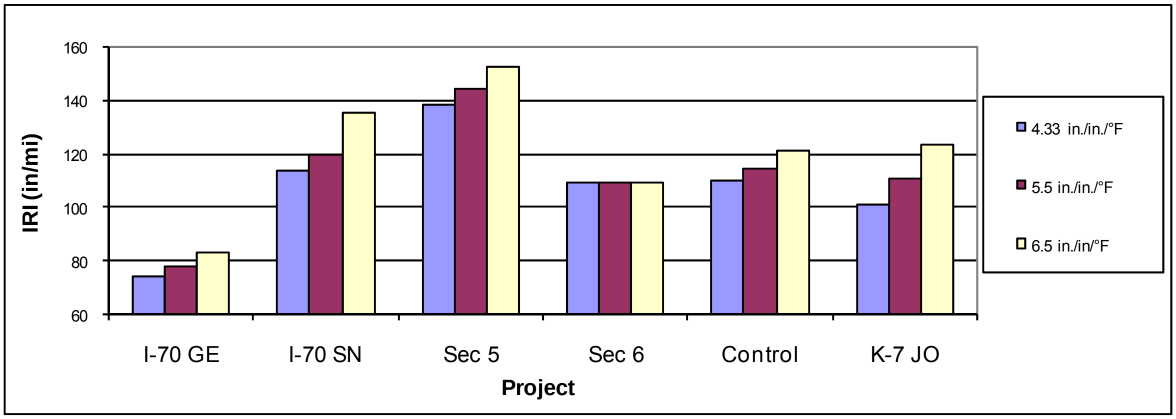
Table 4.4: Comparison of Predicted Response Corresponding to Different Coefficient of Thermal Expansion

Project	IRI (in/mi)			Faulting (in)			% Slabs Cracked		
	4.33 x10^{-6}/ºF	5.5 x10^{-6}/ºF	6.5 x10^{-6}/ºF	4.33 x10^{-6}/ºF	5.5 x10^{-6}/ºF	6.5 x10^{-6}/ºF	4.33 x10^{-6}/ºF	5.5 x10^{-6}/ºF	6.5 x10^{-6}/ºF
I-70 GE	74	78	83	0.006	0.012	0.022	0	0	0
I-70 SN	114	120	135	0.01	0.021	0.035	0	0.8	10.3
Sec 5	138	145	152	0.011	0.023	0.038	0	0	0
Sec 6	109	109	109	0	0	0	0	0	0
Control	110	115	121	0.008	0.017	0.029	0	0	0
K-7 JO	101	110	123	0.014	0.031	0.053	0.2	0.6	1.9

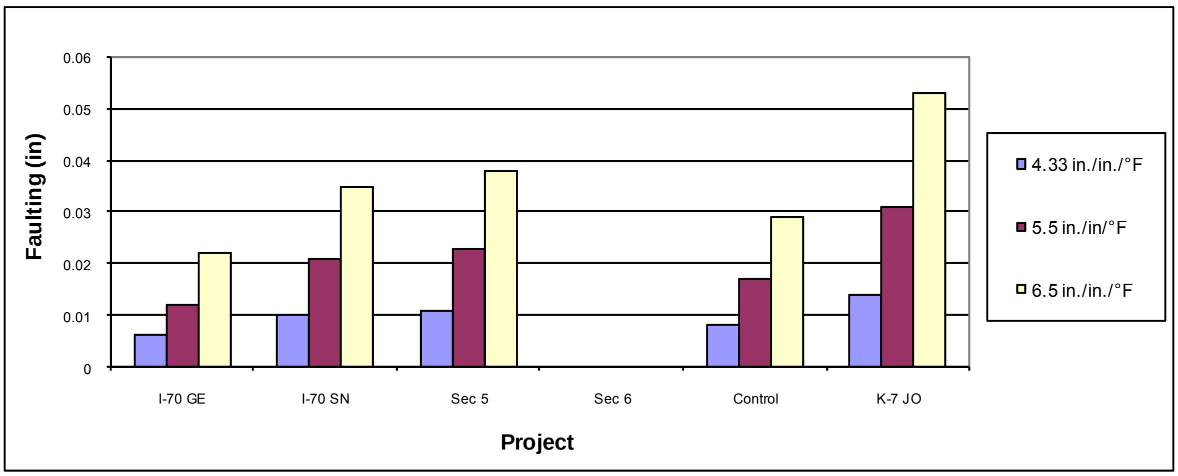
Figure 4.7(c) illustrates the effect of PCC CTE input on predicted percent slabs cracked. Only two projects, I-70 Shawnee County and K-7 Johnson County, appear to be affected by this input. The I-70 Shawnee County project is most severely affected although there was no cracking on this project at the lowest PCC CTE input (4.33 x10^{-6}/^{o}F). Fifty percent increase in this parameter resulted in 10% slabs cracked for this project. Also, as mentioned earlier, this project has the lowest concrete modulus of rupture among all projects studied. Although the amount of cracking is much lower for the K-7 Johnson County project, the increase is also pronounced. For the lowest PCC

CTE value of 4.33 $\times 10^{-6}/^{o}F$, there was only 0.2% slabs cracked. For the highest PCC CTE input, 6.5 $\times 10^{-6}/^{o}F$, cracking increased to 2% – a tenfold increase due to 50% increase in the PCC CTE value. This parameter was found to be extremely sensitive in others studies too (Beam 2003; Coree et al 2005; Nantung et al. 2005).

(a) IRI



(b) Faulting



(c) % Slabs Cracked

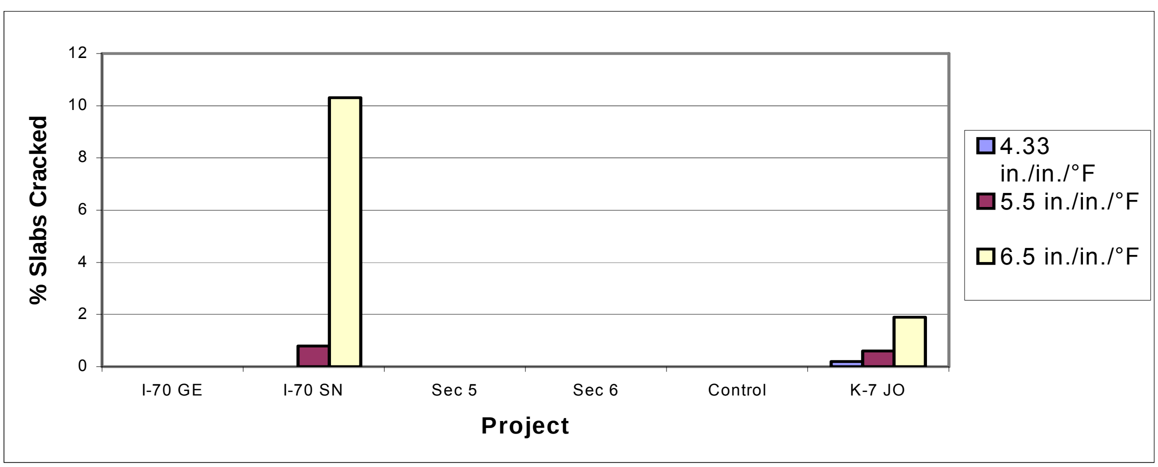


Figure 4.7: Predicted JPCP distresses for different CTE input

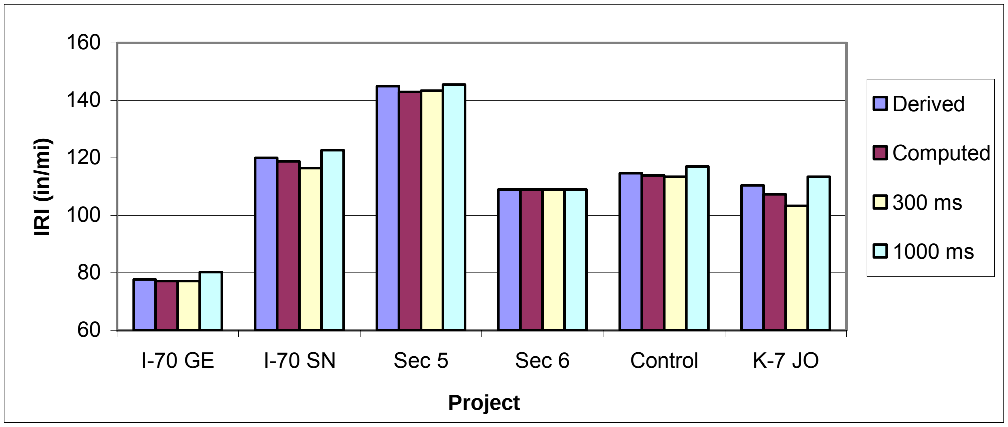
Shrinkage Strain - The MEPDG-predicted 20-year IRI values for the sections were compared at four levels of shrinkage strain – derived, computed, low (300 μm), and high (1,000 μm). Figure 4.8 (a) shows the results. In general, the shrinkage does not greatly affect IRI. The derived, computed, and lower shrinkage strain levels tend to predict similar IRI. There is a slight effect for the K-7 Johnson County project. When the shrinkage strain increased from 300 μm to 1,000 μm, IRI increased from 103 inches/mile to 114 inches/mile. According to the MEPDG algorithm, higher shrinkage strain results in higher faulting. That, in turn, is responsible for increased roughness.

Figure 4.8(b) shows the predicted faulting on all sections corresponding to four levels of shrinkage strain. Higher shrinkage strain results in higher faulting. The effect is most pronounced for the I-70 Shawnee and the K-7 Johnson County projects. When the shrinkage strain increased from 300 μm to 1,000 μm, faulting almost doubled though the faulting values are negligible for all practical purposes. Nevertheless, the faulting is very sensitive to the shrinkage strain.

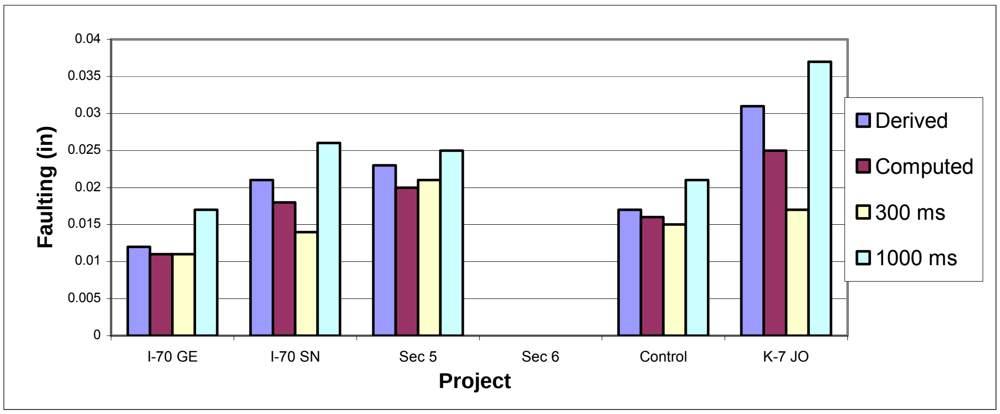
Figure 4.8(c) illustrates the effect of shrinkage strain on predicted percent slabs cracked for the JPCP projects in this study. Only two projects showed cracking, and the effect of shrinkage strain is almost negligible. The I-70 Shawnee County project showed a slight increase in cracking with higher strain. Cracking appears to be fairly insensitive to the shrinkage strain.

This parameter was found to be insensitive in previous studies (Beam 2003; Coree et al 2005; Nantung et al. 2005).

(a) IRI



(b) Faulting



(c) % Slabs Cracked

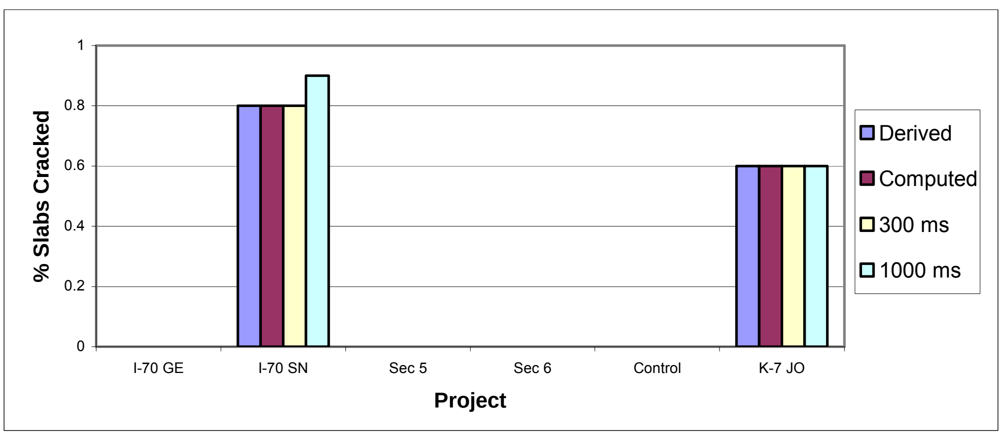


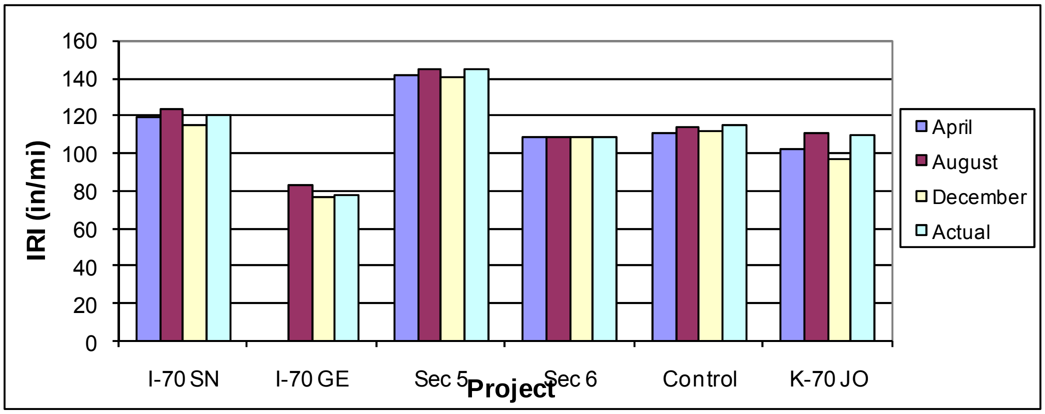
Figure 4.8: Predicted JPCP distresses for different shrinkage strain input

PCC Zero-Stress Temperature - Figure 4.9 shows the predicted JPCP distresses by NCHRP MEPDG corresponding to three probable and one actual construction months for the projects in this study. Three probable construction months were chosen based on an analysis of the mean monthly temperature (MMT) values obtained from the weather database for the years of construction of these projects. The months of April, August, and December were selected to represent high and low MMT or Tz values. October was also chosen but later disregarded since MMT values for this month are very similar to those in April. Actual construction months for the projects, shown in Table 3.1, are as follows: I-70 Geary County: November 1990; I-70 Shawnee County: October 1993; SPS-2's: July 1992; and K-7 Johnson County: September 1995.

