SOUTHERN PLAINS
TRANSPORTATION CENTER

Design Data for Rigid Pavements in New Mexico

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16. ABSTRACT

This study was focused on the characterization of concrete paving mixes being used in New Mexico for further implementation of Pavement ME Design. Seven concrete paving mixes were collected from various districts of New Mexico prepared with different coarse aggregates. Cylinder and beam specimens were tested for compressive strength, elastic modulus, modulus of rupture (MOR) and coefficient of thermal expansion (CTE) to generate level-1 input data for these specific mixes. Inter-conversion models were developed to convert compressive strength into elastic modulus and MOR and it was found that the developed models work better than the default models. CTE of the tested paving mixes also varied over a broad range i.e. 3.7 to 5.9 με/°F. Simulations were conducted in Pavement ME to evaluate the impact of material inputs and other design factors on JPCP and CRCP design/performance. It became evident from the analysis of the simulation results that there is a significant difference in performance predictions between level-1 inputs and default inputs, which necessitated the use of lab tested data for any paving mix to be used in the design process.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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EXECUTIVE SUMMARY

Historically, New Mexico has been a flexible pavement state with a majority of the pavements made with asphalt concrete. Over the years, most of the pavement research conducted in New Mexico was linked to asphalt concrete and there has not been much with regards to rigid pavements. With long service life and little or no maintenance, rigid pavements can be considered a suitable option if economically viable. This study was focused to develop a database of concrete material's inputs including compressive strength, elastic modulus, modulus of rupture (MOR) and coefficient of thermal expansion (CTE) for various concrete paving mixes being used in New Mexico. The impact of these material inputs will be evaluated on the design and performance of rigid pavements by conducting simulations in Pavement ME Design for New Mexico climatic conditions. The Pavement ME default interconversion models will be evaluated with the test data and refined if needed.

For this study, seven concrete paving mixes were collected from various districts of New Mexico. These mixes were prepared with different coarse aggregates having different mineralogy i.e. Limestone, Basalt, Granite, Quartzite and Dolomite. Cylinder and beam specimens from all these paving mixes were tested for compressive strength, elastic modulus, MOR at different age groups to generate time-series data. The database developed with the laboratory testing provides accurate material inputs to the design engineer for designing of rigid pavements. Interconversion models were developed for these concrete mixes to convert compressive strength into elastic modulus and MOR and it was found that the developed models work better than the ME default models for the tested mixes.

The long term compressive strength testing was conducted up to 360 days and it was found that all the concrete mixes have a similar trend with the strength curve getting close to flat in the long term. The 28 days/20 years strength gain factor is an important input in Pavement ME with a value of 1.44. The long-term compressive strength data was extrapolated and 20 years strength gain factors were determined which came in the range of 1.75 to 1.98 and seems higher than the default factor. In line with the previous research, it is suggested that the default value of 1.44 should be used as it is based on actual 20 years testing and extrapolating may have caused some errors giving a higher value for the test data.

CTE testing was conducted for all the paving mixes at the age of 28 days and it was found that the CTE value varies over a broad range i.e. 3.7 to 5.9 με/°F. The impact of coarse aggregate mineralogy on the CTE value was evident with concrete having Limestone aggregate showing the least CTE value in comparison with other aggregates. The CTE of all the mixes were evaluated with the ME default values and it was found that there is a significant difference between the test data and the default data but the tested CTE values lie within the lower and upper bounds of the ME default data which verified the laboratory test data.

Numerous simulations were conducted in Pavement ME Design to evaluate the impact of material inputs, input levels, climate, traffic and other design variables on the design and performance of jointed plain concrete pavement (JPCP) and continuously reinforced concrete pavement (CRCP). It was found that there is a significant difference in predicted performance between level-1 and level-3 inputs thus any pavement should be designed with level-1 material inputs for the specific paving mix to obtain accurate design. The CTE of paving mix has a significant effect on design and performance of any pavement thus using the default value of CTE may result in over/under the designed pavement. Accurately determined CTE value is essential for any pavement design process. Other design factors including traffic volume and climatic conditions have significant effects on design and performance prediction of rigid pavements thus these design factors shall be accurately determined and used in the design process.

CHAPTER 1

INTRODUCTION

1.1 PROBLEM STATEMENT

The design and performance prediction of rigid pavements is based on various input factors including material properties, traffic loads, climatic factors and road-bed soil characteristics. Among the concrete material factors, elastic modulus, modulus of rupture (MOR) and coefficient of thermal expansion (CTE) are the important ones. Various pavement characteristics like thickness, design life, serviceability, and cracking performance depend on these input factors. Accurate determination of these material properties was not a part of the design process until the Mechanistic-Empirical Pavement Design Guide (MEPDG), currently known as AASHTOWare pavement Mechanistic-Empirical (M-E) design software or Pavement ME, was introduced. Pavement M-E design software considers inputs such as traffic, climate, and materials while considering CTE values, elastic modulus, and modulus of rupture of concrete in determining pavement thickness for a given set of performances. The procedure is based on mechanistic-empirical design concepts meaning that firstly the pavement responses such as stresses, strains, and deflections are calculated under axle loads and climatic conditions and then the damage over the design analysis period is determined. The damage is then empirically related to pavement distresses and smoothness based on the field performance of actual projects throughout the United States. It was found that these three material inputs can affect pavement performance quite significantly (Sabih and Tarefder, 2016).

Elastic modulus, MOR, and compressive strength are the basic inputs for designing rigid pavement using Pavement ME Design (ARA, 2004). The CTE of concrete is an important parameter to determine the total stress developed in concrete upon applying traffic and temperature loading. These materials properties are directly used in different subroutines or empirical models in Pavement ME software. For example, the fatigue model of concrete relates MOR with the fatigue life. Therefore, these material inputs should be accurately determined in the laboratory for better prediction of concrete distresses. Currently, New Mexico Department of Transportation (NMDOT) does not have any laboratory test data for designing concrete pavements. Hence, it is important to determine the properties of different concrete mixes to design rigid pavements in New Mexico.

1.2 OBJECTIVES

The main objective of this study is to develop three key input parameters to be used by the pavement ME design software for the design of rigid pavements in New Mexico. Due to climate and material variability between different regions in New Mexico, aggregate/concrete materials from different district locations in New Mexico should be tested for elastic modulus, MOR and CTE. Another major objective of this study is, using Pavement ME Design software, determine the effects of these three material factors on the performances of rigid pavements of different geometry, traffic, and climate conditions.

CHAPTER 2

REVIEW OF CURRENT PRACTICES

2.1 BACKGROUND

A comprehensive literature review of the current state of practice of rigid pavement design, performance, and characterization of concrete materials and effects of material properties of concrete on pavement performance has been conducted. The study found that various material properties of concrete are required for the design of rigid pavements according to Pavement ME Design (previously called AASHTOware Pavement Mechanistic-Empirical (ME) Design guide or MEPDG). Out of these, coefficient of thermal expansion (CTE), elastic modulus and modulus of rupture (MOR) are found to have a substantial impact on rigid pavement design and performance. Accurate determination of these material properties will improve the pavement design process. The variation of CTE, MOR, and elastic modulus affects the performance indicators of rigid pavements such as joint faulting, roughness and transverse cracking in JPCP and punch-outs in CRCP.

Rigid pavements (commonly known as concrete pavements) are composed of a Portland Cement Concrete (PCC) surface course. With high elastic modulus (stiffness) of the PCC layer, the concrete slab itself supplies most of a rigid pavement's structural capacity. Concrete pavements may be either unreinforced (plain) or reinforced depending on how the designer prefers to control the cracking of the pavement. The high modulus of elasticity and rigidity of concrete compared to other road making materials provides a concrete pavement with a reasonable degree of flexural strength. This property leads to externally applied wheel loads being widely distributed. This, in turn, limits the pressures applied to the sub-layers. The concrete layer alone provides the major portion of the load carrying capacity of concrete pavement.

2.2 TYPES OF CONCRETE PAVEMENTS

The different types of concrete pavements, generally used in the United States are Jointed Plain Concrete Pavement (JPCP), Jointed Reinforced Concrete Pavement (JRCP) and Continuously Reinforced Concrete Pavement (CRCP). According to the statistics on new construction pavement types used by the agencies of various states in the United States, JPCP is the most widely used pavement type as being used by 44 states while CRCP is the least used pavement being used by 9 states (NCHRP, 2014). JPCP is unreinforced concrete pavement with transverse joints and longitudinal joints. Dowel bars are provided at transverse joints and tie bars at the longitudinal joints. Transverse joints are used to control the transverse cracking of pavement slab. Dowel bars increases the load transfer efficiency (CADOT, 2015). JPCP can be designed with Pavement ME Design.

Jointed reinforced concrete pavement (JRCP) is constructed with transverse joints and reinforcing steel. The transverse joint spacing ranges between 25 to 50 ft. Reinforcing steel is used to control cracking. Dowel bars help in load transfer across transverse joints. The use of JRCP is lesser as compared to JPCP and CRCP. These pavements cannot be designed using Pavement.

Continuously reinforced concrete pavement (CRCP) is constructed with steel reinforcement and it has no transverse joints. The main reinforcement is in the longitudinal direction with some transverse bars to support the longitudinal bars. Transverse cracks are controlled by continuous longitudinal bars (CADOT, 2015). CRCP is a durable highway which can provide an effective service life of 40 years or more.

2.3 CONCRETE MATERIALS

Concrete is composed of cementitious materials, coarse aggregate, fine aggregate, water, and admixtures. Concrete properties are materials dependent. Section 509 of the "Standard Specifications for Highway and Bridge Construction 2014 edition" defines the required chemical and physical concrete material properties (NMDOT, 2014).

2.3.1 Cementitious Materials

Cementitious material comprises Portland cement and supplementary cementitious materials (SCM). Portland cement consists of lime, iron, silica, and alumina. Different types of cement have varying physical and chemical properties.

SCMs are used to lower the demand for cement. It also improves workability and durability properties.

2.3.2 Aggregate

Aggregates constitute around 70% of the total volume of concrete thus it has a significant effect on the mechanical and thermal properties of concrete. Coarse aggregates crushed rock or gravels that are retained by a No. 4 sieve and fine aggregates are usually sand, passing a No. 4 sieve.

2.3.3 Water

Water with no pronounced taste or odor is used for concrete. Water is tested according to AASHTO T-26. Water with a PH value of from 6.0 to 8.5 should be used. The water to cementitious ratio (w/c) is an important parameter contributing to the concrete strength.

2.3.4 Admixtures

Different types of admixtures are used in concrete to obtain specific concrete mix properties. Some chemical admixtures are used to increase workability or strength gain rate. Air-entraining admixtures create a matrix of air bubbles inside concrete so that water in the concrete can expand when frozen or contract when thawed. Chemical admixtures must comply with AASHTO M 194.

2.4 PAVEMENT MECHANISTIC EMPIRICAL DESIGN

AASHTO design guides for rigid pavements up to 1993, were based on empirical models. Limited mechanistic concepts were employed in 1998 AASHTO design guide. The need for the development of Mechanistic-Empirical based pavement design procedure was recognized (Rao, 2014).

The Pavement ME Design was developed by AASHTO as the standard for rigid pavement design (AASHTO 2008). Pavement analysis and design can be performed using the Pavement ME Design software (Rao et al. 2016).

The Pavement ME Design is based on mechanistic-empirical concepts. The design procedure calculates pavement responses such as stresses, strains, and deflections under axle loads and climatic conditions and then accumulates the damage over the design analysis period. The procedure then empirically relates calculated damage over time to pavement distresses and smoothness based on the performance of actual projects throughout the U.S. Pavement ME Design has simplified the pavement design process and resulted in improved designs (Mallela et al. 2014).

2.4.1 Hierarchical Input Levels for Pavement ME

The input levels in Pavement ME are used to categorize the designer's knowledge of the input parameter. Three levels are available to input the concrete material properties (AASHTO, 2008).

Level 1 input scheme is the most accurate one and consist of laboratory tested data of the specific concrete mixture to be used for the project. This level has the highest testing and data collection costs.

Level 2 inputs are estimated from correlations or regression equations. These input values are calculated from other site-specific data or parameters that are less costly to measure. These values may represent measured regional values.

Level 3 inputs are based on the best estimated or default values. This input level has has the lowest testing and data collection costs, but it may result in erroneous pavement design.

2.5 CHARACTERIZATION OF PORTLAND CEMENT CONCRETE MATERIALS FOR RIGID PAVEMENT DESIGN

Different material properties including elastic modulus, Poisson's ratio, flexural strength, coefficient of thermal expansion, thermal conductivity, ultimate shrinkage, etc. are used to characterize PCC materials within the Pavement ME framework for the design of rigid pavements. Key parameters can be determined for each PCC mixture design through laboratory testing. These key parameters are used by the analytical model for critical response calculations, for damage calculations, and for performance predictions. One of the features of the Pavement ME Design software is its ability to use the default, regional, or site-specific values for materials data inputs. The PCC properties considered for this project are compressive strength, elastic modulus, MOR and CTE. Table 2.1 shows a summary of the requirement of PCC material properties according to various input levels. NCHRP conducted a survey in 2014 regarding the use of default, regional, and site-specific values for various material inputs by various agencies in the United States. According to the results, most agencies were using either the ME default values or regional values. Relatively few agencies indicated the use of site-specific values.

2.6 PERFORMANCE INDICATORS FOR JOINTED PLAIN CONCRETE PAVEMENTS

The performance indicators for JPCP are joint faulting, transverse cracking and IRI. According to the ME Design, the threshold limit for IRI is 172 in/mile, for mean joint faulting is 0.12 in and for slabs cracked with transverse cracking is 15%.

2.6.1 Transverse Joint Faulting

Joint faulting is the differential elevation across the transverse joint. Mean joint faulting of all transverse joints is the parameter predicted by the Pavement ME. The unit of faulting is inches. The major impact of faulting is on ride quality. Faulting is the result of repeated traffic load applications, poor load transfer, moisture beneath pavement slab, erosion of the supporting base/subbase, subgrade material, and upward curling of the slab (Bautista et al. 2008).

Table 2.1. PCC material input levels and data required

Material Property	Level-1	Level-2	Level-3
Compressive Strength	Nil	7, 14, 28, 90 days	28 days
Elastic Modulus	7, 14, 28, 90 days	Nil	Optional
Modulus of Rupture	7, 14, 28, 90 days	Nil	Optional
Coefficient of Thermal Expansion	28 days	Default	Default

2.6.2 Transverse Cracking in Concrete Slabs

There are two types of transverse cracking in JPCP namely top-down transverse cracks and bottom-up transverse cracks. When there is a positive temperature gradient in the pavement slab resulting in downward curling of the slab and the truck axles are near midway between the transverse joints, a critical bending stress occurs at the bottom center of the slab. Repeated loadings in such an arrangement result in fatigue damage, which results in a transverse crack. The factors that affect bottom-up cracking are CTE of concrete, slab thickness, joint spacing and concrete strength (NCHRP, 2003).

When the pavement is exposed to a negative temperature gradient it results in upward curling of the pavement slab. During this condition when the axles load opposite ends of the slab, a tensile bending stress occurs at the top of the slab. Such repeated loadings will result in fatigue damage and initiation of top-down crack in the pavement slab.

2.6.3 International Roughness Index (IRI)

Pavement roughness is an expression of irregularities in the pavement surface which affect the ride quality. Roughness is generally expressed as international roughness index (IRI). IRI is a characteristic of the longitudinal profile of a traveled wheel-track and constitutes a standardized roughness measurement. The recommended units are meters per kilometer (m/km) or inch per mile (in/mile). Pavement ME uses such a performance model to predict IRI. This model considers initial IRI, percentage of slabs with transverse cracking and total joint faulting to predict IRI value (Abd El-Hakim and El-Badawy, 2013). Both an initial IRI and terminal IRI must be selected for any design project. The terminal IRI typically selected is similar to that used in pavement management to establish when roadways require rehabilitation (Mallela et al. 2014).

2.7 EFFECTS OF CONCRETE MATERIAL INPUTS ON PAVEMENT DESIGN

2.7.1 Coefficient of Thermal Expansion (CTE)

CTE is a measure of concrete’s expansion or contraction with a change in temperature. It is usually expressed in micro-strains per unit temperature change. The test method to determine the CTE is AASHTO T-336 (Tanesi et al. 2010). The CTE of PCC ranges from about 3.5 to 6.5 micro-strains/°F and an average value of 5.5 micro-strains/°F is commonly used in the design process. As aggregates are the main component of concrete thus CTE value of concrete also depends upon the type of coarse aggregate. Concrete containing limestone aggregate has a lower CTE than concrete containing siliceous aggregate. S. Jahangirnejad and his team conducted research on CTE of PCC produced with various types of aggregates. They concluded that the magnitude of the measured CTE of PCC varies with aggregate geology. The CTE of hardened cement paste, which is a function of factors such as water to cement ratio, cement fineness, and cement composition, also affects the CTE of concrete (Jahangirnejad et al. 2009). Hak-Chul Shin and Yoonseok Chung found that the measured CTEs at various ages (3, 5, 7, 14, 28, 60, 90 days) fluctuates within 0.2 micro-strain/°F (0.36 micro-strains/°C) and the age of concrete, statistically have no significant effect on CTE (Shin et al. 2011).

2.7.2 Importance of Coefficient of Thermal Expansion in JPCP

CTE of PCC is a very important parameter in concrete pavement design and analysis because the magnitudes of temperature related pavement deformations are directly proportional to this value. These deformations affect the resulting curling stresses in the hardened slab. Accurate values of the CTE are required to predict potential thermally induced movements in a concrete pavement. J. Mallela et al. (2005) found that higher transverse and longitudinal fatigue cracking is caused by higher curling stresses and higher amounts of faulting is caused by loss of slab support due to curling.

J. Tanesi et al. worked to determine the effect of the variability of the CTE test on the predicted pavement performance. He performed a sensitivity analysis by varying the CTE values on a single jointed plain concrete pavement design. He found that with the increase in CTE value, the percentage of cracked slabs also increases (Tanesi et al. 2008).

David K Hein conducted his research and described that thermal expansion and contraction of a concrete pavement can have a significant effect on its performance. Thermal contraction can result in transverse cracking of slabs depending on the joint spacing (Hein, 2012).

Leslie McCarthy et al. found that a difference of 0.5 micro-strain/^{0}C has a significant impact on the service life in terms of number of years prior to exceeding the distress limit for cracking (Mccarthy et al. 2015).

2.7.3 Elastic Modulus

Elastic modulus measures material stiffness and is a ratio of the applied stress to measured strain. It is measured according to ASTM C 469 with a concrete cylinder loaded in longitudinal compression at a relatively slow constant rate. American Concrete Institute (ACI) developed a relation between elastic modulus and compressive strength of concrete (at 28 days), which gives quite satisfactory results for the elastic modulus values of concrete. Typical elastic modulus of normal strength Portland cement plain concrete ranges between $2x10^6$ to $6x10^6$ psi (14 to 41 GPa). In general, the material characteristics affect the elastic modulus in the same manner as the compressive strength. However, the elastic modulus is more sensitive to aggregate characteristics and volumes. The higher the modulus of elasticity of the aggregate, the higher will be the elastic modulus of the concrete. The shape of coarse aggregate particles and their surface characteristics also influence the value of modulus of elasticity of concrete.

2.7.4 Importance of Elastic Modulus in JPCP Design

Elastic modulus of concrete is an important variable in pavement design. It controls the overall slab deflections from traffic loading and slab curling stresses. Historically, in pavement applications, this value was not rigorously estimated. The typical value of 4.2×10^6 psi was assumed during the design of rigid pavement because it was perceived to have little effect. However, newer design methods such as Pavement ME have brought the importance of this parameter to the forefront. As elastic modulus is directly related to concrete strength so concrete with higher elastic modulus behaves in a better way to deal with the curling and loading stresses as compared to the concrete with lower elastic modulus.

2.7.5 Modulus of Rupture (MOR)

The flexural strength or MOR of concrete defines the tensile capacity of concrete. Typically, concrete is not tested under direct tension because the test apparatus and the loading mechanism introduce secondary stresses that are not easy to compensate for in test results. MOR can be determined as the maximum tensile strength at rupture at the bottom of a simply supported concrete beam during a flexural test with third point loading, as standardized in ASTM C-78. This test measures the tensile capacity of the concrete in bending or flexure. MOR is influenced by mix design parameters including water to cement ratio, cement type, cement-content, and aggregate properties.

2.7.6 Importance of Modulus of Rupture in Concrete Pavement

Modulus of rupture is the basis for estimating flexural fatigue in concrete. A true estimation of modulus of rupture would improve the accuracy of cracking prediction. Although modulus of rupture is an important parameter in evaluating the design of rigid pavement, it was not given due importance in the past. With the advent of Pavement ME, a lot of emphases has been given to the accurate determination of modulus of rupture and its use in the design of rigid pavement.

2.8 FACTORS AFFECTING THE COEFFICIENT OF THERMAL EXPANSION (CTE) OF CONCRETE

Concrete CTE is affected by several factors including aggregate mineralogy and volume, moisture content, and types and amount of cement (Sellevold and Bjøntegaard 2006). Concrete age, water-to-cement ratio (w/c), and cylinder size have a slight effect on the concrete CTE (Kohler et al. 2007). Effects of various factors on the CTE of concrete are presented next.

2.8.1 Effect of aggregates

Aggregate mineralogy and volume are major factors that affect concrete CTE. Dettling (1964) found that the concrete with quartzite as coarse aggregate has the highest CTE and the one with limestone has the lowest CTE. Concrete CTE is dependent on the CTE of the aggregate (Mehta and Monteiro 2006). Won (2005) tested the CTE from various aggregate sources. Even the concrete with the same type of aggregates showed different CTE values, which shows the impact of an aggregate's mineralogical composition on CTE. Won (2005) also studied the impact of aggregate volume on the CTE and found that CTE decreases as the volume of coarse aggregate increases because it reduces the volume of the cement paste. As cement paste has higher CTE than aggregate so concrete CTE can be reduced by reducing cement paste volume. Mindess et al. (2002) found that the CTE of any coarse aggregate depends on its silica content. Higher silica content results in higher CTE like river gravel or quartz whereas lower silica content will give lower CTE such as limestone. McCullough et al. (2000) studied the effect of mineralogical composition on the CTE of aggregate and found that an increase in silicon oxide content results in CTE increase. Neville and Brooks (1987) found that the CTE of concrete decreases when aggregate volume increases.

2.8.2 Effect of moisture content and relative humidity

Moisture content and relative humidity have a significant effect on CTE of cement paste and concrete (Chung and Shin 2011). Emanuel and Hulsey (1977) documented the effect of moisture content on the CTE of cement paste. They found that peak CTE occurs at 60 to 70% moisture content. Chung and Shin (2011) found that the peak CTEs for expansion and contraction were obtained at about 65 and 85% relative humidity, respectively. Yeon et al. (2009) found that the maximum CTE of concrete was obtained at 80% relative humidity.

2.8.3 Effect of water-to-cement ratio

Different researchers have a different opinion regarding the effect of water to cement ratio on CTE of concrete. Berwanger and Sarkar (1976) found that CTE of concrete decreases with increased water to cement ratio. Alungbe et al. (1992) conducted experimental work and found that there is no effect of water to cement ratio on the CTE of concrete.

2.8.4 Effect of concrete paste content and composition

The cement paste has a higher CTE than most of the aggregate types thus CTE of concrete increases when cement paste increases (Bonnell and Harper 1950). Hossain at al. (2006) also confirmed these findings. Bonnell and Harper (1950) also studied the effect of cement type on the CTE of concrete and found that blast-furnace cement has higher CTE in comparison to high-alumina cement. Emanuel and Hulsey (1977) observed that when the cement fineness increases so CTE also increases.

CHAPTER 3

MATERIALS COLLECTION AND SPECIMEN PREPARATION

This project consists of laboratory testing of 6 concrete paving mixes from different districts of New Mexico prepared with different coarse aggregates. Concrete cylinder and beam specimens will be cast from these mixes to determine compressive strength, elastic modulus, modulus of rupture (MOR) and coefficient of thermal expansion (CTE). The test results will provide pavement engineers with level-1 inputs to be used in rigid pavement design with Pavement ME design for these specific mixes and an effort will be made to produce level-2 correlations for interconversion of compressive strength into elastic modulus and MOR for New Mexico.
The casting of specimens from 7 concrete paving mixes was completed. 40 cylindrical specimens and 12 beam specimens were prepared from each concrete mix for further laboratory testing. These mixes are indicated as CA-ID-1 to CA-ID-7 for data composition and analysis. The summary of these mixes according to the coarse aggregate type is listed in Table 3.1.

Table 3.1. Summary of Concrete Mixes with Coarse Aggregate Type

Mix ID	Company	Coarse Aggregate Source	Coarse Aggregate Type	Fine Aggregate	Concrete Class	Cement Type	Location
CA-ID-1	PB Materials	Dark Canyon	Dolomite	Grand Falls Sand	AA-HPD	GCC Tijeras	Hobbs
CA-ID-2	K Barnett	Steele pit	Granite	Steele pit	F-LS	GCC Tijeras	Clovis
CA-ID-3	C&E Concrete	Tinaja	Lime stone	Tinaja	F	GCC Tijeras	Grants
CA-ID-4	Vulcan Materials	Placitas	Quartzite	Placitas	F	GCC Tijeras	Santa Fe
CA-ID-5	Jobe Materials	Avispa Quarry	Limestone	Dyer	F-LS	GCC Samaluyuca	Vado
CA-ID-6	Duke City Redimix	South Valley	Basalt	Orona	F	Holcim	Albuquerque
CA-ID-7	BTU Block & Concrete	Watrous pit	Quartzite	Watrous pit	F	GCC Tijeras	Las Vegas

3.1 DETAILS OF PAVING MIXES

ASTM C31-15 and AASHTO T 23-14 were followed for making and curing concrete test specimens including cylinders (4x8 in.) and beams (6x6x22 in.). These standards provide standardized requirements for making, curing, protecting, and transporting concrete test specimens. The pictorial view of cylinder and beam specimen preparation is shown in Figure 3.1 & 3.2.

