CHAPTER 7
PRINCIPLES OF DESIGN FOR RIGID PAVEMENT

7.1 Introduction

Rigid pavement design is based on the mechanistic-empirical (M-E) design concepts. The design procedure utilizes distress and smoothness prediction models developed and calibrated locally. The *MEPDG Design Guide* and the *AASHTO Interim MEPDG Manual of Practice* documents provide a detailed description of the M-E concepts for rigid pavement designs.

The design procedures described in this chapter can be used for design of new or reconstructed rigid pavements. There are no fundamental differences in the pavement design procedure for new alignment and reconstruction, however, the potential reuse of the materials from the existing pavement structure can be an important issue. Refer to **CHAPTER 9: Principles of Design for Pavement Rehabilitation with Rigid Overlay** when rehabilitation designs are necessary with rigid overlays or restoration projects.

The design life for typical thin white topping should be 10 to 20 years for rehabilitations and 30 years for reconstruction. An overview of the proven concrete pavement practices the Colorado Department of Transportation (CDOT) has implemented over the last several years is documented in the Final Research Report CDOT-DTD-R-2006-9, *Implementation of Proven PCCP Practices in Colorado*, dated April 2006 (8).

7.2 M-E Design Methodology for Rigid Pavement

The M-E Design of rigid pavements is an iterative process. The key steps in the design process include the following:

- **Select a Trial Design Strategy**

- **Select the Appropriate Performance Indicator Criteria for the Project:** Establish criteria for acceptable pavement performance (i.e. distress/IRI) at the end of the design period. CDOT criteria for acceptable performance is based on highway functional class and location. The performance criteria is established to reflect magnitudes of key pavement distresses and smoothness that trigger major rehabilitation or reconstruction.

- **Select the Appropriate Reliability Level for the Project:** The reliability is a factor of safety to account for inherent variations in construction, materials, traffic, climate, and other design inputs. The level of reliability selected should be based on the criticality of the design. CDOT criteria for desired reliability is based on highway functional class and location. The desired level of reliability is selected for each individual performance indicator.

- **Assemble All Inputs for the Pavement Trial Design Under Consideration:** Define subgrade support, PCC and other paving material properties, traffic loads, climate, pavement type, and design/construction features. The inputs required to run M-E Design may be obtained using one of three hierarchical levels of effort and need not be consistent for all of the inputs in a given design. A hierarchical level for a given input is selected based on the importance of the project and input, and the resources at the disposal of the designer.

- **Run the M-E Design Software:** The software calculates changes in layer properties, damage, key distresses, and IRI over the design life. The key steps include:

  - **Processing Input** to obtain monthly values of traffic inputs and seasonal variations of material and climatic inputs needed in the design evaluations for the entire design period.

  - **Computing Structural Responses** (stresses and strains) using finite element based pavement response models for each axle type and load and damage-calculation increment throughout the design period.

  - **Calculating Accumulated Distress** and/or damage at the end of each analysis period for the entire design period.

  - **Predicting Key Distresses** (JPCP transverse cracking and joint faulting) at the end of each analysis period throughout the design life using the calibrated mechanistic-empirical performance models.

  - **Predicting Smoothness** as a function of initial IRI, distresses that accumulate over time, and site factors at the end of each analysis increment.

- **Evaluate the Adequacy of the Trial Design:** The trial design is considered "adequate" if none of the predicted distresses/IRI exceed the performance indicator criteria at the design reliability level chosen for the project. If any of the criteria has been exceeded, determine how this deficiency can be remedied by altering material types and properties, layer thicknesses, or other design features.

- **Revise the Trial Design, as Needed:** If the trial design is deemed "inadequate", revise the inputs/trial design and re-run the program. Iterate until all the performance criteria have been met. Once they have been met, the trial design becomes a feasible design alternative.

The design alternatives that satisfy all performance criteria are considered feasible from a structural and functional viewpoint and can be further considered for other evaluations, such as life cycle cost analysis. A detailed description of the design process is presented in the interim edition of the AASHTO *Mechanistic-Empirical Pavement Design Guide Manual of Practice*, AASHTO, 2008.

7.3 Select Trial Design Strategy

7.3.1 Rigid Pavement Layers

Figure 7.1 Rigid Pavement Layers shows a conventional rigid layered system. The PCC slab may be placed over base, subbase, or directly on a prepared subgrade. The base (layer directly beneath the PCC slab) and subbase layers (layer placed below the base layer) may include unbound aggregates, asphalt stabilized granular, cement stabilized, lean concrete, crushed concrete, lime stabilized, recycled asphalt pavement (RAP), and other materials. Base/subbase layers may be dense graded or permeable drainage layers.

Transverse joints are closely spaced in JPCP, typically between 10 and 20 feet, to minimize transverse cracking from temperature and moisture gradients. JPCP may have tied or untied longitudinal joints.

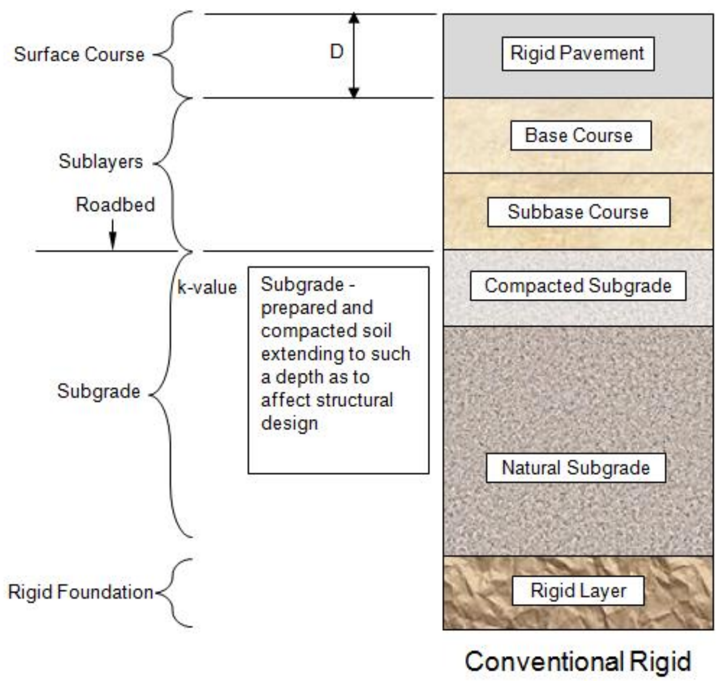

Figure 7.1 Rigid Pavement Layers

7.3.2 Establish Trial Design Structure

The designer must establish a trial design structure (combination of material types and thicknesses). This is done by first selecting the pavement type (see **Figure 7.2 M-E Design Screenshot Showing General Information Performance Criteria and Reliability**). M-E Design automatically provides the top layers of the selected pavement type. The designer may add or remove pavement structural layers and modify layer material type and thickness as appropriate. **Figure 7.3 M-E Design Screenshot of Rigid Pavement Trial Design Structure** shows the pavement layer configuration of a sample rigid pavement and trial design on the left and layer properties of the PCC slab on the right.

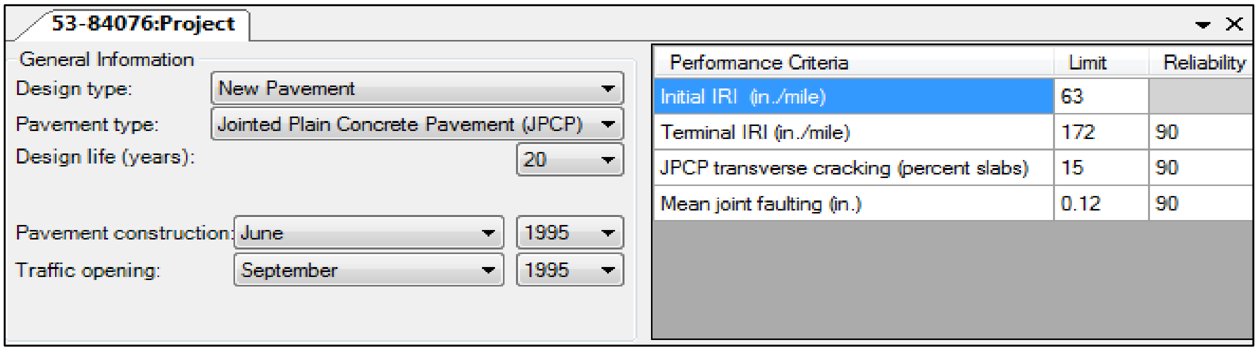

Figure 7.2 M-E Design Screenshot Showing General Information, Performance Criteria, and Reliability

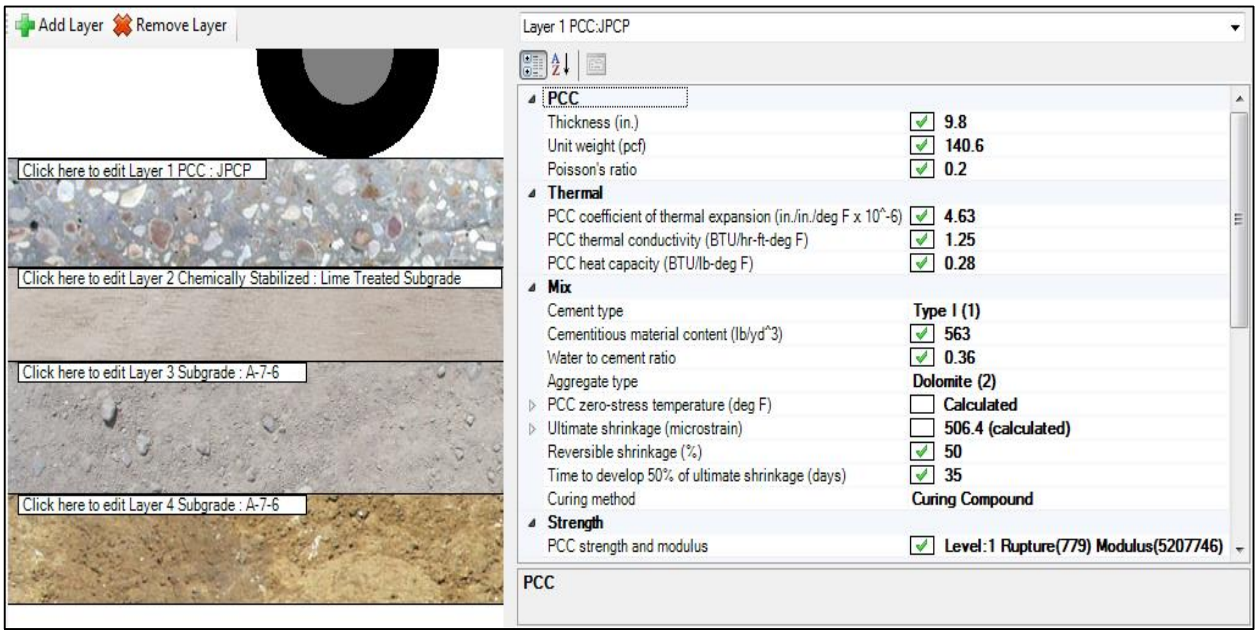

Figure 7.3 M-E Design Screenshot of Rigid Pavement Trial Design Structure

7.4 Select the Appropriate Performance Indicator Criteria for the Project

Table 2.4 Recommended Threshold Values of Performance Criteria for New Construction or Reconstruction Projects presents recommended performance criteria for a rigid pavement design. The designer should enter the appropriate performance criteria based on functional class. An appropriate initial smoothness (IRI) is also required. **<u>For new rigid pavements, the recommended initial IRI is 75 inches/mile.</u>** This recommendation is for regular paving projects and projects with incentive-based smoothness acceptance; the designer may modify this value as needed**. Figure 7.3 M-E Design Screenshot Showing General Information, Performance Criteria, and Reliability** shows performance criteria for a sample rigid pavement trial design. The coefficients of performance prediction models considered in the design of a rigid pavement are shown in **Figure 7.4 Performance Prediction Model Coefficients for Rigid Pavement Designs.**