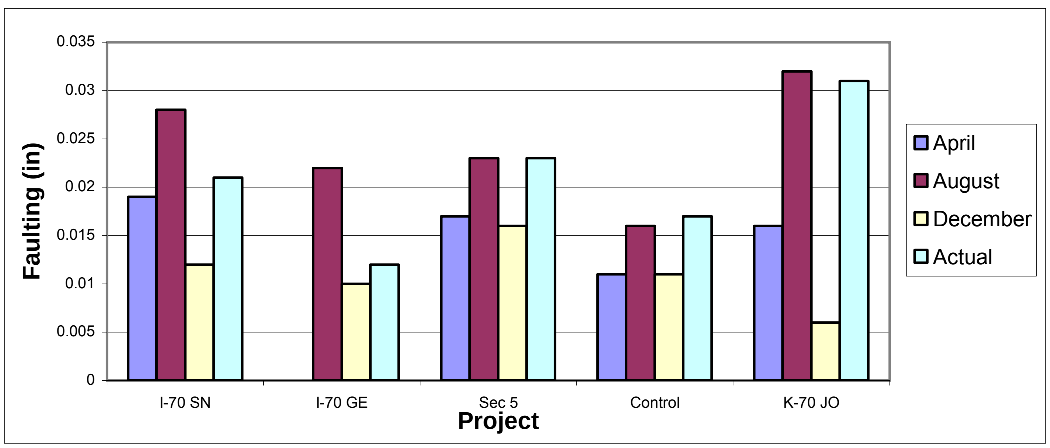
Figure 4.9(a) shows that construction month/Tz does not greatly affect the predicted IRI. For almost all projects, JPCP's constructed in April show slightly lower IRI. However, the initial IRI results in Table 3.1 show that the SPS sections, built in July 1992, have the highest initial or as-constructed IRI among all sections.

Construction months/Tz tends to make the biggest difference in predicted faulting as shown in Figure 4.9(b). It is clear that the pavements constructed in August (with highest MMT/Tz) will have much higher faulting than those constructed in a temperate climate in April or even in at a cold temperature in December. The effect is very pronounced on I-70 Shawnee County and K-7 Johnson County - the projects with lower PCC strength and thinner PCC slab, respectively. The only project which is not affected by this parameter is SPS-2 Section 6. This pavement has a widened lane and that appears to address the higher faulting effect due to construction during the month with high MMT.

(a) IRI



(b) Faulting



(c) % Slabs Cracked

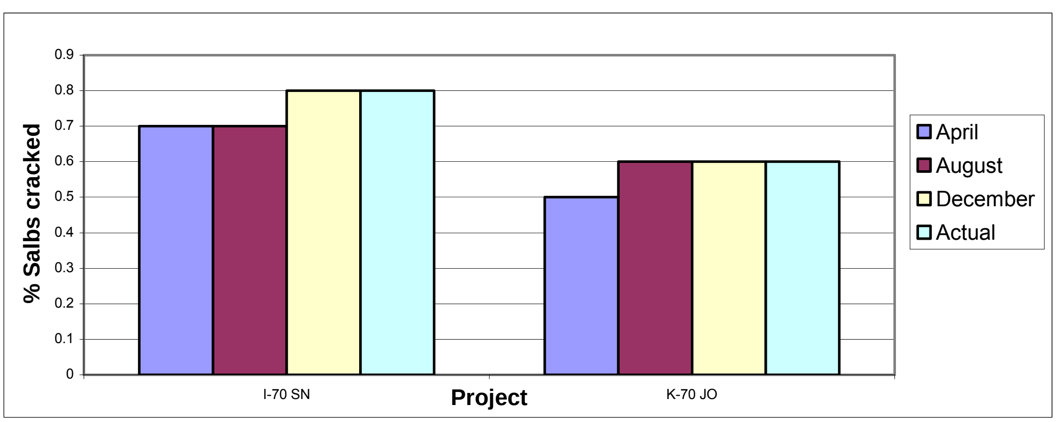


Figure 4.9: Predicted JPCP distresses for different pavement construction months

Figure 4.9(c) shows that predicted slab cracking is not highly affected by the construction month. However, both I-70 Shawnee County and K-7 Johnson County projects, where some cracking was observed, showed slightly less slab cracking for construction during April. Considering all results it appears that April and October are the two best months for JPCP construction (paving) in Kansas. However, previous studies (Beam 2003; Coree et al. 2005) found that the variation in distress corresponding to the changes in PCC zero stress temperature is not that significant.

Soil Class - In this study, subgrade soil class was varied as: A-4 (Silt, ML) and A-7-6 or A-6 (Clay, CL). The properties used for these soils are summarized in Table 3.1.

No variation in IRI was observed for all projects except for the I-70 Shawnee and K-7 Johnson County projects as shown in Figure 4.10 (a). That variation was not significant. When the subgrade soil was changed from to clay to silt, the predicted IRI decreased from 120 to 117 inches/mile and 110 to 106 in/mi for the I-70 Shawnee and K-7 Johnson County projects, respectively. This variation is negligible for all practical purposes.

No significant variation in faulting was observed for all JPCP projects, except for the I-70 Shawnee County and K-7 Johnson County projects. Figure 4.10(b) shows that faulting increased by 0.005 in for both projects, when the soil type was changed from clay to silt. However, these faulting values are negligible for all practical purposes.

Figure 4.10(c) shows the predicted cracking on all sections. With the project-specific inputs, only I-70 Shawnee County and K-7 Johnson County sections showed some insignificant increase in slab cracking (around 1.2% and 0.3%, respectively) for different soil types. However, no variation in cracking was observed on all other projects. Therefore, cracking is not sensitive to the changes in soil type.

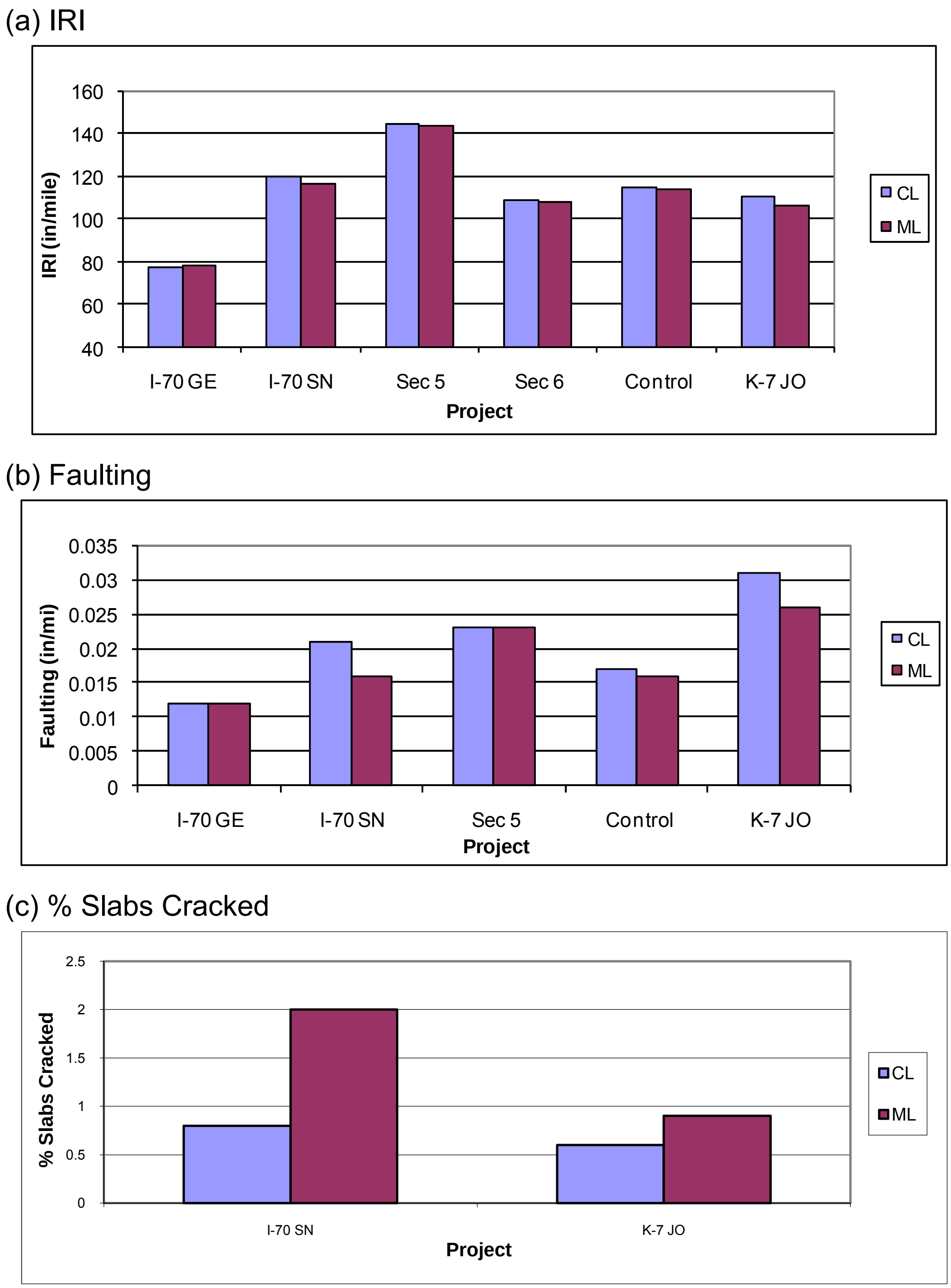


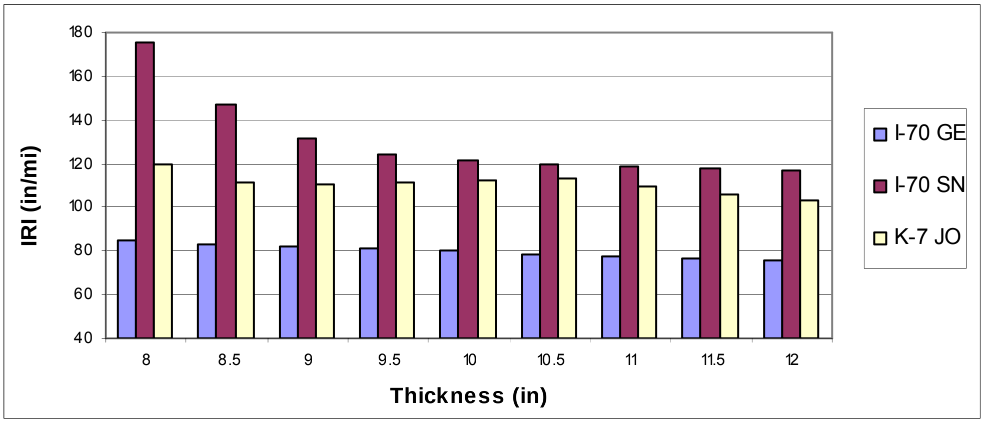
Figure 4.10: Predicted JPCP distresses for different subgrade soil type

4.2.3 Design and Construction Features

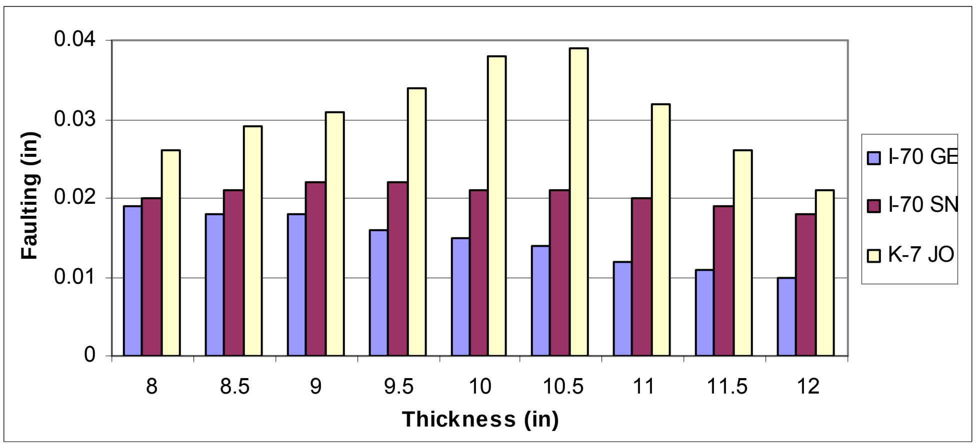
Thickness - The MEPDG-predicted IRI values for the KDOT sections were compared at nine levels of PCC slab thickness input- 8 inches to 12 inches with an increment of 0.5 inch. Figure 4.11(a) shows the results on non-SPS-2 sections. In general, higher slab thickness resulted in lower IRI. The effect is highly pronounced for the I-70 Shawnee County as well as for the Geary County project. When the PCC slab thickness was increased from 8 inches to 12 inches, the predicted IRI decreased from 176 inches/mile to 117 inches/mile for the I-70 Shawnee County project. For the I-70 Geary County project, the predicted IRI decrease was smaller (8.7 inches/mile- from 84.5 to 75.8 inches/mile). The effect on the K-7 Johnson County project was also very high. It is to be noted that the K-7 Johnson County project is different from the other projects because this project has the lowest PCC slab thickness (9 inch). The I-70 Shawnee County project has the lowest 28-day modulus of rupture (473 psi).

Figure 4.12(a) shows the change in predicted IRI with slab thickness for the three SPS-2 projects. For the SPS-2 Section 5 and Control Section, when the slab thickness was increased from 8 inch to 12 inches, the predicted IRI decreased from 151 inches/mile to 145 inches/mile and 124 inches/mile to 115 inches/mile, respectively. SPS-2 Section 5 has the highest modulus of rupture and the Control section has the highest thickness among all projects studied. The effect of change in slab thickness on predicted IRI is not significant for the SPS-2 Section 6, which has a 14-foot widened lane.