Figure 3.1. Cylindrical Specimens Preparation and Curing

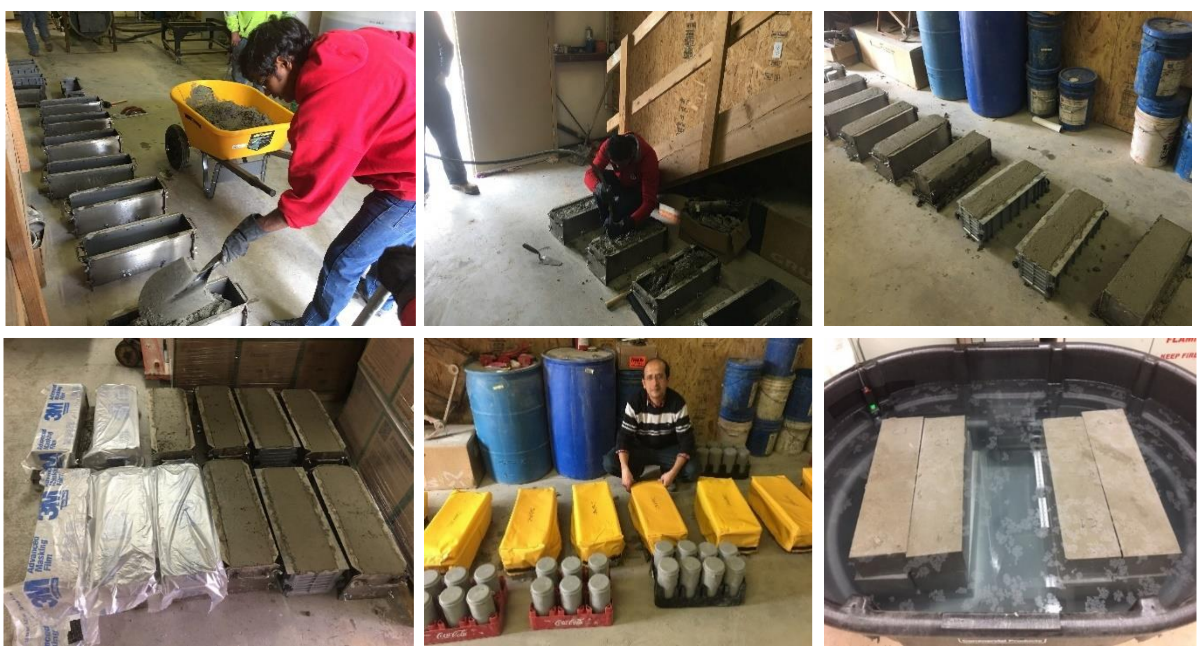

Figure 3.2. Beam Specimens Preparation and Curing

3.1.1 Details of Mix-1 (CA-ID-1)

Concrete specimens comprising 40 cylinders and 12 beams were prepared with the concrete mix–1 on Dec 15, 2016. This concrete mix was supplied by PB Materials for PCCP project at NM-18S and NM 176 which was located 20 miles South of Hobbs, NM. The details of this concrete mix are given in Table 3.2 and 3.3. The specimens were transported to pavement laboratory at UNM, after initial setting and placed in a temperature controlled curing tanks at 75 ^{0}F for final curing.

Table 3.2. Details of Concrete Mix-1

Nomenclature	Remarks
Company	PB Materials
Concrete Type	Class AA-HPD
Cement Source	GCC - Tijeras
Cement, Batch Weight	421 lbs/CuYd
Specific Gravity of Cement	3.15
Fine Aggregate Source	Grand Falls Sand
Specific Gravity of Fine Aggregate	2.64
Absorption of Fine Aggregate	1.00%
Fine Aggregate, Batch Weight	1380 lbs/CuYd
Fineness Modulus of Fine Aggregate	2.94
Coarse Aggregate 1, Source	Dark Canyon Pit, 1 in
Specific Gravity of Coarse Aggregate 1	2.779
Absorption of Coarse Aggregate 1	0.80%
Coarse Aggregate 1, Batch Weight	1507 lbs/CuYd
Coarse Aggregate 2, Source	Dark Canyon Pit, 3/8 in
Specific Gravity of Coarse Aggregate 2	2.778
Absorption of Coarse Aggregate 2	1.00%
Coarse Aggregate 2, Batch Weight	250 lbs/CuYd
Specific Gravity of Fly Ash - Class F	NA
Fly Ash Class F, Batch Weight	NA
Specific Gravity of Fly Ash - Class C	2.6
Fly Ash Class C, Batch Weight	140 lbs/CuYd
Total Pozzolan/Total Cementitious Ratio	25.00%
Water / Cement Ratio	0.57
Daravair (Admixture), Batch Weight	0.5 lbs/CuYd

Nomenclature	Remarks
Daracem 55 (Admixture), Batch Weight	1.4 lbs/CuYd
Slump	4.5 in
Air Content	6.00%

Table 3.3. Mix-1, Gradation Properties of Coarse and Fine Aggregates

Sieve Size	Percent Passing (%) Fine Aggregate	Percent Passing (%) Coarse Aggregate # 1	Percent Passing (%) Coarse Aggregate # 2
1.5 in.	100	100	100
1 in.	100	100	100
0.75 in.	100	88	100
0.5 in.	100	48	100
0.375 in.	100	22	100
No. 4	97	2	16
No. 8	80	1	4
No. 16	67		
No. 30	22		
No. 50	4		
No. 100	1.7		
No. 200	2.5		

3.1.2 Details of Mix-2 (CA-ID-2)

Concrete specimens were prepared from the concrete mix–2 on Jan 23, 2017. This concrete mix was supplied by K Barnett Inc from Clovis, NM. The details of this concrete mix are given in Table 3.4 and 3.5.

Table 3.4. Details of Concrete Mix-2

Item	Remarks
Company	K Barnett & Sons
Concrete Type	Class F-LS
Cement Source	GCC - Tijeras
Cement, Batch Weight	466 lbs/CuYd
Specific Gravity of Cement	3.15
Fine Aggregate Source	Steele Pit

Item	Remarks
Specific Gravity of Fine Aggregate	2.64
Absorption of Fine Aggregate	1.20%
Fine Aggregate, Batch Weight	1215 lbs/CuYd
Fineness Modulus of Fine Aggregate	2.94
Coarse Aggregate 1, Source	Steele Pit, 1 in
Specific Gravity of Coarse Aggregate 1	2.644
Absorption of Coarse Aggregate 1	0.90%
Coarse Aggregate 1, Batch Weight	1163 lbs/CuYd
Coarse Aggregate 2, Source	Steele Pit, 3/4 in
Specific Gravity of Coarse Aggregate 2	2.69
Absorption of Coarse Aggregate 2	0.90%
Coarse Aggregate 2, Batch Weight	624 lbs/CuYd
Specific Gravity of Fly Ash - Class F	2
Fly Ash Class F, Batch Weight	132 lbs/CuYd
Total Pozzolan/Total Cementitious Ratio	22.10%
Water / Cement Ratio	0.49
AT60 (Admixture), Batch Weight	0.4 lbs/CuYd
Daracem 55 (Admixture), Batch Weight	3.7 lbs/CuYd
Slump	1.5 in
Air Content	6.50%

Table 3.5. Mix-2, Gradation Properties of Coarse and Fine Aggregates

Sieve Size	Percent Passing (%) Fine Aggregate	Percent Passing (%) Coarse Aggregate # 1	Percent Passing (%) Coarse Aggregate # 2
1.5 in.	100	100	100
1 in.	100	100	100
0.75 in.	100	76	100
0.5 in.	100	17	85
0.375 in.	100	3	48
No. 4	100	2	2
No. 8	79	1	0
No. 16	57		
No. 30	40		
No. 50	23		
No. 100	7		

Sieve Size	Percent Passing (%) Fine Aggregate	Percent Passing (%) Coarse Aggregate # 1	Percent Passing (%) Coarse Aggregate # 2
No. 200	2.5		

3.1.3 Details of Mix-3 (CA-ID-3)

Concrete specimens were prepared from the concrete mix–3 on Feb 02, 2017. This concrete mix was supplied by C&E Concrete from Grants, NM. The coarse aggregate used in this mix was from Tinaja pit which contains 100% Limestone. The details of this concrete mix are given in Table 3.6 and 3.7.

Table 3.6. Details of Concrete Mix-3

Item	Remarks
Company	C&E Concrete
Concrete Type	Class F
Cement Source	GCC - Tijeras
Cement, Batch Weight	390 lbs/CuYd
Specific Gravity of Cement	3.15
Fine Aggregate Source	Tinaja
Specific Gravity of Fine Aggregate	2.647
Absorption of Fine Aggregate	2.20%
Fine Aggregate, Batch Weight	1505 lbs/CuYd
Fineness Modulus of Fine Aggregate	3.27
Coarse Aggregate Source	Tinaja Pit, 1 in
Specific Gravity of Coarse Aggregate	2.665
Absorption of Coarse Aggregate	1.40%
Coarse Aggregate, Batch Weight	1505 lbs/CuYd
Specific Gravity of Fly Ash - Class F	2.03
Fly Ash Class F, Batch Weight	130 lbs/CuYd
Total Pozzolan/Total Cementitious Ratio	25.00%
Water / Cement Ratio	0.63
Sika Air (Admixture), Batch Weight	0.98 lbs/CuYd
Sika Plastocrete 161 (Admixture), Batch Weight	2.6 lbs/CuYd
Slump	1.5 in
Air Content	7.00%

Table 3.7. Mix-3, Gradation Properties of Coarse and Fine Aggregates

Sieve Size	Percent Passing (%) Fine Aggregate	Percent Passing (%) Coarse Aggregate # 1	Percent Passing (%) Coarse Aggregate # 2
1.5 in.	100	100	NA
1 in.	100	100	NA
0.75 in.	100	95	NA
0.5 in.	100	40	NA
0.375 in.	100	16	NA
No. 4	99	2	NA
No. 8	77	2	NA
No. 16	48		
No. 30	30		
No. 50	14		
No. 100	5		
No. 200	1.8		

3.1.4 Details of Mix-4 (CA-ID-4)

Concrete specimens comprising 40 cylinders and 12 beams were prepared with the concrete mix–4. This concrete mix was from Vulcan Materials in Santa Fe, NM. The details of this concrete mix are given in Table 3.8. The fineness modulus of fine aggregate was 2.67. The coarse aggregate was taken from Placitas pit and comprised of two sizes i.e. 1 in. and 3/4 in. The gradation properties of coarse and fine aggregates are given in Table 3.9.

Table 3.8. Details of Concrete Mix-4

Nomenclature	Remarks
Company	Vulcan Materials
Concrete Type	Class F
Cement Source	GCC - Tijeras
Cement, Batch Weight	510 lbs/CuYd
Specific Gravity of Cement	3.15
Fine Aggregate Source	Placitas Pit
Specific Gravity of Fine Aggregate	2.573
Absorption of Fine Aggregate	0.70%
Fine Aggregate, Batch Weight	1400 lbs/CuYd
Fineness Modulus of Fine Aggregate	2.67
Coarse Aggregate 1, Source	Placitas Pit, 1 in
Specific Gravity of Coarse Aggregate 1	2.587
Absorption of Coarse Aggregate 1	1.30%
Coarse Aggregate 1, Batch Weight	1042 lbs/CuYd

Nomenclature	Remarks
Coarse Aggregate 2, Source	Placitas Pit, 3/4 in
Specific Gravity of Coarse Aggregate 2	2.587
Absorption of Coarse Aggregate 2	1.40%
Coarse Aggregate 2, Batch Weight	330 lbs/CuYd
Specific Gravity of Fly Ash - Class F	2.03
Fly Ash Class F, Batch Weight	217 lbs/CuYd
Specific Gravity of Fly Ash - Class C	NA
Fly Ash Class C, Batch Weight	NA
Total Pozzolan/Total Cementitious Ratio	30.00%
Water / Cement Ratio	0.35
Zyla 610 (Admixture), Batch Weight	2.28 lbs/CuYd
AT 60 (Admixture), Batch Weight	0.5 lbs/CuYd
Slump	0.5 in
Air Content	5.40%

Table 3.9. Mix-4, Gradation Properties of Coarse and Fine Aggregates

Sieve Size	Percent Passing (%) Fine Aggregate	Percent Passing (%) Coarse Aggregate # 1	Percent Passing (%) Coarse Aggregate # 2
1.5 in.	100	100	100
1 in.	100	100	100
0.75 in.	100	94	100
0.5 in.	100	38	100
0.375 in.	100	24	98
No. 4	95	3	11
No. 8	82	2	3
No. 16	70		
No. 30	55		
No. 50	25		
No. 100	6		
No. 200	2		

3.1.5 Details of Mix-5 (CA-ID-5)

Concrete specimens were prepared with the concrete mix–5. This concrete mix was from Jobe Materials in Vado, NM. The details of this concrete mix are given in Table 3.10. The fineness modulus of fine aggregate was 2.72. The coarse aggregate was taken from Avispa Quarry and comprised of two sizes i.e. 1 in. and 3/4 in. The gradation properties of coarse and fine aggregates are given in Table 3.11.

Table 3.10. Details of Concrete Mix-5

Item	Remarks
Company	Jobe Materials
Concrete Type	Class F-LS
Cement Source	GCC - Samaluyuca
Cement, Batch Weight	506 lbs/CuYd
Specific Gravity of Cement	3.15
Fine Aggregate, Source	Dyer sand
Specific Gravity of Fine Aggregate	2.65
Absorption of Fine Aggregate	1.10%
Fine Aggregate, Batch Weight	938 lbs/CuYd
Fineness Modulus of Fine Aggregate	2.72
Coarse Aggregate 1, Source	Avispa Quarry, 1 in.
Specific Gravity of Coarse Aggregate 1	2.709
Absorption of Coarse Aggregate 1	0.30%
Coarse Aggregate 1, Batch Weight	746 lbs/CuYd
Coarse Aggregate 2, Source	Avispa Quarry, 3/4 in.
Specific Gravity of Coarse Aggregate 2	2.697
Absorption of Coarse Aggregate 2	0.40%
Coarse Aggregate 2, Batch Weight	1254 lbs/CuYd
Specific Gravity of Fly Ash - Class F	2.35
Fly Ash Class F, Batch Weight	140 lbs/CuYd
Total Pozzolan/Total Cementitious Ratio	27%
Water / Cement Ratio	0.33
AEA 92 (Admixture), Batch Weight	1 lbs/CuYd
X-15 A&B (Admixture), Batch Weight	4 lbs/CuYd
Slump (ASTM C 143)	2 in
Air Content (ASTM C 231-B)	7.50%

Table 3.11. Mix-5, Gradation Properties of Coarse and Fine Aggregates

Sieve Size	Percent Passing (%) Fine Aggregate	Percent Passing (%) Coarse Aggregate # 1	Percent Passing (%) Coarse Aggregate # 2
1.5 in.	100	100	100
1 in.	100	90	100
0.75 in.	100	12	98
0.5 in.	100	1	67
0.375 in.	100	1	48
No. 4	92	1	21
No. 8	84	1	8
No. 16	75		

Sieve Size	Percent Passing (%) Fine Aggregate	Percent Passing (%) Coarse Aggregate # 1	Percent Passing (%) Coarse Aggregate # 2
No. 30	55		
No. 50	19		
No. 100	3		
No. 200	1	0.3	3.6

3.1.6 Details of Mix-6 (CA-ID-6)

Concrete specimens were prepared with the concrete mix–6. This concrete mix was from Duke City Redimix in Albuquerque, NM. The details of this concrete mix are given in Table 3.12. The specimens were transported to pavement laboratory at UNM, after initial setting, and placed in temperature-controlled curing tanks at 75 ^{0}F for final curing. The fineness modulus of fine aggregate was 2.57. The coarse aggregate was taken from South Valley and comprised of two sizes i.e. 7/8 in. and 3/8 in. The gradation properties of coarse and fine aggregates are given in Table 3.13.

Table 3.12. Details of Concrete Mix-6

Item	Remarks
Company	Duke City Redi Mix
Concrete Type	Class F
Cement Source	Holcim
Cement, Batch Weight	390 lbs/CuYd
Specific Gravity of Cement	3.15
Fine Aggregate, Source	Orona
Specific Gravity of Fine Aggregate	2.56
Absorption of Fine Aggregate	2.10%
Fine Aggregate, Batch Weight	1220 lbs/CuYd
Fineness Modulus of Fine Aggregate	2.57
Coarse Aggregate 1, Source	South Valley, 7/8 in.
Specific Gravity of Coarse Aggregate 1	2.74
Absorption of Coarse Aggregate 1	2.20%
Coarse Aggregate 1, Batch Weight	970 lbs/CuYd
Coarse Aggregate 2, Source	Southvalley, 3/8 in.
Specific Gravity of Coarse Aggregate 2	2.77
Absorption of Coarse Aggregate 2	2.30%
Coarse Aggregate 2, Batch Weight	940 lbs/CuYd
Specific Gravity of Fly Ash - Class F	2.01
Fly Ash Class F, Batch Weight	116 lbs/CuYd
Total Pozzolan/Total Cementitious Ratio	27%
Water / Cement Ratio	0.4
Sika Air (Admixture), Batch Weight	0.5 lbs/CuYd

Item	Remarks
Plastocrete (Admixture), Batch Weight	2.5 lbs/CuYd
Slump	0.75 in
Air Content	3.50%

Table 3.13. Mix-6, Gradation Properties of Coarse and Fine Aggregates

Sieve Size	Percent Passing (%) Fine Aggregate	Percent Passing (%) Coarse Aggregate # 1	Percent Passing (%) Coarse Aggregate # 2
1.5 in.	100	100	100
1 in.	100	100	100
0.75 in.	100	97	100
0.5 in.	100	50	100
0.375 in.	100	26	97
No. 4	93	4	32
No. 8	86	2	6
No. 16	79		
No. 30	61		
No. 50	20		
No. 100	4		
No. 200	0.8		

3.1.7 Details of Mix-7 (CA-ID-7)

Concrete specimens comprising 40 cylinders and 12 beams were prepared with the concrete mix–7. This concrete mix was obtained from BTU block and concrete in Las Vegas, NM. The details of this concrete mix are given in Table 3.14. The fineness modulus of fine aggregate was 2.57. The coarse aggregate was taken from South Valley and comprised of two sizes i.e. 7/8 in. and 3/8 in. The gradation properties of coarse and fine aggregates are given in Table 3.15.

Table 3.14. Details of Concrete Mix-7

Item	Remarks
Company	BTU Block & Concrete
Concrete Type	Class F
Cement Source	GCC - Tijeras
Cement, Batch Weight	405 lbs/CuYd

Item	Remarks
Specific Gravity of Cement	3.15
Fine Aggregate, Source	Watrous pit
Specific Gravity of Fine Aggregate	2.63
Absorption of Fine Aggregate	0.80%
Fine Aggregate, Batch Weight	1340 lbs/CuYd
Fineness Modulus of Fine Aggregate	2.67
Coarse Aggregate 1, Source	Watrous pit, 3/8 in.
Specific Gravity of Coarse Aggregate 1	2.619
Absorption of Coarse Aggregate 1	1.10%
Coarse Aggregate 1, Batch Weight	135 lbs/CuYd
Coarse Aggregate 2, Source	Watrous pit, 3/4 in.
Specific Gravity of Coarse Aggregate 2	2.622
Absorption of Coarse Aggregate 2	1.00%
Coarse Aggregate 2, Batch Weight	1470 lbs/CuYd
Specific Gravity of Fly Ash - Class F	2.01
Fly Ash Class F, Batch Weight	108 lbs/CuYd
Total Pozzolan/Total Cementitious Ratio	25%
Water / Cement Ratio	0.43
Micro Air (Admixture), Batch Weight	0.78 lbs/CuYd
Polyheed 997 (Admixture), Batch Weight	2.8 lbs/CuYd
Slump	1 in
Air Content	4.50%

Table 3.15. Mix-7, Gradation Properties of Coarse and Fine Aggregates

Sieve Size	Percent Passing (%) Fine Aggregate	Percent Passing (%) Coarse Aggregate # 1	Percent Passing (%) Coarse Aggregate # 2
1.5 in.	100	100	100
1 in.	100	100	100
0.75 in.	100	86	100
0.5 in.	100	51	100
0.375 in.	100	32	98

Sieve Size	Percent Passing (%) Fine Aggregate	Percent Passing (%) Coarse Aggregate # 1	Percent Passing (%) Coarse Aggregate # 2
No. 4	99	6	21
No. 8	84	3	2
No. 16	70	-	-
No. 30	50	-	-
No. 50	24	-	-
No. 100	6	-	-
No. 200	0.9	-	-

CHAPTER 4

COMPRESSIVE STRENGTH AND ELASTIC MODULUS TESTING

4.1 COMPRESSIVE STRENGTH RESULTS

The compressive strength of concrete cylinders (4x8 in.) was determined according to ASTM C39-16 and AASHTO T 22-14 at the age of 7, 14, 28, 90, 180 and 360 days. This test method covers the determination of compressive strength of cylindrical concrete specimens such as molded cylinders. The method consists of applying a compressive axial load to molded cylinders at a rate of 28 to 42 psi/sec until failure occurs. The compressive strength of the specimen is calculated by dividing the maximum load attained during the test by the cross-sectional area of the specimen. The pictorial view of compressive strength testing is shown in Figure 4.1. The summary of compressive strength results for the 7 concrete mixes is tabulated in Table 4.1.

Figure 4.1. Pictorial View of Compressive Strength Testing

Table 4.1. Summary of Compressive Strength Results

	Compressive Strength (psi)	7 Days	14 Days	28 Days	90 Days	180 Days	360 Days
CA-ID-1	Test-1	6009	6664	7723	8962	9591	10103
CA-ID-1	Test-2	6007	6256	7696	9234	9400	10247
CA-ID-1	Test-3	5809	6525	7664	9156	9491	10159
CA-ID-1	Average	5941	6482	7694	9117	9494	10170
CA-ID-2	Test-1	4894	5221	5823	6590	7457	7754
CA-ID-2	Test-2	4737	5348	5787	6331	7398	7612
CA-ID-2	Test-3	4443	5298	5685	7214	7291	7485
CA-ID-2	Average	4691	5289	5765	6711	7382	7617
CA-ID-3	Test-1	4258	4849	5785	7674	7388	7897
CA-ID-3	Test-2	4394	5043	5481	6804	7528	7602
CA-ID-3	Test-3	4074	4545	5325	6879	7491	7816
CA-ID-3	Average	4242	4812	5530	7119	7469	7772
CA-ID-4	Test-1	3298	3901	4730	5178	5637	6114
CA-ID-4	Test-2	3160	3477	4218	5223	5793	5932
CA-ID-4	Test-3	3186	3597	4318	5181	5691	6121
CA-ID-4	Average	3215	3658	4422	5194	5707	6056
CA-ID-5	Test-1	3597	4301	4805	5873	6652	7147
CA-ID-5	Test-2	3529	4408	4788	6093	6726	6848
CA-ID-5	Test-3	3724	4245	4693	6176	6592	6914
CA-ID-5	Average	3617	4318	4762	6047	6657	6970
CA-ID-6	Test-1	4117	5095	5021	6208	7548	-
CA-ID-6	Test-2	4308	4629	5413	6809	6877	-
CA-ID-6	Test-3	4080	4853	5571	6976	7941	-
CA-ID-6	Average	4168	4859	5335	6664	7455	-
CA-ID-7	Test-1	3370	4219	4776	5446	-	-
CA-ID-7	Test-2	3743	4004	4730	5350	-	-
CA-ID-7	Test-3	3514	4279	4707	5931	-	-
CA-ID-7	Average	3542	4167	4738	5576	-	-

4.1.1 Analysis of Compressive Strength Test Data

The comparison of compressive strength results is shown in Figure 4.2. The results are consistent with a constantly increasing trend with a concrete age. The 28 days compressive strength values range from 4,422 to 7,694 psi whereas the strength at 14

days ranges from 3658 psi to 6482 psi. These values far exceed the minimum NMDOT specification requirement of 3,000 psi.
The average 28-day PCC compressive strength values in the LTPP database range from 3,034 to 7,611 psi with an average value of 5,239 psi (Rao et al., 2012). NMDOT mixes have a higher than average compressive strength value compared to the mixes used in the sections included in the national calibration of the ME design models.