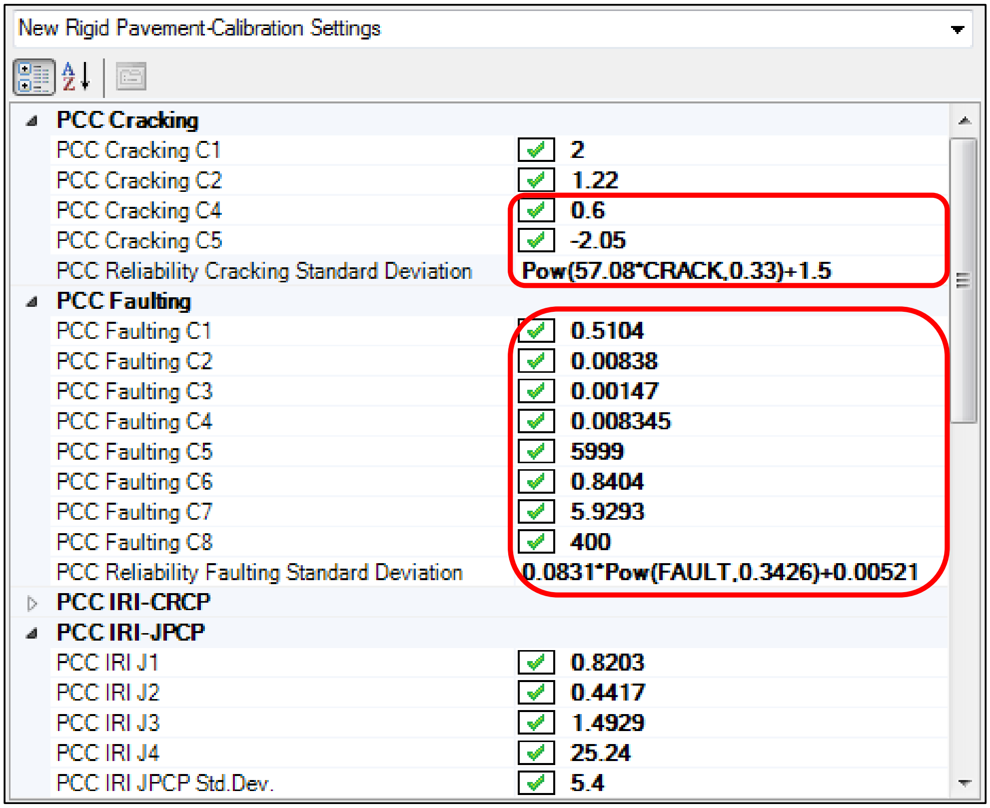

Figure 7.4 Performance Prediction Model Coefficients for Rigid Pavement Designs

7.5 Select the Appropriate Reliability Level for the Project

Table 2.3 Reliability (Risk) presents recommended reliability levels for rigid pavement designs. The designer should select an appropriate reliability level based on highway functional class and location (see **Figure 7.3 M-E Design Screenshot Showing General Information, Performance Criteria, and Reliability**).

7.6 Assemble the M-E Design Inputs

7.6.1 General Information

7.6.1.1 Design Period

<u>The design period for new rigid pavement construction and reconstruction is 20 or 30 years. It is recommended a 30-year design period be used for rigid pavements.</u> Selection of a design period other than 10, 20, or 30 years needs to be supported by a LCCA or other overriding considerations.

7.6.1.2 Project Timeline

The following inputs are required to specify the project timeline in the design (see **Figure 7.3 M-E Design Screenshot Showing General Information, Performance Criteria and Reliability**).

- Pavement construction month and year
- Traffic open month and year

The designer may select the most likely month and year when the PCC surface layer is scheduled to be placed, and when the pavement section is scheduled to be opened to traffic. Changes to the surface layer material properties due to time and environmental conditions are considered beginning from the construction date. Due to warping, curling and other factors, if the actual month(s) of construction is unknown then the month of August should be used.

7.6.1.3 Identifiers

Identifiers are helpful in documenting the project location and recordkeeping. M-E Design allows designers to enter site or project identification information, such as the location of the project (route signage, jurisdiction, etc.), identification numbers, beginning and ending milepost, direction of traffic, and date.

7.6.1.4 Traffic

Several inputs are required for characterizing traffic for M-E Design and are described in detail in **Section 3.1 Traffic.**

7.6.1.5 Climate

The climate input requirements for M-E design are described in detail in **Section 3.2 Climate**.

7.6.1.6 Pavement Layer Characterization

As shown in **Figure 7.1 Rigid Pavement Layers**, a typical rigid pavement design comprises of the following pavement layers: PCC, treated and/or unbound aggregate base, and subgrade. The inputs required by the M-E Design software for characterizing these layers are described in the following sections.

7.6.1.7 Portland Cement Concrete

The inputs required for PCC layer characterization are divided into three categories (see **Figure 7.5 PCC Layer and Material Properties in M-E Design**).

- **General and Thermal Properties:** This category includes layer thickness, Poisson's ratio, Coefficient of Thermal Expansion (CTE), thermal conductivity, and heat capacity.

- **PCC Mix-Related Properties**: This category includes cement type (Types I, II, or III), cement content, water/cement (or w/c) ratio, aggregate type, PCC zero-stress temperature, ultimate shrinkage at 40 percent relative humidity, reversible shrinkage, and curing method.

- **Strength and Stiffness Properties**: This category includes modulus of rupture (flexural strength), static modulus of elasticity, and/or compressive strength.

These inputs are required for predicting pavement responses to applied loads, long-term strength and elastic modulus, and effect of climate (temperature, moisture, and humidity) on PCC expansion and contraction. **Table 7.1 PCC Material Inputs and Recommendations for New JPCP Designs** presents recommendations for inputs used in PCC material characterization for a new JPCP design. Level 1 inputs of typical CDOT PCC mixtures may be used for Levels 2 and 3 (see **APPENDIX G**). Refer to **Table 2.6 Selection of Input Hierarchical Level** for selection of an appropriate hierarchical level for material inputs.

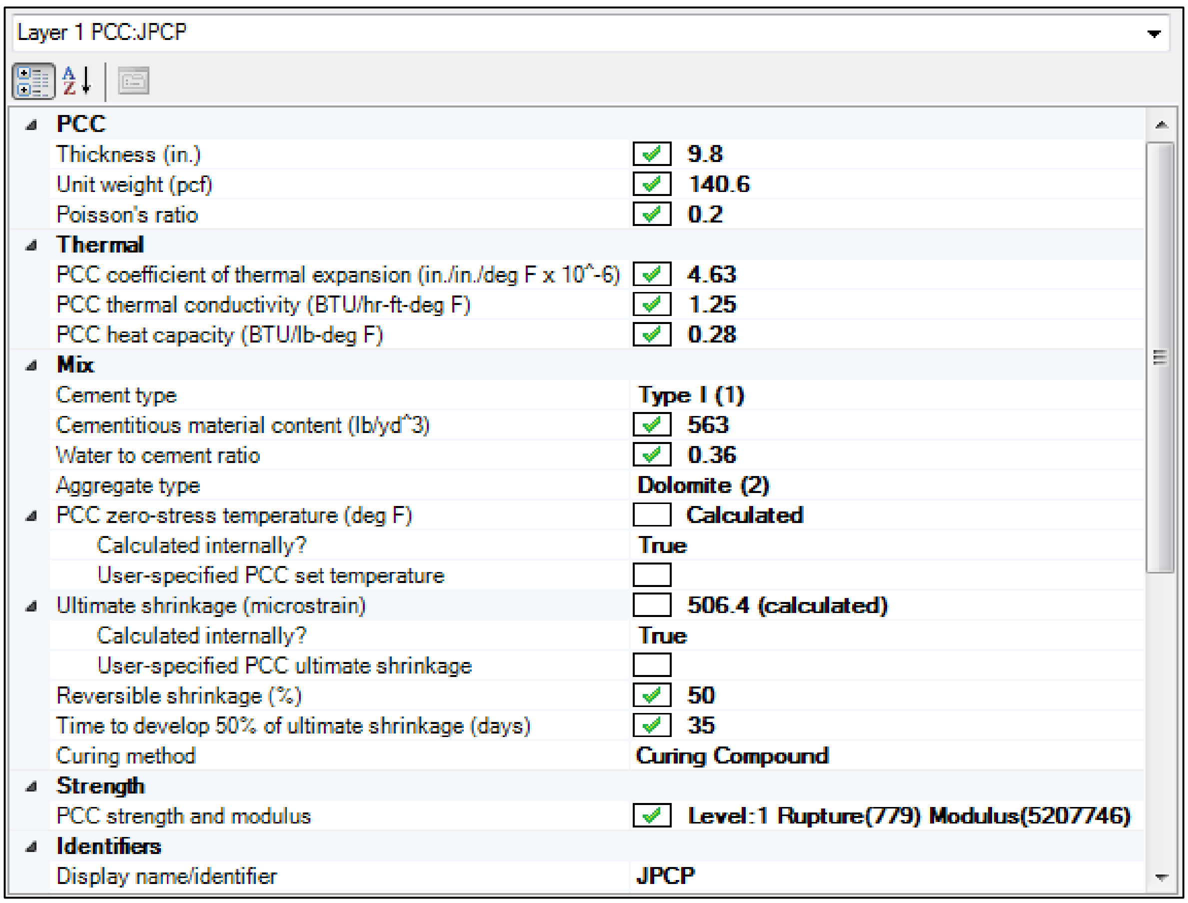

Figure 7.5 PCC Layer and Material Properties in M-E Design

Table 7.1 PCC Material Inputs and Recommendations for New JPCP Design

Input Property (Strength)	Input Hierarchy		
	Level 1	**Level 2**	**Level 3**
Elastic Modulus	Mix specific values (ASTM C 469)		Use typical values from **APPENDIX G**. Select a mix that is closest to the project. Use a default ratio of 1.20 for 20-year / 28-day strength gain of elastic modulus and flexural strength.
Flexural Strength	Mix specific values (AASHTO T 97)		
Compressive Strength		Mix specific values (AASHTO T 22)	
Unit Weight	Mix specific values (AASHTO T 121)	**APPENDIX G**	
Poisson's Ratio	Mix specific values (ASTM C 469)	**APPENDIX G**	
Coefficient of Thermal Expansion	Mix specific values (AASHTO TP 60)	**APPENDIX G**	
Surface Shortwave Absorptivity	0.85		
Thermal Conductivity	1.25		
Heat Capacity	0.28		
Cement Type	Mix specific values	Typical values from the CDOT PCC input library. Select a mix that is closest to the project.	
Cementitious Material Content	Mix specific values	Typical values from the CDOT PCC input library. Select a mix that is closest to the project.	
Water to Cement Ratio	Mix specific values	Typical values from the CDOT PCC input library. Select a mix that is closest to the project.	
Curing Method	Select an appropriate method based on Section 412.14 of *CDOT Standard Specifications for Road and Bridge Construction*		
PCC Zero-stress Temperature	Internally calculated		
Ultimate Shrinkage	Internally calculated		
Reversible Shrinkage	50 percent		
Time to Develop 50 Percent of Ultimate Shrinkage	35 days		

7.6.1.8 Asphalt Treated Base Characterization

The asphalt treated base layer is modeled as a HMA layer. The material input requirements are identical to those of a conventional HMA layer as described in **Section 6.6.4.1 Asphalt Concrete**

Characterization, with an exception to indirect tensile strength and creep compliance values. For JPCP designs, no sub-layering is done within the asphalt treated base layer.

7.6.1.9 Chemically Stabilized Base Characterization

Refer to **Section 5.4.1 Characterization of Treated Base in M-E Design** for treated base characterization.