(a) IRI



(b) Faulting



(c) % Slabs Cracked

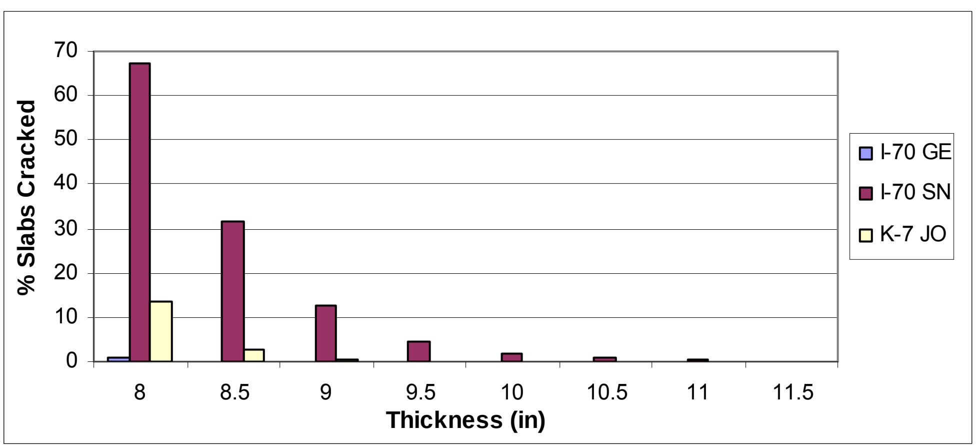
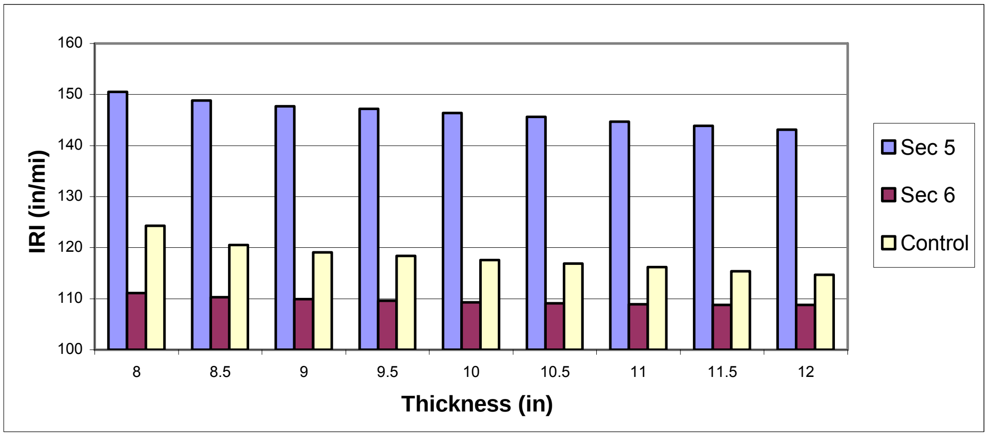


Figure 4.11: Predicted JPCP distresses on KDOT sections for varying thickness

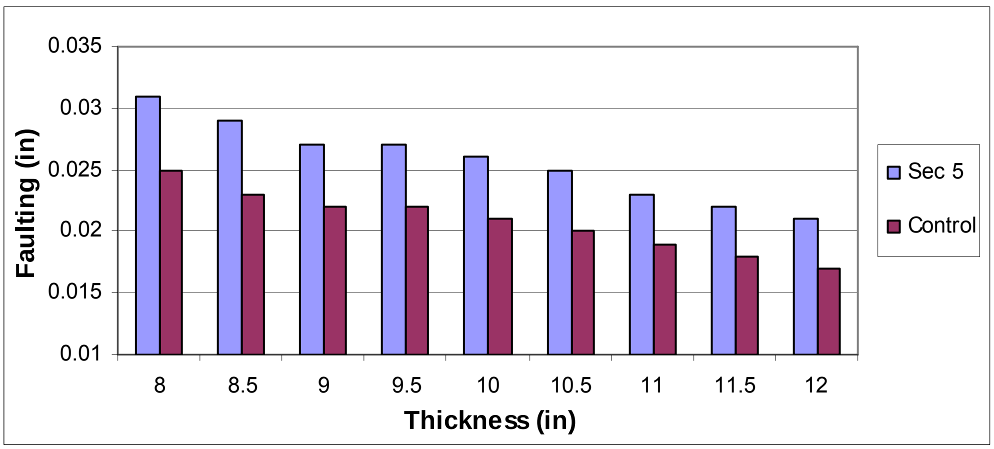
It appears that the effect of thickness is more pronounced on the PCC pavements with thinner slab or low strength. However, this sensitivity was not observed for a JPCP with widened lane.

Figure 4.11(b) and 4.12(b) show the predicted faulting on all KDOT and SPS-2 sections. Significant changes in faulting values were observed for all projects except the SPS-2 Section 6. No faulting was observed for that particular project due to its widened lane effect. The effect of varying thickness is most significant for the I-70 Shawnee County and K-7 Johnson County projects. For the K-7 Johnson County project faulting increased with increasing thickness up to a certain point and then reduced with increasing thickness. A similar trend was also observed for the I-70 Shawnee County project. The highest faulting observed in the K-7 Johnson and I-70 Shawnee County projects were 0.039 inch at 10.5 inch PCC slab thickness and 0.022 inches at 9 inch thickness, respectively. The NCHRP MEPDG suggests that with increasing slab thickness (in order to reduce slab cracking for heavier traffic), dowel diameter be increased to control joint faulting. This may result in a small increase in predicted joint faulting due to a reduction in effective area of the dowel bar relative to the slab thickness (NCHRP 2004). Therefore, since the K-7 Johnson County project has the lowest dowel diameter (1.125 inch) compared to others, predicted faulting is greater at higher thickness. However, after a certain thickness level (nearly 11 inches), distresses get compensated for higher thickness as shown in Figure 4.11 (b). Though the dowel diameter for the I-70 Shawnee County project (1.375 inch) is not that low but the project has the lowest modulus of rupture compared to others. That may have resulted in a trend similar to the K-7 Johnson County project. For the I-70 Geary County project, as the thickness increased the predicted faulting decreased. When the thickness was increased from 8 inches to 12 inches, faulting decreased from 0.02 inch to 0.01 inch.

(a) IRI



(b) Faulting



(c)% Slabs Cracked

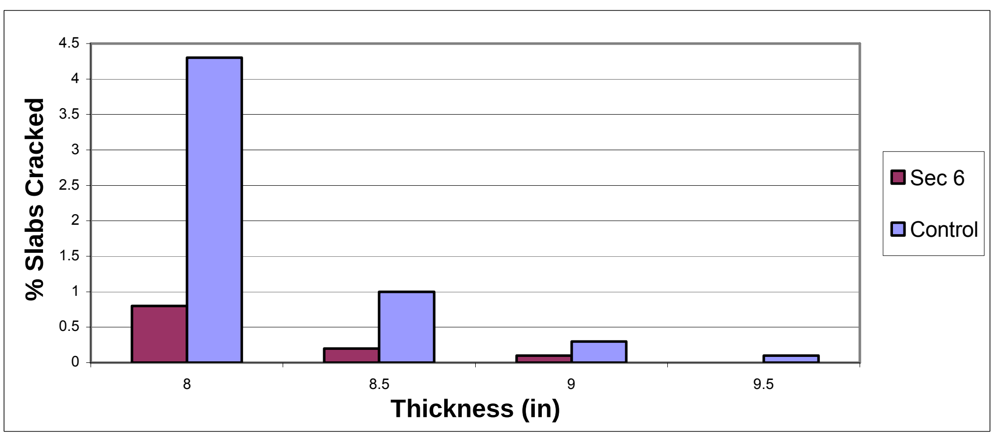


Figure 4.12: Predicted JPCP distresses on SPS-2 Sections for varying thickness

The SPS-2 Section 5 and SPS-2 Control section show similar trends of decreasing faulting with thickness, as illustrated in Figure 4.12(b). When the thickness increased from 8 inches to 12 inches, the predicted faulting decreased from 0.025 inches to 0.017 inches and 0.031 inches to 0.021 inches for the SPS-2 Control section and SPS-2 Section 5, respectively. No faulting was observed on Section 6, which has a 14 foot widened lane. However, all predicted faulting values are negligible for all practical purposes.

Figures 4.11(c) and 4.12(c) illustrate the effect of thickness on predicted percent slabs cracked for the JPCP projects in this study. Among the three KDOT sections, I-70 Shawnee County and K-7 Johnson County projects showed significant change in cracking with thickness. Figure 4.11(c) shows the when that thickness was increased from 8 inches to 12 inches, percent slabs cracked decreased from 67% to 0.1% for the I-70 Shawnee County project. For the K-7 Johnson County project, the predicted percent slabs cracked decreased from 14% to 0.1% when the slab thickness was increased from 8 inches to 9.5 inches. After that thickness level, no cracking was observed for that project. Among the three SPS-2 projects, the SPS-2 Control section showed higher percentage of cracking (4.3%) at an 8 inch thickness, and no cracking was observed at 10 inches as shown in Figure 4.12(c).

Cracking on SPS-2 Section 6 decreased almost similar to the SPS-2 Control Section (0.8% at 8 inches), and then reduced to none at a 9.5 inch thickness. No cracking was observed on the SPS-2 Section 5 which has the highest modulus of rupture among all projects. Therefore, thickness significantly influences cracking on projects with lower PCC strength.

Dowel Diameter - In this study, dowel diameter was varied at three levels of input: 1 inch (low), 1.25 inches (average) and 1.5 inches (high). The predicted MEPDG distresses are tabulated in Table 4.5. Figure 4.13 (a) shows the predicted IRI results. In general, higher dowel diameter resulted in lower IRI. The effect is most pronounced for the I-70 Shawnee County and K-7 Johnson County projects as well as for the three SPS-2 projects. When the dowel diameter was increased from 1.0 to 1.5 inch, the predicted IRI decreased from 161 inches/mile to 117 inches/mile for the I-70 Shawnee County project. For the K-7 Johnson County project, predicted IRI decrease was almost similar (36 inches/mile- from 133 to 97 inches/mile). It is to be noted that these projects are different from the other projects because the I-70 Shawnee County project has the lowest 28-day modulus of rupture (473 psi) and the K-7 Johnson County project has the lowest PCC slab thickness (9 inches). Similar trends were also observed for the three SPS-2 projects. For the SPS-2 Section 5, when the dowel diameter was increased from 1.0 inch to 1.5 inches, the predicted IRI decreased from 194 inches/mile to 145 inches/mile. This section has the highest modulus of rupture among all projects studied. Dowel size effect is also significant for the SPS-2 Control section and Section 6, which have the highest slab thickness and 14-foot widened lane, respectively. With increasing dowel diameter from 1.0 inch to 1.5 inch, IRI decreased from 151 inches/mile to 115 inches/mile and 129 inches/mile to 109 inches/mile for these sections, respectively. It appears the effect of dowel diameter is more pronounced on the PCC pavements with thinner or thicker slab, and very high or low strength. It also is to be noted that that dowel diameter variation does not have any significant effect on the predicted IRI for the

I-70 Geary County project. That section has a slab thickness of 11 inches with a modulus of rupture of 690 psi.

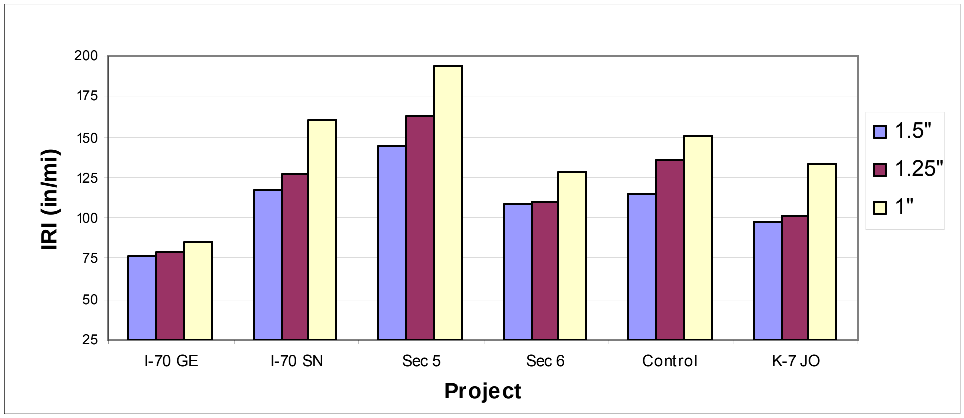
Table 4.5: Comparison of Predicted Response Corresponding to Varying Different Dowel Diameter

Project	IRI (in/mi)			Faulting (in)			% Slabs Cracked		
	1.5"	1.25"	1"	1.5"	1.25"	1"	1.5"	1.25"	1"
I-70 GE	77	80	86	0.011	0.016	0.028	0	0	0
I-70 SN	117	128	161	0.015	0.036	0.099	0.8	0.8	0.8
Sec 5	145	163	194	0.023	0.059	0.0117	0	0	0
Sec 6	109	111	129	0	0.003	0.038	0	0	0
Control	115	135	151	0.017	0.056	0.087	0	0	0
K-7 JO	97	101	133	0.006	0.014	0.075	0.6	0.6	0.6

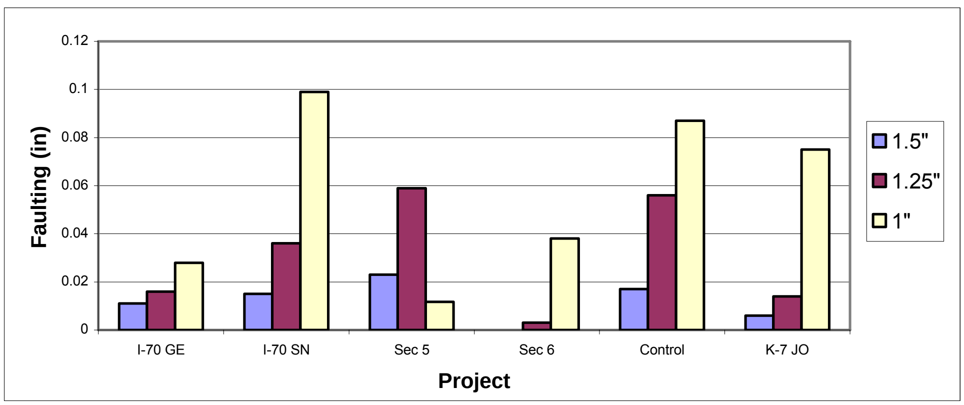
Figure 4.13(b) shows the predicted faulting on all sections corresponding to three levels of dowel diameter. Significant changes in faulting values were observed for all projects. The effect of varying dowel diameter is most significant for the I-70 Shawnee County project, SPS-2 Section 5, SPS-2 Control section and the K-7 Johnson County project. For the I-70 Shawnee County and K-7 Johnson County projects, when dowel diameter was increased from 1.0 inch to 1.5 inches, the predicted faulting decreased from 0.10 to 0.015 inches and 0.08 to 0.006 inches, respectively. For Section 5, which has a very high cement factor (862 lbs/yd^3), the effect is even more significant. For that section faulting decreased from 0.12 to 0.02 inch with an increase in dowel diameter from 1.0 to 1.5 inch. With increasing dowel diameter, the predicted faulting on the SPS-2 Control section decreased from 0.09 to 0.02 inch. It was also observed that with 1.5-inch dowels, no faulting was observed for the KDOT Section 6, which has a 14 foot

widened lane. However, faulting was present for smaller diameter dowels. The predicted faulting values are negligible for all practical purposes.