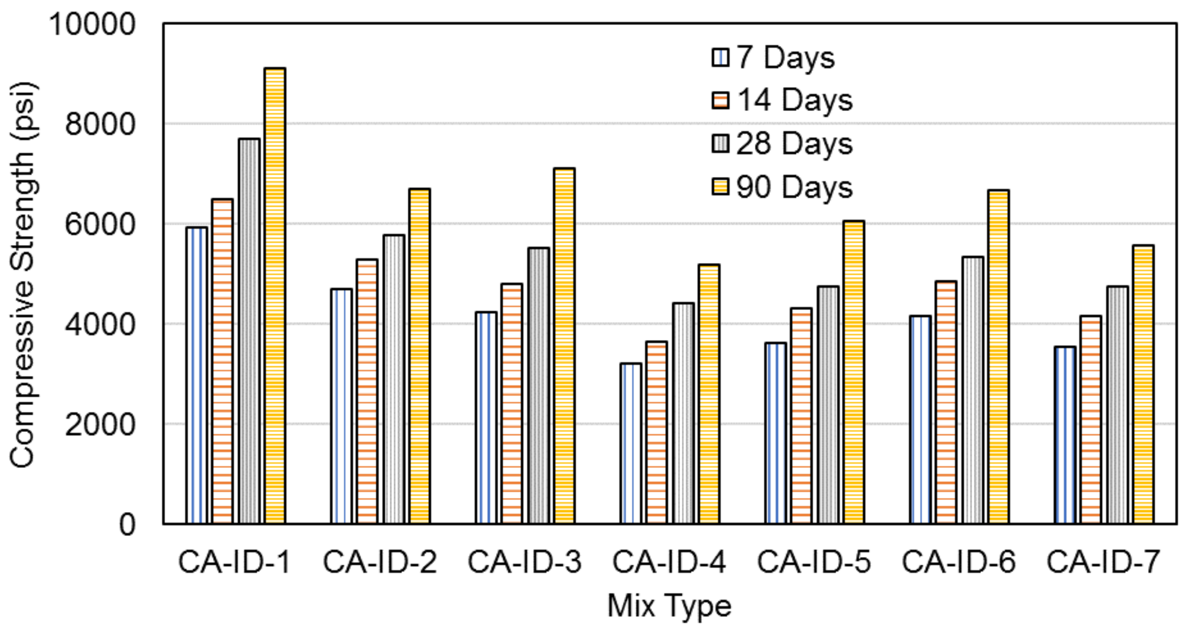

Figure 4.2. Comparison of Compressive Strength Results

4.1.2 Long Term Compressive Strength Results

The long term compressive strength testing of 5 paving mixes was completed (CA-ID-1 to CA-ID-5) up to 360 days. The comparison of long term strength gain is presented in Figure 4.3 which shows that the trend of long term strength gain for all mixes is quite similar and the all the curves become nearly flat near 360 days age.

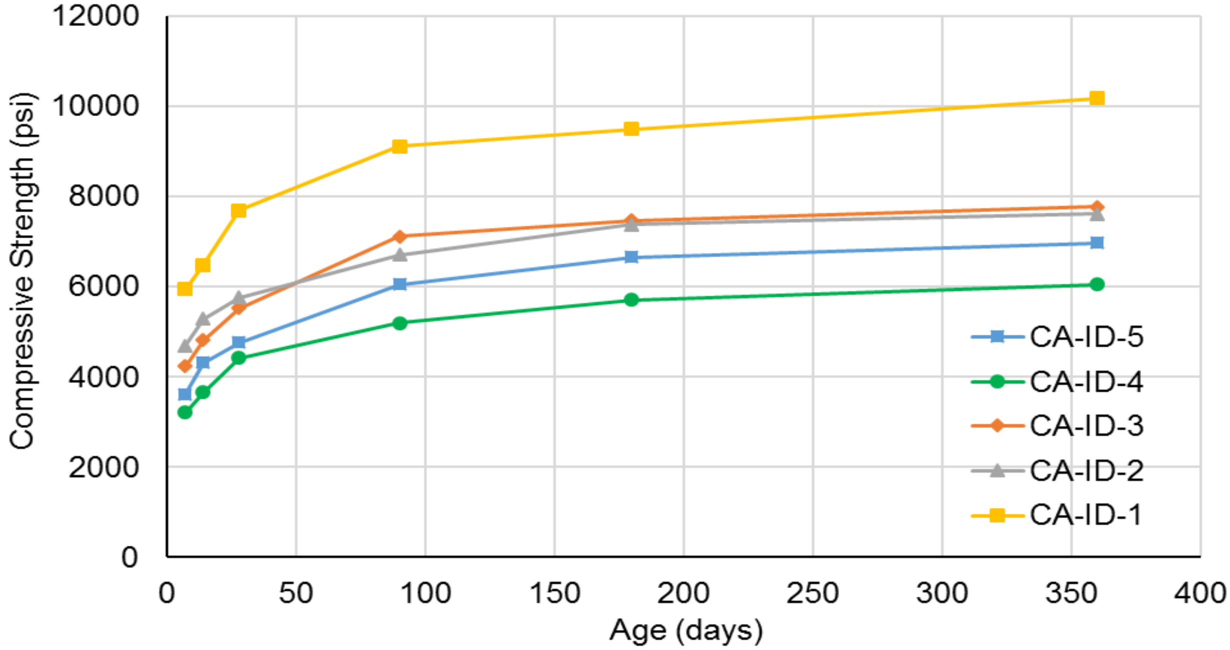

Figure 4.3. Comparison of Long Term Compressive Strengths

4.1.3 Strength Gain Ratios of Compressive Strength

Strength gain ratio of paving concrete for 28 days to 20 years is an important input in Pavement ME Design. The strength gain ratios were determined for the tested mixes by extrapolating the long-term strength data and the comparison is tabulated in Table 4.2. The 28 days/20 years strength ratio ranges from 1.75 to 1.98 for the tested mixes whereas the ME default ratio is 1.44. It is suggested that the ME default ratio may be used in pavement design as it is based on actual laboratory testing of concrete mixes up to 20 years age.

Table 4.2. Summary of Strength Gain Ratios

Age (days)	ME Default	CA-ID-1	CA-ID-2	CA-ID-3	CA-ID-4	CA-ID-5
7	0.85	0.77	0.81	0.77	0.73	0.76
14	0.93	0.84	0.92	0.87	0.83	0.91
28	1.00	1.00	1.00	1.00	1.00	1.00
90	1.12	1.18	1.16	1.29	1.17	1.27
7300 (20 years)	1.44	1.78	1.75	1.98	1.90	1.91

4.2 ELASTIC MODULUS TESTING AND RESULTS

Concrete cylindrical specimens were tested according to ASTM C-469-14 which covers the determination of chord modulus of elasticity (Young's modulus) of molded concrete cylinders when under longitudinal compressive stress. This test method provides stress to strain ratio value for hardened concrete at the prescribed age. The specimen is placed, with the strain-measuring equipment attached, on the lower platen of the testing machine. The axis of the specimen is aligned with the center of thrust of the spherically-seated upper bearing block. The specimen is loaded at least three times. During the first loading, the data is not recorded. The calculations are based on the average of the results of the subsequent loadings. The load is applied continuously and without shock. The load is applied at a constant rate within the range of 35 ± 7 psi/s. The specimen is loaded until the applied load is 40% of the average ultimate load of the companion specimens.

The pictorial view of elastic modulus testing is shown in Figure 4.4. The summary of elastic modulus results for all the mixes is tabulated in Table 4.3. The comparison of these results is shown in Figure 4.5. The 28-day elastic modulus values range from 4.62 to 6.49 E6 psi, with an average of 5.5 E6 psi. These values can be considered high relative to the average value of 4.38 E6 psi corresponding to LTPP sections used in the national calibration of rigid pavement models for ME design.

Figure 4.4. Pictorial View of Elastic Modulus Testing

Table 4.3. Summary of Elastic Modulus Results

	Elastic Modulus (E6 Psi)	7 Days	14 Days	28 Days	90 Days
CA-ID-1	Test-1	5.92	6.16	6.49	7.28
CA-ID-1	Test-2	5.98	6.23	6.41	7.23
CA-ID-1	Test-3	6.05	6.14	6.3	7.36
CA-ID-1	Average	5.98	6.18	6.40	7.29
CA-ID-2	Test-1	4.37	4.69	5.41	6.27
CA-ID-2	Test-2	4.37	4.54	5.38	6.32
CA-ID-2	Test-3	4.33	4.53	5.46	6.17
CA-ID-2	Average	4.36	4.59	5.42	6.25
CA-ID-3	Test-1	3.62	4.31	4.62	5.23
CA-ID-3	Test-2	3.58	4.19	4.71	5.21
CA-ID-3	Test-3	3.56	4.17	4.76	5.19
CA-ID-3	Average	3.59	4.22	4.70	5.21
CA-ID-4	Test-1	3.61	3.75	4.36	4.62
CA-ID-4	Test-2	3.67	3.68	4.29	4.63
CA-ID-4	Test-3	3.64	3.68	4.31	4.55
CA-ID-4	Average	3.64	3.70	4.32	4.60
CA-ID-5	Test-1	3.78	3.97	4.29	4.81
CA-ID-5	Test-2	3.81	3.94	4.13	4.79
CA-ID-5	Test-3	3.76	4.01	4.15	4.76
CA-ID-5	Average	3.78	3.97	4.19	4.79
CA-ID-6	Test-1	2.53	2.91	3.18	4.45
CA-ID-6	Test-2	2.52	2.99	3.13	4.51
CA-ID-6	Test-3	2.54	3.02	3.15	4.39
CA-ID-6	Average	2.53	2.97	3.15	4.45
CA-ID-7	Test-1	3.25	3.33	3.98	4.45
CA-ID-7	Test-2	3.26	3.29	3.95	4.46
CA-ID-7	Test-3	3.23	3.3	3.98	4.51
CA-ID-7	Average	3.25	3.31	3.97	4.47

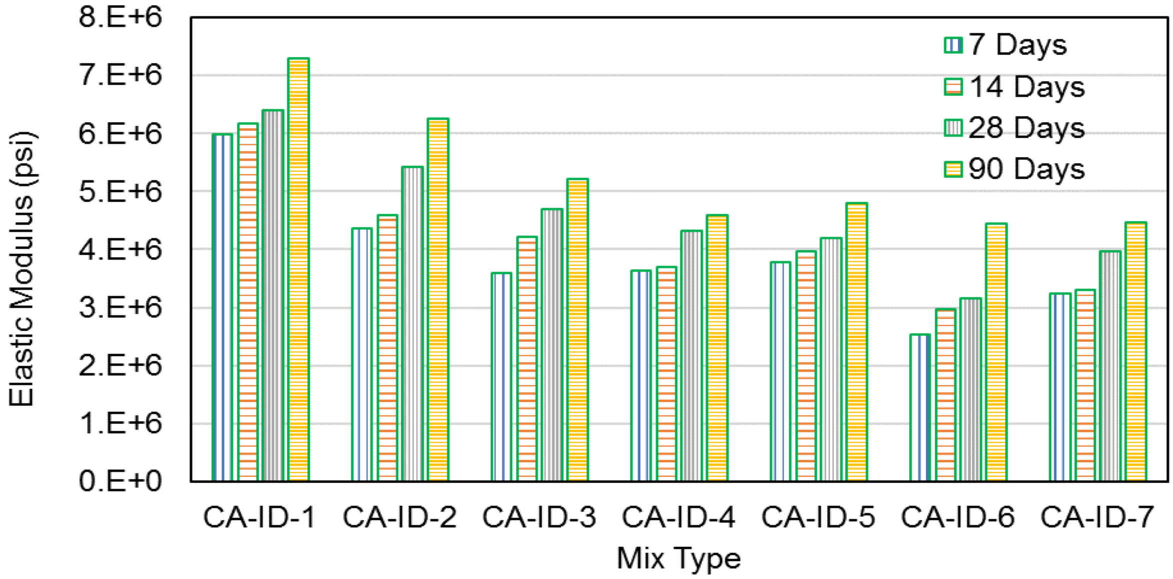

Figure 4.5. Comparison of Elastic Modulus Results

4.3 ME DESIGN MODEL FOR INTERCONVERSION OF COMPRESSIVE STRENGTH INTO ELASTIC MODULUS

4.3.1 ME Design Model for Interconversion of Compressive Strength into Elastic Modulus

The global calibration of the Pavement ME design distress models utilized several level 2 and level 3 inputs based on the best information available from the literature and LTPP database. PCC elastic modulus correlation was borrowed from the American Concrete Institute (ACI) model. The model is as follows:

$$E_c = 57000\, f'_c{}^{0.5} \qquad (1)$$

where E_c = modulus of elasticity in psi; f'_c is compressive strength in psi

4.3.2 Proposed Models for NMDOT Mixes

With the help of tested data of 6 mixes i.e. CA-ID-1 to 6, two inter-conversion models have been proposed. The first model is 0.5 power model similar to the ME design model, designated as Model-1, and the second model is a power model shown as model-2. The regression analysis of these two models is presented in Figure 4.6 and 4.7 respectively.

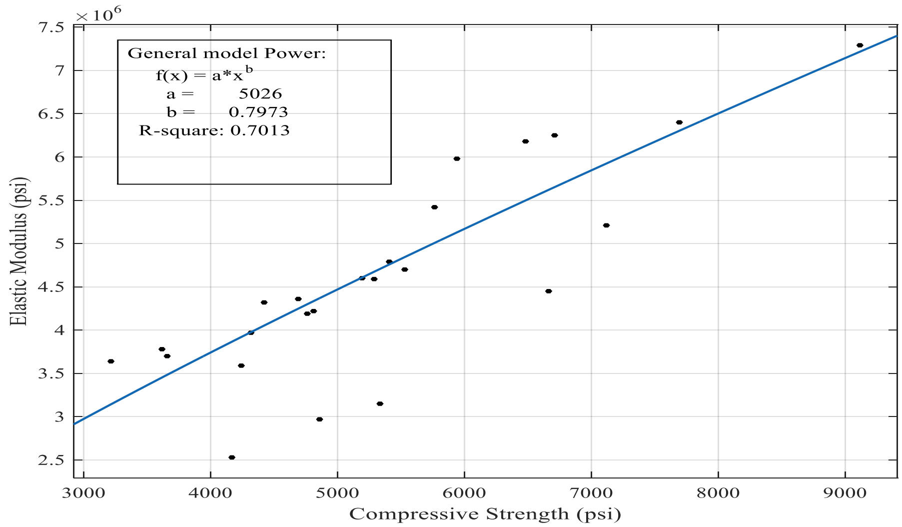

Figure 4.6. Power Model for Elastic Modulus

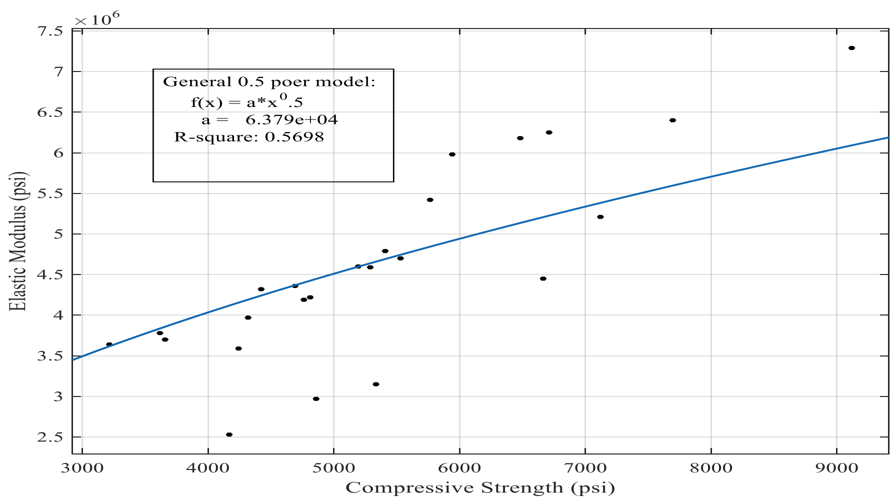

Figure 4.7. 0.5 Power Model for Elastic Modulus

The two models are presented as follows:

$$E_c = 63790\, f'_c{}^{0.5} \qquad \text{Model-1}$$

$$E_c = 5026\, f'_c{}^{0.7973} \qquad \text{Model-2}$$

The proposed power model has a R^2 (coefficient of determination) value of 0.70 and the 0.5 power model gives R^2 value of 0.57, which shows that both the models fit the data well but the power model works better than the 0.5 power model.

4.3.3 Analysis of Proposed Models

The analysis of the proposed model was conducted with reference to the ME design model by determining the predicted values of elastic modulus based on both the models in comparison with the measured values and the results are tabulated in Table 4.4.

Table 4.4. Comparison of Predicted Models and ME Design Model

Concrete Age (Days)	CA-ID	Compressive Strength (psi)	Measured Elastic Modulus (psi)	Power Model Prediction	0.5 Power Model Prediction	ME Default Model Prediction
7	1	5941	5.98E+06	5.13E+06	4.92E+06	4.39E+06
7	2	4691	4.36E+06	4.25E+06	4.37E+06	3.90E+06
7	3	4242	3.59E+06	3.92E+06	4.15E+06	3.71E+06
7	4	3215	3.64E+06	3.14E+06	3.62E+06	3.23E+06
7	5	3617	3.78E+06	3.45E+06	3.84E+06	3.43E+06
7	6	4168	2.53E+06	3.87E+06	4.12E+06	3.68E+06
14	1	6482	6.18E+06	5.50E+06	5.14E+06	4.59E+06
14	2	5289	4.59E+06	4.68E+06	4.64E+06	4.15E+06
14	3	4812	4.22E+06	4.34E+06	4.43E+06	3.95E+06
14	4	3658	3.70E+06	3.48E+06	3.86E+06	3.45E+06
14	5	4318	3.97E+06	3.98E+06	4.19E+06	3.75E+06
14	6	4859	2.97E+06	4.37E+06	4.45E+06	3.97E+06
28	1	7694	6.40E+06	6.30E+06	5.60E+06	5.00E+06
28	2	5765	5.42E+06	5.01E+06	4.84E+06	4.33E+06
28	3	5530	4.70E+06	4.85E+06	4.74E+06	4.24E+06
28	4	4422	4.32E+06	4.05E+06	4.24E+06	3.79E+06
28	5	4762	4.19E+06	4.30E+06	4.40E+06	3.93E+06
28	6	5335	3.15E+06	4.71E+06	4.66E+06	4.16E+06
90	1	9117	7.29E+06	7.22E+06	6.09E+06	5.44E+06
90	2	6711	6.25E+06	5.65E+06	5.23E+06	4.67E+06
90	3	7119	5.21E+06	5.93E+06	5.38E+06	4.81E+06
90	4	5194	4.60E+06	4.61E+06	4.60E+06	4.11E+06
90	5	5408	4.79E+06	4.76E+06	4.69E+06	4.19E+06
90	6	6664	4.45E+06	5.62E+06	5.21E+06	4.65E+06

Further analysis of the predicted elastic modulus values was carried out with the measured values and the results have been plotted as shown in Figure 4.8, which clearly shows that the proposed models give better results for NMDOT mixes when compared to the ME design default model.

In comparison of the two proposed models, the power model works better as the predicted values from this model are closer to the measured elastic modulus values as compared to the 0.5 power model.

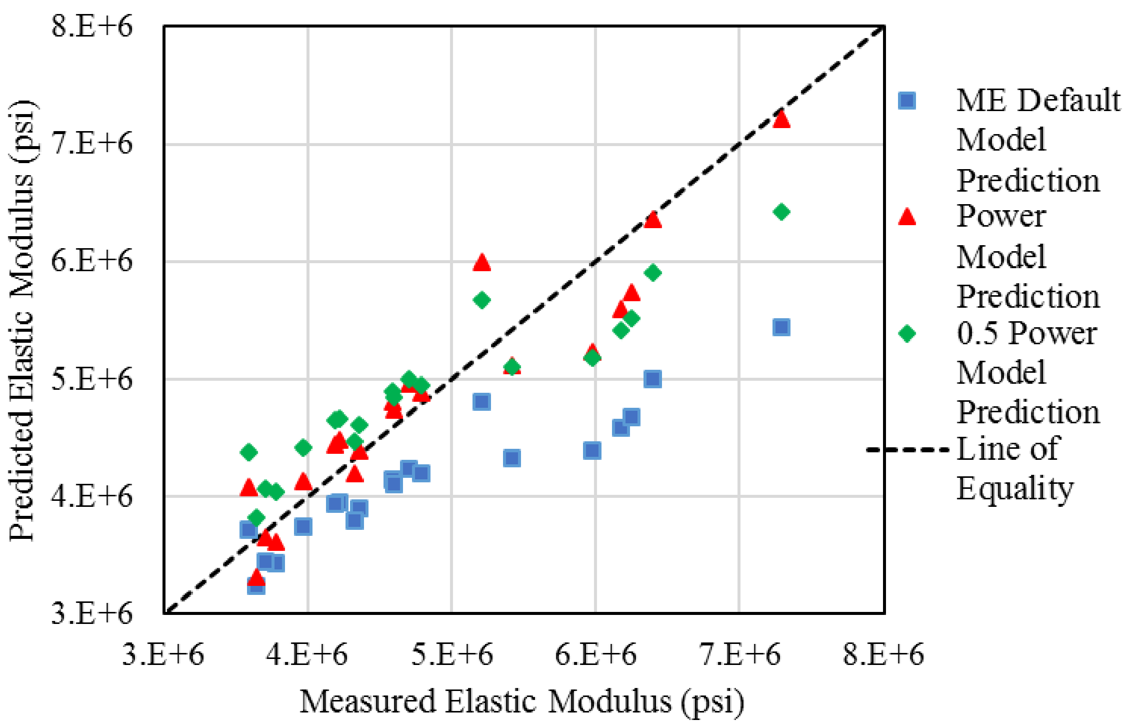

Figure 4.8. Comparison of Measured and Predicted Results

CHAPTER 5

MODULUS OF RUPTURE/FLEXURAL STRENGTH TESTING OF CONCRETE BEAMS

5.1 TESTING METHODOLOGY

This test is conducted per the specifications of ASTM C78-15 and AASHTO T 97-14. This test method covers the determination of the flexural strength of concrete using a simple beam with third-point loading. The results are calculated and reported as the modulus of rupture in psi. The specimen is loaded continuously and without shock. The load shall be applied at a constant rate to the breaking point. The load is applied at a rate that constantly increases the maximum stress on the tension face between 125 and 175 psi/min until rupture occurs. For a beam with dimensions of 6x6x22 in. the loading rate should be between 20 to 30 lbs/sec.

5.2 TEST RESULTS AND DISCUSSION

The beam specimens of all the paving mixes were tested for MOR at 7, 14, 28 and 90 days. The pictorial view of MOR testing is shown in Figure 5.1 and the summary of the results of the 6 mixes is tabulated in Table 5.1. The results follow a similar increasing trend and flexural strength increases with concrete age.

Figure 5.1. Pictorial View of MOR testing

Table 5.1. Summary of MOR Values

	Flexural Strength (psi)	7 Days	14 Days	28 Days	90 Days
CA-ID-1	Test-1	797	896	942	1113
CA-ID-1	Test-2	861	895	963	1076
CA-ID-1	Test-3	756	894	930	967
CA-ID-1	Average	805	895	945	1052
CA-ID-2	Test-1	829	821	963	910
CA-ID-2	Test-2	792	854	881	937
CA-ID-2	Test-3	831	862	894	904
CA-ID-2	Average	817	846	913	917
CA-ID-3	Test-1	606	688	799	985
CA-ID-3	Test-2	622	686	768	971
CA-ID-3	Test-3	691	724	795	849
CA-ID-3	Average	640	699	787	935
CA-ID-4	Test-1	626	621	677	784
CA-ID-4	Test-2	579	675	728	774
CA-ID-4	Test-3	577	626	665	773
CA-ID-4	Average	594	641	690	777
CA-ID-5	Test-1	555	657	643	782
CA-ID-5	Test-2	543	587	656	788
CA-ID-5	Test-3	567	583	678	844
CA-ID-5	Average	555	609	659	805
CA-ID-6	Test-1	606	668	751	867
CA-ID-6	Test-2	557	655	642	904
CA-ID-6	Test-3	634	658	758	809
CA-ID-6	Average	599	660	717	860
CA-ID-7	Test-1	479	504	560	687
CA-ID-7	Test-2	461	506	566	722
CA-ID-7	Test-3	468	497	546	698
CA-ID-7	Average	469	502	557	702

The average values are plotted in Figure 5.2. The results are consistent with a constantly increasing trend. The 28-day flexural strengths are in the range of 557 psi to 945 psi with an average value of 751 psi. These values are characteristic of high strength mixes. A typical 28-day PCC flexural strength value used in rigid pavement design is 650 psi. The target flexural strength of the LTPP sections, which represent the newly constructed rigid pavement experiments nationwide, was 550 psi for the low strength PCC mixes. The 28-day flexural strength values reported in the LTPP database range from 489 to 1006 psi with an average of 735 psi (Rao et al., 2012). It is evident that the NMDOT mixes show evidence of high 28-day flexural strengths.

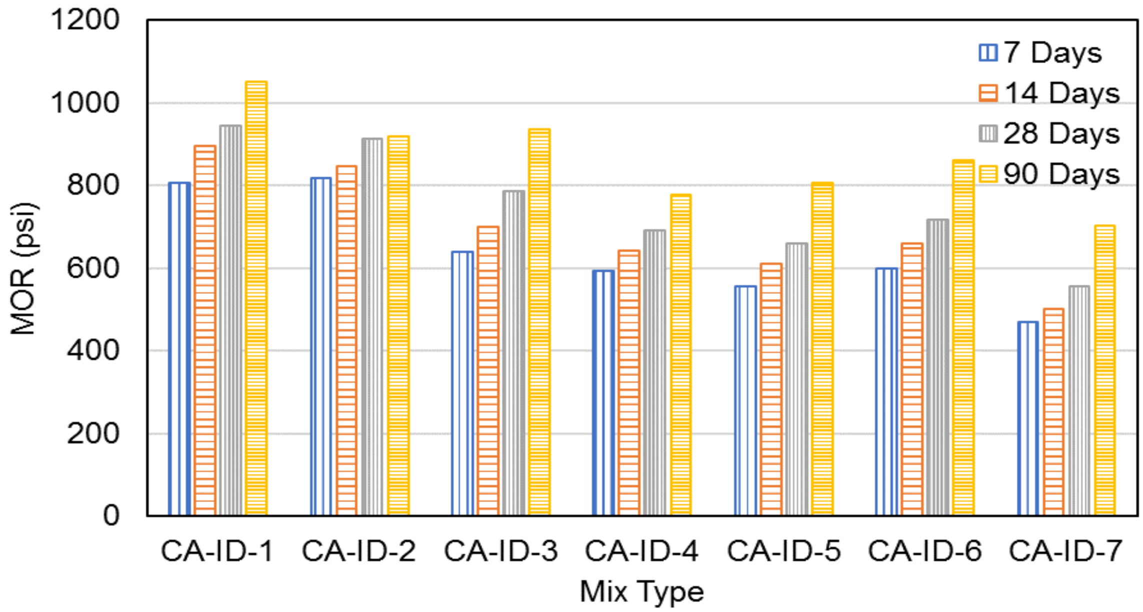

Figure 5.2. Comparison of MOR Test Results

5.3 INTER-CONVERSION DEFAULT MODEL FOR ME DESIGN

ME design distress models utilized several level 2 and level 3 inputs based on the best information available from the literature and LTPP database. PCC flexural strength model for ME design software is based on the Portland Cement Association (PCA) and LTPP studies. The model uses the general model form used in literature for flexural strength estimation and the correlation is expressed as:

$$MR = 9.5\, f'_c{}^{0.5} \qquad (4)$$

where MR is flexural strength in psi; f'_c is compressive strength in psi

5.4 PROPOSED MOR MODELS FOR NMDOT MIXES

Based on the test data of MOR testing, two inter-conversion models have been proposed for NMDOT paving mixes i.e. 0.5 power model and power model. The regression analysis for these two models is presented in Figure 5.3 & 5.4 respectively.