7.6.1.10 Unbound Material Layers and Subgrade Characterization

Refer to **Section 5.3.1 Unbound Layer Characterization in M-E Design** for unbound aggregate base layer characterization; and refer to **Section 4.4 Subgrade Characterization for M-E Design** for subgrade characterization.

7.6.2 JPCP Design Features

JPCP design features and construction practices influence long-term performance. The common design features considered in M-E Design (see **Figure 7.6 M-E Design Screenshot of JPCP Design Features**) include the following:

- Surface shortwave absorptivity: Refer to **Table 7.1 PCC Material Inputs and Recommendations for New JPCP Design**
- Joint spacing: Refer to **Section 7.10 Joint Spacing (L**)
- PCC-base contact friction: Refer to **Section 7.11 Slab/Base Friction**
- Permanent curl/warp effective temperature difference: Refer to **Section 7.12 Effective Temperature Differential (°F)**
- Widened slab: Refer to **Section 7.14 Lane Edge Support** Condition
- Dowel bars: Refer to **Section 7.13 Dowel Bars (Load Transfer Devices) and Tie Bars**
- Tied shoulders: Refer to **Section 7.13 Dowel Bars (Load Transfer Devices) and Tie Bars** and **Section 7.14 Lane Edge Support Condition**
- Base type and erodibility index: Refer to **Section 7.15 Base Erodibility**
- Sealant type: Refer to **Section 7.16 Sealant Type**

7.7 Run M-E Design

Designers should examine all inputs for accuracy and reasonableness prior to running M-E Design. The designer will run the software to obtain outputs required for evaluating whether the trial design is adequate. After a trial run has been successfully completed, M-E Design will generate a report in form of a PDF and/or Microsoft Excel file, see **Figure 7.7 Sample Rigid Pavement Design PDF Output Report**. The report contains the following information: inputs, reliability of design, materials and other properties, and predicted performance.

After the trial run is complete, the designer should examine all inputs and outputs for accuracy and reasonableness. The output report also includes the estimates of material properties and other properties on a month-by-month basis over the entire design period in either tabular or graphical form. For a JPCP pavement trial design, the report provides the following:

- PCC flexural strength/modulus of rupture
- PCC elastic modulus
- Unbound material resilient modulus
- Subgrade k-value
- Cumulative trucks (FHWA Class 4 through 13) over the design period

Once again, the designer should examine the above mentioned parameters to assess their reasonableness before accepting a trial design as complete.

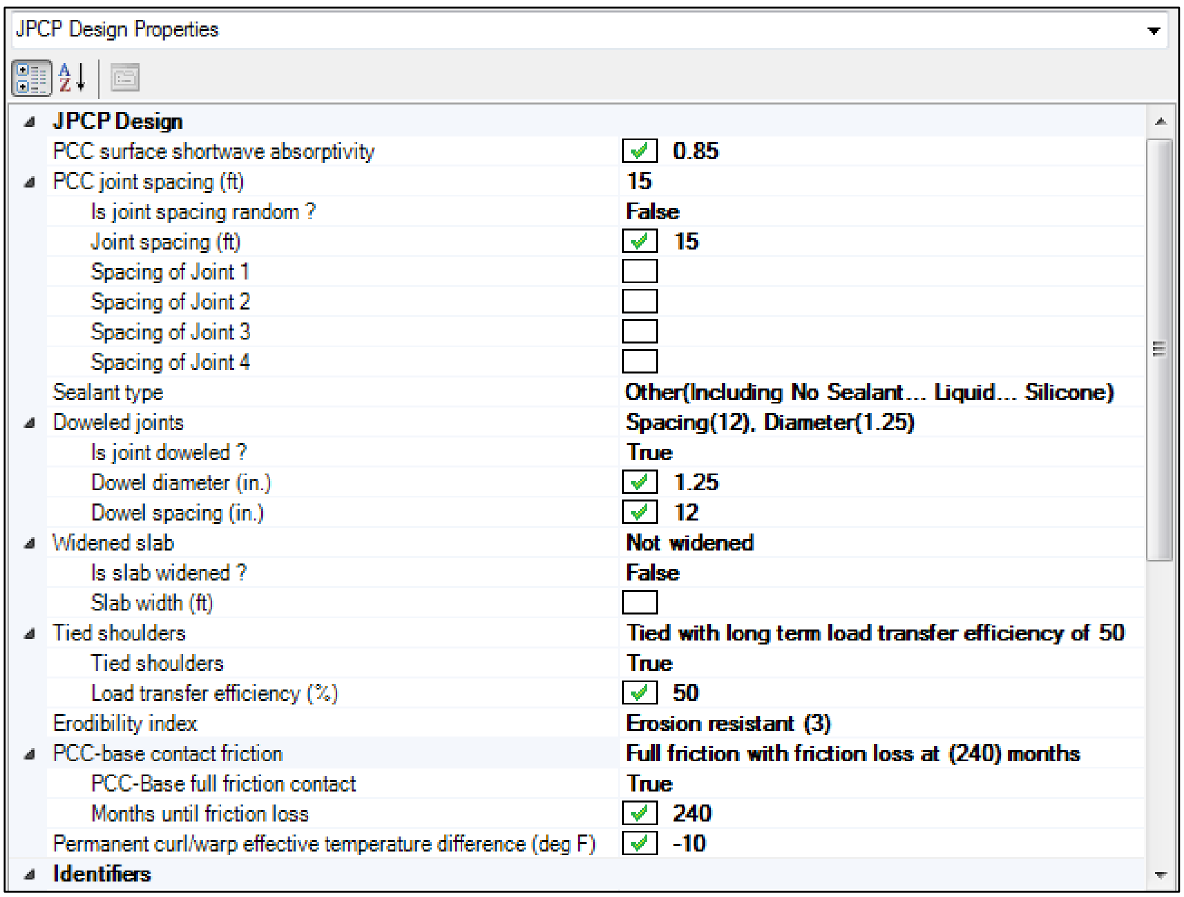

Figure 7.6 M-E Design Screenshot of JPCP Design Features

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Design Inputs

Design Life: 20 years
Design Type: Jointed Plain Concrete Pavement (JPCP)

Existing construction: -
Pavement construction: June, 1995
Traffic opening: September, 1995

Climate Data Sources (Lat/Lon): 39.909, -105.117; 39.833, -104.658; 39.702, -104.752; 39.57, -104.849; 39.783, -104.55; 40.452, -105.001; 40.436, -104.618

Design Structure

Layer type	Material Type	Thickness (in.):
PCC	JPCP	9.8
Cement_Base	Lime Treated Subgrade	8.5
Subgrade	A-7-6	12.0
Subgrade	A-7-6	Semi-infinite

Joint Design:	
Joint spacing (ft)	15.0
Dowel diameter (in.)	1.25
Slab width (ft)	12.0

Traffic

Age (year)	Heavy Trucks (cumulative)
1995 (initial)	619
2005 (10 years)	2,625,540
2015 (20 years)	6,251,140

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	172.00	95.59	90.00	100.00	Pass
Mean joint faulting (in.)	0.12	0.03	90.00	100.00	Pass
JPCP transverse cracking (percent slabs)	15.00	1.92	90.00	100.00	Pass

Figure 7.7 Sample Rigid Pavement Design PDF Output Report

7.8 Evaluate the Adequacy of the Trial Design

The output report of a rigid pavement trial design includes the monthly accumulation of the following key distress types at their mean values and chosen reliability for the entire design period:

- **Joint Faulting**: This is an indicator of erosion of sublayers and the effectiveness of joint LTE. A critical value is reached when joint faulting results in excess roughness, which is unacceptable to drivers and difficult to remove by re-texturing.
  - The designer should examine the results to evaluate if the performance criteria for joint faulting are met at the desired reliability. If joint faulting has not been met, the trial design is deemed unacceptable and revised accordingly to produce a satisfactory design.
  - The output report also includes the monthly accumulation of the following secondary distress types and smoothness indicators at their mean values and chosen reliability values for the entire design period.
- **Percent Slabs Cracked**: This is the mean predicted transverse cracks that form from fatigue damage at the top and bottom of the slab. Beyond a critical threshold, the rate of cracking accelerates and may require significant repairs and lane closures.

- **IRI**: This is a function of joint faulting and slab cracking along with climate and subgrade factors. A high IRI indicates unacceptable ride quality.

  - The designer should examine the results to evaluate if the performance criteria for percent slabs cracked and IRI meet the **minimum of 27 years** at the desired reliability.

  - If any of the criteria have not been met, the trial design is deemed unacceptable and revised accordingly to produce a satisfactory design.

Another important output is the reliability levels of each performance indicator at the end of the design period. If the reliability value predicted for the given performance indicator is greater than the target/desired value, the trial design passes for that indicator. If the reverse is true, then the trial design fails to provide the desired confidence and performance indicator will not reach the critical value during the pavement's design life. In such an event, the designer needs to alter the trial design to correct the problem.

The strategies for modifying a trial design are discussed in **Section 7.9 Modifying Trial Designs**. The designer can use a range of thicknesses to optimize the trial design and make it more acceptable. Additionally, the software allows the designer to perform a sensitivity analysis for key inputs. The results of the sensitivity analysis can be used to further optimize the trial design if modifying PCC thickness alone does not produce a feasible design alternative. A detail description of the thickness optimization procedure and sensitivity analysis is provided in the *Software HELP Manual*.

7.9 Modifying Trial Designs

An unsuccessful trial design may require revisions to ensure all performance criteria are satisfied. The trial design is revised by systematically modifying the design inputs. The design acceptance in M-E Design is distress-specific; in other words, the designer needs to first identify the performance indicator that failed to meet the performance targets and modify one or more design inputs that has a significant impact on a given performance indicator accordingly. The impact of design inputs on performance indicators is typically obtained by performing a sensitivity analysis.

The strategies to produce a satisfactory design by modifying design inputs can be broadly categorized into:

- Pavement layer considerations:
  - Increasing layer thickness
  - Modifying layer type and layer arrangement
  - Foundation improvements

- Pavement material improvements:
  - Use of higher quality materials
  - Material design modifications
  - Construction quality

Remember, when modifying the design inputs, the designer needs to be aware of input sensitivity to various distress types. Changing a single input to reduce one distress may result in an increase in another distress. **Table 7.2 Modifying Rigid Pavement Trial Designs** presents a summary of inputs that may be modified to optimize trial designs and produce a feasible design alternative.

Table 7.2 Modifying Rigid Pavement Trial Designs

Distress/IRI	Design Inputs that Impact
Transverse Cracking	• Increase slab thickness • Increase PCC strength • Minimize permanent curl/warp through curing procedures that eliminate built-in temperature gradient • PCC tied shoulder (separate placement or monolithic placement). • Widened slab (1 to 2 feet) • Use PCC with a lower coefficient of thermal expansion
Joint Faulting	• Increase slab thickness • Reduce joint width over analysis period • Increase erosion resistance of base (specific recommendations for each type of base) • Minimize permanent curl/warp through curing procedures that eliminate built-in temperature gradient • PCC tied shoulder • Widened slab (1 to 2 feet)
IRI	• Require more stringent smoothness criteria and greater incentives • Increase slab thickness • Ensure PCC has proper entrained air content • Decrease joint spacing • Widen the traffic lane slab by 2 feet • Use a treated base (if nonstabilized dense graded aggregate was specified) • Increase diameter of dowels

Figure 7.8 Sensitivity of JPCP Transverse Cracking to PCC Thickness through **Figure 7.19 Sensitivity of JPCP IRI to Design Reliability** presents sensitivity plots of a sample rigid pavement trial design showing the effects of key inputs, such as traffic volume, PCC thickness, PCC coefficient of thermal expansion, and design reliability on key distresses. **Note:** The plots do not cover the effects of all key factors on rigid pavement performance; other significant factors are not shown herein.