(a) IRI



(b) Faulting



(c) % Slabs Cracked

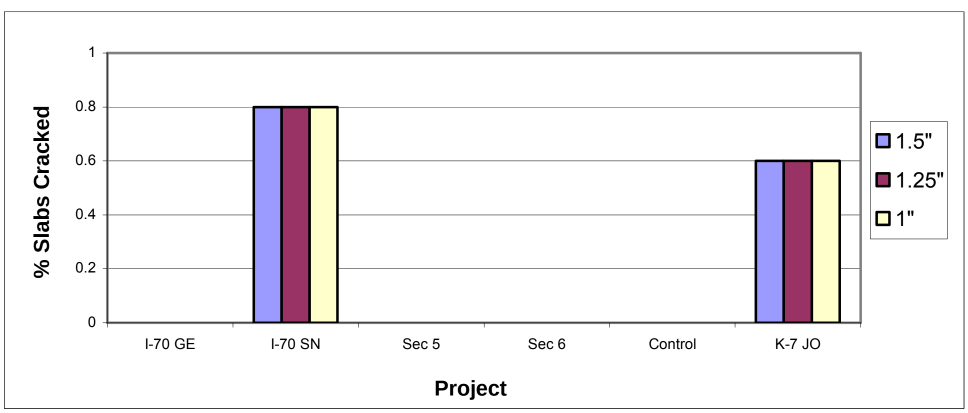


Figure 4.13: Predicted JPCP distresses for different dowel diameters

Figure 4.13(c) illustrates the effect of dowel diameter on predicted percent slabs cracked for the JPCP projects in this study. Only two projects, I-70 Shawnee County and K-7 Johnson County, appear to have this distress irrespective of dowel size. Very small amounts of cracking of 0.8% and 0.6 % were observed for the Shawnee County and Johnson County projects, respectively. Dowel size does not appear to affect slab cracking.

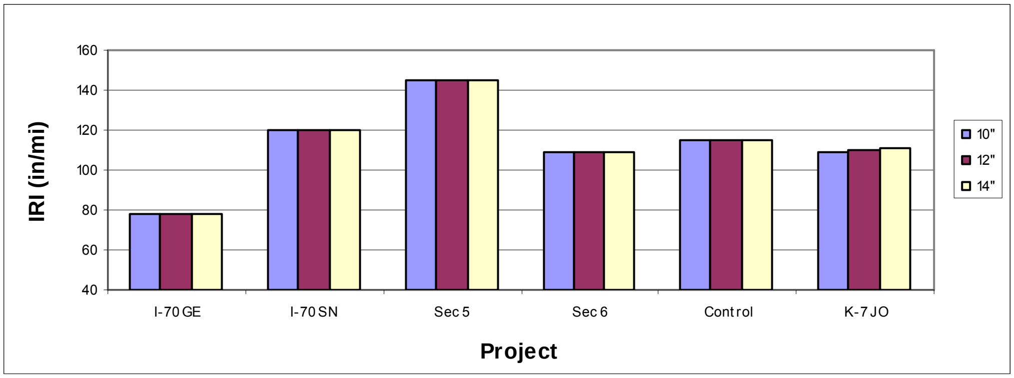
Dowel Spacing - Figure 4.14(a) shows the predicted IRI on all sections corresponding to three levels of dowel spacing-10, 12, and 14 inches. No variation in IRI was observed for all projects except K-7 Johnson County. That variation was not significant. When the dowel spacing was increased from 10 inches to 14 inches, the predicted IRI increased from 109 to 111 inches/mile. This much variation is negligible for all practical purposes.

Figure 4.14(b) shows the predicted faulting on all sections corresponding to three levels of dowel spacing. However, no variation was observed for almost all projects. When the dowel spacing increased from 10 inches to 14 inches, the predicted faulting increased from 0.029 inches to 0.032 inches for the K-7 Johnson County project. The predicted faulting values were negligible for all practical purposes.

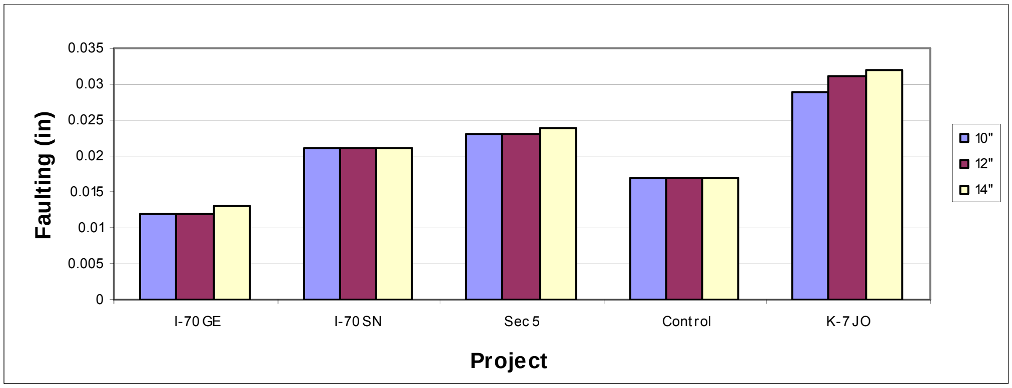
Figure 4.14(c) shows the predicted cracking on all sections. With the project-specific inputs, only I-70 Shawnee County and K-7 Johnson County sections showed some insignificant amounts of slab cracking. However, no variation in cracking amount was observed with varying dowel bar spacing. Therefore, cracking is not sensitive to the changes in dowel bar spacing.

Beam (2003) and Nantung et al. (2005) found that dowel diameter is a sensitive design input, whereas dowel spacing is insensitive. Coree et al. (2005) have shown that dowel diameter does not influence the predicted percent slabs cracked.

(a) IRI



(b) Faulting



(c) % Slabs Cracked

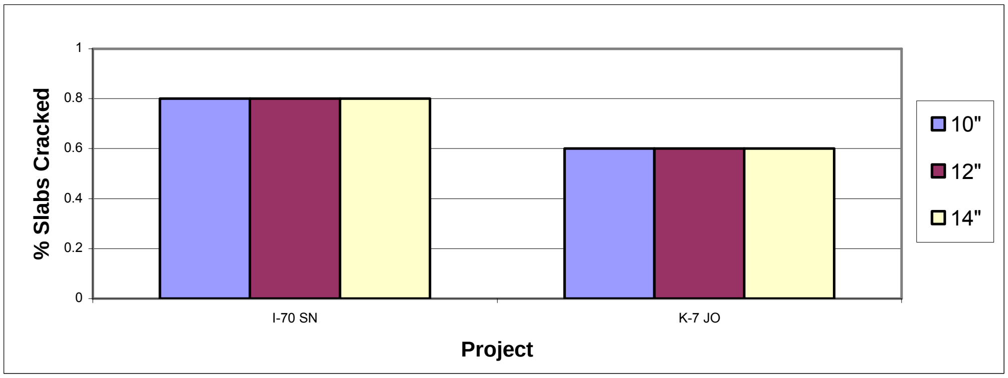


Figure 4.14: Predicted JPCP distresses for different dowel spacing

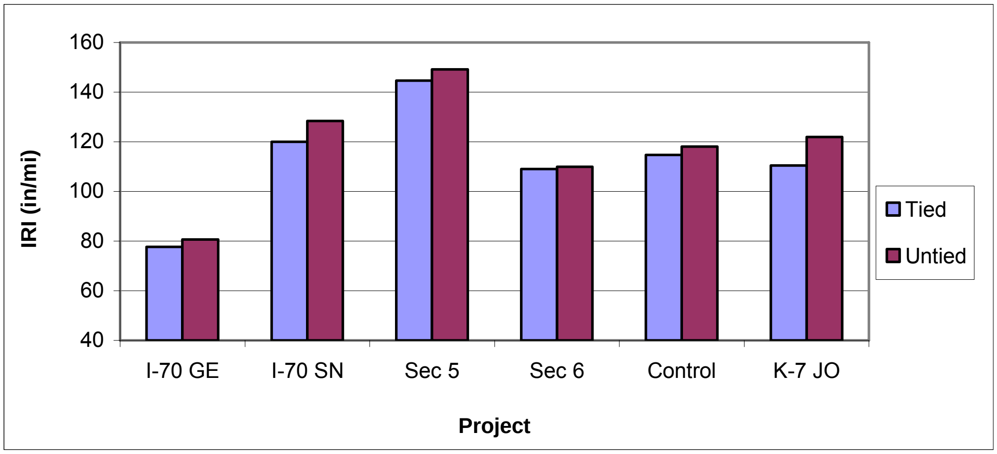
Shoulder Type - Table 4.6: Comparison of Predicted Responses Corresponding to Tied and Untied Shoulders

Tied PCC shoulders can significantly improve JPCP performance by reducing critical deflections and stresses (NCHRP 2004). For tied concrete shoulders, the long-term Load Transfer Efficiency (LTE) value between the lane and the tied shoulder must be provided. In this study, 60 percent LTE was considered for the projects with monolithically constructed, tied PCC shoulder. MEPDG-predicted IRI values, tabulated for all projects in Table 4.6, were compared for tied and untied shoulders. The effect was prominent for the same three sections (I-70 Shawnee, SPS-2 Section 5, and K-7 Johnson County) as was with the dowel size as shown in Figure 4.15 (a). When the PCC shoulder was untied on the K-7 Johnson County project, the predicted IRI increased from 110 inches/mile to 122 inches/mile. This project showed the greatest effect. The SPS-2 Section 6 did not show any change in IRI mainly due to the fact that this project has a widened lane of 14 feet.

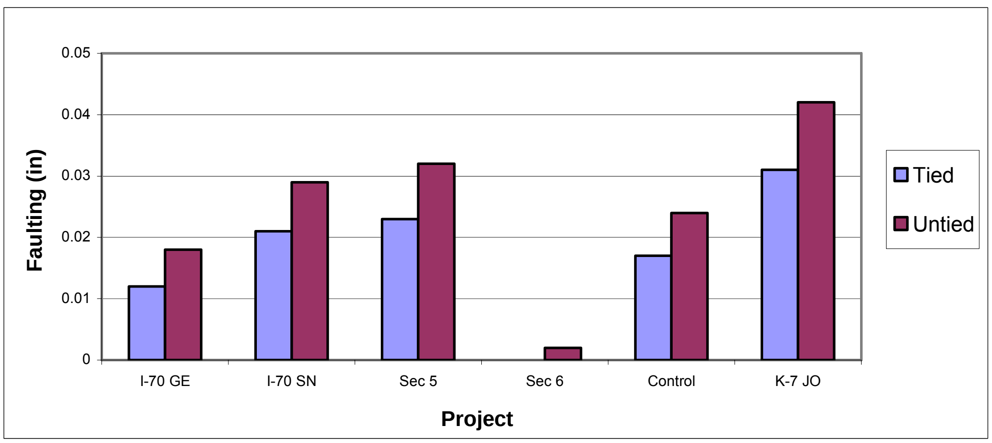
Table 4.6: Comparison of Predicted Responses Corresponding to Tied and Untied Shoulders

Project	**IRI (in/mi)**		**Faulting (in)**		**% Slabs Cracked**	
	Tied	Untied	Tied	Untied	Tied	Untied
I-70 GE	78	81	0.012	0.018	0	0
I-70 SN	120	128	0.021	0.029	0.8	5.6
Sec 5	145	149	0.023	0.032	0	0
Sec 6	109	110	0	0.002	0	0
Control	115	118	0.017	0.024	0	0
K-7 JO	110	122	0.031	0.042	0.6	7.9

(a)IRI



(b) Faulting



(c) % Slabs Cracked

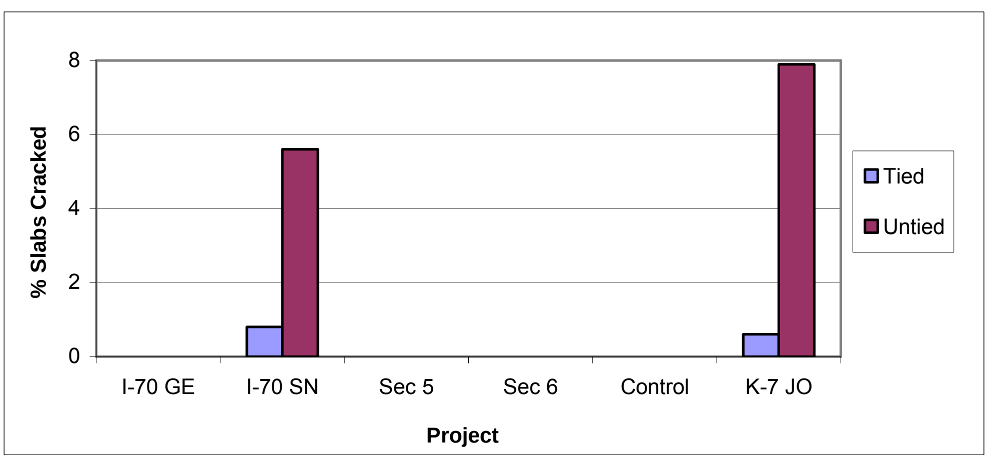


Figure 4.15: Predicted JPCP distresses for different shoulder type

Figure 4.15(b) illustrates the effect of untied shoulders on predicted faulting for the JPCP projects in this study. Effect was significant for all projects. When the shoulders were untied, the SPS-2 Section 6 also showed some faulting. However, with tied shoulders this project did not show any faulting. MEPDG-predicted faulting increased from 0.03 inches to 0.04 inches when the shoulders were untied for the K-7 Johnson County project. Similar effect was also observed for all other projects.