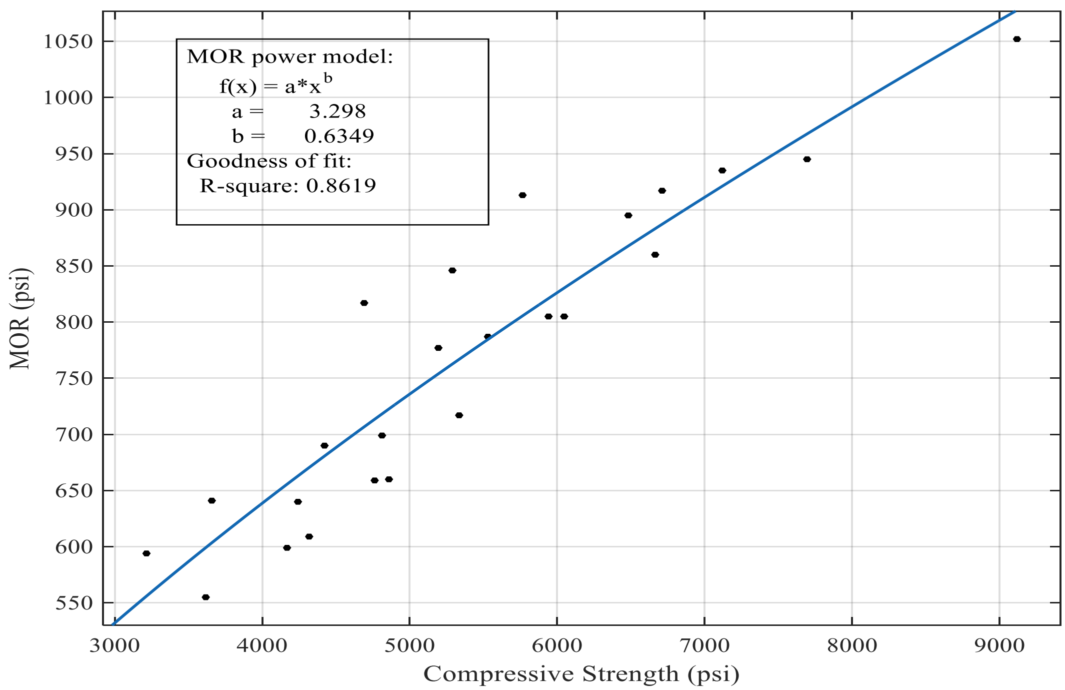

Figure 5.3. Power Model for MOR Prediction

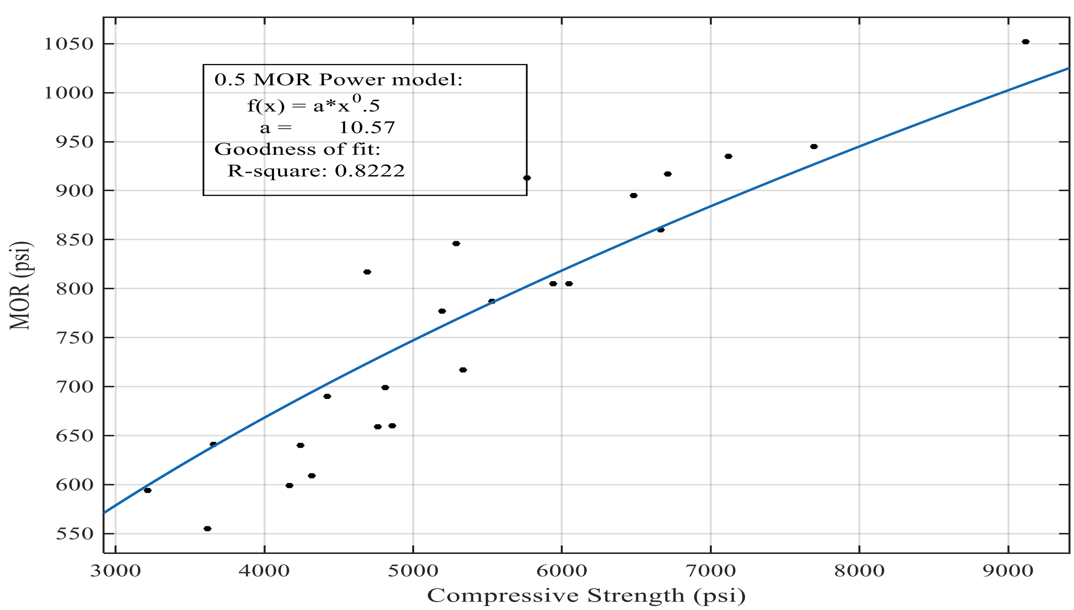

Figure 5.4. 0.5 Power Model for MOR Prediction

The proposed models are presented as Model 3 and 4 respectively. The 0.5 power model presents the R^2 value of 0.82 while the power model gives R^2 value of 0.86, which shows that both the models are a good fit to the experimental data, but the power model is slightly better than the 0.5 power model.

$$MR = 10.7\, f'_c{}^{0.5} \qquad \text{Model-3}$$

$$MR = 3.45\, f'_c{}^{0.6308} \qquad \text{Model-4}$$

5.5 ANALYSIS OF PROPOSED MODELS

The analysis of the proposed model was conducted by determining the predicted MOR values with reference to the ME default model in comparison with the experimental values and the results are tabulated in Table 5.2.

Table 5.2. Comparison of Proposed Model vs ME Default Model

Concrete Age (Days)	CA-ID	Compressive Strength (psi)	Measured Flexural Strength (psi)	Power Model Prediction (psi)	0.5 Power Model Prediction (psi)	ME Default Model Prediction (psi)
7	1	5941	805	820.87	814.71	732.24
7	2	4691	817	706.54	723.95	650.66
7	3	4242	640	662.82	688.43	618.74
7	4	3215	594	555.85	599.33	538.66
7	5	3617	555	599.03	635.70	571.34
7	6	4168	599	655.45	682.40	613.32
14	1	6482	895	867.57	851.00	764.85
14	2	5289	846	762.47	768.71	690.89
14	3	4812	699	718.06	733.23	659.00
14	4	3658	641	603.33	639.29	574.57
14	5	4318	609	670.33	694.57	624.26
14	6	4859	660	722.50	736.80	662.21
28	1	7694	945	967.32	927.15	833.30
28	2	5765	913	805.35	802.55	721.31
28	3	5530	787	784.34	786.03	706.46
28	4	4422	690	680.54	702.89	631.73
28	5	4762	659	713.31	729.41	655.57
28	6	5335	717	766.67	772.04	693.89
90	1	9117	1052	1077.36	1009.26	907.09
90	2	6711	917	886.91	865.90	778.25
90	3	7119	935	920.77	891.83	801.55
90	4	5194	777	753.74	761.77	684.66
90	5	6047	805	830.14	821.95	738.74

Concrete Age (Days)	CA-ID	Compressive Strength (psi)	Measured Flexural Strength (psi)	Power Model Prediction (psi)	0.5 Power Model Prediction (psi)	ME Default Model Prediction (psi)
90	6	6664	860	882.96	862.86	775.52

Further analysis of the predicted MOR values was carried out with the measured values and the results have been plotted as shown in Figure 5.5, which clearly shows that the proposed models give better results for NMDOT mixes when compared to the ME design default model.

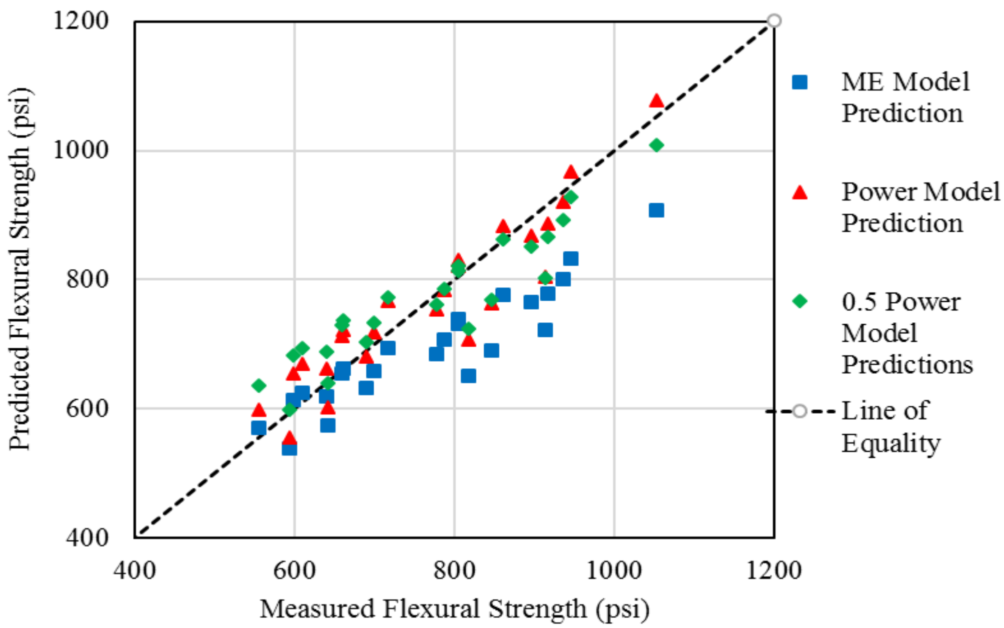

Figure 5.5. Comparison of Measured and Predicted MOR Results

CHAPTER 6

COEFFICIENT OF THERMAL EXPANSION TESTING OF CONCRETE CYLINDERS

6.1 CTE TEST RESULTS

The concrete cylinders were tested for determination of coefficient of thermal expansion (CTE) per AASHTO T 336-11. This method determines the CTE of a cylindrical concrete specimen, maintained in a saturated condition, by measuring the length change of the specimen due to a specified temperature change. The measured length change is corrected for any change in length of the measuring apparatus, and the CTE is then calculated by dividing the corrected length change by the temperature change and then the specimen length. The CTE of one expansion or contraction test segment of a concrete specimen is calculated and reported in micro strains/°F. The pictorial view of CTE testing is shown in Figure 6.1 and the summary of the results for the 6 mixes at the ages of 28 days is tabulated in Table 6.1.

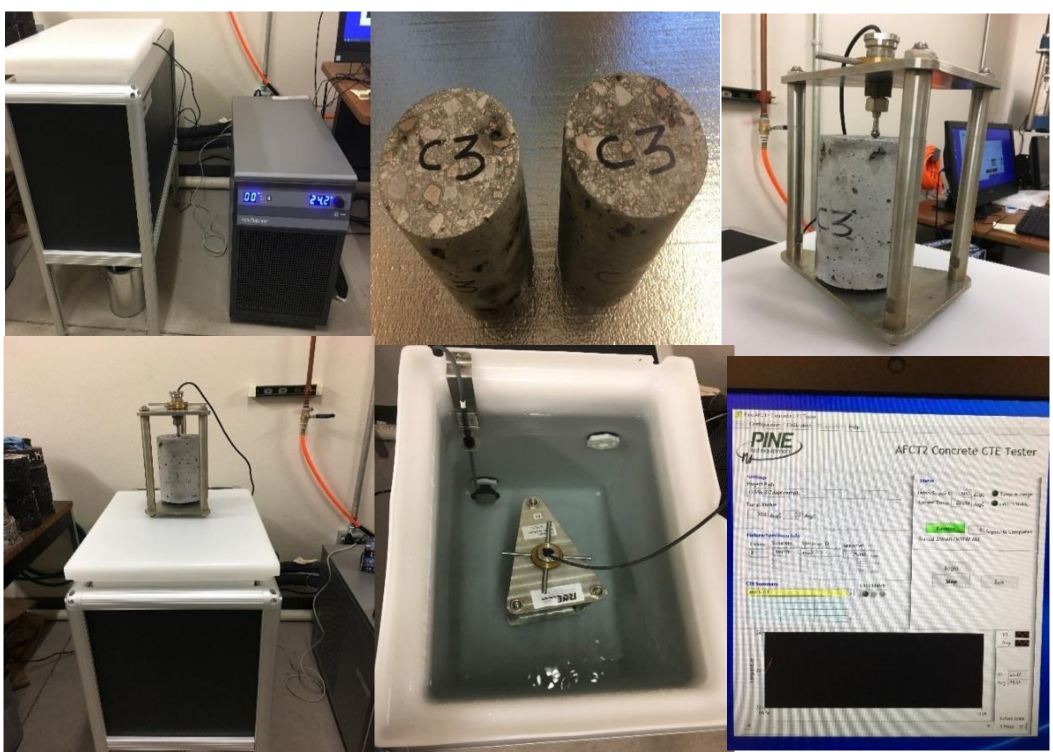

Figure 6.1. Pictorial View of CTE Testing

Table 6.1. 28 Days CTE with Coarse Aggregate Petrography

Mix Type	CA-ID-1	CA-ID-2	CA-ID-3	CA-ID-4	CA-ID-5	CA-ID-6	CA-ID-7
Coarse Aggregate Type	Dolomite	Granite	Limestone	Quartzite	Limestone	Basalt	Quartzite
Average CTE (E-6 in/in/°F)	5.25	5.39	3.71	5.09	4.09	4.37	5.95

The comparison is shown in Figure 6.2. The CTE property in NMDOT mixes varies over a fairly large range. The CTE values range from 3.71 to 5.95 E-6 in/in/°F. The CTE of CA-ID-1, 2, 4 and 6 are consistently higher than that of CA-ID-3 and 5 which are 100% limestone. This confirms the findings from the literature review that Limestone has the lowest CTE value as compared to other minerals. The impact of coarse aggregate on the CTE values is evident with these results. For the tested mixes the standard deviation values for the tested specimens (same lab, same mix design) are within 0.1 E-6 in/in/^{0}F which shows excellent repeatability.

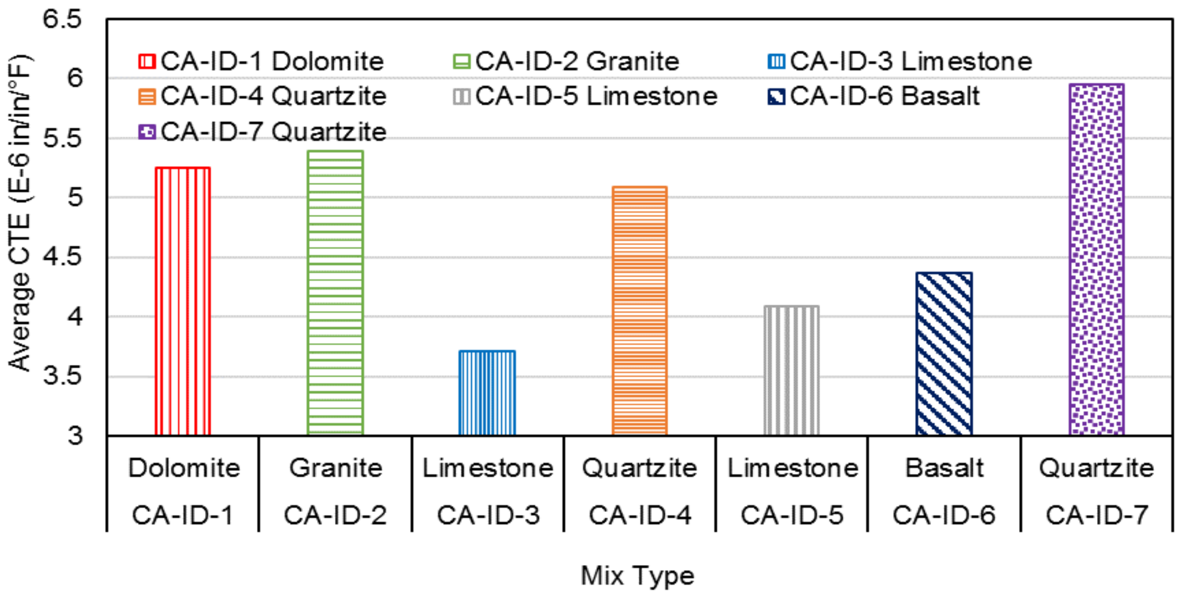

Figure 6.2. Comparison of CTE Test Results

6.2 DEFAULT CTE DATA OF PAVEMENT ME DESIGN

Pavement ME design uses default CTE data for design and analysis purposes. CTE default data is based on coarse aggregate type, which was established based on testing and petrography performed under the LTPP program. The CTE values that were originally generated using the AASHTO TP-60 provisional test procedure was revised and made consistent with the AASHTO T 336 procedure. The recommended default CTE values as per the ME design database for all aggregate types are summarized in Table 6.2.

6.3 COMPARISON OF CTE TEST DATA WITH DEFAULT CTE VALUES

The CTE test data was compared with the ME default data and the results are presented in Figure 6.3. The CTE value of CA-ID- 1 & 2 with dolomite and granite respectively are higher than the ME default values whereas the CTE test data of four mixes is lower than the default values. Thus, using ME default CTE values will lead to inaccuracy in rigid pavement design. The test data is within the lower bound and upper bound of the ME default data which indicates that the tested CTE values are in the normal range.

Table 6.2. ME Design Default CTE Average Data (ARA, 2011)

Primary aggregate origin	Primary aggregate class	PCC CTE, 10 E-6/˚F Average	PCC CTE, 10 E-6/˚F Standard deviation	Number of test sections
Igneous (Extrusive)	Basalt	4.4	0.5	18
Igneous (Plutonic)	Diabase	5.2	0.5	21
Igneous (Plutonic)	Granite	4.8	0.6	69
Metamorphic	Schist	4.4	0.4	17
Sedimentary	Chert	6.1	0.6	25
Sedimentary	Dolomite	5.0	0.7	30
Sedimentary	Limestone	4.4	0.7	160
Sedimentary	Quartzite	5.2	0.5	9
Sedimentary	Sandstone	5.8	0.5	7

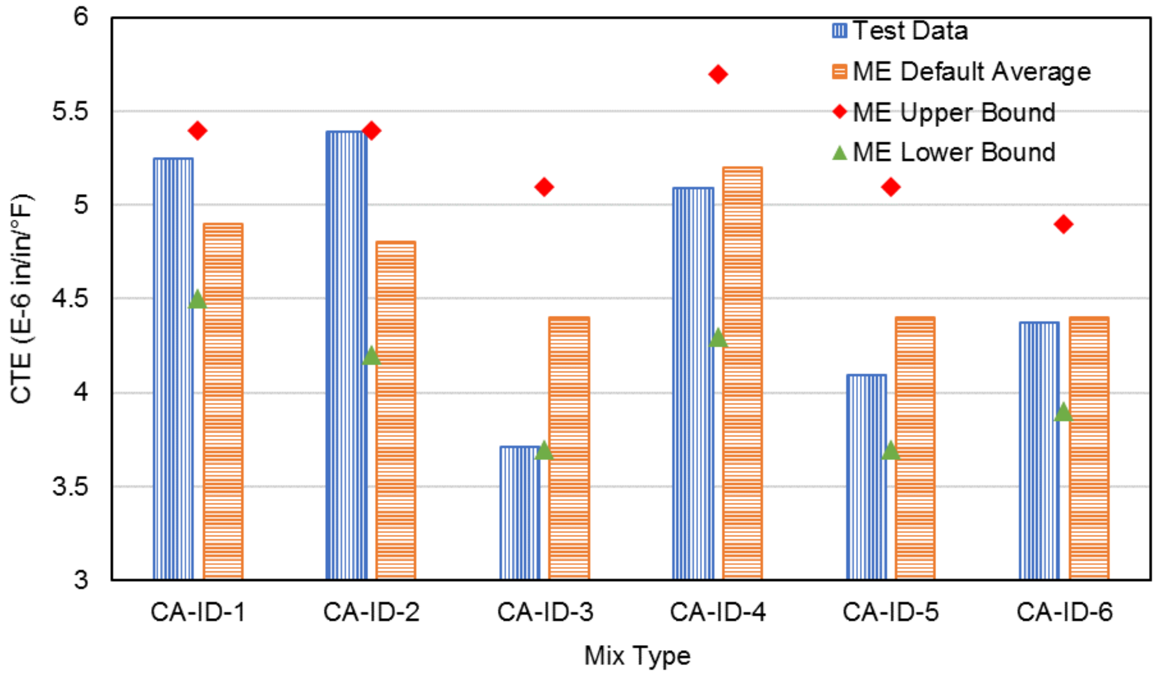

Figure 6.3. Comparison of CTE Test Data with ME Default Data

CHAPTER 7

PAVEMENT ME SIMULATIONS TO EVALUATE THE IMPACT OF THE MATERIAL INPUTS

7.1 IMPACT OF INPUT LEVELS ON JPCP DESIGN

As described earlier that Pavement ME design has 3 levels of inputs which the designer can use based on the available data and accuracy required. Level-1 has the highest level of accuracy with all the tested data for the specific project including CTE, MOR, and elastic modulus. While level-3 has the lowest level of accuracy in the design by using the default values of CTE and using compressive strength parameter at 28 days.

7.1.1 Simulation Methodology

The simulations were conducted in pavement ME design for CA-ID 2, 3, 4 & 5 to contrast the impact of level 1 and level-3 inputs on JPCP performance. The lab tested data for concrete strength properties including MOR and elastic modulus and CTE were used for level-1 design while for level-3 design default CTE value and compressive strength input was used. Other major design inputs including design life, traffic volume, climate, etc. were considered constant as shown in Table 7.1, to compare the effects of input levels.

Table 7.1. JPCP Design Parameters for Simulation Work

Parameter	Value
Design Life	30 years
Design Thickness	10 in
Dowel Diameter & Spacing	1.25 in @ 12 in
Joint Spacing	15 ft
Slab Width	12 ft
Climate Station	Albuquerque, NM
Initial IRI	63 in/mile
Terminal IRI	172 in/mile
Threshold Transverse Cracking (% of Slabs)	15 %
Terminal Mean Joint Faulting	0.12 in
Reliability	90%
Modulus of Rupture of Concrete	Per CA-ID

Parameter	Value
Elastic Modulus of Concrete	Per CA-ID
Poisson's Ratio	0.2
Aggregate Type	Per CA-ID
AADTT	4000
Number of Lanes	2
Base Course	Non-Stabilized
Base Course Thickness	6 in
Base Course Resilient Modulus	25000 Psi

For CA-ID-2 (granite aggregate), the tested CTE value was 5.4 E-6 in/in/°F and ME default value was 4.8 E-6 in/in/°F. For CA-ID-3 (limestone aggregate), the lab value was 3.71 E-6 in/in/°F and ME default value was 4.4 E-6 in/in/°F. For CA-ID-4 (Quartzite aggregate), the lab value was 5.09 E-6 in/in/°F and ME default value was 5.2 E-6 in/in/°F. The comparative results are tabulated in Table 7.2.

7.1.2 Analysis of Simulation Results

The analysis of simulation results was conducted to quantify the effects of input levels variation on pavement performance indicators.

7.1.2.1 Effects on Transverse Cracking

The comparison for transverse cracking is presented in Figure 7.1, which shows that there is significant variation in transverse cracking between the results of level 1 and level 3 inputs. The change in transverse cracking with input levels is 39 to 54% which is highly significant. With these results, it is evident that the pavement must be designed with the accurately tested level-1 inputs for the paving mix to be used so that the designed pavement can last for the entire service life. The ME default CTE data will not produce an accurate design for NMDOT paving mixes.

Table 7.2. Simulation Results of Impact of Input Levels on Pavement Performance

CA-ID	Level-1 Transverse cracking (%)	Level-1 Joint faulting (in)	Level-1 IRI (in/mile)	Level-3 Transverse cracking (%)	Level-3 Joint faulting (in)	Level-3 IRI (in/mile)
2	13.04	0.14	165.9	55.1	0.1	179.5
3	3.79	0.07	114.6	43.35	0.09	158.9
4	68.87	0.11	197.68	115.7	0.11	240.8
5	20.11	0.07	130.5	74.26	0.08	186.1

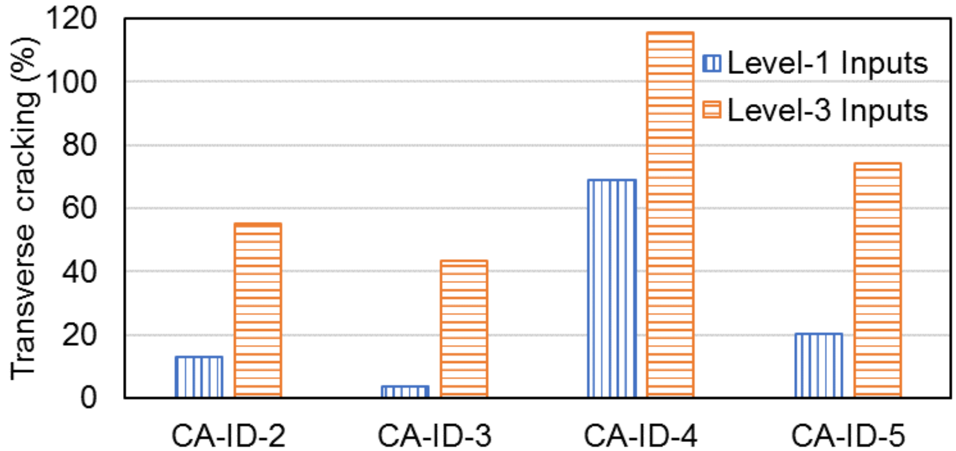

Figure 7.1. Impact of Input Levels on Transverse Cracking

7.1.2.2 Effects on Joint Faulting

The comparative summary for joint faulting is presented in Figure 7.2. It is evident that there is not much significant impact on joint faulting between the two input levels. The difference in joint faulting values ranges between 0.01 to 0.04 in. For further accuracy in the design, level-1 inputs should be used.