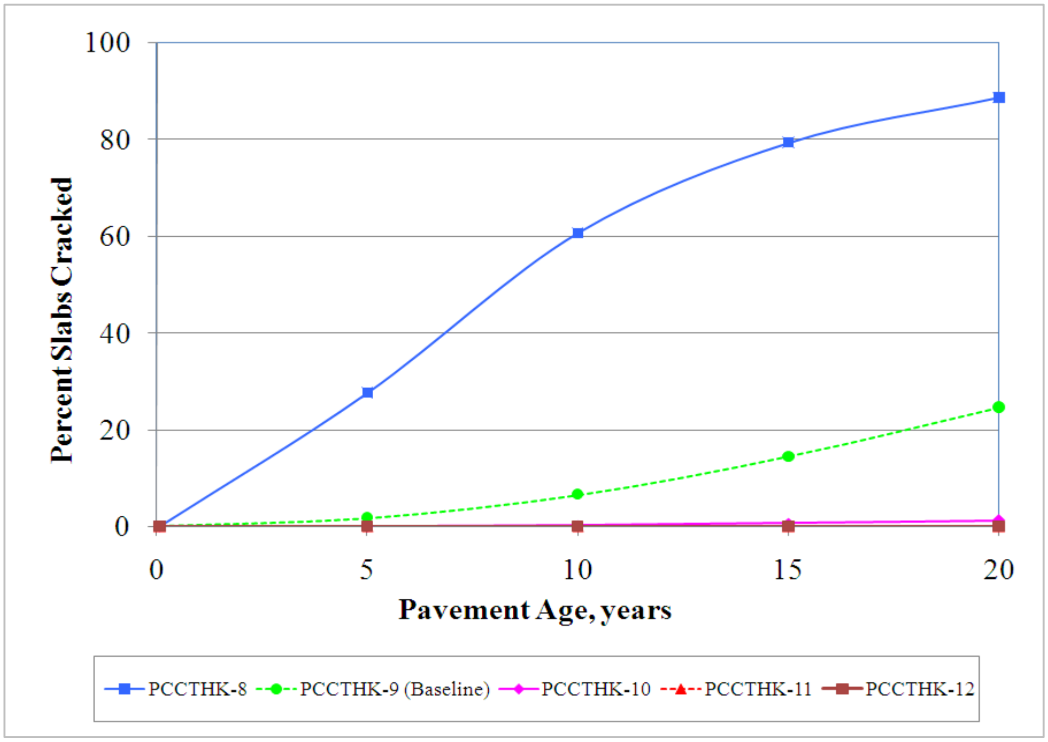

Figure 7.8 Sensitivity of JPCP Transverse Cracking to PCC Thickness

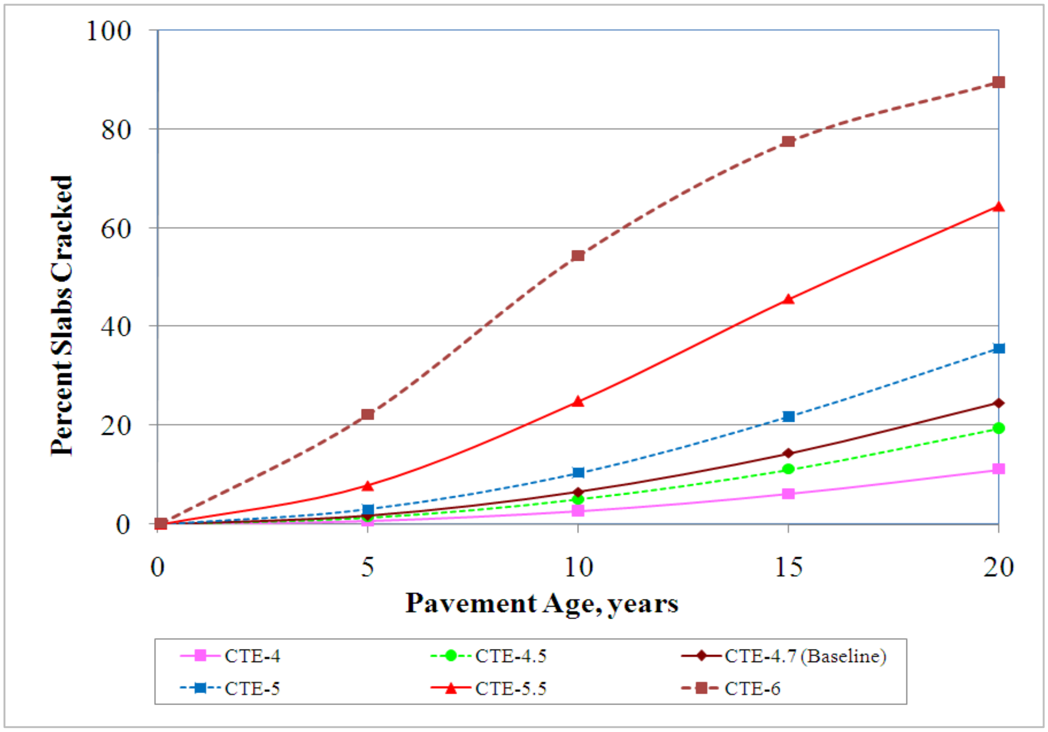

Figure 7.9 Sensitivity of JPCP Transverse Crackling to PCC Coefficient of Thermal Expansion