The effect of tied shoulders was very pronounced on slab cracking as shown in Figure 4.15(c). For untied shoulders, percent slabs cracked increased from 0.8% to 5.6% and from 0.6% to 8% for the Shawnee and Johnson County projects, respectively.

Widened Lane - In this study, MEPDG-predicted IRI's for all sections were compared at two different lane widths- 12 and 14 feet. Table 4.7 tabulates and Figure 4.16 illustrates the results. In all projects, a widened lane resulted in lower IRI as shown in Figure 4.16(a). On the I-70 Shawnee County project, when the lane width increased from 12 feet to 14 feet, the predicted IRI decreased from 120 inches/mile to 109 inches/mile. For the SPS-2 Section 5 and Section 6, the decrease was almost similar (about 11 inches/mile). The lowest decrease, about 7 inches/mile, was observed for the I-70 Geary county project and the SPS-2 Control section. The effect is most pronounced for the K-7 Johnson County project. When the lane width was increased from 12 feet to 14 feet, the predicted IRI decreased from 111 inches/mile to 94 inches/mile. This project has the lowest PCC slab thickness (9 inches).

Widened lane also has significant effect on faulting. Figure 4.16(b) illustrates the predicted faulting on all sections corresponding to 12 foot lane width. When the lane width was increased to 14 feet, no faulting was observed for almost all sections except the SPS-2 Section 5 and Control Sections. Insignificant amount of faulting was observed for those projects with 14 foot lane width too. The K-7 Johnson County project showed significant variation with the change in lane width. When the lane width increased from 12 feet to 14 feet, the predicted faulting decreased from 0.03 inches to zero. Similar trend was observed for all other sections.

Table 4.7: Comparison of Predicted Response Corresponding to Varying Lane Width

Project	**IRI (in/mi)**		**Faulting (in)**		**% Slabs Cracked**	
	12 ft	14 ft	12 ft	14 ft	12 ft	14 ft
I-70 GE	78	71	0.012	0	0	0
I-70 SN	120	109	0.021	0	0.8	0.5
Sec 5	145	133	0.023	0.002	0	0
Sec 6	120	109	0.02	0	0	0
Control	115	106	0.017	0.001	0	0
K-7 JO	110	94	0.031	0	0.6	0.2

Figure 4.16(c) shows the effect of lane width on predicted percent slabs cracked for the JPCP projects in this study. Only two projects, I-70 Shawnee County and K-7 Johnson County, appeared to be affected by this input though the predicted cracking values were negligible. When a widened lane was used, percent slabs cracked decreased by about 0.3% to 0.4% for the Shawnee County and Johnson County projects, respectively. Widened lane appears to reduce cracking insignificantly for the projects with lower strength and thinner slabs.

(a) IRI

(b) Faulting

(c) % Slabs Cracked

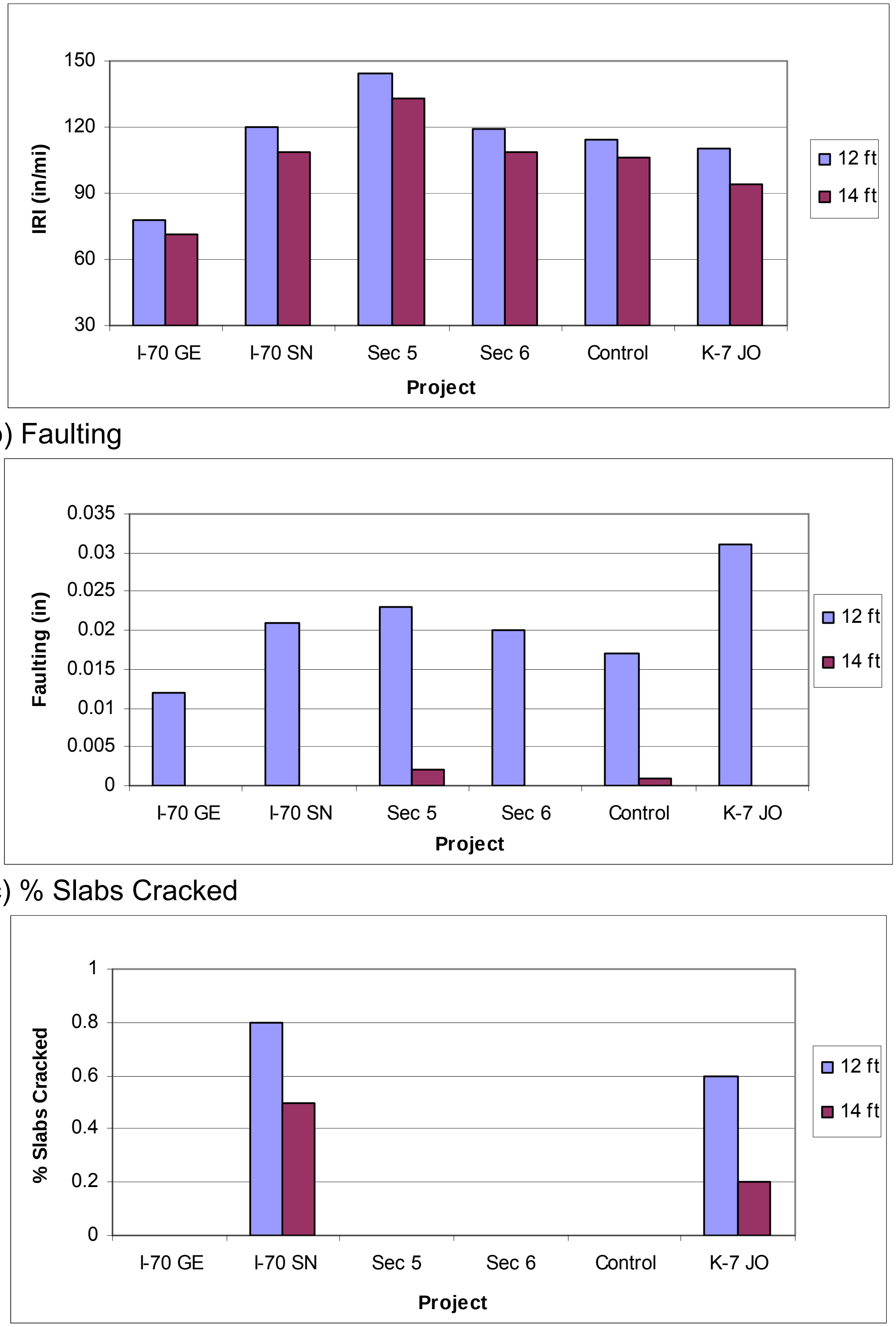


Figure 4.16: Predicted JPCP Distresses for Different Lane Widths

(a) IRI

(b) Faulting

(c) % Slabs Cracked

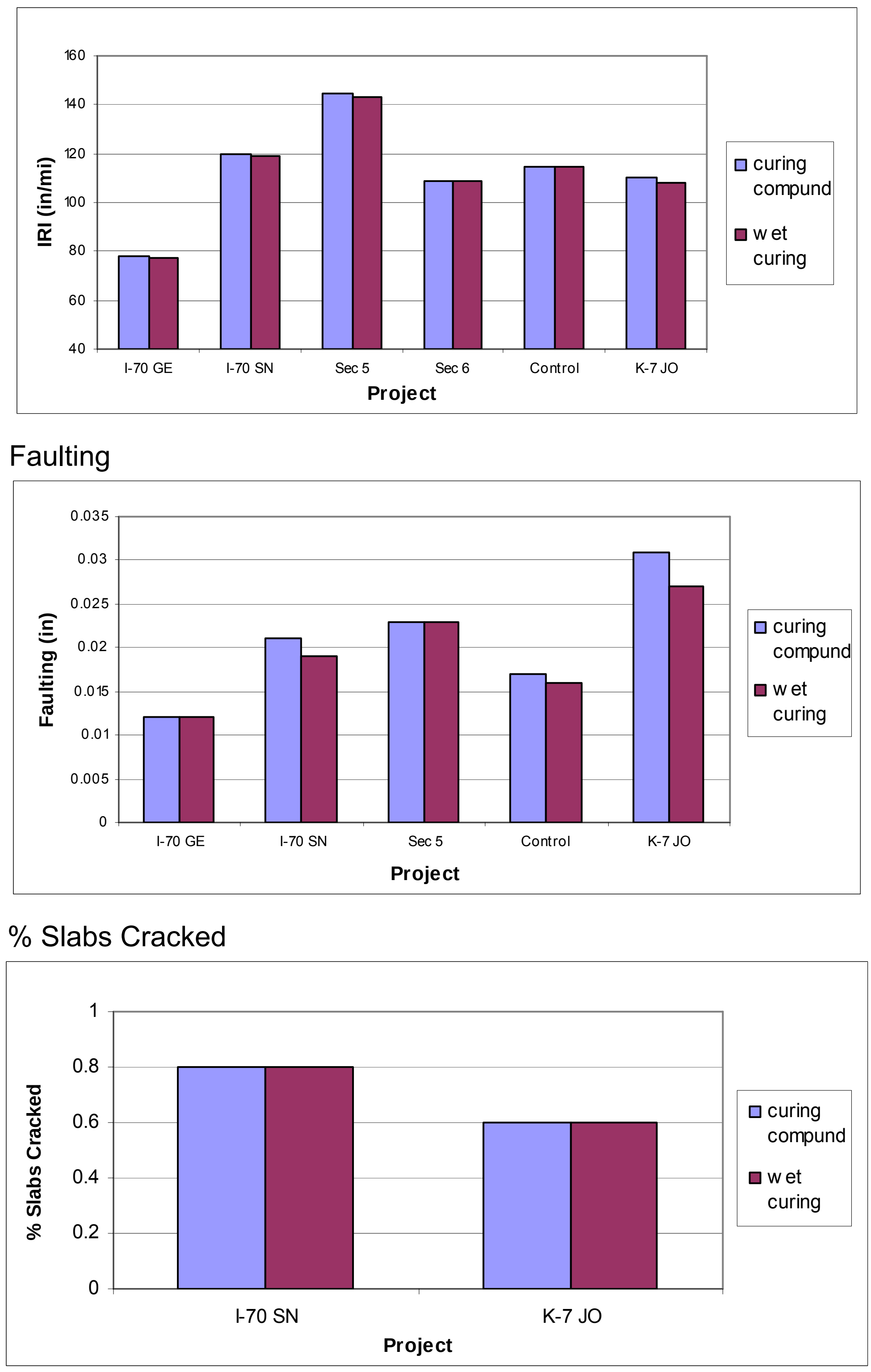


Figure 4.17: Predicted JPCP distresses for Curing type

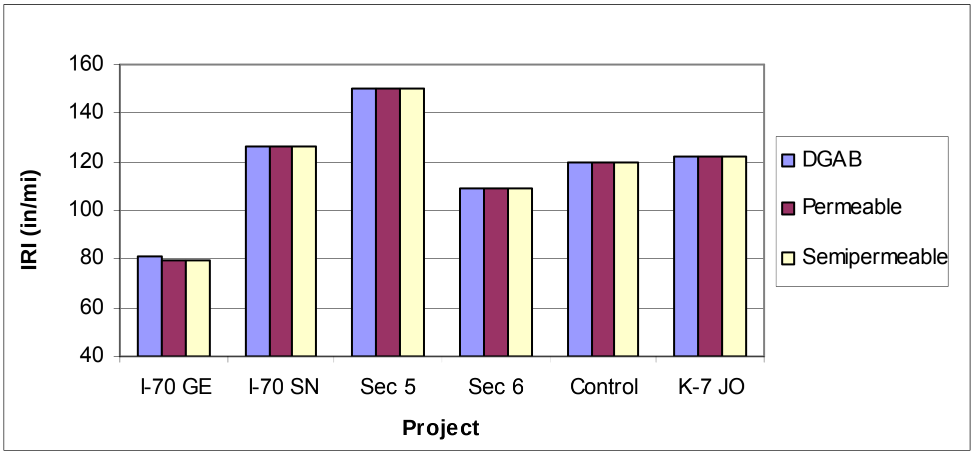
Curing Method - Distresses were predicted using MEPDG for two different curing methods – sprayed curing compound and wet curing. No effect was observed for IRI, percent slabs cracked and faulting on most of the projects as shown in Figure 4.17. Only the K-7 Johnson County project showed slightly lower faulting with wet curing. Thus this factor did not appear to affect the MEPDG-predicted distresses on the study sections. This parameter was also found to be insensitive by Coree et al. (2005).

Base Type - Sensitivity analysis was done toward two types of base – stabilized and granular. Stabilized bases are asphalt-treated base (ATB), Portland cement-treated base (PCTB) and Bound drainable base (BDB). Dense graded aggregate base (DGAB), permeable base and semi-permeable base are under the granular base category.

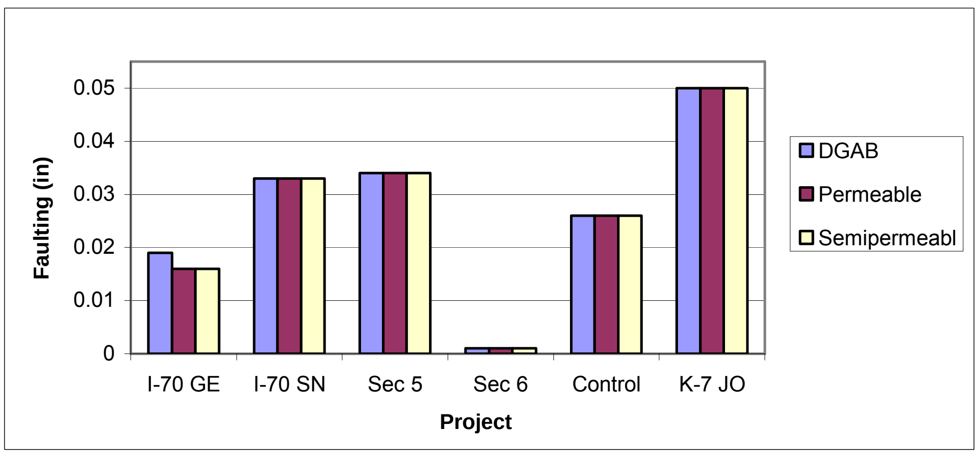
Asphalt treated bases (ATB) use the same aggregates as in the granular bases, but mixed with an asphaltic binder. Typically two to three percent asphalt binder is added in Kansas. Granular base (DGAB) consists of untreated dense-graded aggregate, such as crushed stone. Unbound granular base properties such as, gradation and plasticity index are the required inputs in this category. BDB is similar to PCTB except for aggregate gradation and permeability requirements.