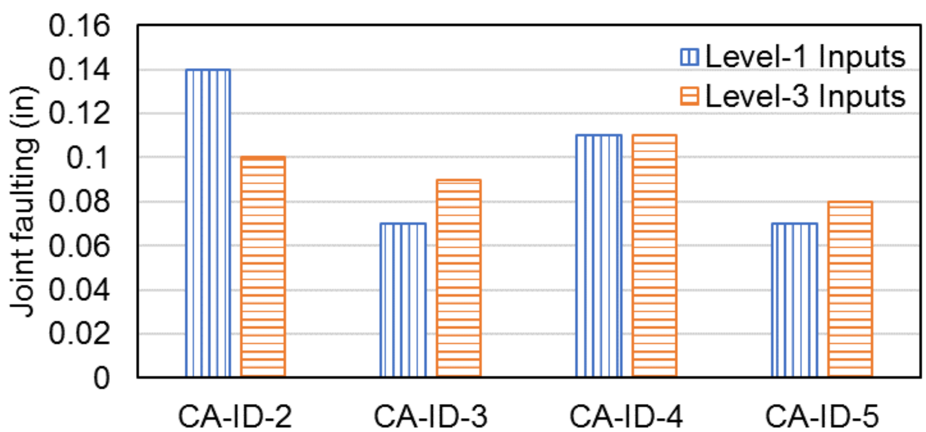

Figure 7.2. Impact of Input Levels on Joint Faulting

7.1.2.3 Effects on Pavement Roughness

The comparison is presented in Figure 7.3, which shows that there is significant variation in IRI between the two input levels. The difference ranges between 13 to 55 in/mile. The high values correspond to the difference in transverse cracking as IRI is dependent on transverse cracking parameter along with other factors. These results necessitate the importance of using level-1 inputs while designing concrete pavement.

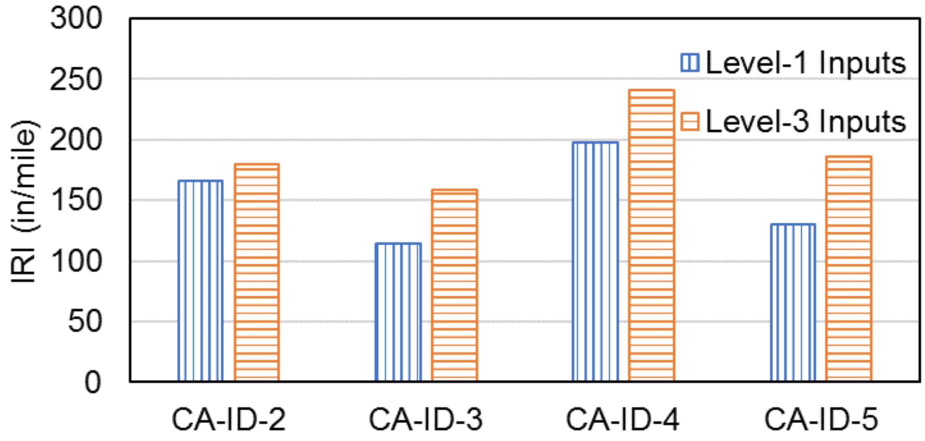

Figure 7.3. Impact of Input Levels on IRI

7.1.2.4 Percent Change in Performance Parameters

To compare the impact of input levels on the performance parameters, the % change in transverse cracking, faulting and IRI was determined and the comparison is shown in Figure 7.4. The results show that transverse cracking has the highest % change with 40 to 91% change as compared to faulting and IRI. While IRI is the next significantly affected parameter with the change of 8 to 42%.

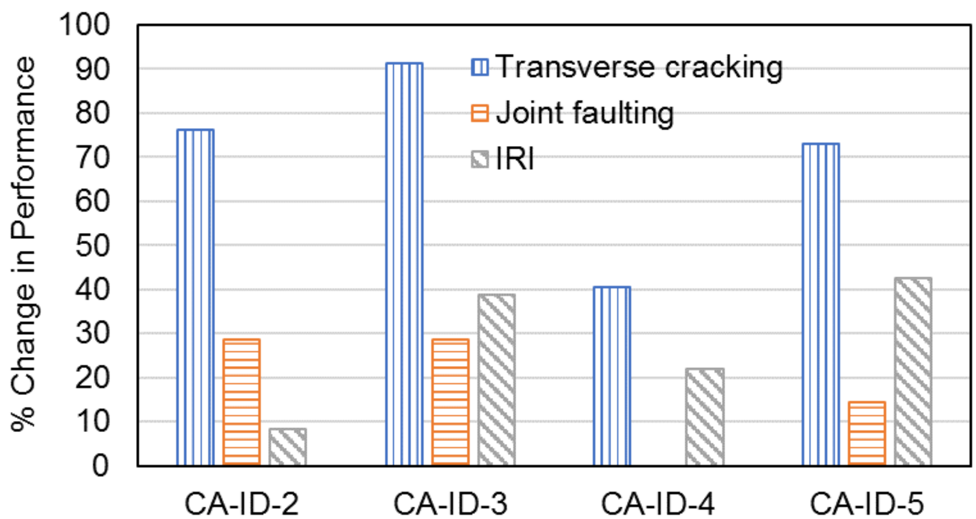

Figure 7.4. % Change in Performance between Input Levels

7.2 EFFECTS OF BASE COARSE MODULUS ON JPCP PERFORMANCE

The effects of base coarse modulus on performance parameters was evaluated for CA-ID-2, 3 & 5 and the results are presented in Table 7.3. The base coarse modulus was varied from 20 ksi to 40 ksi and the results show that there is no effect on joint faulting while there is a marginal effect on transverse cracking ranging between 0.2 to 0.5%. The comparison of results for transverse cracking is shown in Figure 7.5.

Table 7.3. Summary of Effects of Base Coarse Modulus on Pavement Performance

CA-ID	Input Level	Base Coarse MR (ksi)	Cracking (%)	Faulting (in)	IRI (in/mile)
2	1	20	3.36	0.14	159.1
2	3	20	2.18	0.11	143.8
2	1	30	3.21	0.14	159.2
2	3	30	2.18	0.11	143.82
2	1	40	3.13	0.14	159.5
2	3	40	2.18	0.11	143.7
3	1	20	0.96	0.07	118.03
3	3	20	1.92	0.09	132.57
3	1	30	0.96	0.07	117.8
3	3	30	1.92	0.09	132.34
3	1	40	0.96	0.07	117.63
3	3	40	1.92	0.09	132.3
5	1	20	4.41	0.08	124.9
5	3	20	5.35	0.09	131.8
5	1	30	4.19	0.08	124.5
5	3	30	5.05	0.09	131.4
5	1	40	3.95	0.08	124.29
5	3	40	4.8	0.09	131.2

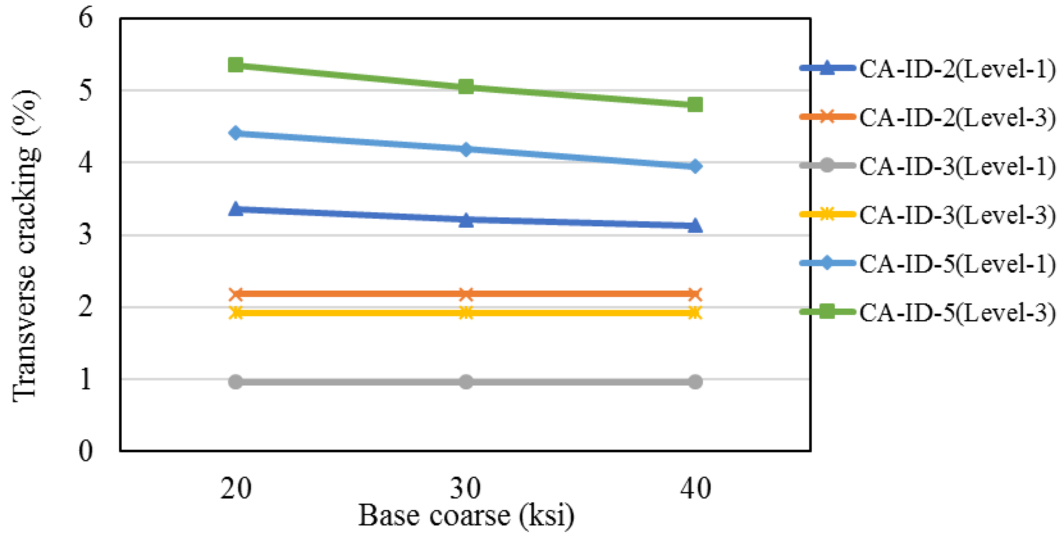

Figure 7.5. Impact of Base Coarse Modulus on Transverse Cracking

7.3 EFFECTS OF JOINT SPACING ON JPCP PERFORMANCE

The effects of joint spacing on pavement performance parameters were evaluated by conducting simulations with level-1 inputs used for all the paving mixes and varying the joint spacing between 12 to 17 feet. The summary of results is presented in Table 7.4. The results show that there is a significant impact of joint spacing on all the performance parameters. The maximum impact of joint spacing is on transverse cracking with % change of 30 to 122% with a unit change in joint spacing, while joint faulting is affected with % change of 6 to 20%. This shows that joint spacing has significant effects on JPCP and by modifying the joint spacing the adverse effects of higher CTE concrete can be minimized.

7.3.1 Comparison of Effects of Joint Spacing

The comparison of the effects of joint spacing on cracking, faulting and IRI is shown in Figure 7.6, 7.7, and 7.8 respectively. The results show that joint spacing has a significant impact on all the performance parameters and as the joint spacing increases the pavement distresses also increases with all other design factors being constant. Thus, pavement performance improves when joint spacing is reduced with less transverse cracking, less faulting, and lower IRI values.

Table 7.4. Summary of Effects of Joint Spacing on Pavement Performance

CA-ID	Joint Spacing (ft)	Transverse cracking (%)	Joint faulting (in)	IRI (in/mile)
2	12	2.52	0.11	153.88
2	13	4	0.12	156.7
2	14	6.5	0.13	85.26
2	15	13.04	0.14	165.9
2	16	28.97	0.15	180.2
2	17	63.22	0.16	211.9
3	12	1.92	0.05	112.5
3	13	2.5	0.06	113.7
3	14	3.05	0.06	114.37
3	15	3.79	0.07	114.64
3	16	4.57	0.07	114.74
3	17	5.45	0.07	114.88
4	12	11.53	0.09	143.2
4	13	22.55	0.1	153.85
4	14	42.23	0.1	172.52
4	15	68.87	0.11	197.68
4	16	94.53	0.12	221.57
4	17	112.18	0.12	237.71
5	12	6.25	0.06	118.67
5	13	9.2	0.06	121.49
5	14	13.76	0.07	125.22
5	15	20.11	0.07	130.55
5	16	28.06	0.08	137.22
5	17	36.39	0.08	144.42

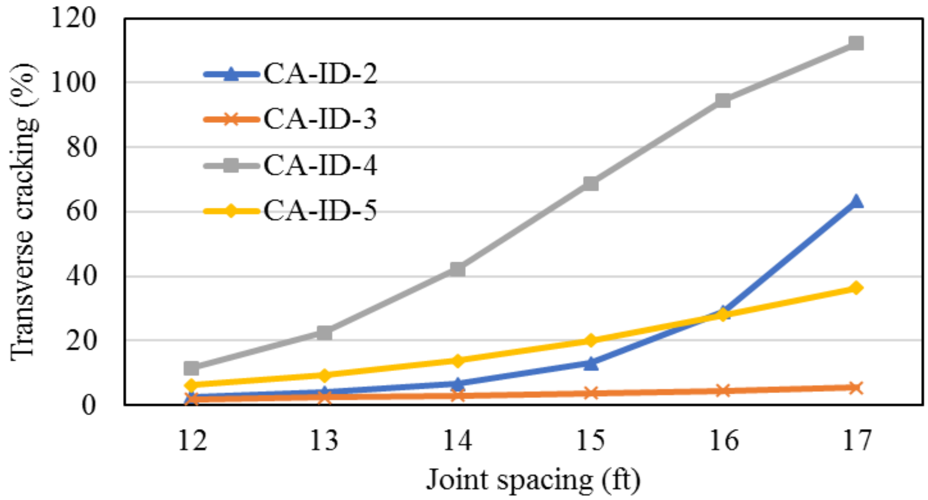

Figure 7.6. Impact of Joint Spacing on Transverse Cracking

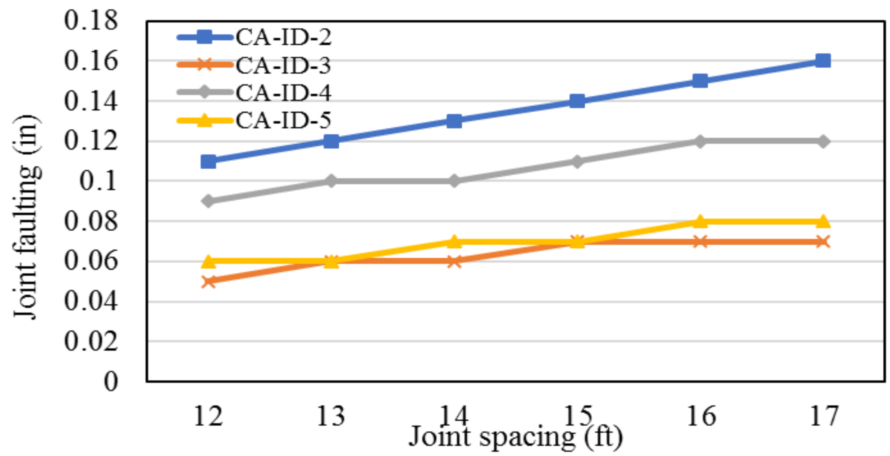

Figure 7.7. Impact of Joint Spacing on Faulting

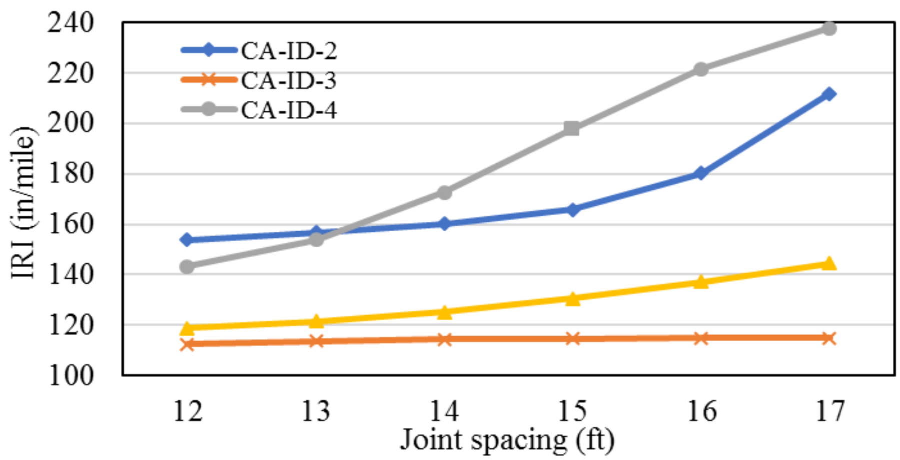

Figure 7.8. Impact of Joint Spacing on Pavement Roughness

7.4 EFFECTS OF LEVEL-1 INPUTS ON PAVEMENT THICKNESS

The effects of level-1 inputs including CTE and strength data were evaluated with regards to the thickness of pavement slab on the performance parameters. The summary of the simulation results is presented in Table 7.5. With regards to transverse cracking the distress decreases as the slab thickness increases and for the simulated design scenario the slab thickness range of 9 to 10 in is suited with transverse cracking below the threshold after design life of 30 yrs. The comparison of all the mixes with various slab thickness and transverse cracking is shown in Figure 7.9.

Table 7.5. Summary of Effects of Slab Thickness on Pavement Performance

	Slab Thickness (in)	CA-ID-2	CA-ID-3	CA-ID-4	CA-ID-5
Transverse Cracking (%)	6	120	96	122	121
Transverse Cracking (%)	7	88.7	19	118	96
Transverse Cracking (%)	8	13.04	4	69	20
Transverse Cracking (%)	9	3.13	0.9	11	4
Transverse Cracking (%)	10	1.92	0.9	4	0.9
Transverse Cracking (%)	11	0.96	0.9	2	0.9
Transverse Cracking (%)	12	0.96	0.9	0.9	0.9
Joint Faulting (in)	6	0.11	0.04	0.07	0.04
Joint Faulting (in)	7	0.13	0.06	0.1	0.06
Joint Faulting (in)	8	0.14	0.07	0.11	0.07

	Slab Thickness (in)	CA-ID-2	CA-ID-3	CA-ID-4	CA-ID-5
Joint Faulting (in)	9	0.14	0.07	0.12	0.08
Joint Faulting (in)	10	0.14	0.07	0.12	0.09
Joint Faulting (in)	11	0.14	0.08	0.13	0.09
Joint Faulting (in)	12	0.16	0.09	0.15	0.11

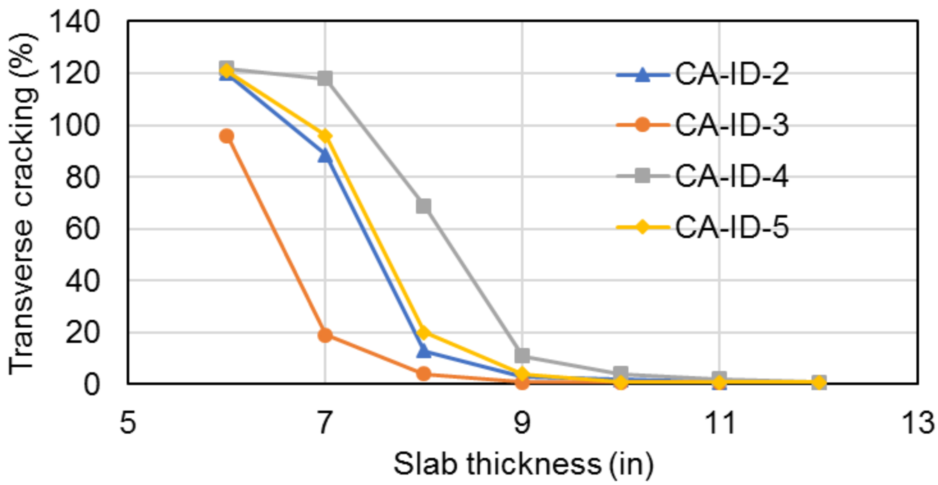

Figure 7.9. Impact of slab thickness on transverse cracking

The comparison of slab thickness vis-à-vis joint faulting for all the paving mixes is shown in Figure 7.10. It is evident from these results that all the mixes show a similar trend that joint faulting increases with increase in slab thickness. This increase in faulting is due to the dowel size as the dowel size was kept constant for all the simulations to obtain the effects of slab thickness on the joint faulting. By increasing the dowel size, faulting distress can be minimized.

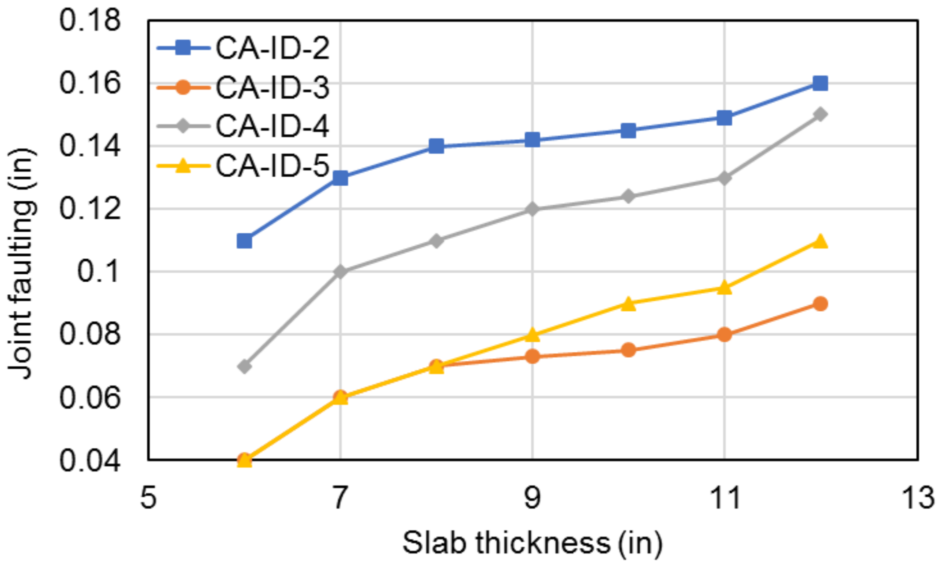

Figure 7.10. Impact of slab thickness on joint faulting

7.5 IMPACT OF CTE INPUT ON JPCP DESIGN

The simulations were conducted in pavement ME design for CA-ID 2, 3 & 4 to contrast the impact of CTE on JPCP performance. The concrete strength properties including MOR and elastic modulus were kept constant for each mix to observe the effects of CTE only. The comparative results of the three mixes are tabulated in Table 7.6. For CA-ID-2 (granite aggregate), the tested CTE value was 5.4 E-6 in/in/°F and ME default value was 4.8 E-6 in/in/°F. For CA-ID-3 (limestone aggregate), the lab value was 3.71 E-6 in/in/°F and ME default value was 4.4 E-6 in/in/°F. For CA-ID-4 (Quartzite aggregate), the lab value was 5.09 E-6 in/in/°F and ME default value was 5.2 E-6 in/in/°F.

Table 7.6. Simulation Results of CTE Impact on Pavement Performance

Performance Prediction	Lab CTE (E-6 in/in/°F)	ME Default CTE (E-6 in/in/°F)
CA-ID-2	5.4	4.8
Transverse Cracking (% slabs)	24.7	8.14
Joint Faulting (in.)	0.13	0.11
IRI (in/mile)	171.14	144.73
CA-ID-3	3.71	4.4
Transverse Cracking (% slabs)	4.2	9.3
Joint Faulting (in.)	0.07	0.1

Performance Prediction	Lab CTE (E-6 in/in/°F)	ME Default CTE (E-6 in/in/°F)
IRI (in/mile)	119.8	136.1
CA-ID-4	5.09	5.2
Transverse Cracking (% slabs)	18.8	23.5
Joint Faulting (in.)	0.12	0.12
IRI (in/mile)	158.3	164.79

7.5.1 Analysis of Simulation Results

The analysis of simulation results was conducted to quantify the effects of CTE variation on pavement performance indicators.

7.5.1.1 Effects on Transverse Cracking

The comparison for transverse cracking is presented in Figure 7.11, which shows that there is significant variation in transverse cracking between the tested CTE values and the default CTE values. The primary indication is that higher CTE value leads to higher transverse cracking. With these results, it is evident that the pavement must be designed with the accurately tested CTE value for the paving mix to be used so that the designed pavement can last for the entire service life. The ME default CTE data will not produce an accurate design for NMDOT paving mixes.

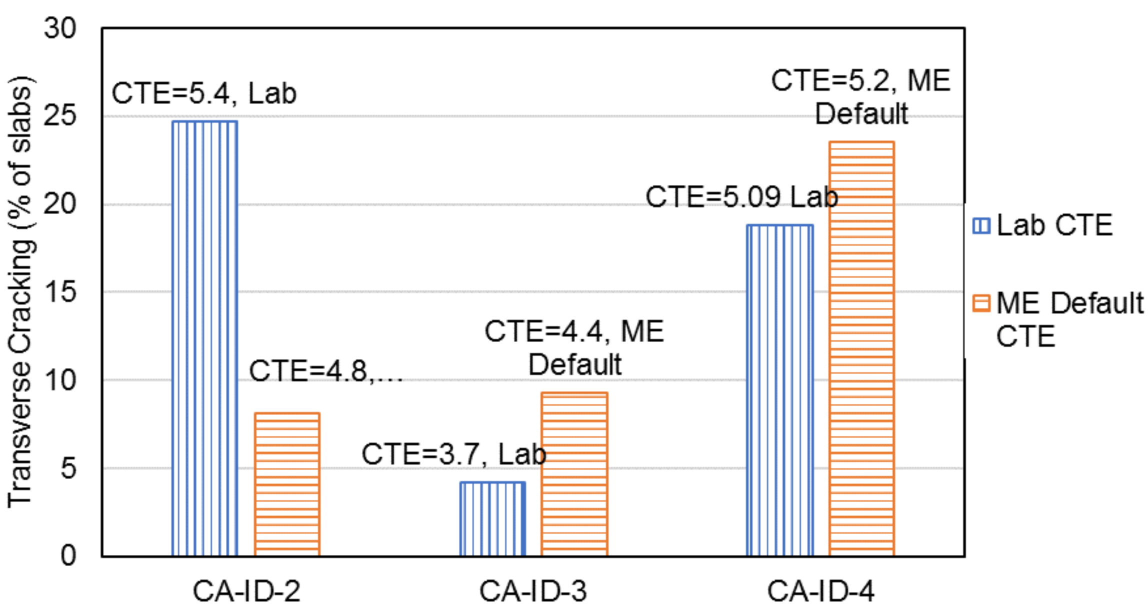

Figure 7.11. Impact of CTE Test Results on Transverse Cracking

7.5.1.2 Effects on Joint Faulting

The comparative summary for joint faulting is presented in Figure 7.12. It is evident that there is a significant impact on joint faulting between the tested CTE values and the default CTE values. Apparently higher CTE value leads to higher joint faulting. With these results, it is evident that the pavement must be designed with the accurately tested CTE value so that the designed pavement can last for the entire service life. The ME default CTE data may not produce an accurate design for NMDOT paving mixes and there is a need to generate a database of CTE results for NMDOT paving mixes to obtain precision in pavement design and performance predictions.