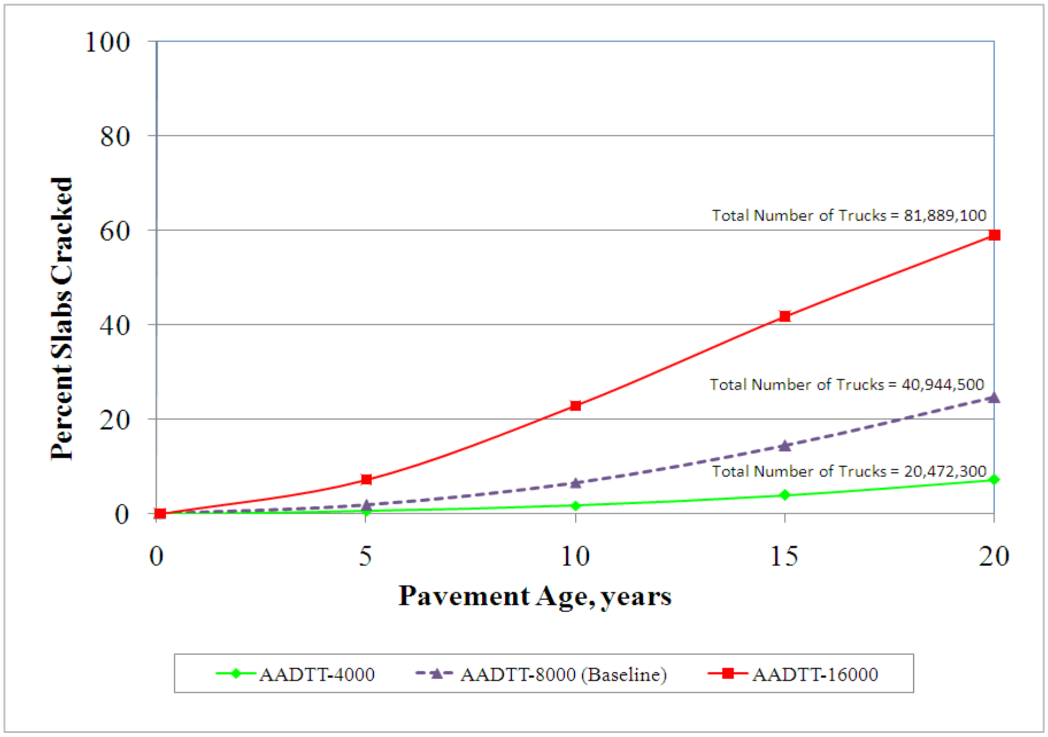

Figure 7.10 Sensitivity of JPCP Transverse Cracking to Traffic Volume

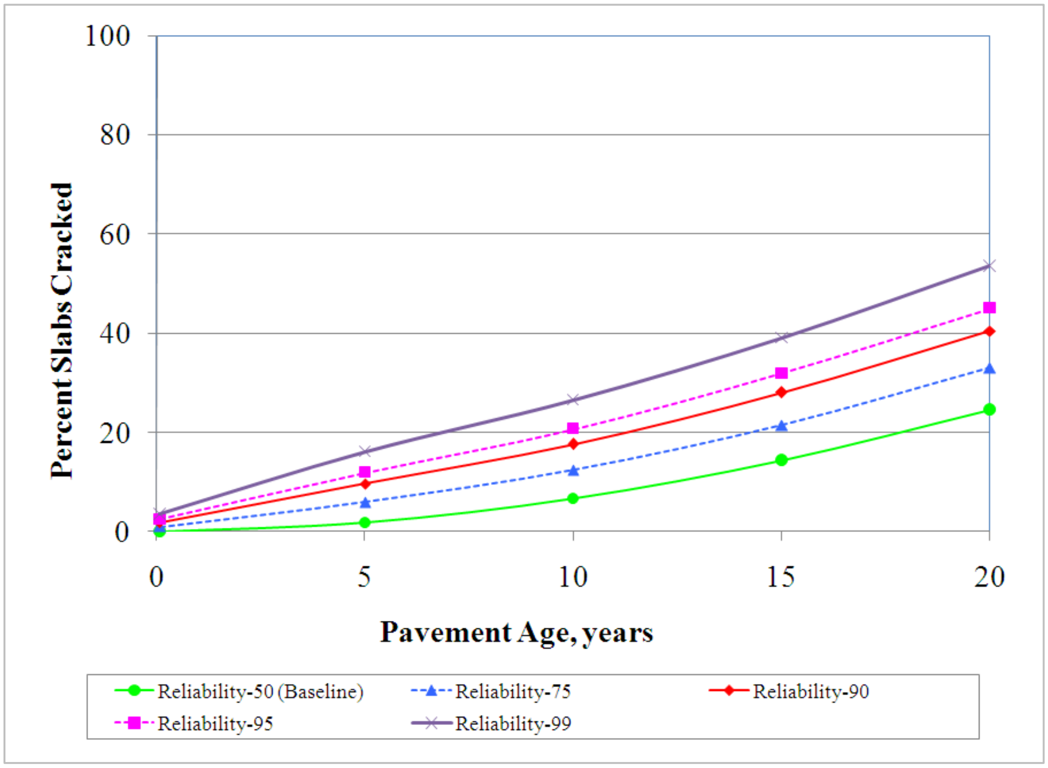

Figure 7.11 Sensitivity of JPCP Transverse Cracking to Design Reliability

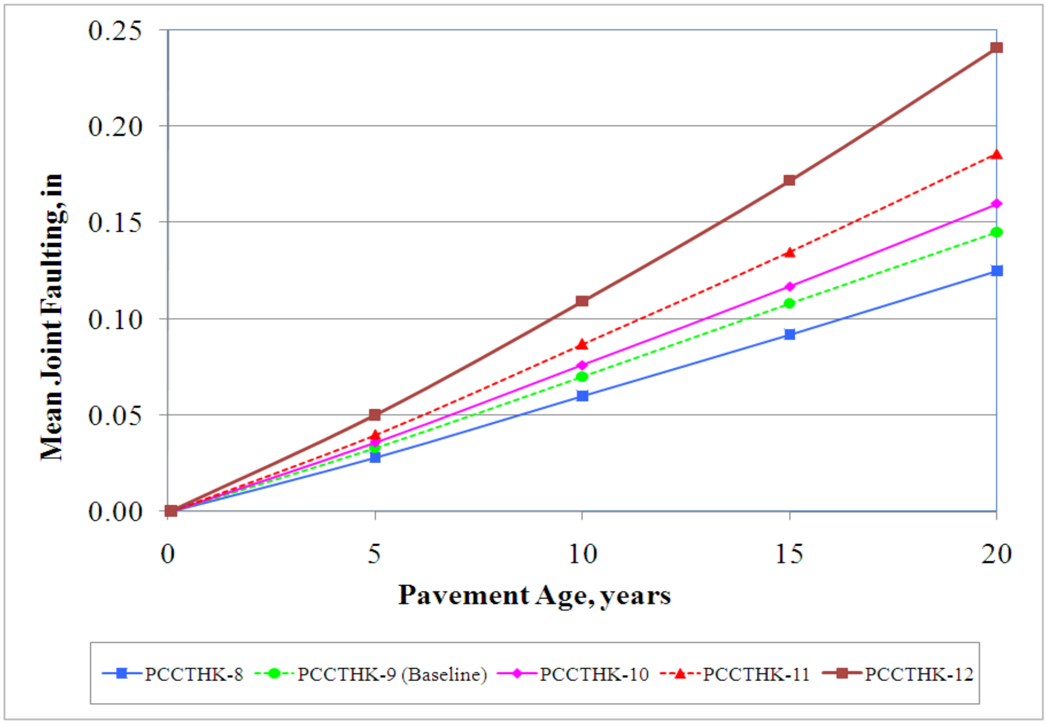

Figure 7.12 Sensitivity of JPCP Faulting to PCC Thickness

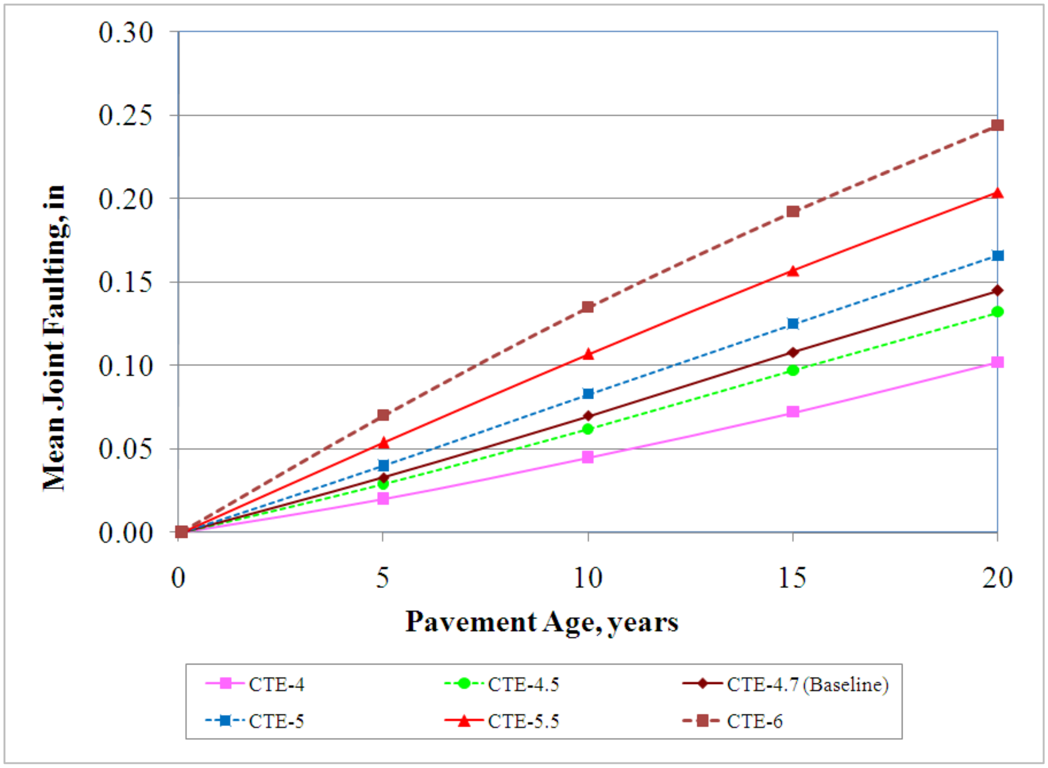

Figure 7.13 Sensitivity of JPCP Faulting to PCC Coefficient of Thermal Expansion

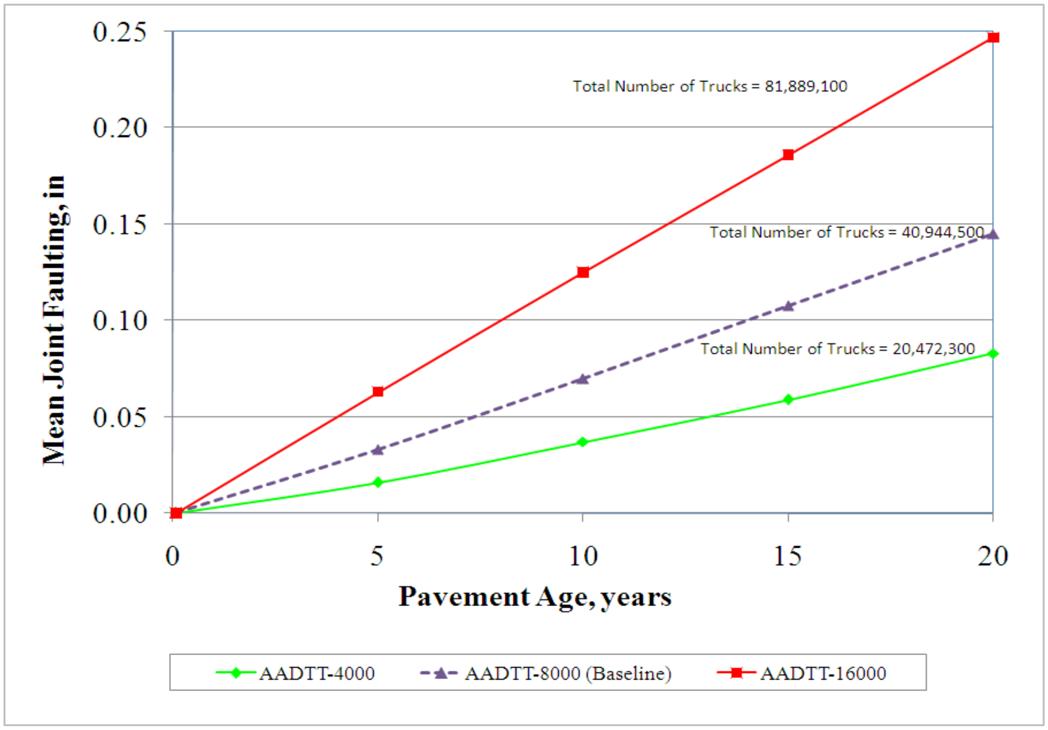

Figure 7.14 Sensitivity of JPCP Faulting to Traffic Volume

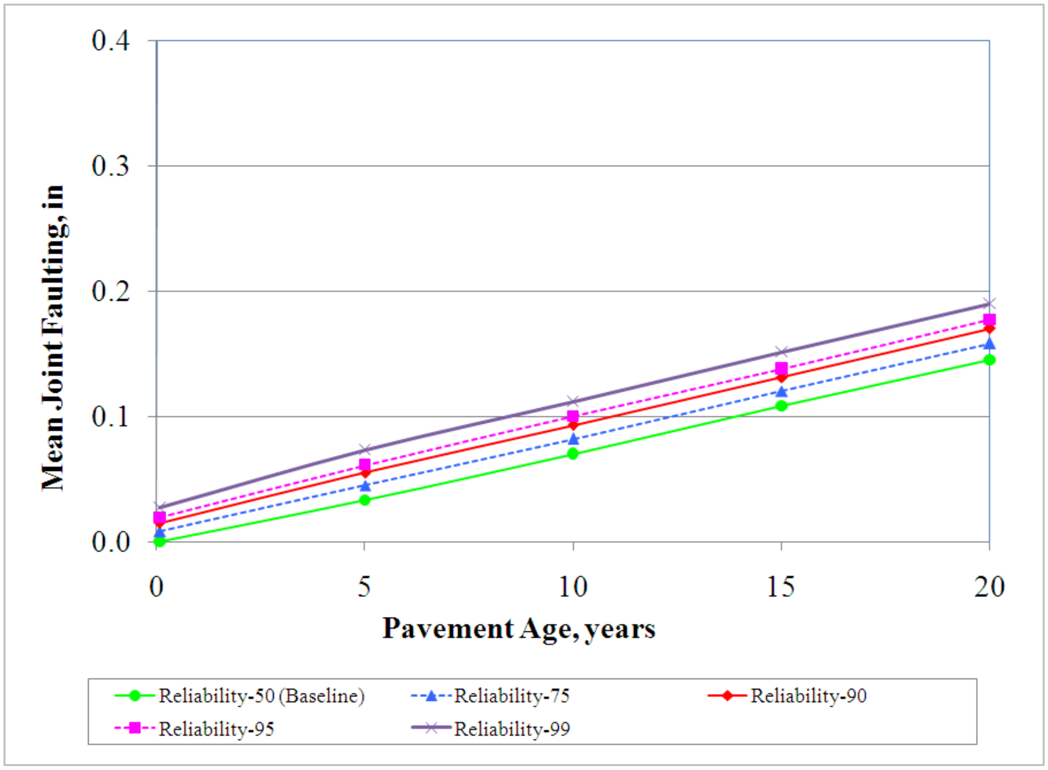

Figure 7.15 Sensitivity of JPCP Faulting to Design Reliability

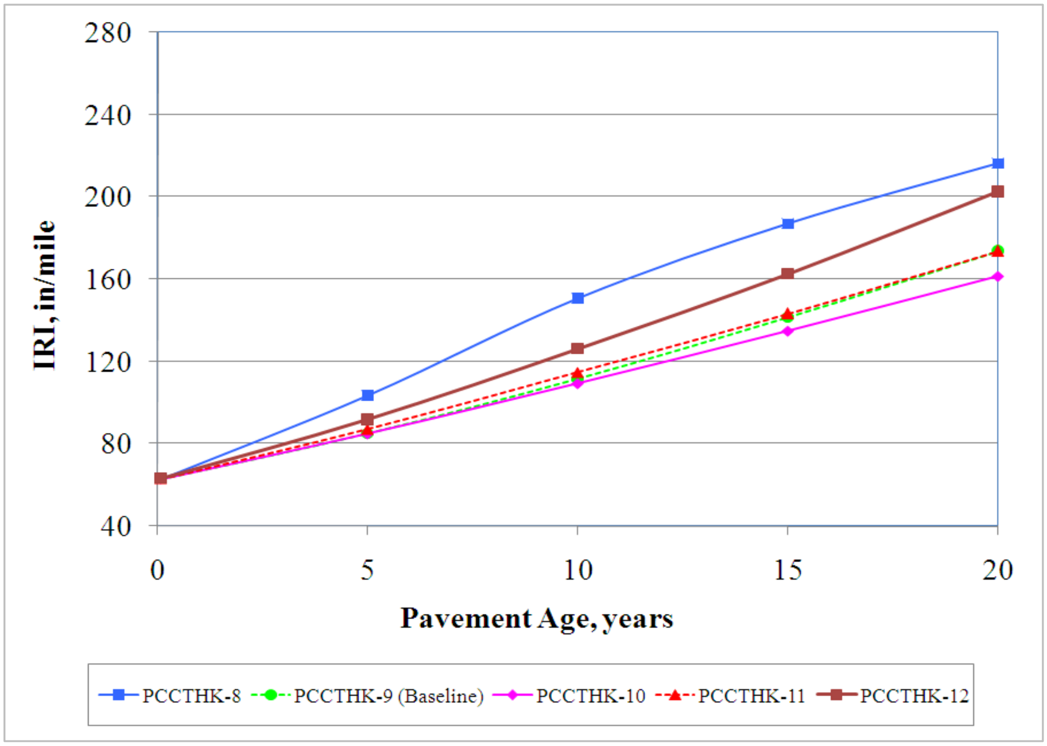

Figure 7.16 Sensitivity of JPCP IRI to PCC Thickness

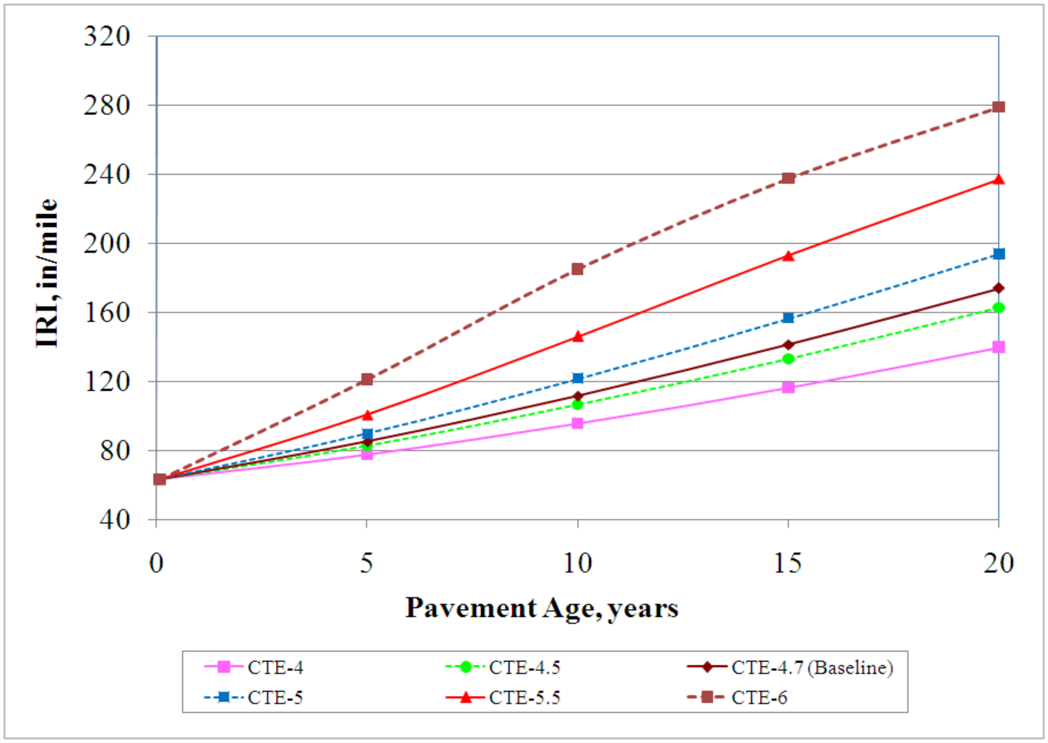

Figure 7.17 Sensitivity of JPCP Faulting to PCC Coefficient of Thermal Expansion

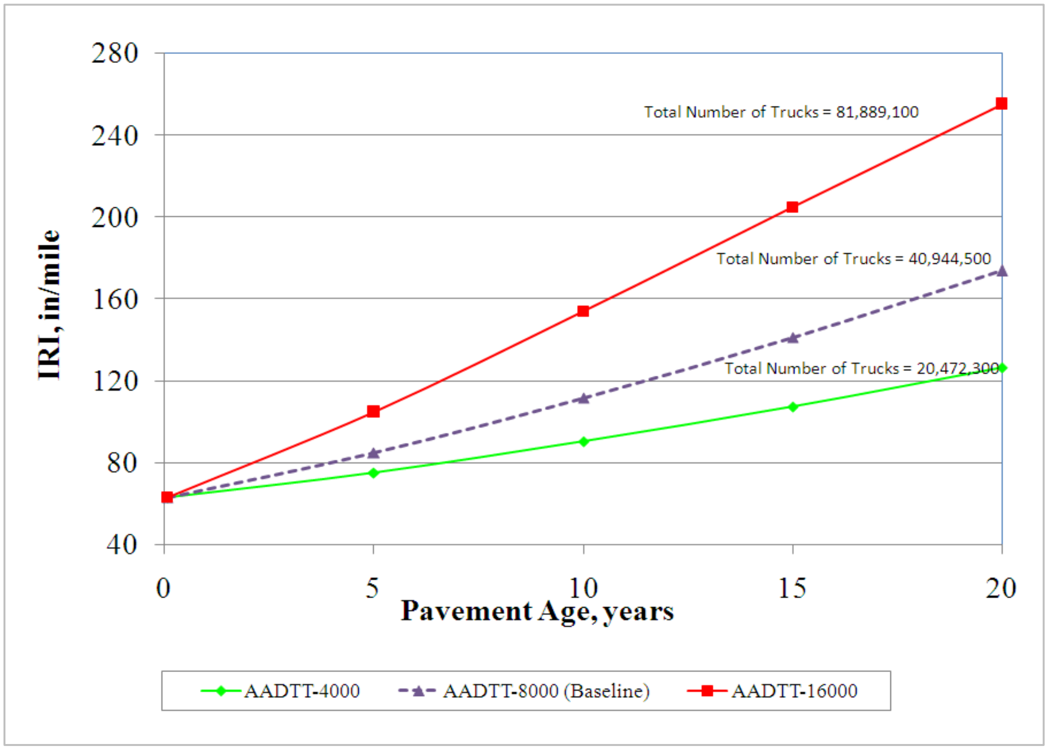

Figure 7.18 Sensitivity of JPCP IRI to Traffic Volume

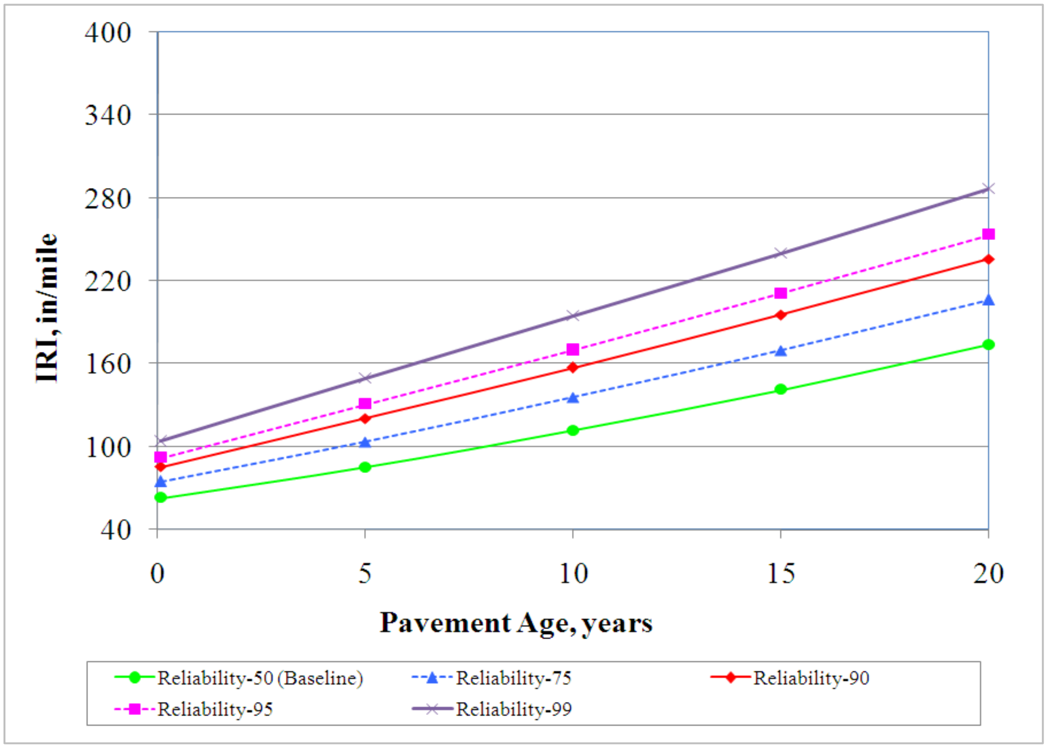

Figure 7.19 Sensitivity of JPCP IRI to Design Reliability

7.10 Joint Spacing (L)

In general, the spacing of both transverse and longitudinal contraction joints depends on local conditions of materials and environment, whereas expansion and contraction joints are primarily dependent on layout and construction capabilities. For contraction joints, when a positive temperature gradient, or base frictional resistance increases; the spacing increases as the concrete tensile strength increases. Spacing is also related to the slab thickness and joint sealant capabilities.

Determination of the required slab thickness includes an input for joint spacing. As joint spacing increases, stresses due to thermal curling and moisture warping increase. CDOT designs their PCCP using the Jointed Plain Concrete Pavement (JPCP) method. For a detailed illustration, see CDOT's current Standard Plan Sheet M-412-1. **CDOT uses a joint spacing of 15 feet maximum for concrete pavement thicknesses over 6 inches, 12 feet maximum for concrete thicknesses of 6 inches or less, and a minimum of 8 feet for any full depth pavement**.

7.11 Slab/Base Friction

The time over which full contact friction exists between the PCC slab and the underlying layer (usually the base course) is an input in M-E Design. This factor indicates (1) whether or not the PCC slab/base interface has full friction at construction, and (2) how long full friction will be available at the interface if present after construction. This factor is a significant input in JPCP cracking predictions since a monolithic slab/base structure is obtained when full friction exists at the interface.