Granular Base - In this study, MEPDG-predicted IRI's for the sections were compared for two different granular base types- permeable and semi-permeable. Figure 4.18 (a) shows the predicted IRI values for granular bases. The predicted IRI remained unaffected by the granular base type.

(a)IRI



(b) Faulting



(c)% Slabs Cracked

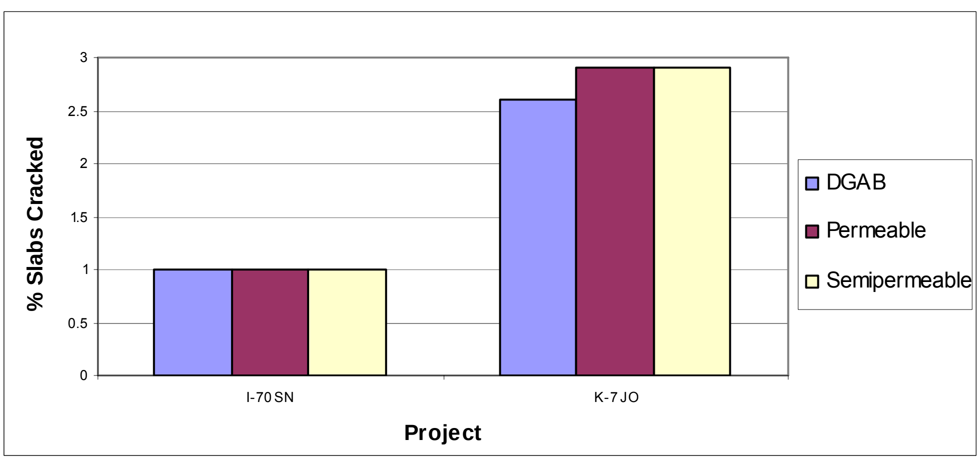


Figure 4.18: Predicted JPCP distresses for different granular bases

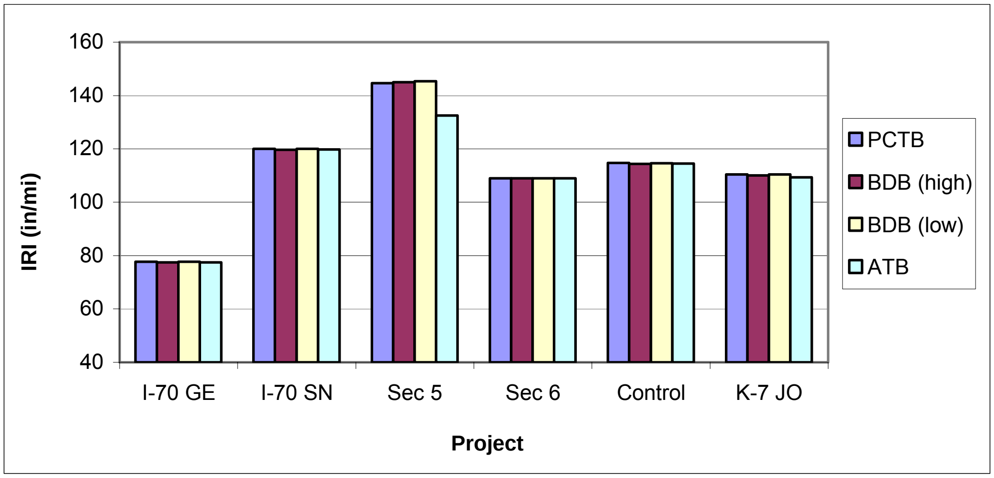
Figure 4.18 (b) illustrates the predicted faulting for all projects corresponding to different granular base type. Again, the predicted faulting is largely unaffected by granular base type. Cracking also remains somewhat unaffected by the granular base type as shown in Figure 4.18 (c).

Stabilized Base - MEPDG-predicted IRI values for the sections were compared for three different stabilized base types- PCTB, BDB, and ATB. The BDB modulus was varied at two levels –low and high. Figure 4.19(a) shows the results. No significant variation was observed for all projects except SPS-2 Section 5 that has the highest 28-day modulus of rupture. The IRI decreased from 145 inches/mile with BDB to 133 inches/mile with ATB for this section.

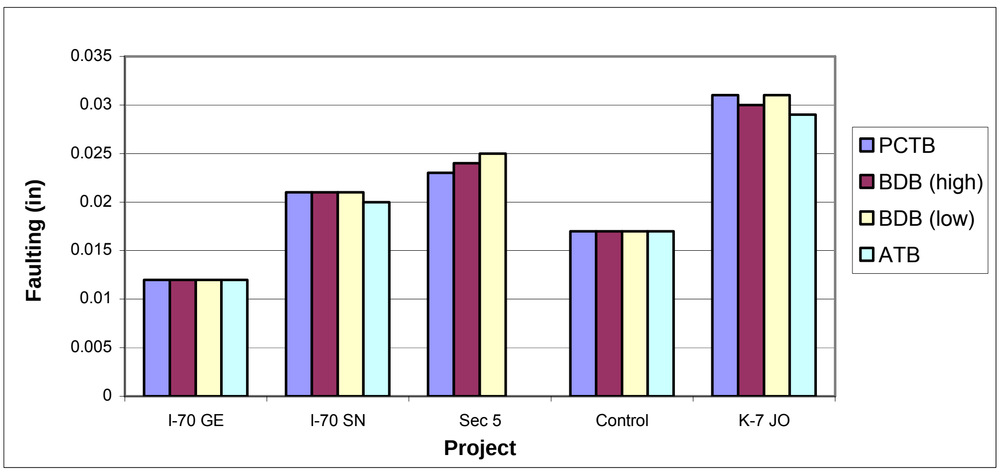
Figure 4.19(b) shows that the faulting on the SPS-2 Section 5 project was somewhat reduced by ATB. Reduced faulting with ATB was also observed for the K-7 Johnson County and I-70 Shawnee County projects. However, the predicted faulting values were negligible for all practical purposes.

MEPDG-predicted cracking was evaluated for all projects as shown in Figure 4.19(c). Negligible variation in cracking with respect to the treated base type was observed on the I-70 Shawnee County and the K-7 Johnson County projects. For K-7 Johnson County, ATB showed lower cracking compared to other bases, whereas completely reverse phenomenon was observed for the I-70 Shawnee County project. Also, high modulus BDB showed the lowest amount of cracking for the Shawnee County project.

(a) IRI



(b) Faulting



(c)% Slabs Cracked

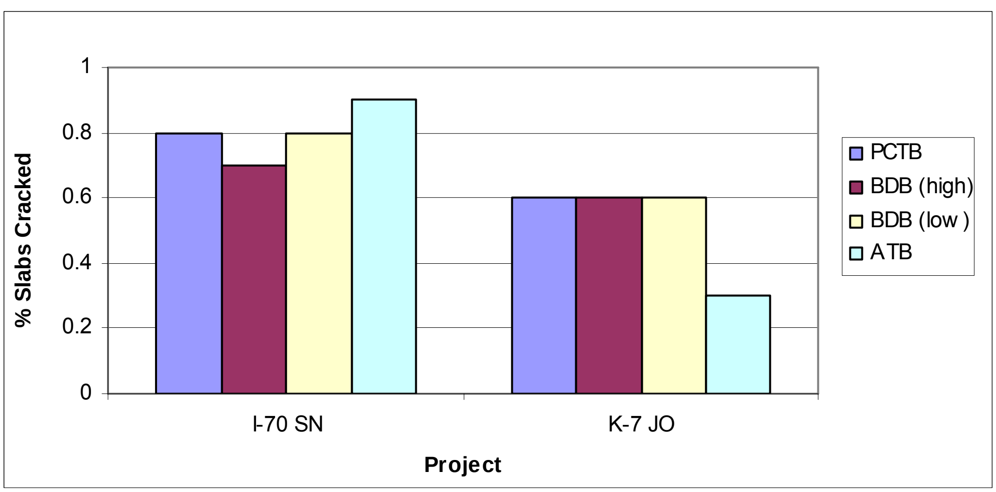


Figure 4.19: Predicted JPCP distresses for stabilized bases

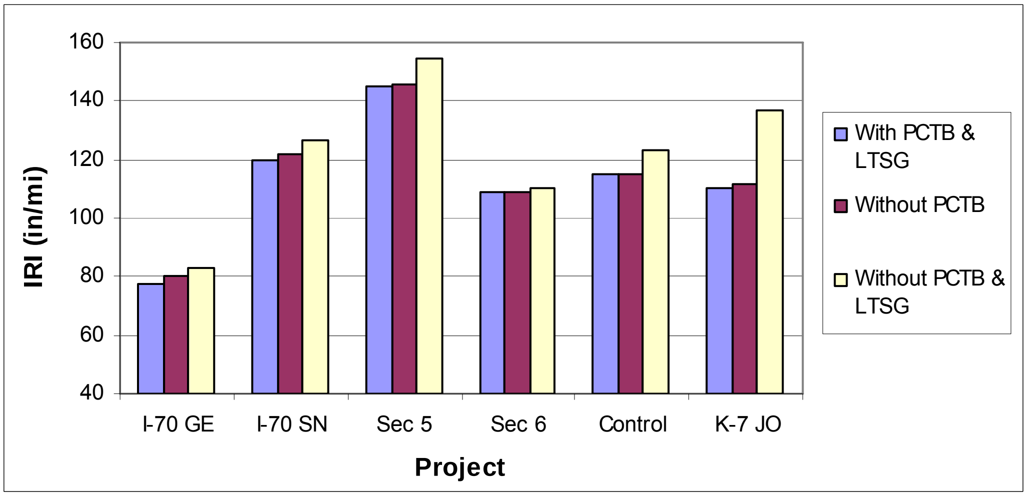
4.2.4 Alternative Design

The 1993 AASHTO Design Guide was known to be fairly insensitive to the changes in the modulus of subgrade reaction of the foundation layer (AASHTO 1993). In this study, MEPDG sensitivity analysis was done for all KDOT and SPS-2 projects toward three alternate design strategies involving the JPCP foundation layer. First, the projects were analyzed with a Portland cement-treated base (PCTB) and a lime-treated subgrade (LTSG). Then the projects were analyzed without PCTB but with LTSG. Finally, the projects were assumed to be built directly on the compacted natural subgrade (no subgrade modification with lime). Typical JPCP distresses, IRI, faulting, and percent slabs cracked, were calculated and compared after 20 years. Table 4.8 tabulates the results.

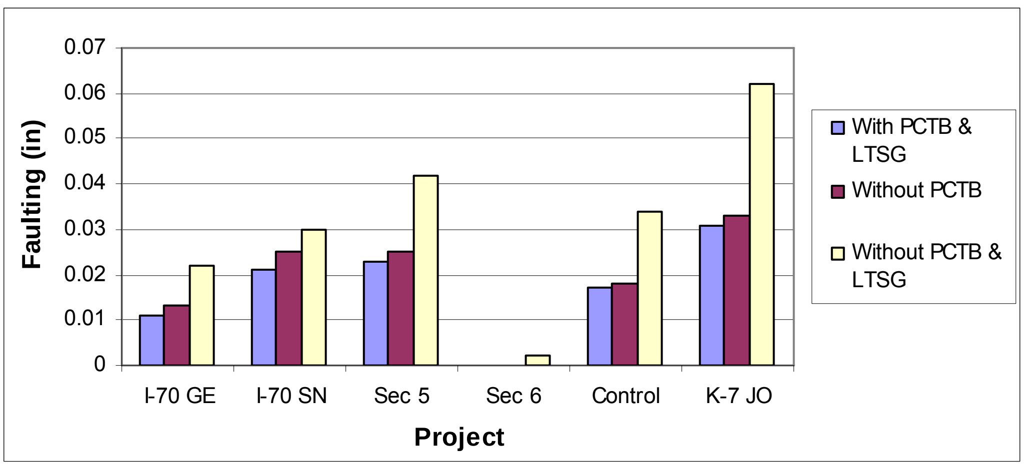
Table 4.8: Comparison of Predicted Responses Corresponding to Different Design Strategy

Project	IRI (in/mi)			Faulting (in)			% Slabs Cracked		
	With PCTB &	Without PCTB	Without PCTB &LTSG	With PCTB & LTSG	With out PCT	Without PCTB &LTSG	With PCTB & LTSG	Without PCTB	Without PCTB &LTSG
I-70 GE	77	80	83	0.011	0.013	0.022	0	0	0
I-70 SN	120	122	127	0.021	0.025	0.03	0.8	0.8	3.2
Sec 5	145	146	155	0.023	0.025	0.042	0	0	0
Sec 6	109	109	110	0	0	0.002	0	0	0
Control	115	115	123	0.017	0.018	0.034	0	0	0
K-7 JO	110	111	137	0.031	0.033	0.062	0.6	0.7	13.5

(a) IC



(b) Faulting



(c) % Slabs cracked

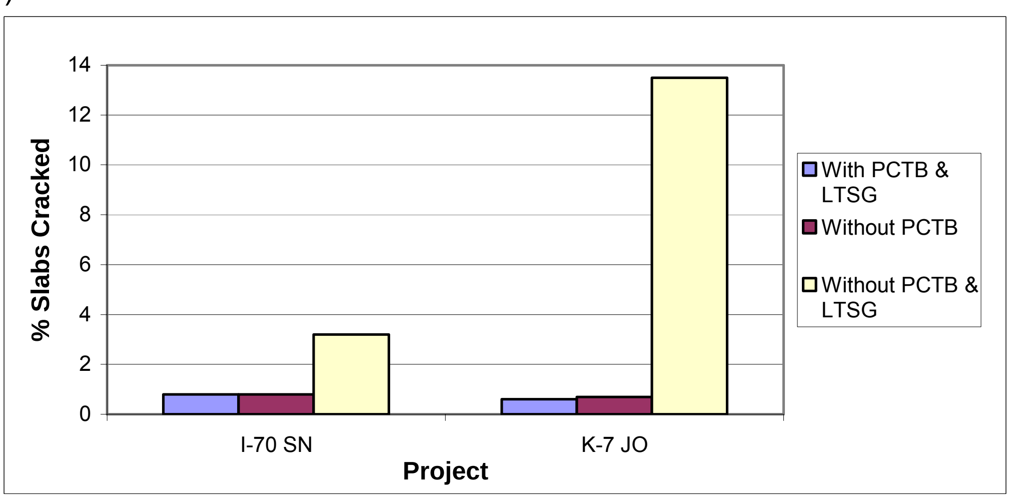


Figure 4.20: Predicted JPCP distresses for alternative designs

The MEPDG-predicted IRI values for the sections were compared for different design alternatives. Figure 4.20(a) shows the results. In general, for pavements without any treated base/subbase, the effect is not that pronounced on IRI. The projects without PCTB showed almost similar IRI compared to those with PCTB. However, IRI increased markedly for the projects built directly on the compacted subgrade. The effect is most pronounced for the K-7 Johnson County and SPS-2 Section 5 projects. On the K-7 Johnson County project, without any treated base and subgrade, predicted IRI would increase from 110 inches/mile to 137 inches/mile. It is to be noted that this project has the lowest PCC slab thickness (9 inches) among all projects. The effect is also significant for the SPS-2 Section 5. For the SPS-2 Section 5, without any treated base and subgrade, the predicted IRI increased from 145 inches/mile to 155 inches/mile. This section has the highest modulus of rupture among all projects studied. No effect on IRI was noticed for the SPS-2 Section 6 which has a widened lane.