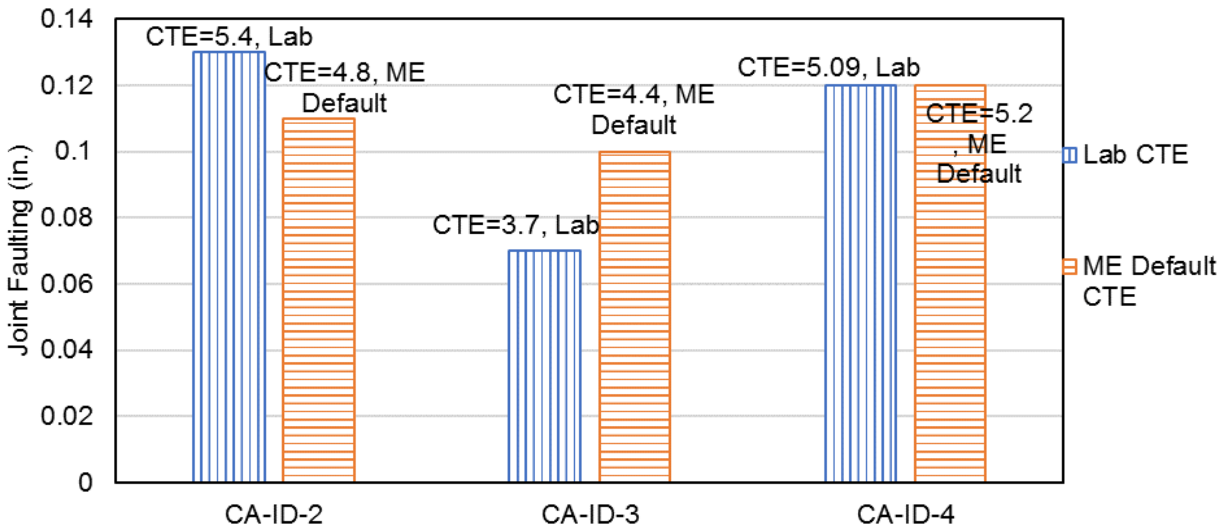

Figure 7.12. Impact of CTE Test Results on Joint Faulting

7.5.1.3 Effects on Pavement Roughness

The comparison is presented in Figure 7.13, which shows that there is significant variation in IRI between the tested CTE values and the default CTE values. Higher CTE value leads to higher pavement roughness. It is evident that the accurately tested CTE values are essential for accurate pavement design. The ME default CTE data will not produce an accurate design for NMDOT mixes. These results also confirm the requirement of generating a database of CTE values for the paving mixes to be used in NM.

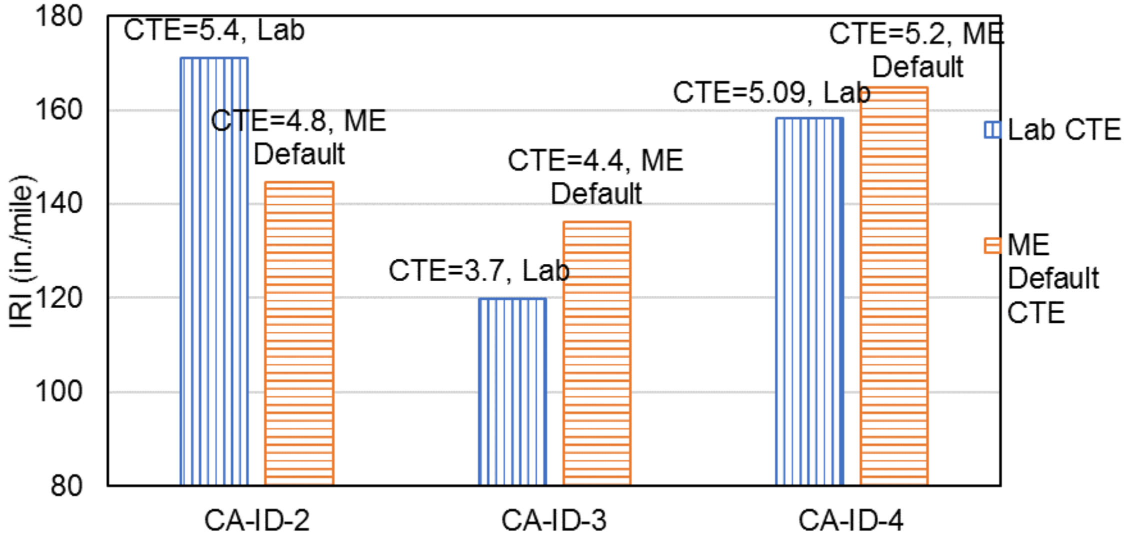

Figure 7.13. Impact of CTE Test Results on IRI

7.6 JPCP DESIGN WITH INPUTS FROM TEST DATA, DEVELOPED MODELS AND ME DESIGN DEFAULT MODELS

The level-1 inputs of concrete strength (MOR and elastic modulus) comprise the actual lab test data of the specific paving mix whereas level-2 inputs can be obtained from the ME design default models or the local calibrated models for NMDOT mixes. Out of these, level-1 inputs produce the most accurate design. The simulations were conducted in pavement ME design to contrast the effects of these three types of inputs of MOR and elastic modulus data with the design parameters presented earlier in Table 10 while keeping the CTE constant for each specific mix to analyze the effects of MOR & elastic modulus data on pavement performance indicators. The summary of performance predictions for the four paving mixes is tabulated in Table 7.7.

Table 7.7. Summary of Simulation Results of Impact of MOR & Elastic Modulus Data on Pavement Performance

Performance Prediction	Level-1	Level-2 (Power Model)	Level-2 (ME Default Model)
CA-ID-1			
Transverse Cracking (% slabs)	1.92	0.96	1.92
Joint Faulting (in.)	0.13	0.13	0.13
IRI (in/mile)	152.1	152.2	152.34
CA-ID-2			
Transverse Cracking (% slabs)	1.92	2.5	3.9

Performance Prediction	Level-1	Level-2 (Power Model)	Level-2 (ME Default Model)
Joint Faulting (in.)	0.13	0.13	0.13
IRI (in/mile)	156.3	156.4	155.9
CA-ID-3			
Transverse Cracking (% slabs)	0.96	0.96	0.96
Joint Faulting (in.)	0.07	0.07	0.07
IRI (in/mile)	119.3	119.5	119.3
CA-ID-4			
Transverse Cracking (% slabs)	3.68	3.21	6.09
Joint Faulting (in.)	0.12	0.12	0.12
IRI (in/mile)	148.8	148.9	149

7.6.1 Analysis of Simulation Results

The simulation results were analyzed by comparing the performance predictions obtained from the three sets of MOR and elastic modulus inputs for each of the four mixes. The comparison is displayed in Figures 7.14, 7.15, and 7.16.

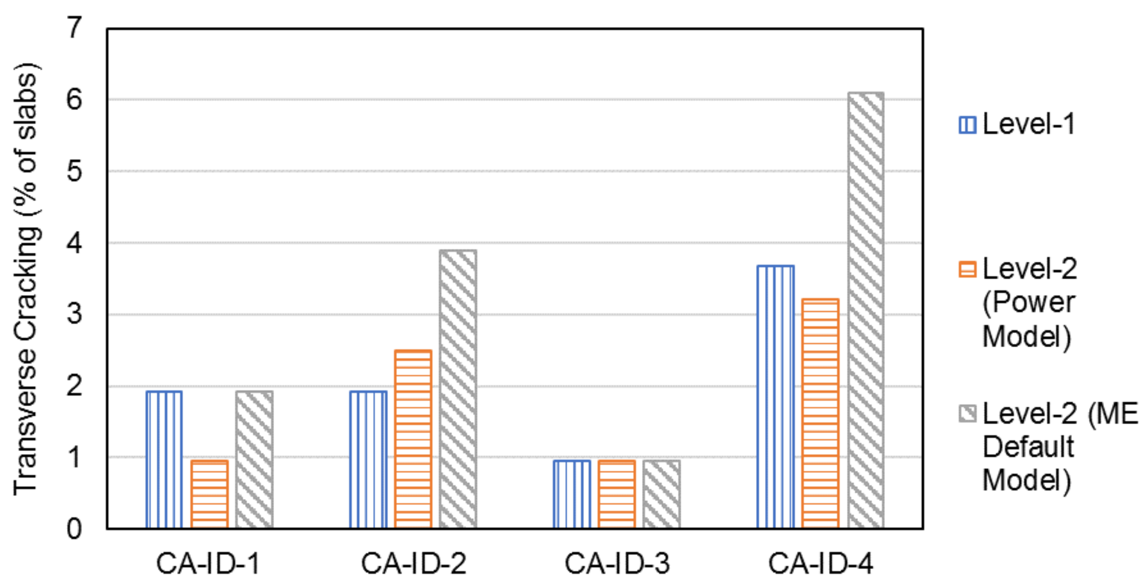

Figure 7.14. Impact of Test Data on Transverse Cracking

It is evident from the comparison that the level of input data has a significant effect on the transverse cracking performance of the simulated pavement. The results of the mixes CA-ID-2 and CA-ID-4 indicate that the predicted transverse cracking with the input data from power model is much closer to the prediction with the level-1 inputs as compared to the predictions with the ME default models. For CA-ID-1, which is a high-performance mix, the prediction of transverse cracking with level-1 inputs is similar to the prediction with the ME default models. For CA-ID-3, the performance prediction of all three input levels is similar. With this in view, it can be concluded that level-1 inputs are the best data for pavement design while the use of developed models (specifically for NMDOT mixes) works better than the ME default models for all the mixes except the mix CA-ID-1.

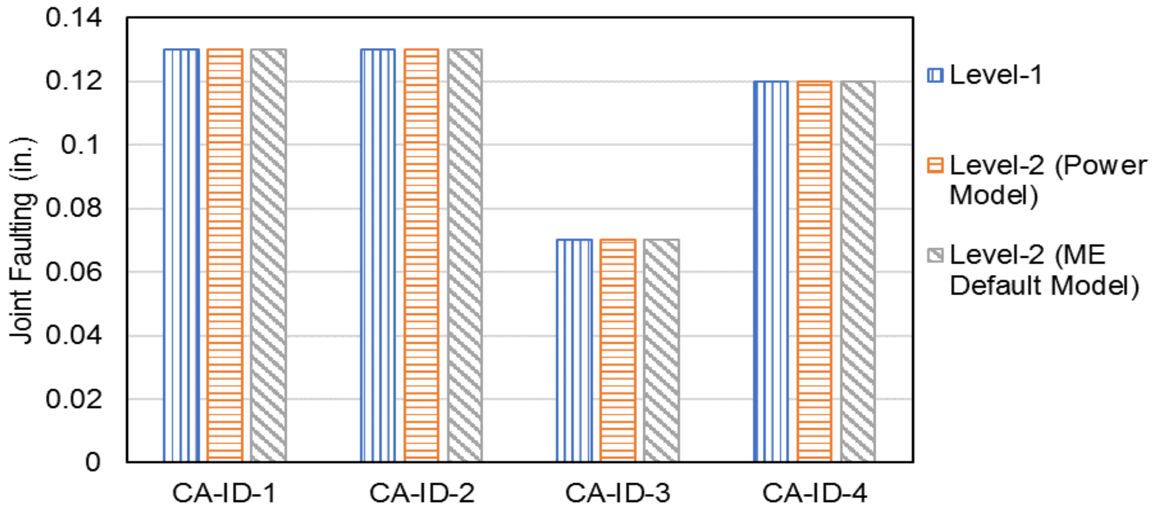

Figure 7.15. Impact of Test Data on Joint Faulting

With regards to joint faulting and pavement roughness, there is no significant difference between the performance predictions obtained with level-1 inputs, power models and ME default models.

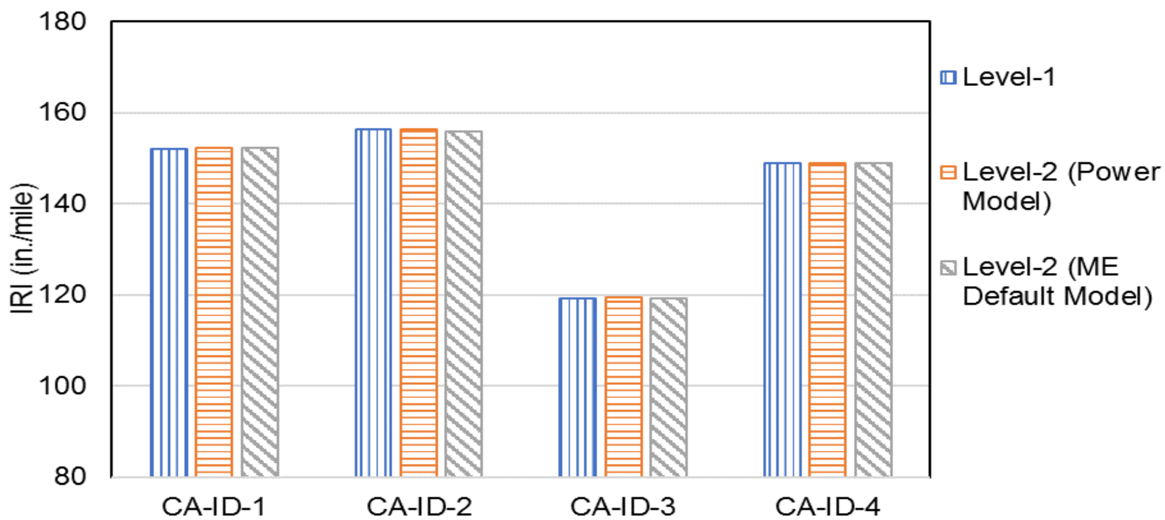

Figure 7.16. Impact of Test Data on Pavement Roughness

7.7 IMPACT OF TRAFFIC VOLUME ALONG WITH INPUT LEVELS ON JPCP PERFORMANCE

The simulations were conducted in pavement ME design for CA-ID 2, 3 & 6 to contrast the impact of various traffic volumes along with level 1 and level-3 inputs on JPCP performance. The lab tested data for concrete strength properties including MOR and elastic modulus and CTE were used for level-1 design while for level-3 design default CTE value and compressive strength input was used. Traffic volume ranged from AADTT of 2000 to 8000. JPCP thickness was 10 in and dowel diameter was 1.25 in. Other design parameters were kept constant to compare the effects of traffic and input levels. The comparative results are tabulated in Table 7.8.

Table 7.8. Simulation Results of Impact of Traffic and Input Levels on Pavement Performance

CA-ID	AADTT	Level-1 Transverse Cracking (%)	Level-1 Joint Faulting (in)	Level-3 Transverse Cracking (%)	Level-3 Joint Faulting (in)
2	2000	0.96	0.24	19.38	0.28
2	3000	0.96	0.34	32.39	0.39
2	4000	1.92	0.44	46.11	0.5
2	5000	1.92	0.53	58.97	0.6
2	6000	1.92	0.63	70.16	0.71
2	7000	2.18	0.72	79.5	0.81
2	8000	2.37	0.81	87.09	0.92
3	2000	2.37	0.19	14.41	0.25
3	3000	2.76	0.27	23.64	0.35
3	4000	3.21	0.36	33.98	0.44
3	5000	3.68	0.44	44.73	0.54
3	6000	4.14	0.52	54.93	0.64
3	7000	4.61	0.6	64.2	0.74
3	8000	5.09	0.68	72.33	0.83
6	2000	2.96	0.24	16.68	0.25
6	3000	3.73	0.34	27.67	0.35
6	4000	4.53	0.44	39.7	0.45
6	5000	5.32	0.53	51.53	0.54
6	6000	6.14	0.63	62.34	0.64
6	7000	7.01	0.73	71.74	0.74
6	8000	8.04	0.82	79.68	0.83

7.7.1 Analysis of Simulation Results

The analysis of simulation results was conducted to quantify the effects of traffic volume and input levels variation on pavement performance indicators.

7.7.1.1 Effects on Transverse Cracking

The comparison for terminal values of transverse cracking for CA-ID-2, 3 and 6 is presented in Figure 7.17, 7.18 and 7.19, which shows that there is significant variation in transverse cracking between the results of level 1 and level 3 inputs for all the mixes and all traffic volumes. The variation in transverse cracking with input levels is up to 85% which is highly significant. With these results, it is evident that the pavement must be designed with the accurately tested level-1 inputs for the paving mix to be used so that the designed pavement can last for the entire service life. The ME default CTE data and level-3 inputs will not produce an accurate design for NMDOT paving mixes. The increase in traffic volume also impacts the terminal transverse cracking of JPCP and with level-1 inputs, the increase in cracking ranges up to 5% with an increase in AADTT from 2000 to 8000. This increase is not significant when compared with the impact of material properties input levels on cracking.

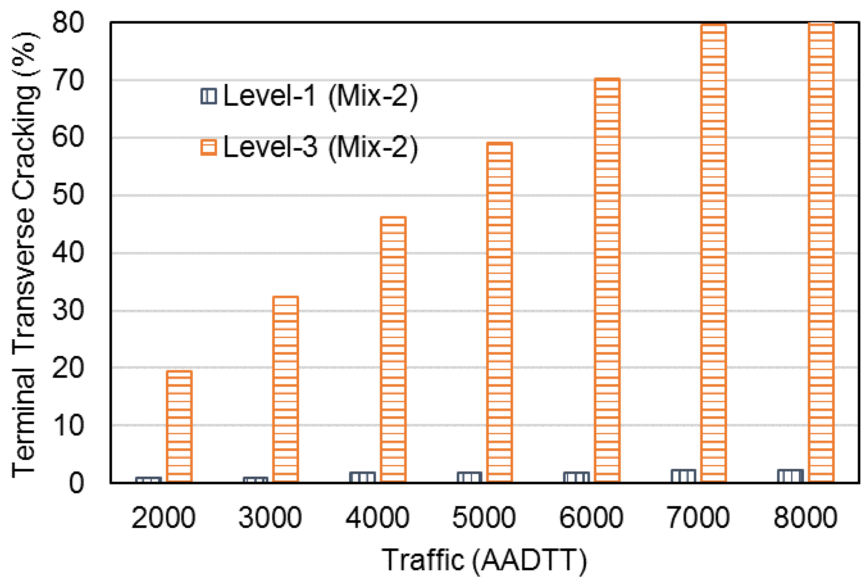

Figure 7.17. Impact of Traffic volume and Input Levels on Transverse Cracking for CA-ID-2

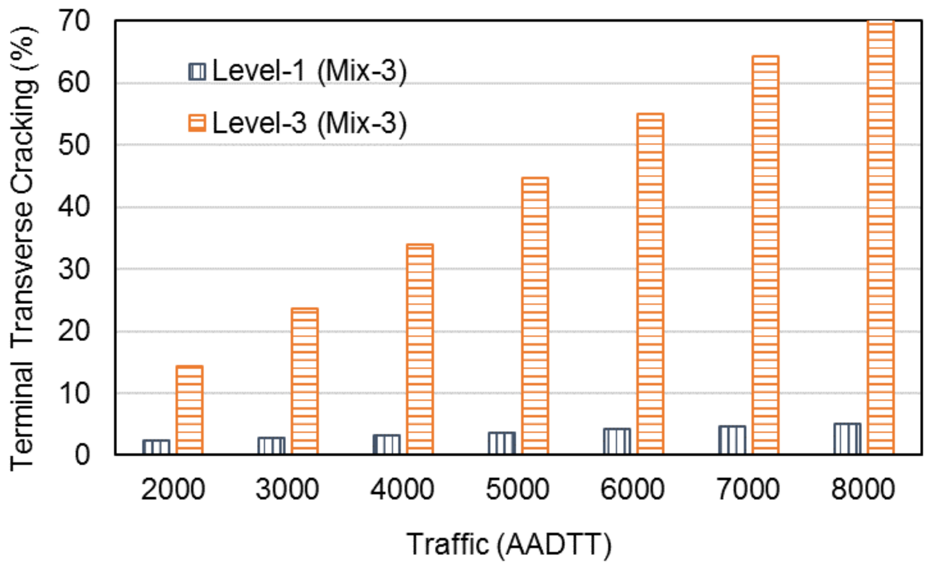

Figure 7.18. Impact of Traffic volume and Input Levels on Transverse Cracking for CA-ID-3

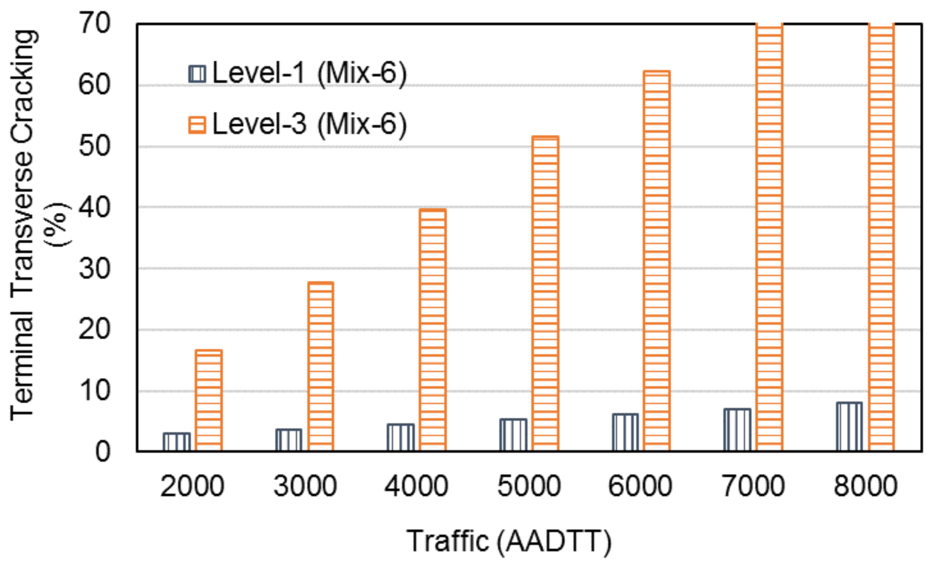

Figure 7.19. Impact of Traffic volume and Input Levels on Transverse Cracking for CA-ID-6

7.7.1.2 Effects on Joint Faulting

The comparative summary for terminal joint faulting for the impact of material inputs and traffic volume is presented in Figures 7.20, 7.21, and 7.22. It is evident that there is a significant impact on joint faulting between the two material input levels. The difference in terminal joint faulting values ranges between 0.01 to 0.15 in. Also, there is a significant impact of traffic volume increase on joint faulting ranging up to 0.58 in with level-1 material inputs. This shows the importance of using accurate concrete material inputs and traffic volume to be used for JPCP design.