Global calibration of JPCP performance prediction models show full contact friction exists over the life of the pavements for all base types, with the exception of cement treated or lean concrete base. Therefore, it is recommended the designer set the "months to full contact friction" between the JPCP and the base course equal to the design life of the pavement for unbound aggregate, asphalt stabilized, and cementitious stabilized base courses.

For cement treated or lean concrete base, the months of full contact friction may be reduced if attempts are made to debond the base from the PCC slab. The age at which debonding occurs can be confirmed through construction specifications and/or historical records. If no efforts are made to debond the interface, the designer is recommended to use 10 years of full interface friction.

The inputs required for M-E Design software are as follows:

- Presence or absence of PCC-Base full-friction contact
- Months until friction loss
    - Use the design life (in months) for asphalt treated and aggregate base types
    - Use 120 months for lean concrete and cement treated base

7.12 Effective Temperature Differential (°F)

An effective temperature differential includes the effects of temperature, precipitation, and wind. Wind is considered because if moist, it has an influence on the surface. Wind may be drier at the surface of the slab creating a larger differential. The same concept may be applied to temperature differences.

Curling is slab curvature produced by a temperature gradient throughout the depth of the slab. Warping is moisture-induced slab curvature. As shown in **Figure 7.20 Curling and Warping**, a positive gradient occurs when temperature and/or moisture levels at the top of a PCC slab are higher than at the bottom of the PCC slab, resulting in downward curvature. In contrast, negative gradients occur when the temperature and moisture in the slab are greater at the bottom, resulting in upward slab curvature. Curling and warping actions may offset or augment each other. During summer days, curling may be counteracted by warping. During summer nights, the curling and warping actions may compound each other. Gradients, as shown in **Figure 7.20 Curling and Warping** are primarily non-linear in nature (5).