Figure 4.20(b) shows that the faulting values on the K-7 Johnson County and SPS-2 Section 5 projects are markedly affected by the changes in design. Without PCTB and LTSG, faulting increased from 0.03 inches to 0.06 inches on the K-7 Johnson County project. This increasing trend is similar for all other projects. Although SPS-2 Section 6 with a widened lane and tied PCC shoulder did not have any faulting before, without PCTB and LTSG, faulting started to appear on this section. However, the amount is insignificant.

Figure 4.20(c) shows that only two projects, I-70 Shawnee County and K-7 Johnson County, had some cracking. Cracking increases dramatically on the K-7 Johnson County project when the projects were built without PCTB and LTSG. When the K-7 Johnson County project was designed to be built without any treated base and treated subgrade, the percent slabs cracked increased from 0.6% to 13.5%. In fact, the K-7 Johnson County project failed in reliability level for that design aspect.

It appears NCHRP MEPDG correctly predicts that the treated base and treated subgrade have far reaching effect on the performance of JPCP. Clayey and silty subgrades are subjected to erosion during “pumping.” Therefore, as shown in this analysis, JPCP’s should not be built in Kansas directly on compacted natural subgrade consisting of clayey and silty soils.

CHAPTER 5 - SURFACE TYPE SELECTION

5.1 Introduction

The design procedure recommended by the American Association of State Highway and Transportation Officials (AASHTO) is based on the results of the AASHO road test in the late 1950's and early 1960's. The first guide was published in 1961 and was subsequently revised in 1972 and 1981. The guide was revised again in 1986 and then another version was released in 1993. The 1986 and 1993 guides included some further modifications based on theory and experience. Currently these versions are widely used and KDOT uses the 1993 version. Due to the limitations of this guide, as mentioned earlier, a design guide based as fully as possible on the mechanistic principles was developed under the National Cooperative Highway Research Program (NCHRP) (NCHRP 2004). The procedure is capable of developing mechanistic-empirical design while accounting for local environment conditions, local materials, and actual highway traffic distribution by means of axle load spectra. Since this procedure is very sound and flexible and considerably surpasses the capabilities of any currently available pavement design and analysis tools, it is expected that it will be adopted by AASHTO as the new AASHTO design guide for pavement structures. However, a comparison between these guides is necessary to find the effect on the design process.

5.2 Test Sections

Five in-service Jointed Plain Concrete Pavement (JPCP) projects in Kansas were reanalyzed as equivalent JPCP and asphalt concrete (AC) projects using the NCHRP MEPDG. Four of the projects are located on Interstate Route 70, and the other one on Kansas state route K-7. Table 5.1 tabulates the project features of these sections. Four projects have a 6-inch lime treated subgrade (LTSG) to reduce the plasticity and/or to control moisture susceptibility. The reported subgrade modulus in Table 5.1 was computed by the MEPDG software based on the correlation equation involving plasticity index and gradation. According to the Unified Soil Classification all projects have silty clay (CL) soils.

Table 5.1: Project Features of the Study Sections

Project ID	Route	County	Mile Post Limit	Year Built	Subgrade Soil Type	Subgrade Modulus (psi)	Initial AADT	% Truck
K-2611-01	I-70	Geary	0-7	1990	A-6	9,746	9,200	18
K-2609-01	I-70	Dickinson	20-22.6	1992	A-6	6,928	11,970	22.3
K-3344-01	I-70	Shawnee	9-10	1993	A-7-6	6,268	36,000	5
K-3382-01	K-7	Johnson	12-15	1995	A-7-6	7,262	13,825	7
K-5643-01	I-70	Wabaunsee	0-5.2	2001	A-6	7,101	18,000	20.5

5.3 1993 AASHTO Design Guide Inputs

5.3.1 Common Inputs for AC and PCC Pavements

Design Traffic. The design procedures are based on the cumulative expected 18-kip (80-kN) equivalent single axle load (ESAL) on the design lane over the design period. Equivalent axle load factors (EALF), developed during the AASHO Road Test, were used to compute the ESALs and these EALF's depend on the type of pavement.

Serviceability. Initial and terminal present serviceability indices (PSI) must be established to compute the change in serviceability to be used in the design equations. Typical values for initial PSI used in Kansas are 4.2 for AC pavements and 4.5 for PCC pavements. The terminal PSI of 2.5 was used for all projects in this study.

Reliability. Reliability is a means of incorporating some degree of certainty into the design process to ensure that the design alternative will serve the design traffic. In Kansas, the reliability level chosen is a function of the functional classification of the roadway for which the pavement design is intended. Application of the reliability concept requires the selection of a standard deviation that is representative of the local conditions. Typical Guide recommended values are 0.49 for AC pavements and 0.39 for PCC pavements. These values were used in this study.

Design Period. The Kansas Department of Transportation (KDOT) uses a design period of 10 years for AC pavements and 20 years for PCC pavements for the initial section design, and these values were used in this study.

Subgrade Properties. For AC pavements, the effective roadbed soil resilient modulus (M_r) is determined. For PCC pavements the effective modulus of subgrade reaction (*k*) is determined using the equation: $k = M_r / 19.4$. In this study, no seasonal variation was considered.

5.3.2 Specific Design Inputs for AC Pavements

Structural Number (SN). It is a function of layer thicknesses, layer coefficients, and drainage coefficients inputs. SN is actually an output that is derived from the AASHTO flexible pavement design equation as a function of the following inputs.

1) *Layer Coefficient:* The layer coefficient, the relative ability of a given material to function as flexible pavement layer material, is a function of the modulus. In Kansas, the following layer coefficients are used new pavement design: Superpave surface: 0.42 (1/3*AC thickness, up to 4 inches); Superpave base: 0.34; and LTSG: 0.11.

2) *Drainage Coefficient.* Depending on the quality of drainage and the availability of moisture, drainage coefficients are applied to granular bases and subbases to modify the layer coefficients. The drainage coefficient for flexible pavement design in Kansas is always 1.0.

5.3.2.1 Selection of Layer Thicknesses

Once the design SN for an initial pavement structure is computed, it is necessary to select a set of thicknesses so that the computed SN will be greater than the required SN. Many combinations of layer thicknesses are acceptable, so the cost effectiveness along with the construction and maintenance constraints must be considered (Huang 2004). However, in this study, cost effectiveness was not done so the minimum thickness derived from the computed SN was selected as the optimum thickness. It is important to note that minimum thicknesses may vary depending on local practices and conditions.

5.3.3 Specific Design Inputs for PCC Pavements

Effective Modulus of Concrete. The elastic modulus of concrete can be determined according to the procedure described in ASTM C469 or correlated with the concrete compressive strength. In this study, the correlated value was used.

Concrete Modulus of Rupture. The modulus of rupture required by the design procedure is the mean value after 28 days determined using the third-point loading flexural test, as specified in AASHTO T97 or ASTM C78. The modulus of rupture values listed in Table 3 was used in this study for the 1993 AASHTO design guide. In general, Kansas uses a 28-day modulus of rupture value of 600 psi in design.

Load Transfer Coefficient. The load transfer coefficient (J) indicates the ability of the loaded slab to transfer some of the applied traffic load across the joint to the adjacent slab. PCC pavements that are dowel jointed and have tied PCCP shoulders are considered to have a J factor of 2.8 in Kansas.

Drainage Coefficient The drainage coefficient (C_d) has the same effect as the load transfer coefficient (J). An increase in C_d is equivalent to a decrease in J, both causing an increase in the design traffic. The drainage coefficient used in Kansas JPCP design is 1.0 or 1.2 depending upon whether positive subsurface drainage is present or not. A value of 1.0 was used in this study.

5.4 MEPDG Design Inputs

5.4.1 Hierarchical Design Inputs

The hierarchical approach is used for the design inputs in MEPDG. This approach provides the designer with several levels of "design efficacy" that can be related to the class of highway under consideration or to the level of reliability of design desired. The hierarchical approach is primarily employed for traffic, materials, and environmental inputs (NCHRP 2004). In general, three levels of inputs are provided with Level 1 being the highest practically achievable level of reliability. Level 2 is the input level expected to be used in routine design. Level 3 typically is the lowest class of design. For a given design, it is permissible to mix different levels of input.

5.4.2 MEPDG Design Features

In the mechanistic-empirical (M-E) design, the key outputs are the individual distress quantities. For instance, for jointed plain concrete pavements, MEPDG analysis predicts distresses, such as faulting, transverse cracking, and smoothness in terms of

International Roughness Index (IRI). A reliability term has been incorporated in MEPDG for each predicted distress type to come up with an analytical solution, which allows the designer to design a pavement with an acceptable level of distress at the end of design life. The chosen failure criteria are associated with this design reliability. The failure criteria and design reliability are also required inputs for the MEPDG analysis although the designer and the agency have the control over these values. The design can fail if the predicted distress is greater than the allowable amount or if the predicted distresses are unacceptable. In this study, the design reliability used for all projects was 90%, and the corresponding failure criterion for IRI was 164 inches/mile. Other distress failure criteria, chosen according to the MEPDG default for both AC and PCC pavements, are summarized in Table 5.2. In the later part of the study, the AC failure criteria were changed. Table 5.2 also shows those criteria.

Project specific input parameters for the MEPDG AC and PCC pavement analysis in this study are given in Tables 5.3 and 5.4, respectively. Some of the inputs used in this study were similar for both AC and PCC pavements except for the structural details. Important traffic inputs like monthly and hourly truck distribution, axle load spectra, and truck class distribution were similar for both pavement types and were derived from an analysis of Weigh-In-Motion (WIM) and/or Automatic

Table 5.2: Failure Criteria for AC and PCC Pavements in MEPDG Analysis

Distress Type	MEPDG default	Revised MEPDG
AC Pavements		
Longitudinal Cracking (ft/mile)	1000	500
Alligator Cracking (%)	25	10
Transverse Cracking (ft/mile)	1000	500
Fatigue Fracture (%)	25	10
Permanent Deformation (AC Only, in)	0.25	0.25
Permanent Deformation (Total Pavement, in)	0.75	0.75
PCC Pavements		
Transverse Cracking (% slabs cracked)	15	15
Mean Joint faulting (in)	0.12	0.12

Vehicle Classification (AVC) data in Kansas. A brief description of the input parameters is given below:

General Information. The general information inputs include design life, construction month, traffic opening month, pavement type (AC/JPCP), initial smoothness (IRI), etc. All the AC sections in this study were analyzed for a 10-year design life. All PCC sections were jointed plain concrete pavements (JPCP) and were analyzed for 20-year design life.

Traffic. Traffic data is one of the key elements required for the MEPDG analysis. The basic required information is AADT for the year of construction, percentage of trucks in the design direction and in the design lane, operational speed and traffic growth rate. The traffic growth rates in MEPDG can be linear or exponential. Project-specific linear traffic growth rates varied from 1 to about 7%. Ninety-five percent trucks were assigned in the design direction based on the level 3 default input. Truck percentage in the design lane varied from 5% to 23%. For this study, some other required traffic inputs were derived from the Design Guide level 3 or default values.

Climate. The seasonal damage and distress accumulation algorithms in the MEPDG design methodology require hourly data for five weather parameters such as air temperature, precipitation, wind speed, percentage sunshine and relative humidity (NCHRP, 2004). The design guide recommends that the weather inputs be obtained from weather stations located near the project site. In this study, project specific virtual weather stations were created via interpolation of the climatic data from the selected physical weather stations.

Structural Details (AC Pavements). The flexible pavement sections consisted of varying thicknesses of the AC layer as shown in Table 5.3. Four of the AC sections were on a 6-inch LTSG, and the last layer was the natural compacted subgrade. The fifth project had an AC layer placed directly on the compacted natural subgrade. The inputs required for the AC layer were layer thickness, PG binder grade, gradation, Superpave mixture volumetric properties, Poisson’s ratio, reference temperature, etc.

Table 5.3: Input Parameters for MEPDG Flexible Pavement Design

Input	Design Value				
	(I-70 GE)	(I-70 DK)	(I-70 SN)	(K-7 JO)	(I-70 WB)
General Information					
Pavement construction date	Nov, 90	Oct, 92	Oct, 93	Sep, 95	May, 01
Traffic					
Initial two-way AADTT	1,656	2,790	1,800	968	3,690
No. of lanes in design direction	2	2	2	2	2
Traffic growth factor (%)	1..2	3.5	3	6.7	3.3
Design lane width (ft)	12	14	12	12	12
AC Layer					
AC Layer thickness (in)	12	12	12	10	13
Reference Temperature (^{0}F)	690	617	473	537	675.6
Total Unit Weight (pcf)	145	145	145	145	145
Air Voids (%)	4	4	4	4	4
Poisson's ratio	0.35	0.35	0.35	0.35	0.35
Treated Subgrade					
Subgrade type	N/A	LTSG	LTSG	LTSG	LTSG
Subgrade modulus (psi))	N/A	45,000	45,000	45,000	45,000
Unit weight (pcf)	N/A	121.2	120.9	120.9	120.8
Poisson's ratio	N/A	0.4	0.4	0.4	0.4
Compacted Subgrade					
Subgrade soil type	A-6	A-6	A-7-6	A-7-6	A-7-6
Subgrade Modulus (psi)	9,746	6,928	6,268	7,262	7,101
Plasticity index, PI	15.8	26	25.7	19.9	20.15
Percent passing %200 sieve	71.8	78.1	93.3	94.3	96.7
% passing #4 sieve	100	100	100	100	100
D_{60} (mm)	0.001	0.001	0.001	0.001	0.001

The software computed dynamic modulus (E*) using the default Witczak's predictive equation that takes into account gradation, volumetric properties, asphalt binder grade, and reference temperature (NCHRP 2004). The gradation and binder type and contents were selected according to the Superpave mixture types and PG binders used in the AC layers of the projects.