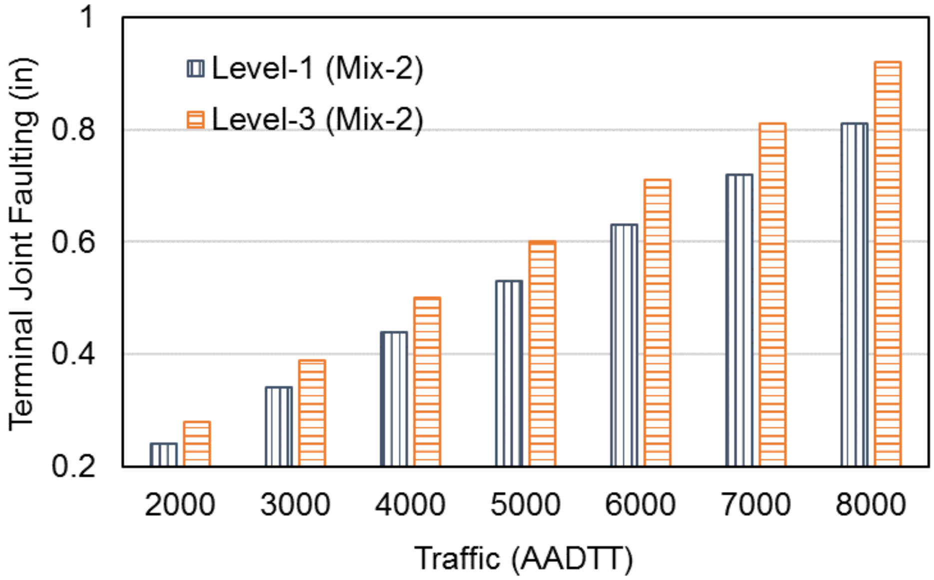

Figure 7.20. Impact of Traffic volume and Input Levels on Joint Faulting for CA-ID-2

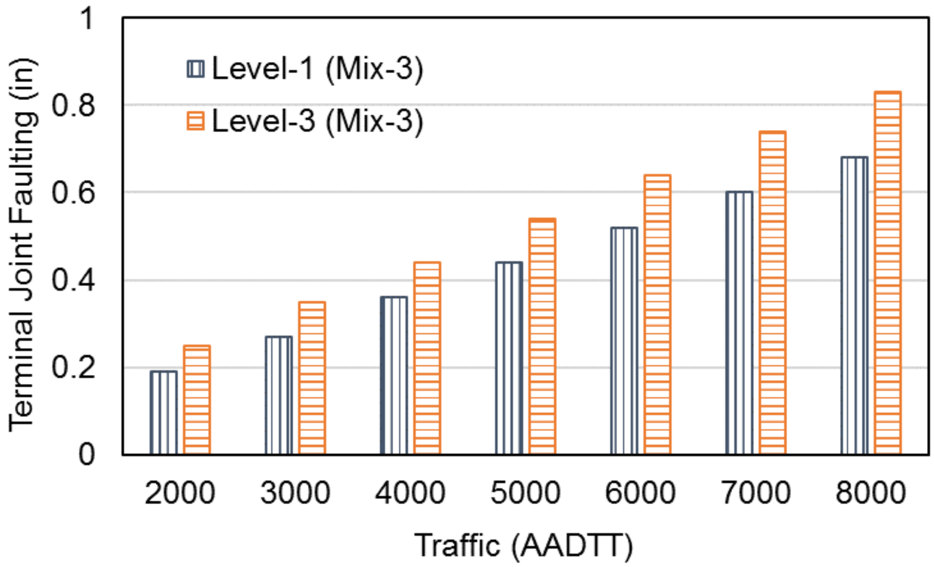

Figure 7.21. Impact of Traffic volume and Input Levels on Joint Faulting for CA-ID-3

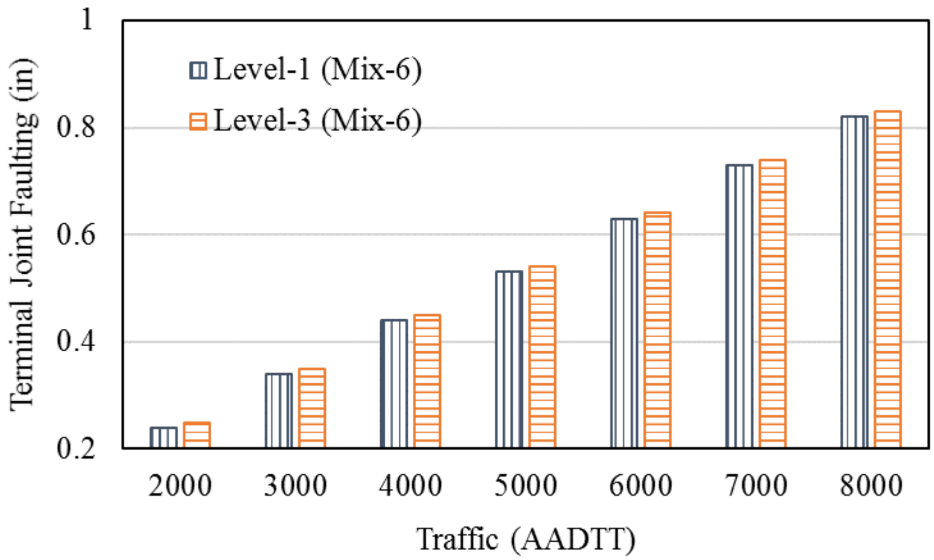

Figure 7.22. Impact of Traffic volume and Input Levels on Joint Faulting for CA-ID-6

7.8 EFFECTS OF DOWEL SIZE ON JPCP DESIGN AND PERFORMANCE

The effects of dowel size on performance parameters were evaluated and the results are presented in Table 7.9. The design thickness was kept constant at 8 in for these simulations. The dowel size was varied from 1 to 1.5 in and the results show that there is no effect on transverse cracking while there is a significant effect on joint faulting ranging between 0.32 to 0.41 in. The comparison of results is shown in Figures 7.23 and 7.24. The dowel bars have an important role in joint load transfer and minimizing the effects of thermal curling thus with an increase in dowel size, joint faulting can be curtailed.

Table 7.9. Summary of Effects of Dowel Size on Pavement Performance

Mix Type	Dowel Size (in)	Transverse Cracking %)	Joint Faulting (in)
2	1	1.92	0.46
2	1.25	1.92	0.13
2	1.5	1.92	0.07
3	1	3.21	0.36
3	1.25	3.21	0.06
3	1.5	3.21	0.04
6	1	4.53	0.44
6	1.25	4.53	0.07
6	1.5	4.53	0.05

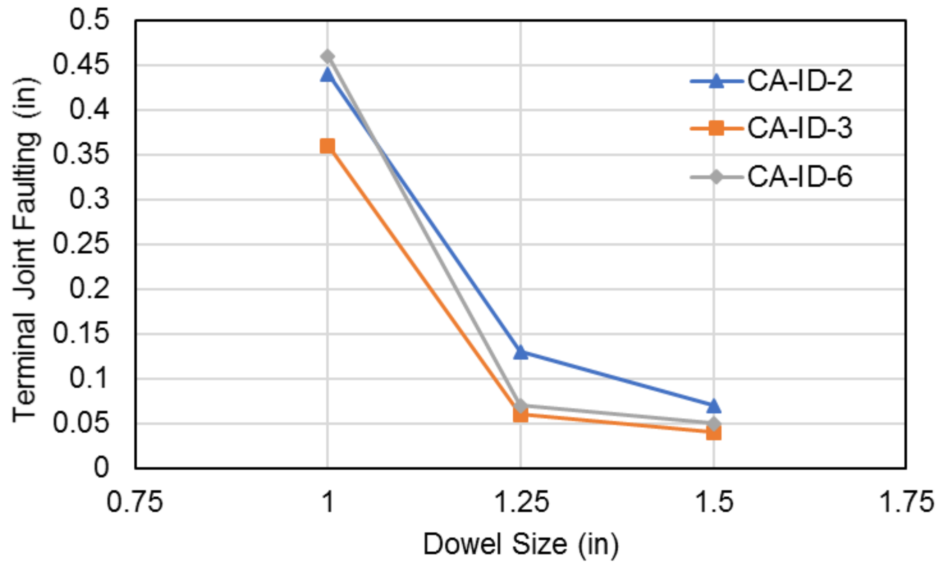

Figure 7.23. Impact of Dowel Size on Joint Faulting

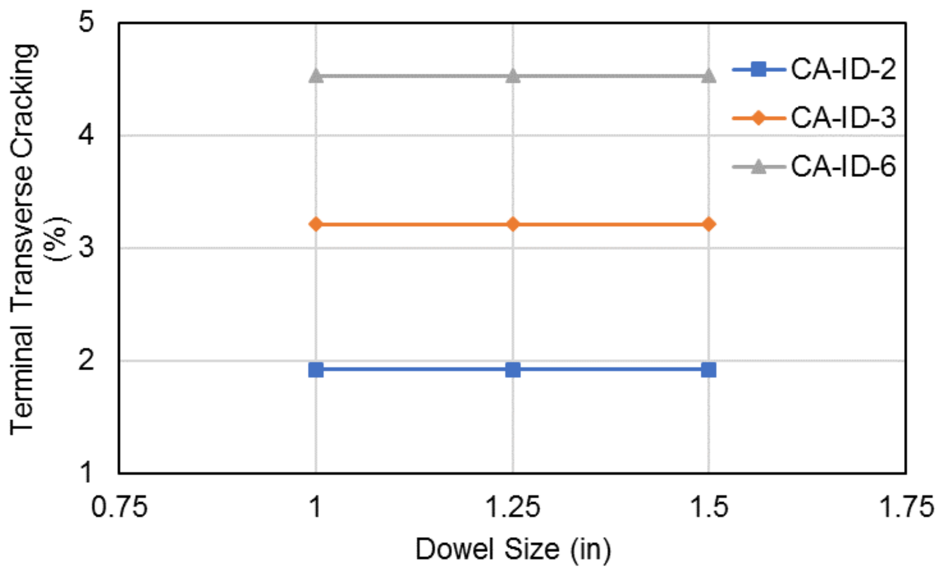

Figure 7.24. Impact of Dowel Size on Transverse Cracking

7.9 EFFECTS OF CLIMATIC CONDITIONS ON PAVEMENT PERFORMANCE

The effects of various climatic conditions on pavement performance were evaluated by conducting simulations with level-1 inputs for all the paving mixes and with 5 different climates for each mix (representing 5 districts of NM). The slab thickness was kept constant at 10 in and the dowel size was kept as 1.25 in. The summary of climate data of these districts is given in Table 7.10, which shows that there is not much difference between the mean annual air temperature between these climates but there is a significant difference between the freezing index and the freeze-thaw cycles for some of the districts. The summary of results is presented in Table 7.11. The results show that there is a significant impact of climatic conditions on transverse cracking and joint faulting.

Table 7.10. Summary of Effects of Climatic Conditions on Pavement Performance

Climate Details	Albuquerque	Santa Fe	Roswell	Deming	Las Vegas
Mean Annual Air Temperature (F)	58.18	52.41	61.57	62.15	50.47
Freezing Index (F - Days)	65.06	259.16	57.21	27.64	329.77
Annual Freeze Thaw Cycles	80.46	144.41	75.58	74.68	145.44

Table 7.11. Summary of Effects of Climatic Conditions on Pavement Performance

	Albuquerque	Albuquerque	Santa Fe	Santa Fe	Roswell	Roswell	Deming	Deming	Las Vegas	Las Vegas
Mix ID	Cracking	Faulting	Cracking	Faulting	Cracking	Faulting	Cracking	Faulting	Cracking	Faulting
2	1.92	0.13	3.62	0.22	3.95	0.15	3.95	0.13	3.36	0.21
3	3.21	0.06	5.6	0.1	6.76	0.07	6.59	0.06	6.49	0.11
4	59.04	0.1	107.9	0.18	116.01	0.12	116.17	0.1	112.4	0.18
5	14.98	0.07	40.75	0.12	54.62	0.08	52.98	0.07	50.09	0.12
6	4.53	0.07	8.81	0.12	11.54	0.08	11.07	0.07	10.62	0.13

7.9.1 Comparison of Effects of Climatic Conditions

The comparison of the effects of climatic conditions on transverse cracking and joint faulting are shown in Figures 7.25 to 7.28 for CA-ID-2 and CA-ID-4 respectively. The results show that climatic conditions have a significant impact on both the performance parameters. As the freezing index and freeze-thaw cycles increases, the performance of JPCP decreases with increase in cracking and joint faulting. With these results, it is evident that JPCP design be performed with the specific climate conditions for any specific project site.

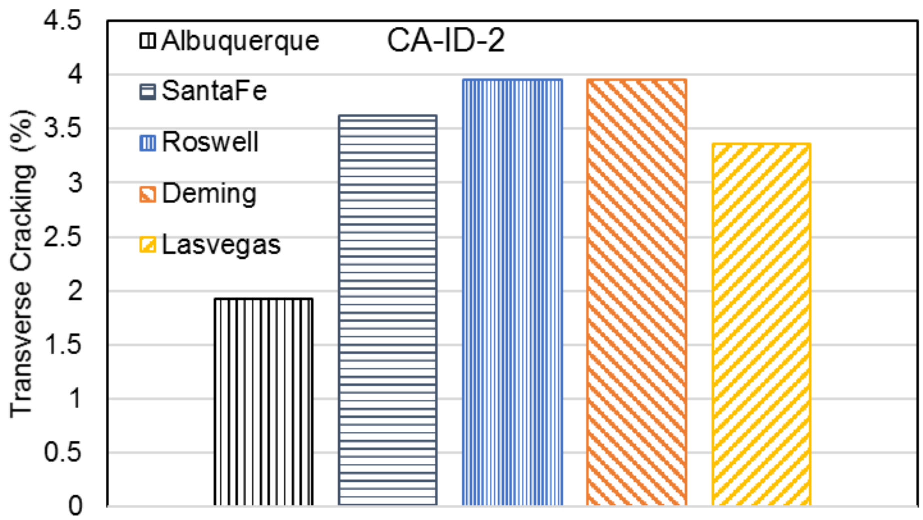

Figure 7.25. Impact of Climatic Conditions on Transverse Cracking for CA-ID-2

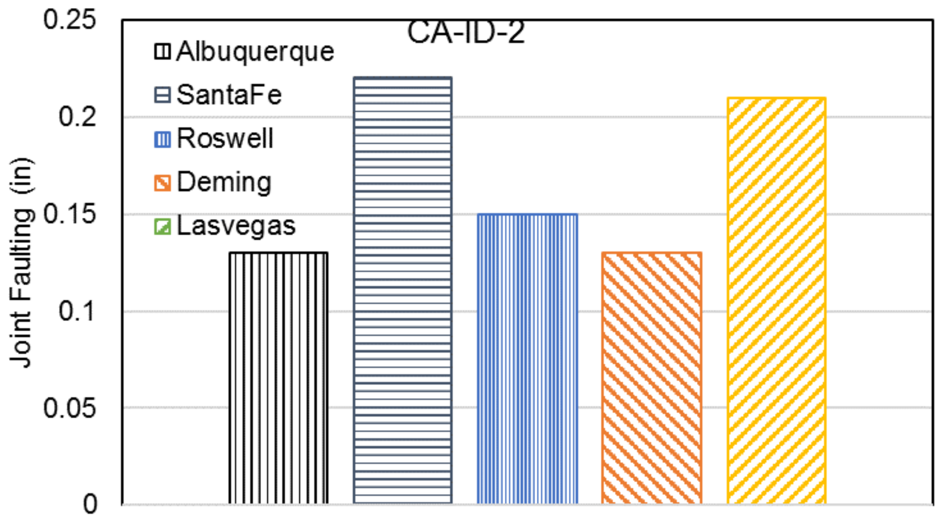

Figure 7.26. Impact of Climatic Conditions on Joint Faulting for CA-ID-2

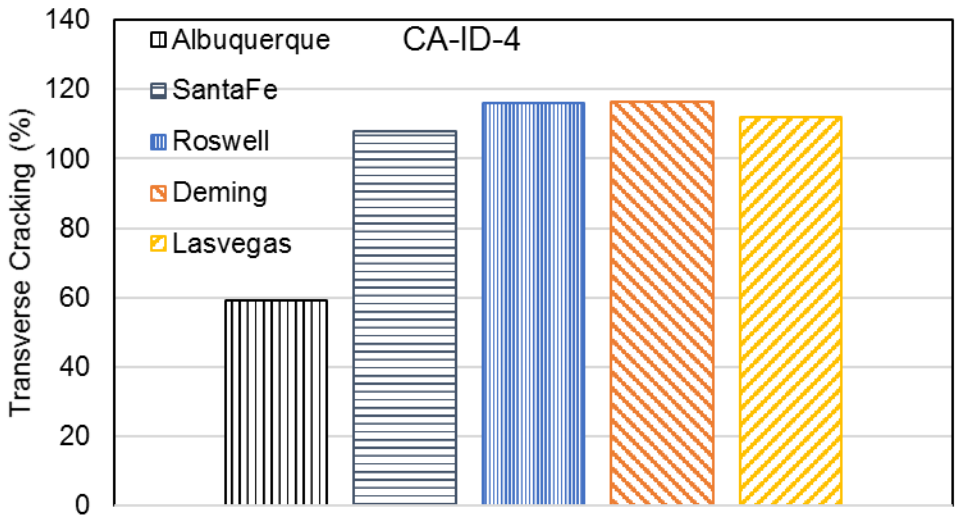

Figure 7.27. Impact of Climatic Conditions on Transverse Cracking for CA-ID-4

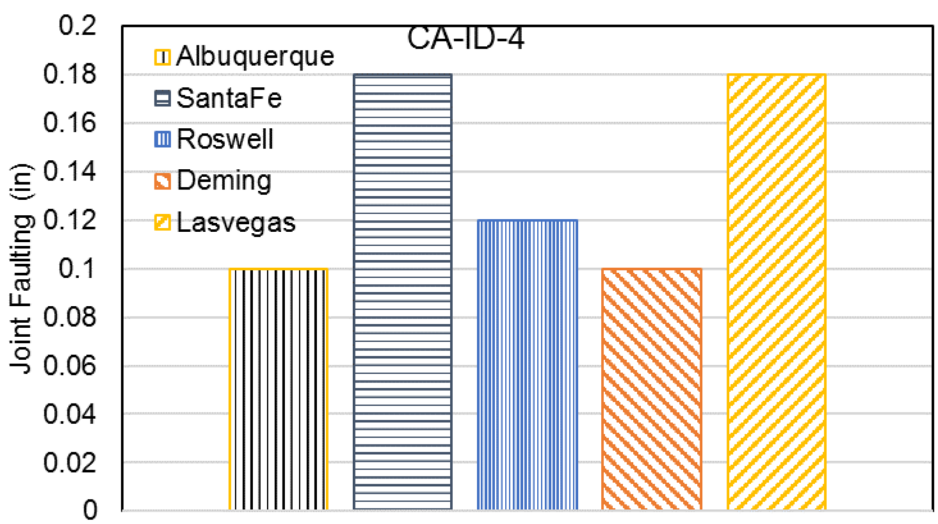

Figure 7.28. Impact of Climatic Conditions on Joint Faulting for CA-ID-4

7.10 IMPACT OF INPUT LEVELS ON CRCP PERFORMANCE

The simulations were conducted in pavement ME design for all the paving mixes (CA-ID 1 to CA-ID-6) to contrast the impact of level 1 and level-3 inputs on CRCP performance. The lab tested data for concrete mechanical properties including MOR and elastic modulus and CTE were used for level-1 design simulations while, for level-3 design, default CTE value and compressive strength input were used. CRCP thickness was 10 in and other design parameters were kept constant (as per Table 7.12) to compare the effects of input levels. The comparative results are tabulated in Table 7.13.

Table 7.12. CRCP Design Parameters for Simulation Work

Parameter	Value
Design Life	30 years
Design Thickness	10 in.
Shoulders	Tied PCC Shoulders
Steel Reinforcement (%)	0.72
Steel Bar Diameter	3/4 in. (#6 bar)
Steel Depth	3.5 in.
Initial IRI	63 in/mile
Threshold IRI	172 in/mile
Threshold Punch-outs	10 per mile
Reliability	90%
Modulus of Rupture of Concrete	As per CA-ID
Elastic Modulus of Concrete (28 days)	As per CA-ID
Poisson's Ratio	0.2
Climate Station	Albuquerque, New Mexico
AADTT	4000
Traffic ESALS	$30x10^6$
Base Course Thickness	6 in.
Base Course Resilient Modulus	40,000 psi

Table 7.13. Simulation Results of Impact of Input Levels on CRCP Performance

Mix ID	IRI Level-1	IRI Level-3	Punch outs Level-1	Punch outs Level-3
CA-ID-1	87.8	88.2	1	1
CA-ID-2	87.9	126.4	1	23.6
CA-ID-3	91.4	130.5	4.14	25.6
CA-ID-4	177.4	244.1	48.9	81.7
CA-ID-5	148.1	178.6	34.43	49.5
CA-ID-6	100.7	137.2	10	29.1

7.10.1 Analysis of Simulation Results

The analysis of simulation results was conducted to quantify the effects of input levels variation on pavement performance indicators.

7.10.1.1 Effects on Pavement Roughness/IRI

The comparison for terminal values of transverse cracking for CA-ID-1 to CA-ID-6 is presented in Figure 7.29, which shows that there is significant variation in IRI values between the results of level 1 and level 3 inputs for all the mixes. The variation in IRI with input levels ranges from 1 to 66.7 in/mile which is highly significant. With these results, it is evident that the CRCP must be designed with the accurately tested level-1 inputs for the paving mix to be used so that the designed pavement can last for the entire service life. The ME default CTE data and level-3 inputs will not produce an accurate design for NMDOT paving mixes.

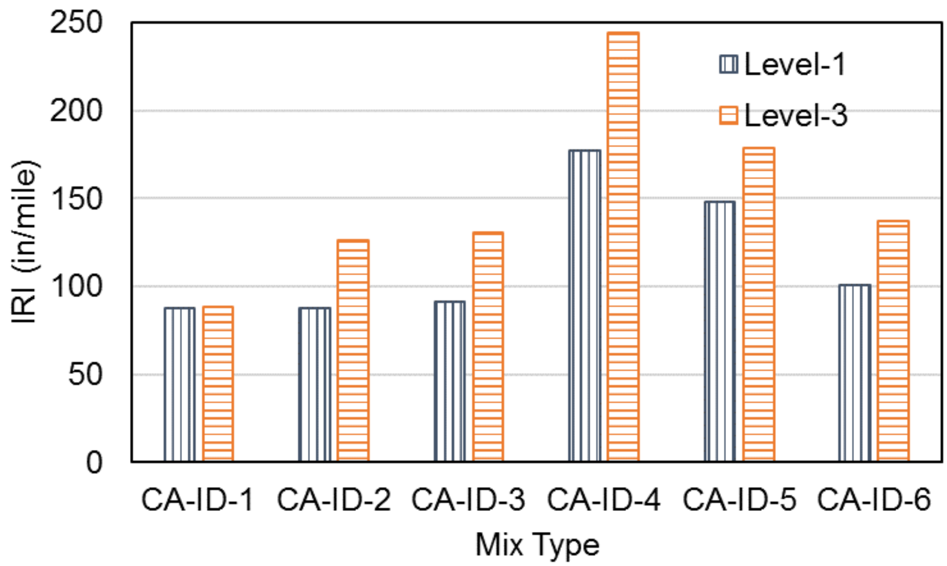

Figure 7.29. Impact of Input Levels on IRI

7.10.1.2 Effects on Punch Outs

The comparative summary for CRCP punch outs for the impact of material input levels is presented in Figure 7.30. It is evident that there is a significant impact on punch outs between the two material input levels. The difference in terminal punch out values ranges between 15.1 to 32.8 per mile. This shows the importance of using accurate concrete material inputs to be used for CRCP design.

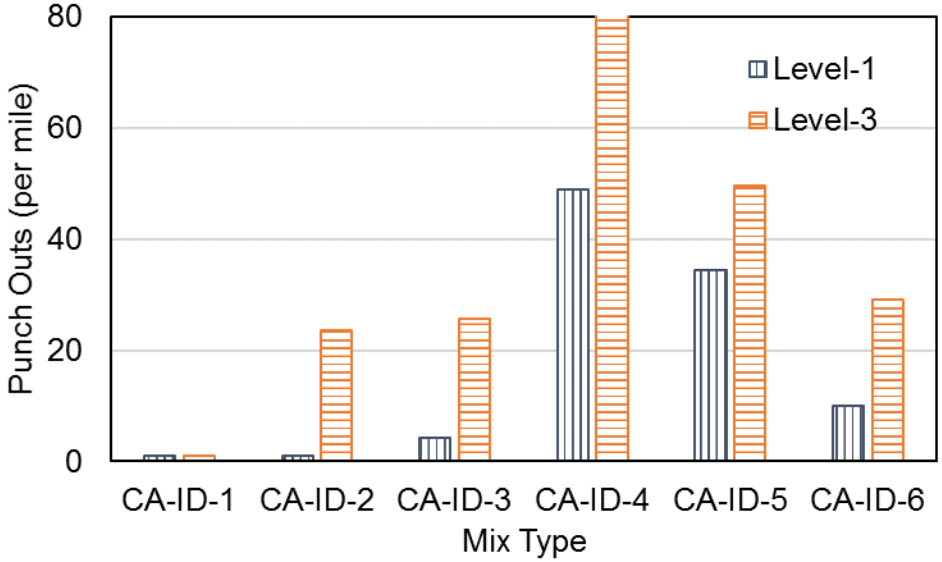

Figure 7.30. Impact of Input Levels on CRCP Punch Outs

7.10.1.3 Percent Change in CRCP Performance with Level-1 & Level-3 Inputs

The simulation results were analyzed to compare the percentage change in performance parameters including IRI and punch outs with level-1 and level-3 inputs. The results are presented in Table 7.14 and Figure 7.31.

Table 7.14. % age Change in CRCP Performance with Level-1 and Level-3 Inputs

Mix Type	% Change in IRI between Level-1 and Level-3	% Change in Punch Outs between Level-1 and Level-3
CA-ID-1	1.0	1.0
CA-ID-2	43.8	95.8
CA-ID-3	42.8	83.8
CA-ID-4	37.6	40.1
CA-ID-5	20.6	30.4
CA-ID-6	36.2	65.6

The results show that the %age change in IRI ranges from 1 to 43.8% while the percentage change in punch outs ranges from 1 to 95.8%. It indicates that the impact of material input levels is more significant on the CRCP punch outs then the pavement roughness. The CRCP designed with level-3 material inputs may be under-designed or over-designed and to obtain accurate CRCP design, level-1 material inputs should be used.