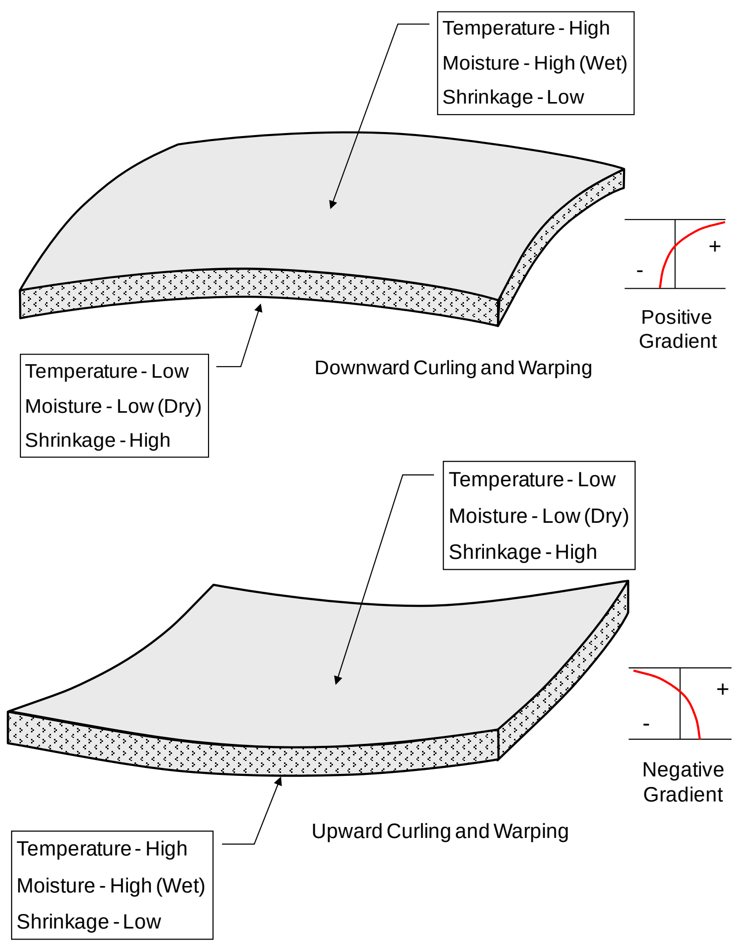

Figure 7.20 Curling and Warping

The magnitude of thermal and moisture gradients within a pavement are influenced by factors of daily temperature and relative humidity conditions, base layer type, slab geometry with constraints, shrinkage characteristics, and concrete mixture characteristics. The key characteristics of concrete mixtures that influence pavement response to thermal gradients are the coefficient of thermal expansion, thermal conductivity, and specific heat (5).

Paving operations are often performed during the morning and daytime of hot sunny days, a condition that tends to expose the newly paved slabs to a high temperature differential from the intense solar radiation and heat of hydration. Depending on the exposure conditions, a significant amount of positive temperature gradient may be present at the time of hardening. On the other hand, shrinkage occurs when the surface drying and bottom moisture wicken into the base/subbase. This resultant condition has been termed the "zero-stress temperature gradient" and is permanently locked into the slab at the time of construction. The permanent components of curling and warping are considered together and are indistinguishable. Creep occurs over time and negates the effects of the permanent curvature, but only a portion of the permanent curling and warping actually affects the long term pavement response (7). Refer to CDOT Final Research Report CDOT-DTD-R-2006-9, *Implementation of Proven PCCP Practices in Colorado*, dated April 2006 (8) for additional discussion on curling.

M-E Design's recommended value for permanent curl/warp is -10°F (obtained through optimization) for all new and reconstructed rigid pavements in all climatic regions. This is an equivalent linear temperature difference from top to bottom of the slab.

7.13 Dowel Bars (Load Transfer Devices) and Tie Bars

Load transfer is used to account for the ability of a concrete pavement structure to transfer (distribute) load across discontinuities, such as joints or cracks. Load transfer devices, aggregate interlock, and the presence of tied longitudinal joints along with tied shoulders all have an effect.

All new rigid pavements, new construction and reconstruction, including ramps, auxiliary lanes, acceleration/deceleration lanes, and urban streets will require epoxy coated smooth dowel bars in the transverse joints for load transfer. Smooth dowel bars aid the transfer of load across joints and allow thermal contraction in the PCCP. Since these transverse joints must be allowed to expand and contract, deformed tie bars should never be used as load transfer devices in the transverse direction. Most pavements should be dowelled.

If the pavement has shoulders, the shoulders must be portland cement concrete and tied to the travel lanes. Two major advantages of using tied portland cement concrete shoulders is the reduction of slab stress and increased service life. Concrete shoulders of three feet or greater may be considered a tied shoulder. Pavements with monolithic or tied curb and gutter that provide additional stiffness and keep traffic away from the edge may be treated as a tied shoulder. Studies have shown that on interstate projects, increasing the outside slab an additional two feet is equivalent to a tied shoulder. In a typical situation with 12-foot lane widths, the paint stripe is placed at 12 feet and the longitudinal joint is sawed and tied at 14 feet. Requiring the longitudinal joint to coincide with the lane line is recommended in urban locations. 14-foot longitudinal joints may not be appropriate for ramps, since ramps are usually much thinner in comparison to the main line pavement.

Dowel bar diameter and tie bar size versus thickness of concrete pavement and type of base is tabulated and noted in *CDOT Standard Plans, M & S Standard Drawing,* July 2012, M-412-1, Sheet 5, Reinforcing Size Table (9). The table is reproduced in **Table 7.3 Reinforcing Size Table**.

Table 7.3 Reinforcing Size Table

Pavement Thickness (T) (inches)	Dowel Bar Diameter (inches)
T < 8	1
8 ≥ T ≤ 10	1.25
10 > T ≤ 15	1.50

- Tie bars for longitudinal joints shall conform to AASHTO M 284 and shall be Grade 60, epoxy-coated, and deformed.
- Tie bar length is to be 30 inches and spaced at 36 inches on center.
- Tie bar size is No. 5 when pavement is placed on unbound bases.
- Tie bar size is No. 6 when pavement is placed on lime treated soil, asphalt treated, cement treated, milled asphalt, or recycled asphalt pavement bases.

Dowel bars for transverse joints shall conform to AASHTO M 254 for the coating and to ASTM A 615, Grade 60 for the core material and shall be epoxy-coated, smooth, and lightly greased, pre-coated with wax or asphalt emulsion, or sprayed with an approved material for their full length.

Details illustrating dowel placement tolerances are shown on *CDOT Standard Plans, M & S Standard Drawing,* July 2012, M-412-1, Sheet 1 (9). Dowel bar placement is at $^{T}/_{2}$ depth (see **Figure 7.21 Details of Dowel Bar Placement)**.

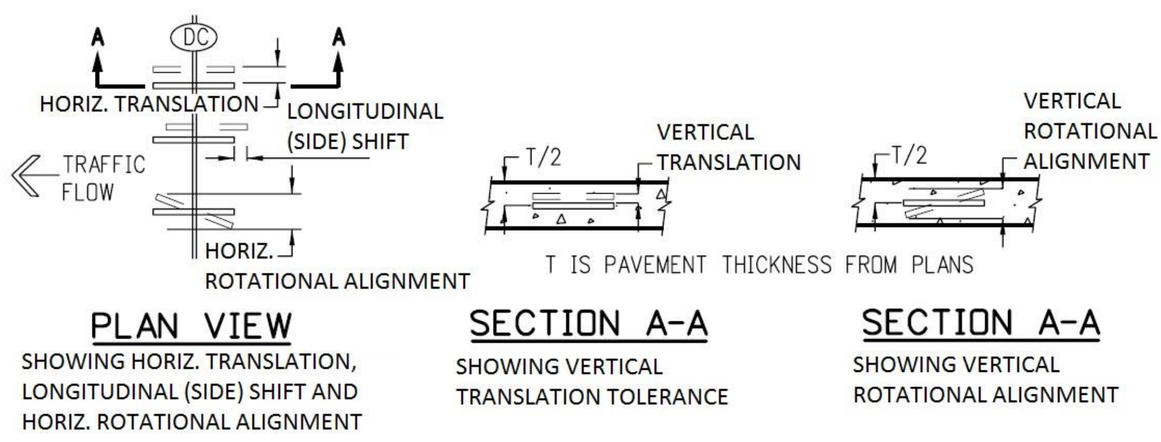

Figure 7.21 Details of Dowel Bar Placement

The tolerances are referenced in Subsection 412.13 of the CDOT *Standard Specification for Road and Bridge Construction*, 2011 (23) and as revised. The tolerance table is reproduced in **Table 7.4 Dowel Bar Target Placement Tolerances**. Tolerances are based on *NCHRP Report 637, Guidelines for Dowel Alignment in Concrete Pavements* (22).

Table 7.4 Dowel Bar Target Placement Tolerances

Position	Tolerance (inches)
Horizontal and Vertical Translation	1
Longitudinal (Side) Shift Translation	3
Horizontal and Vertical Rotational Alignment	1.5

For tied concrete shoulders, M-E Design requires the input of the long-term or terminal deflection load transfer efficiency (LTE) between the lane (PCC outer lane slab) and shoulder's longitudinal joint. The LTE is defined as the ratio of deflections of the unloaded and loaded slabs. The higher the LTE, the greater the support provided by the shoulder to reduce critical responses of the mainline slabs. Typical long-term deflection LTE are:

- 50 to 70 percent for a monolithically constructed and tied PCC shoulder
- 30 to 50 percent for a separately constructed tied PCC shoulder
- Untied concrete shoulders or other shoulder types that do not provide significant support, therefore a low LTE value should be used.

7.14 Lane Edge Support Condition (E)

- Conventional lane width (12 feet) with free edge
- Conventional lane width (12 feet) with tied concrete shoulder
- Wide slab (i.e. 14 feet) with conventional traffic lane width (12 feet)

Refer to CDOT Final Research Report CDOT-DTD-R-2006-9, *Implementation of Proven PCCP Practices in Colorado*, dated April 2006 (8), and *Evaluation of Premature PCCP Longitudinal Cracking in Colorado*, Final Research Report CDOT-DTD-R-2003-1, dated January 2003 (11) for additional discussion on widen slabs.

7.15 Base Erodibility

The erodibility index allows the designer to select the base's resistance to erosion. The potential for base or subbase erosion (layer directly beneath the PCC layer) has a significant impact on the initiation and propagation of pavement distress. Different base types are classified based on 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

Rigorous definitions of the material types that qualify under these various categories are presented in **Table 7.5 Material Types and Erodibility Class**.

Table 7.5 Material Types and Erodibility Class

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Erodibility Class	Material Description and Testing
1	(a) Lean concrete with approximately 8 percent cement; or with long-term compressive strength > 2,500 psi. (> 2,000 psi. at 28-days), and a granular subbase layer or a stabilized soil layer, or a geotextile fabric placed between the treated base and subgrade, otherwise Class 2. (b) Hot mixed asphalt concrete with 6 percent asphalt cement that passes appropriate stripping tests (see **Figure 2.2.8**) and aggregate tests; and a granular subbase layer or a stabilized soil layer, otherwise Class 2. (c) Permeable drainage layer; asphalt treated aggregate (see **Figure 2.2.8** and **Table 2.2.57** for guidance) or cement treated aggregate (see **Table 2.2.58** for guidance) and an appropriate granular or geotextile separation layer placed between the treated permeable base and subgrade.
2	(a) Cement treated granular material with 5 percent cement manufactured in plant, or long-term compressive strength 2,000 to 2,500 psi (1,500 to 2,000 psi at 28-days) and a granular subbase layer or a stabilized soil layer, or a geotextile fabric placed between the treated base and subgrade; otherwise Class 3. (b) Asphalt treated granular material with 4 percent asphalt cement that passes the appropriate stripping test and a granular subbase layer or a treated soil layer, or a geotextile fabric placed between the treated base and subgrade; otherwise Class 3.
3	(a) Cement-treated granular material with 3.5 percent cement manufactured in plant, or long-term compressive strength 1,000 to 2,000 psi (750 psi to 1,500 at 28-days). (b) Asphalt treated granular material with 3 percent asphalt cement that passes appropriate stripping test.
4	Unbound crushed granular material having dense gradation and high quality aggregates.
5	Untreated soils (PCC slab placed on prepared/compacted subgrade)

7.16 Sealant Type

Sealant type applied for transverse joints is a key input used in a joint spalling model which is used for predicting JPCP smoothness. The sealant options are liquid, silicone, and preformed, however, for M-E Design the designer <u>should use a silicone sealant</u>.

7.17 Concrete Pavement Minimum Thickness

The minimum thickness requirement may be changed on a project to project bases depending on traffic, soil conditions, bases, etc. (see **Table 7.6 Minimum Thickness for Highways, Roadways and Bicycle Paths**).