Structural Details (PCC Pavements). In this study, the baseline rigid pavement structure of four projects for the new design methodology is a three-layer JPCP construction consisting of a PCC slab over an LTSG. A 4-inch-thick Portland Cement Treated Base (PCTB) was used later in the analysis. The last layer is the natural compacted subgrade. The fifth project did not have an LTSG. The inputs required for the PCC layer were layer thickness, modulus of rupture (MR), material unit weight, etc. The MR values shown in Table 5.4 are the actual mean values obtained during construction. The inputs required for the base were layer thickness, mean modulus of elasticity of the layer, among others.

5.4.3 Miscellaneous

The thermo-hydraulic properties required as inputs were derived from the design guide level 3 values or determined based on correlations. County soil reports were used to estimate the ground water table depth for all sections.

Table 5.4: Input Parameters for MEPDG Rigid Pavement Baseline Design

Input	Design Value				
	(I-70 GE)	**(I-70 DK)**	**(I-70 SN)**	**(K-7 JO)**	**(I-70 WB)**
General Information					
Pavement construction date	Nov. 90	Oct. 92	Oct. 93	Sep. 95	May. 01
Traffic					
Initial two-way AADTT	1,656	2,790	1,800	968	3,690
No. of lanes in design direction	2	2	2	2	2
Traffic growth factor (%)	1..2	3.5	3	6.7	3.3
Design lane width (ft)	12	14	12	12	12
PCC Layer					
PCC Layer thickness (in)	9	11	11.5	10.5	10
Modulus of Rupture (MR) (psi)	690	617	473	537	675.6
Material Unit Weight (pcf)	140	139.2	142	142	141.3
Cement Type	I	I	I	I	I
Cement Content (lb/yd^3)	653	532	630	623	647.7
Poisson's ratio	0.20	0.20	0.20	0.20	0.2
Aggregate Type	Limestone	Limestone	Limestone	Limestone	Limestone
Co-eff of thermal exp (in./in./°Fx10^{-6})	5.5	5.5	5.5	5.5	5.5
Water-cement ratio (w/c)	0.44	0.35	0.411	0.46	0.437
Treated Subgrade					
Subgrade type	N/A	LTSG	LTSG	LTSG	LTSG
Subgrade modulus (psi))	N/A	45,000	45,000	45,000	45,000
Unit weight (pcf)	N/A	121.2	120.9	120.9	120.8
Poisson's ratio	N/A	0.4	0.4	0.4	0.4
Compacted Subgrade					
Subgrade soil type	A-6	A-6	A-7-6	A-7-6	A-7-6
Subgrade Modulus (psi)	9,746	6,928	6,268	7,262	7,101
Plasticity index, PI	15.8	26	25.7	19.9	20.15
Percent passing #200 sieve	71.8	78.1	93.3	94.3	96.7
Percent passing #4 sieve	100	100	100	100	100
D_{60} (mm)	0.001	0.001	0.001	0.001	0.001

5.5 Analysis and Results

5.5.1 Analysis Procedure

Initial sections were designed using the 1993 AASHTO design guide. These sections were then reanalyzed using the new MEPDG software. No base layer was used. All sections were built directly on LTSG wherever applicable. The analysis was done at 90% reliability for all sections and the terminal IRI was 164 inches per mile. The terminal IRI value was chosen based on the limits on this distress imposed by the KDOT Pavement Management System. The initial IRI was assumed to be 63 inches per mile and is the default value. The other distress failure criteria for the AC and PCC sections appear in Table 5.2. If the section passed all criteria for the smoothness (IRI) and other distresses (shown in Table 5.2), the thickness was reduced by 0.5 inch and the analysis was redone. This process was repeated until the section failed to pass in one of the failure criteria. The section that was eventually obtained was taken to be the equivalent MEPDG section as shown in Table 5.5.

5.5.1.1 AC Sections

The AC mixtures were designed following the Superpave mix design procedure. The AC layer was assumed to be built in three distinct sublayers (surface, base, and base) for all projects. The designation of the mixtures, shown in Table 5.5, follows the KDOT nomenclature for the Superpave mixes. In Kansas, a superpave mix is designated as "SM." The numeric following SM indicates the nominal maximum aggregate size (NMAS) in the mix in mm. The alphabet "A" immediately after that specifies the aggregate gradation i.e. it indicates that the gradation passed above the maximum density line in the finer sand sizes. Thus the gradation is finer and it allows inclusion of more sandy materials in the mix. In general, the surface mixture is a 3/8 inch (9.5 mm) NMAS mixture, and the binder and base mixtures are ¾ inch (19 mm) mixtures. For each sublayer, a distinct performance grade (PG) binder is used. The binder grades have been shown in parentheses in Table 5.5. In this analysis, the thickness of the base layer was altered mostly because of the lower temperature requirement of the PG binder in other layers (top 4 inches).

Table 5.5: Equivalent AC and PCC Pavement Sections obtained in MEPDG Analysis

Section	**(a) ACP Sections**									
	Shawnee		**Geary**		**Johnson**		**Dickinson**		**Wabaunsee**	
	1993*	M-E^	1993*	M-E^	1993*	M-E^	1993*	M-E^	1993*	M-E^
SM-9.5T (PG70-28)	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
SM-19A (PG70-28)	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
SM-19A (PG64-22)	8.0	3.0	8.0	4.5	6.0	3.0	8.0	3.0	9.0	4.0
TOTAL AC	**12.0**	**7.0**	**12.0**	**8.5**	**9.0**	**7.0**	**12.0**	**7.0**	**13.0**	**8.0**
LTSG	6.0	6.0	None	None	6.0	6.0	6.0	6.0	6.0	6.0
Section	**(b) PCCP Sections**									
	1993*	M-E^	1993*	M-E^	1993*	M-E^	1993*	M-E^	1993*	M-E^
PCCP	11.5	10.0	9.0	9.0	10.5	8.5	11.0	8.0	10.0	11.0
LTSG	6.0	6.0	None	None	6.0	6.0	6.0	6.0	6.0	6.0

* 1993 AASHTO ^ MEPDG

The results obtained using the MEPDG default criteria are shown in Table 5.5. Considerable thinner AC sections were obtained in the MEPDG analysis for all projects. The reduction in thickness varied from 2 inches for the K-7 Johnson County project to 5 inches for the I-70 projects in Shawnee, Dickinson, and Wabaunsee Counties. For the I-70 Geary County project, the reduction was 3.5 inches. These values tend to defy the historical precedence of providing asphalt pavement thicknesses on the heavily traveled corridor like the Interstate routes. It was concluded that the NCHRP MEPDG default criteria allowed far more distress quantities than that would be allowed on major highways in Kansas. This might have resulted in thinner AC pavement sections on all projects. Thus the MEPDG default failure criteria for the asphalt pavements were then altered (shown in Table 1) in order to establish the effects on the resulting sections, and the analysis was repeated for each project for a 10-year design period.

The thicknesses of the Johnson, Shawnee, and Wabaunsee County projects increased by 0.5 inches where as the Dickinson and Geary County projects showed a one-inch increase. It appears that the design AC pavement sections obtained in the MEPDG analysis are fairly sensitive to the failure criteria chosen.

5.5.1.2 PCC Sections

As mentioned earlier, all PCC sections in this study are JPCP with 15 foot joint spacing and dowelled joints built directly on the subgrade. The dowel diameter was estimated as one eighth of the slab thickness in inches. Sections where the PCC slab thickness was less than 10 inches, 1.25-inches diameter dowels were used. The concrete strength, in terms of 28-day modulus of rupture, ranged from 470 psi to 690 psi as shown in Table 2. All PCC sections have 12 foot lane widths with tied concrete shoulders except the Dickinson county project, which is a specific pavement study (SPS) section of the Long Term Pavement Performance (LTPP) program. This SPS-2 section has a widened lane of 14 feet with tied PCC shoulders. The results of the MEPDG analysis showed that for a 20-year design period, thinner PCC sections were also obtained for most projects although not to the extent that was observed in the AC analysis. No change in thickness was observed for the I-70 Geary County project as shown in Table 5.5. For the I-70 Wabaunsee county project, the initial section did not meet one of the criteria and the PCC slab thickness was 1.0 inch thicker than that obtained using the 1993 AASHTO guide.

In Kansas, a 4-inch thick Portland Cement Treated Base (PCTB) is generally used under the PCC slab. In the 1993 Design Guide method, Kansas assumes a k value of 110 MPa/m for this base on a lime treated subgrade (LTSG). However, the resulting slab thickness remains unchanged. In this study, all sections were reanalyzed with a 4 inch PCTB using the MEPDG methodology to find the effect of this base layer. The modulus of PCTB was assumed to be 500,000 psi and the unit weight of the PCTB layer materials was 135 pcf. The PCC slab thickness was reduced by 0.5 to 1 inch because of the PCTB base. Thus, the PCC slab thickness for the I-70 Wabaunsee County project with PCTB remained unchanged from that obtained using the 1993 AASHTO design guide.

CHAPTER 6 - CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

Kansas rigid pavement analysis results following the Mechanistic-Empirical Pavement Design Guide (MEPDG) have been presented in this report. Design analysis following MEPDG was done for eight in-service concrete pavement projects in Kansas. The predicted distresses were compared with the measured values. A sensitivity analysis of JPCP design following MEPDG was also done with respect to key input parameters used in the design process. Some alternative JPCP designs were also evaluated with the MEPDG analysis. The interaction of selected significant factors through statistical analysis was identified to find the effect on current KDOT specifications for rigid pavement construction. Based on the results of this study the following conclusions may be drawn:

1. For most projects in this study, the predicted IRI was similar to the measured values. MEPDG analysis showed minimal or no faulting and it was confirmed by visual observation. Cracking was predicted only on projects with lower flexural strength or lower slab thickness.
2. Predicted JPCP roughness (IRI) by MEPDG is very sensitive to varying thickness. Lower PCC slab thickness results in higher JPCP faulting. Variation in thickness also affects the predicted cracking.
3. Predicted JPCP roughness (IRI) and faulting by MEPDG are not very sensitive to the PCC compressive strength. However, slab cracking is affected by strength, and cracking decreases with increasing strength.

4. Predicted JPCP roughness (IRI) by MEPDG is very sensitive to varying dowel diameter. Lower dowel diameter results in higher JPCP faulting. However, variation in dowel diameter does not affect predicted cracking. No significant effect on IRI, faulting and slab cracking was observed for dowels spaced from 10 to 14 inches.
5. Effect of tied shoulder on predicted JPCP roughness, faulting, and percent slabs cracked is very pronounced. The distresses are markedly reduced by tied PCC shoulder. No faulting was observed for a JPCP with widened lane that also had tied PCC shoulder. Reduced roughness and lower cracking amount were also obtained for the project with a widened lane.
6. According to the MEPDG analysis, JPCP designs without treated base and subgrade show significant increase in predicted distresses. There are no marked differences in performance with respect to treated base type, although asphalt treated base (ATB) appeared to be beneficial in a few cases.
7. No significant variation on predicted distresses was observed for different soil type. However, clay soil predicts slightly higher distresses compare to the silty soil.
8. Effect of curing method on the predicted distresses is not very prominent though there are indications that wet curing may reduce faulting.
9. The effect of PCC CTE input on predicted roughness is more pronounced on JPCP's with thinner slabs or lower strength; however the level of input is not defined in the software. A combination of high cement factor and higher PCC CTE would result in higher JPCP faulting. In general, faulting is sensitive to the

PCC CTE values. However, no faulting was observed for a JPCP with widened lane that also had tied PCC shoulder. PCC CTE has a very significant effect on percent slabs cracked. PCC CTE does not affect the predicted IRI for a JPCP with widened lane and tied PCC shoulder.

10. In general, the shrinkage does not greatly affect predicted IRI. The higher shrinkage strain results in higher faulting. Cracking appears to be fairly insensitive to the shrinkage strainch
11. MEPDG predicted IRI and percent slabs cracked are fairly insensitive to the zero-stress temperature but the faulting is severely affected. However, a JPCP section (SPS-2 Section 6) with widened lane and tied PCC shoulder did not show any faulting even for the highest zero-stress temperature. April and October are the best months for JPCP construction (paving) in Kansas.
12. Lower PCC slab thickness would result in higher JPCP faulting for a given traffic input. However, the predicted faulting values in this project were negligible for all practical purposes.
13. Monthly adjustment factors for the truck traffic are necessary in Kansas since traffic is heavier during the winter and spring months (December through April). Truck traffic type distributions for some functional classes in Kansas are dissimilar to those in MEPDG default. In contrast to the MEPDG default axle load spectra, Kansas has a higher percentages of trucks distributed in the lower axle load categories. Predicted JPCP roughness (IRI) by MEPDG is very sensitive to thickness at varying traffic level. However, traffic inputs studied in this project did not affect the predicted IRI for a JPCP with widened lane and tied PCC shoulder.

MEPDG traffic input causes more JPCP slab cracking than the Kansas input. Effects of higher AADT and truck traffic on predicted roughness, faulting and percent slabs cracked is more pronounced on the JPCP pavements with thinner slabs or lower strength. Variations in truck type do not affect predicted distresses on JPCP.

14. For the AC sections, the MEPDG procedure resulted in much thinner sections when compared to the sections obtained following the 1993 AASHTO design guide. However, these results were found to be sensitive to the failure criteria chosen.
15. Four of the PCC sections, designed using the 1993 AASHTO guide, were thicker than those analyzed following the NCHRP MEPDG. The thickness of the fifth project was the same in both cases.
16. The stabilized base layer in the PCC pavement was found to affect the resulting PCC slab thickness.

6.2 Recommendations

The following recommendations are made for further studies:

1. The AASHTO TP-60 tests can produce very valuable inputs in the MEPDG JPCP design process. This test should be implemented. A precision and bias statement should be developed.
2. Traffic data analysis needs to be extensive.
3. Based on the statistical analysis, each MEPDG JPCP design analysis should be studied for sensitivity toward PCC strength. An Upper Specification Limit (USL) also should be considered for strength in KDOT PWL specifications.

4. Interactions of several other key input parameters such as, dowel diameter, coefficient of thermal expansion, etc. with the PCC thickness and strength need to be studied in future research.

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