Table 7.6 Minimum Thicknesses for Highways, Roadways, and Bicycle Paths

Design Truck Traffic	Portland Cement Concrete Pavement (inches)
Greater than 1,000,000	8.0
Less than or equal to 1,000,000 for driveways, multi-use sidewalks, bicycle paths, and maintenance pavement	6.0
Sidewalks (pedestrian only) [1]	4.0
Note: [1] Per Standard Plan No. M-609-1, Curb, Gutters and Sidewalks of CDOT's *M&S Standards*, July 2012.	

7.18 Concrete Pavement Texturing, Stationing, and Rumble Strips

- **Texture:** Final surface of the pavement shall be uniformly textured with a broom, burlap drag, artificial turf, or diamond ground to obtain a specified average texture depth of the panel being greater than 0.05 inches. Refer to CDOT Final Research Report CDOT-2012-10, *Assessment of Concrete Pavement Texturing Methodologies in Colorado*, dated October 2012 (25), and CDOT Final Research Report CDOT-DTD-R-2005-22, *PCCP Texturing Methods*, dated January 2005 (12).

- **Stationing**: Stationing shall be stamped into the outside edge of the pavement at 500-foot intervals on each outside mainline shoulder as shown on *CDOT Standard Plans, M & S Standard Drawing*, July 2012, Standard Plan No. M-412-1, Concrete Pavement Joints.

- **Rumble Strips:** When rumble strips are installed, they shall be of the style and location as shown on *CDOT Standard Plans, M & S Standard Drawing*, July 2012, Standard Plan Sheet No. M-614-1, Rumble Strips.

7.19 Concrete Pavement Materials Selection

Concrete pavement is a construction paving material that consists of cement (commonly portland cement), other cementitious materials (fly ash), aggregate (gravel and sand), water, and chemical admixtures. The concrete solidifies and hardens after mixing and placement due to a chemical process known as hydration. The water reacts with cement, which bonds the other components together, eventually creating a hard stone-like material.

CDOT designates a concrete pavement mix as a Class P. **Table 7.7 Concrete Classification** shows the specified mix properties. Class E is a fast track mix that may be substituted for Class P. Class P and E are defined in Section 601 Structural Concrete and 701 Hydraulic Cement of *CDOT Standard Specification for Road and Bridge Construction*, 2011 (23) and as revised.

Table 7.7 Concrete Classification

Concrete Class	Required Field Compressive Strength (psi)	Minimum Cementitious Content (lbs/yd^3)	Air Content Percent Range (Total)	Maximum Water Cement Ratio
P	4,500 at 28 days	520	4-8	0.44
E	4,500 at 28 days	520	4-8	0.44
Note: Table taken from Standard Special Provision: *Revision of Sections 105, 106, 412, 601 and 709 Conformity to the Contract of Portland Cement Concrete Pavement and Dowel Bars and Tie Bars for Joints*, dated April 30, 2015				

7.19.1 Understanding pH in Concrete Mixes

A brief explanation of pH is presented in **Section S.1.4.2 pH Scale** in the **SUPPLEMENT** chapter. When applied to pavement design, freshly poured concrete can have a pH of 11 to 13 making it very alkaline. This high initial alkalinity helps resist corrosion, but as concrete ages, the pH can drop to around 8 increasing the degradation of steel reinforcement and load transfer devices. The high alkalinity of concrete can also affect the performance of fresh and hardened concrete when admixtures are used.

7.19.2 Alkali Aggregate Reactivity

The high alkalinity of concrete can cause serious problems when interacting with different parts of the mix, namely alkali-silica and alkali-carbonate reactions. Alkali-silica reactivity (ASR) is the process in which certain minerals in the aggregate along with the presence of moisture are broken down by the highly alkaline environment of concrete. This process produces a gel-like substance that expands adding tensile forces to the concrete matrix, which then leads to external cracking of the concrete slab (13). The cracking allows more water to infiltrate creating more gel and more expansion. Ultimately, the concrete destroys itself. The ASR chemical reaction is expressed in equation **Eq. 7-1** (15).

$$\mathbf{SiO_2 + 2NaOH + H_2O \rightarrow NA_2SiO_3 + 2H_20} \qquad \textbf{Eq. 7-1}$$

$$\text{silica} + \text{alkali} + \text{water} \rightarrow \text{alkali-silica gel}$$

Alkali-carbonate reactivity (ACR) is much less common than ASR, but it does have similar expansive properties that occur within the aggregate and deteriorate concrete pavement. The ACR reaction is dependent on certain types of clay rich, or impure, dolomitic limestones rarely used in concrete because of their inherently weak structure (14). The ACR chemical reaction known as dedolomitization is represented in equation **Eq. 7-2** (15). The cracking pattern is shown in **Figure 7.22 Idealized Sketch of Cracking Pattern in Concrete Mass Caused By Internal Expansion**.

$$\mathbf{CaMg(CO_3)_2 + 2NaOH \rightarrow Mg(OH_2) + CaCO_3 + NaCO_3} \qquad \textbf{Eq. 7-2}$$
$$\text{Dolomite} + \text{Alkali} \rightarrow \text{Brucite} + \text{Calcite} + \text{Sodium Carbonate}$$

"Sandgravel" aggregates in parts of Kansas, Nebraska, Colorado, and Wyoming, especially those from the Platte, Republican, and Laramie Rivers, have been involved in the deterioration of concrete (17). In 1983 a team was formed to evaluate the concrete pavement condition in Colorado and to recommend rehabilitation methods for these pavements. This team identified that one-third of the pavements inspected suffered from ASR (19). A follow up study conducted in 1987 focused on the cause of ASR in Colorado. The study concluded that aggregates in the Denver Metro area showed no signs of ASR reaction, but aggregate from the Three Bells pit near Windsor demonstrated rapid signs of expansion. This study led CDOT to modify its specifications and require low alkali cement for all concrete pavement, it also identified the need for Class F fly ash in areas were reactive aggregates have been a problem (20).

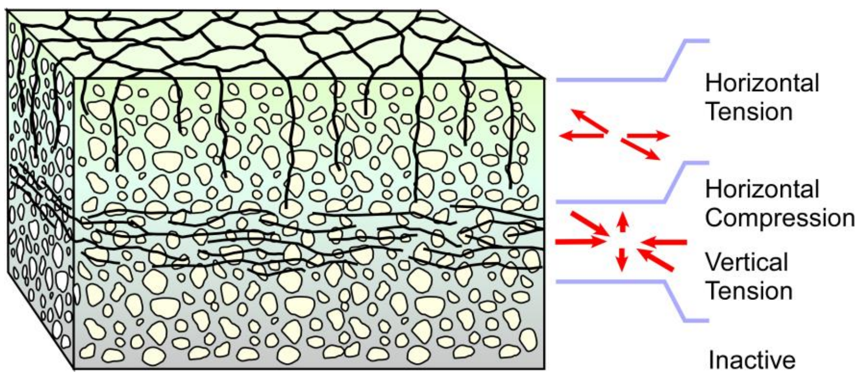

Figure 7.22 Idealized Sketch of Cracking Pattern in Concrete Mass Caused by Internal Expansion
(Figure 93, *Petrographic Methods of Examining Hardened Concrete: A Petrographic Manual*, July 2006)

7.19.3 Sulfate Resistant Concrete Pavement

Sulfates may be found in soil and water and are referred to as "alkali". The sulfates in soils and water are the main source of external sulfate attack on concrete pavement. Although the mechanism of sulfate attack is complex, it is primarily thought to be caused by two chemical reactions: 1) the formation of gypsum through the combination of sulfate and calcium ions, and/or 2) the formation of ettringite through the combination of sulfate ions and hydrated calcium aluminate (18). Ettringite ($Ca_6[Al(OH)_6]^2(SO_4)_3 \cdot 26H_2O_2$) is a high-sulfate, calcium sulfo-aluminate mineral which naturally occurs in curing concrete. The problem appears when ettringite forms after the concrete has set, this is known as Delayed Ettringite Formation (DEF). This process is extremely harmful, because as ettringite crystals form they expand and create internal tensile stresses in the cement matrix (21). These stresses will cause the concrete to crack, but may not be apparent for 3-10 years (18).

Sulfate attack is a chemical reaction between sulfates and the calcium aluminate (C_3A) in cement, resulting in surface softening (22) (see **Figure 7.23 Sulfate Attack**). Steps taken to prevent the development of distress due to external sulfate attack include minimizing the tri-calcium aluminate content in the cement or reducing the quantity of calcium hydroxide in the hydrated cement paste though the use of pozzolanic materials. It is also recommended that a w/c ratio less than 0.45 be used to help mitigate external sulfate attack (18).

Severity levels of potential exposure to sulfate attack have been developed. **Table 7.8 Requirements to Protect Against Damage to Concrete by Sulfate Attack from External Sources of Sulfates** shows the classification levels of potential exposure. Concrete pavement mix designs must provide protection against sulfate attack, thus cementitious material requirements are modified. As the severity of potential exposure increases, the cementitious material requirements become more stringent and the water cement ratio becomes less stringent. Refer to Section 601 Structural Concrete of CDOT *Standard Specification for Road and Bridge Construction*, 2011 (23) and as revised for additional cementitious material requirements.

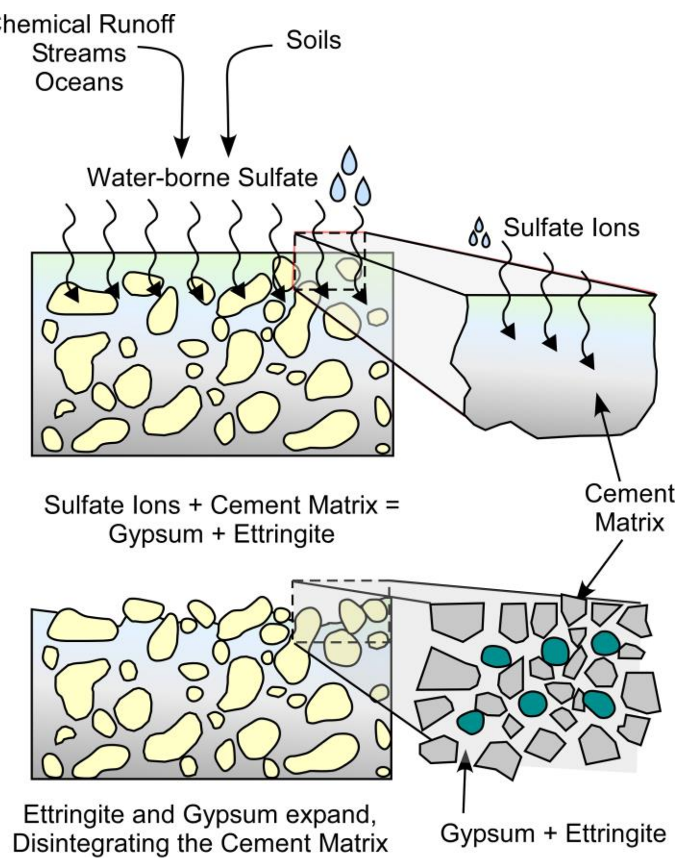

Figure 7.23 Sulfate Attack
*(*Figure 5-18*, Integrated Materials and Construction Practices for Concrete Pavement: State-of-the-33 Practice Manual)*

Table 7.8 Requirements to Protect Against Damage to Concrete by Sulfate attack from External Sources of Sulfates

Severity of Potential Exposure	Water-soluble Sulfate (SO_4), Percent Dry Soil	Sulfate (SO_4) in Water (ppm)	Maximum Water Cement Ratio	Cementitious Material Requirements
Class 0	0.00 to 0.10	0 to 150	0.50	Class 0
Class 1	0.11 to 0.20	150 to 1,500	0.50	Class 1
Class 2	0.21 to 2.00	1,501 to 10,000	0.45	Class 2
Class 3	2.01 or greater	10,001 or greater	0.40	Class 3

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