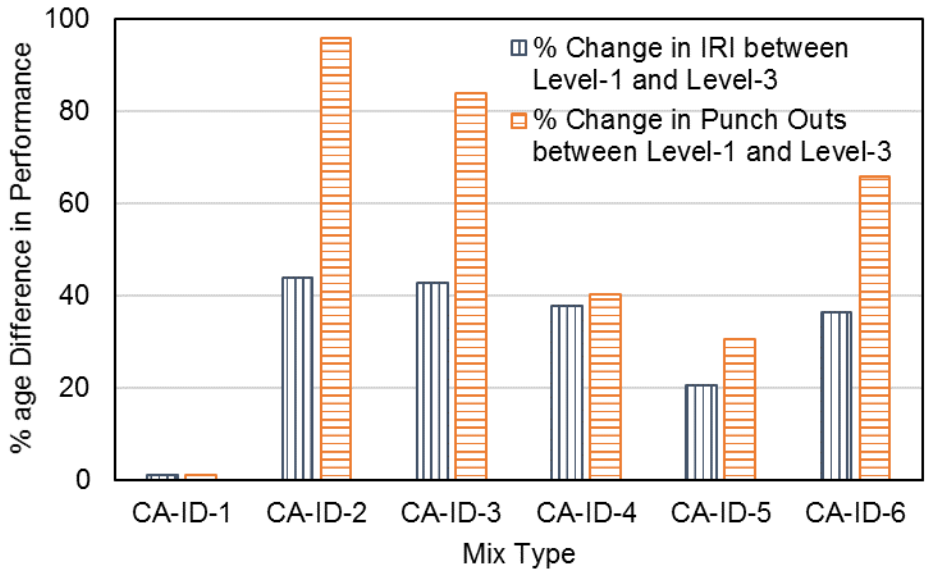

Figure 7.31. Percent Change in CRCP Performance Between Level1 and Level-3 Inputs

7.11 EFFECTS CTE VARIATION ON CRCP PERFORMANCE

The effects of CTE variation was evaluated on the design and performance of CRCP by conducting simulations in pavement ME design software. The CTE of paving concrete was varied from 3.5 to 6.5 µƐ/˚F while other design factors were kept as constant including the concrete mechanical properties. MOR was kept constant at 690 psi and elastic modulus was kept at 4.2 E6 psi. Other design factors are kept the same as shown in Table 36. The simulation results are presented in Table 7.15.

Table 7.15. Summary of Effects of CTE on CRCP Performance

CTE (µƐ/˚F)	IRI (in/mile)	PO (per mile)
3.5	93.9	5.9
4	96.8	7.7
4.5	101.2	10.3
5	107.4	13.6
5.5	116.8	18.2
6	129.6	25.2

CTE (μƐ/˚F)	IRI (in/mile)	PO (per mile)
6.5	147.8	35.3

7.11.1 Effect of CTE Variation on IRI

The results of CRCP performance with regards to IRI are presented in Figure 7.32 which shows an increasing trend in pavement roughness with an increase in CTE of concrete. The IRI value increases from 93 to 148 in/mile with a change in CTE from 3.5 to 6.5 μƐ/˚F. This increase in IRI is not very significant in comparison to the punch outs as the highest value of IRI distress is less than the generally considered threshold value of 173 in/mile.

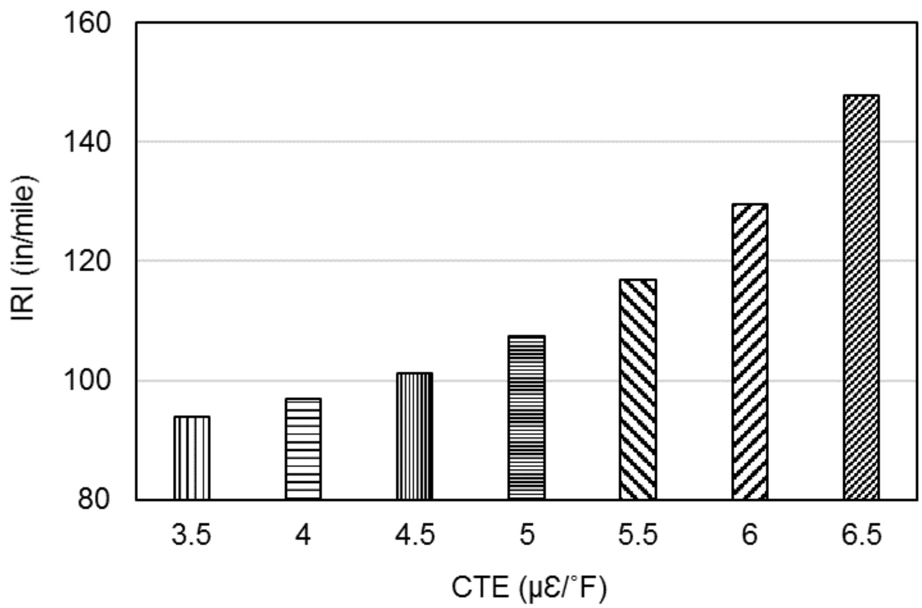

Figure 7.32. Impact of CTE Variation on IRI

7.11.2 Effect of CTE Variation on Punch Outs

The performance for various CTE values at the end of 30 years design life for punch outs are shown in Figure 7.33. It is evident that the magnitude of punch outs in the designed CRCP increases with increase in CTE of concrete. This increase ranges from 1.8 to 10.1 per mile with an increase in CTE from 3.5 to 6.5 μƐ/˚F and the threshold value of punch out is generally considered as 10 per mile. It shows that CTE of concrete has a significant effect on punch out the distress of CRCP. The punch out distress is dependent on cumulative fatigue damage due to loading and slab bending while slab bending is directly related to temperature curling of pavement slab which is a function of CTE value. Higher CTE values result in higher curling and bending and higher punch outs.

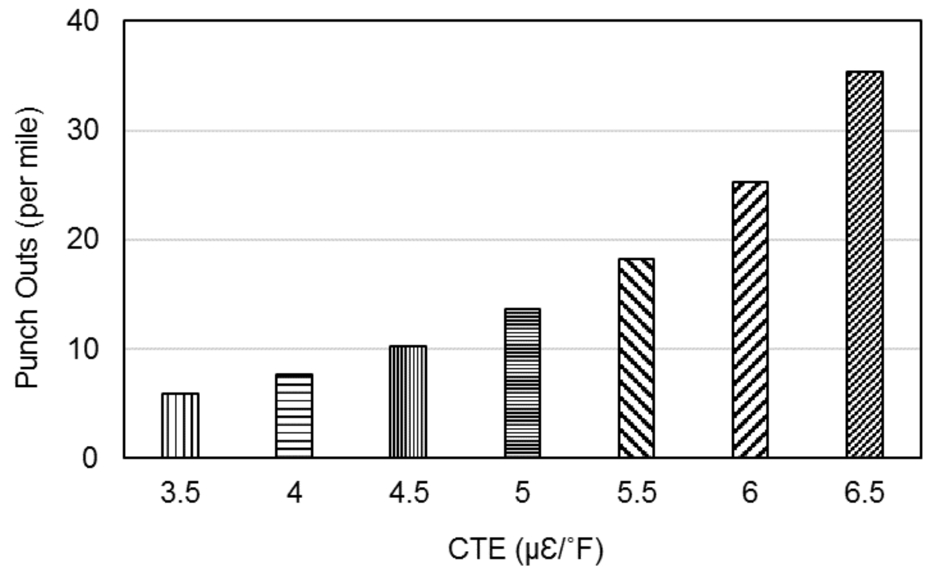

Figure 7.33. Impact of CTE Variation on Punch Outs

7.11.3 Percent Increase in CRCP Distresses

The comparison of CRCP distresses is conducted by determining the percent increase in punch out and IRI for various ranges of CTE increase. The results are shown in Figure 7.34. It is evident from these results that the impact of CTE increase on punch out is the most significant with % increase of 30 to 40% while the IRI is affected with a % increase of 3 to 14%. It is also pertinent to mention that each % increase in distress plotted in Figure 11 is for the difference in CTE of 0.5 µƐ/˚F but as the CTE value increases the percent increase in the distress increases which means that higher CTE values are detrimental for the CRCP performance.

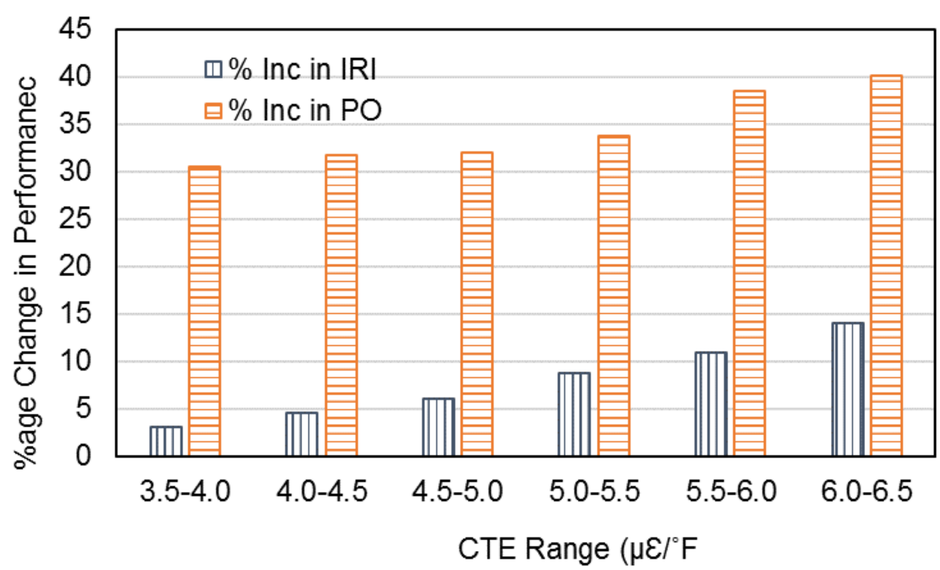

Figure 7.34. Percent Increase in CRCP distresses with a change in CTE

7.12 EFFECTS OF TRAFFIC VOLUME ON CRCP PERFORMANCE

The effects of traffic volume on CRCP performance were evaluated by conducting simulations with level-1 inputs for 2 paving mixes i.e. CA-ID-3 and CA-ID-6 and with varying traffic volumes ranging from AADTT of 2000 to 9000 (14.9 to 67.2 million ESALs) for each mix. The slab thickness was kept constant at 10 in. and other design factors were kept constant as shown in Table 7.12. The summary of results is presented in Table 7.16 and 7.17 for both the paving mixes.

Table 7.16. Summary of Effects of Traffic Volume on CRCP Performance for CA-ID-3

AADTT	ESALs (E6)	IRI (in/mile)	Punch Outs (per mile)
2000	14.9	87.8	0.05
3000	22.4	87.9	0.44
4000	29.8	91.4	4.14
5000	37.3	94.7	6.4
6000	44.8	97.8	8.3
7000	52.2	100.4	9.8
8000	59.7	102.6	11.1
9000	67.2	105.2	12.5

Table 7.17. Summary of Effects of Traffic Volume on CRCP Performance for CA-ID-6

AADTT	ESALs (E6)	IRI (in/mile)	Punch Outs (per mile)
2000	14.9	87.8	0.25
3000	22.4	91.9	4.5
4000	29.8	100.7	10
5000	37.3	106.7	13.4
6000	44.8	112.1	16.2
7000	52.2	117	18.9
8000	59.7	122.4	21.5
9000	67.2	126.4	23.6

7.12.1 Comparison of Effects of Traffic Volume for CA-ID-3

The comparison of the effects of traffic volume on punch outs and IRI are shown in Figures 7.35 and 7.36 for CA-ID-3. The results show that the traffic volume has a significant impact on both the performance parameters. As the traffic volume increases, the performance of CRCP decreases with increase in IRI and punch outs. With these results, it is evident that CRCP design be performed with the specific traffic volume for any specific project.

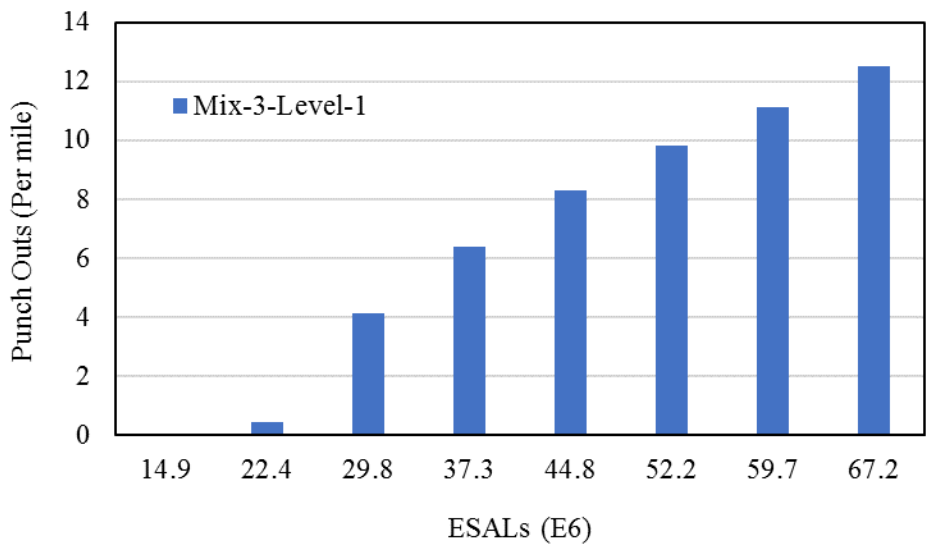

Figure 7.35. Impact of Traffic Volume on Punch Outs for CA-ID-3

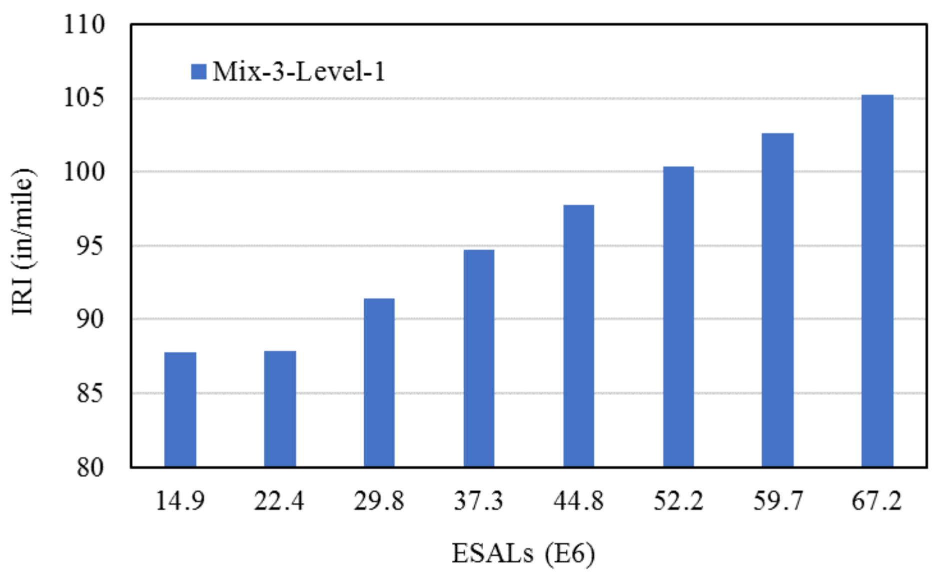

Figure 7.36. Impact of Traffic Volume on IRI for CA-ID-3

7.12.2 Comparison of Effects of Traffic Volume for CA-ID-6

The comparison of the effects of traffic volume on punch outs and IRI are shown in Figures 7.37 and 7.38 for CA-ID-6. The results show that the traffic volume has a significant impact on both the performance parameters. As the traffic volume increases, the performance of CRCP decreases with increase in IRI and punch outs. These results are similar to the results for CA-ID-3.

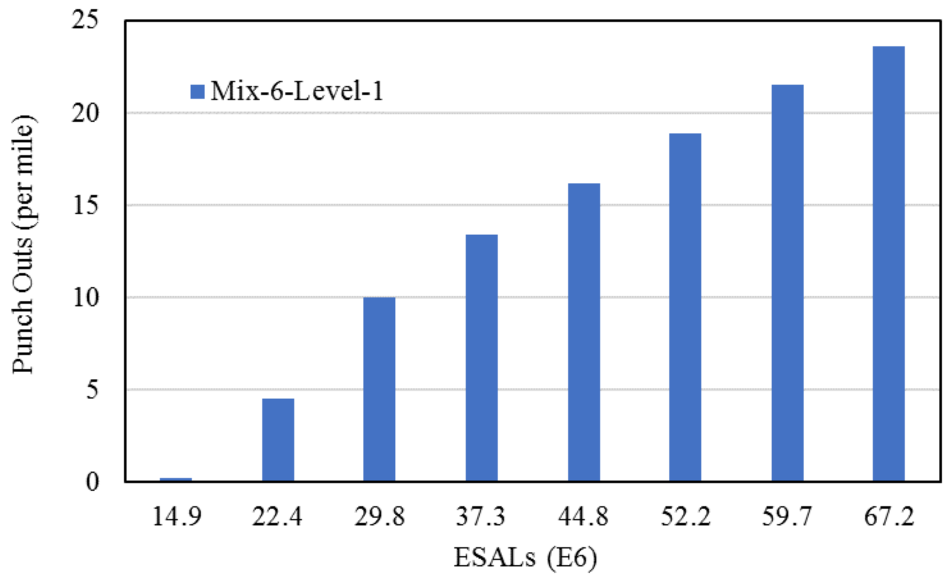

Figure 7.37. Impact of Traffic Volume on Punch Outs for CA-ID-6

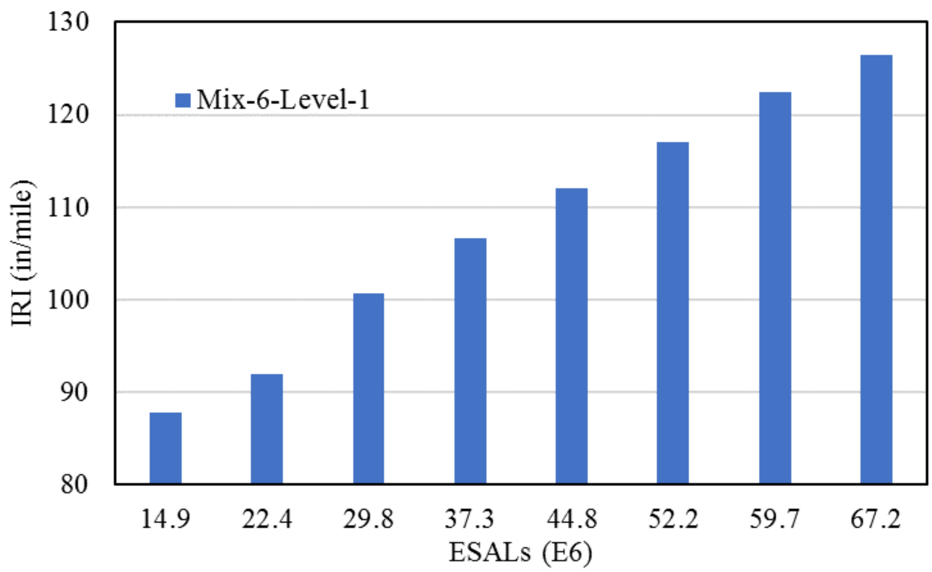

Figure 7.38. Impact of Traffic Volume on IRI for CA-ID-6

7.12.3 Percent Increase in IRI and Punch Outs with Increase in Traffic Volume

Percentage increase in CRCP distresses were determined for every 1000 increase in AADTT to evaluate the impact of traffic volume on pavement performance. The results are presented in Figures 7.39 and 7.40. The results show that the traffic volume has a more significant impact on punch outs with a % increase of 8.9 to 88.6% with increased traffic volume whereas the increase in IRI is in the range of 2.2 to 9.6%.

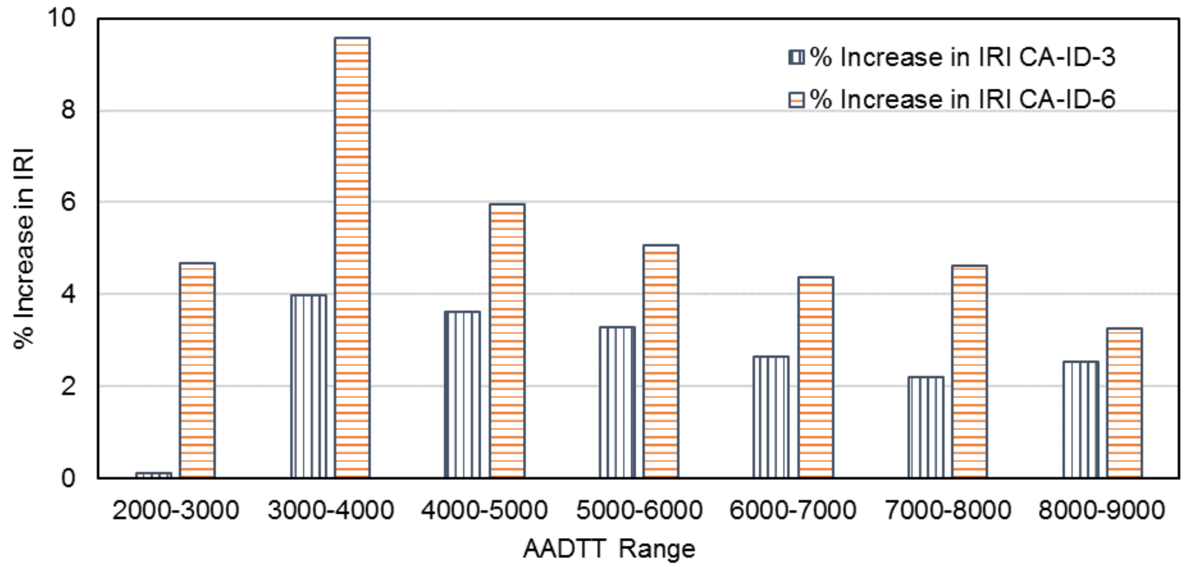

Figure 7.39. Percent Increase in IRI with Traffic Volume

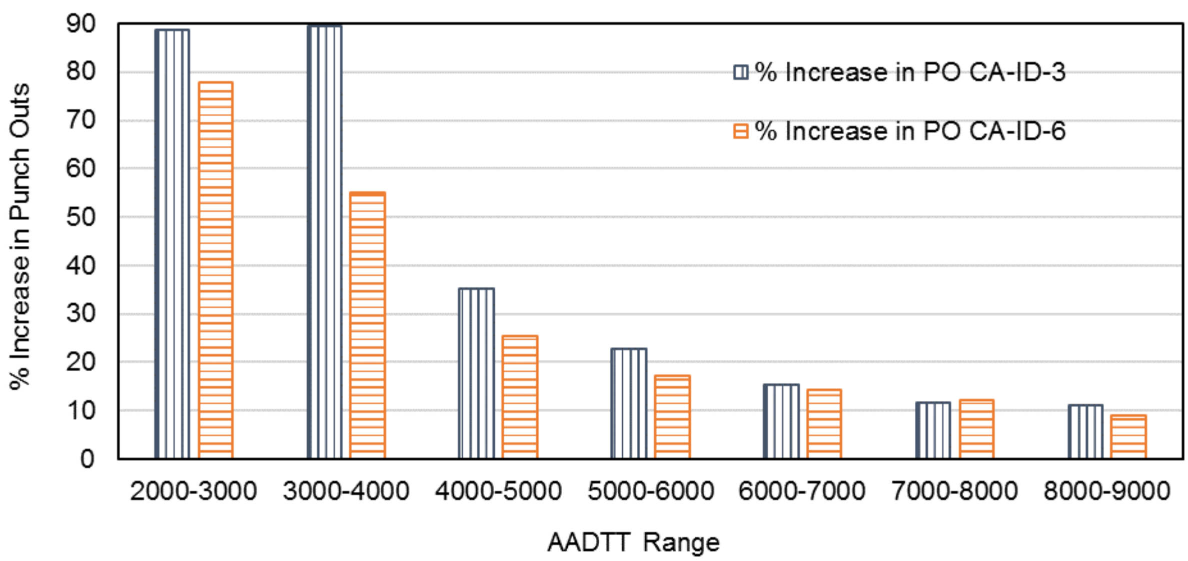

Figure 7.40. Percent Increase in Punch Outs with Traffic Volume

CHAPTER 8

JPCP DESIGN THICKNESS CHARTS/TABLES

8.1 GENERAL

The objective is to provide a Pavement Design Engineer with sufficient information so that the necessary input data can be developed and proper engineering principles applied to design the thickness of new Jointed Plain Concrete Pavement (JPCP). It is the responsibility of the Pavement Design Engineer to ensure that the designs produced conform to Department policies, procedures, standards, guidelines, and sound engineering practices.
The following definitions relate to the 2015 AASHTO Mechanistic-Empirical Pavement Design Guide and the Pavement ME Design software that is used for calculating rigid pavement thickness.

8.1.1 Reliability (R%)

The use of Reliability (R%) allows the Pavement Design Engineer to tailor the design to match the needs of the project. It is the probability of achieving the design life that the Department desires for that facility. The mechanistic-empirical models are based on smoothness, faulting, and transverse cracking failure mechanisms. Recommended values of R% range from 80% to 95%. For the design tables, Reliability value of 90% has been used.

8.1.2 Traffic Loading Forecasts (Equivalent Single Axle Loads - ESALs)

The number of heavy trucks and the equivalent 18-kip axle loads (ESALs) are forecast for the type of facility and its expected traffic growth. These design tables have been developed for various traffic volumes ranging from AADTT of 345 to 11500 (3 to 100 million ESALs).

8.1.3 Climate Region

Temperature gradients through the slab thickness can significantly affect the load induced stresses and performance of concrete pavements. Analysis using the Pavement ME software has shown there are significant differences in the impact of climate on rigid pavement design. The appropriate climate region for the project location being designed must be selected from the design tables or the appropriate climate files selected if using the AASHTOWare Pavement ME software. Six New Mexico districts have been chosen as different climatic regions to develop the design tables. The summary of climate data of these districts is given in Table 8.1.

Table 8.1. Summary of Climatic Conditions of Various Districts

Climate Details	Albuquerque	Santa Fe	Roswell	Deming	Las Vegas	Grants
District ID	D-3	D-5	D-2	D-1	D-4	D-6
Latitude	35.042	35.617	33.308	32.262	35.654	35.511
Longitude	-106.616	-106.089	-104.541	-107.721	-105.143	-108.789
Mean Annual Air Temperature (F)	58.18	52.41	61.57	62.15	50.47	50.17
Freezing Index (F - Days)	65.06	259.16	57.21	27.64	329.77	384.08
Annual Freeze Thaw Cycles	80.46	144.41	75.58	74.68	145.44	176.13

8.1.4 Initial Smoothness (IRI)

The initial smoothness (International Roughness Index - IRI) is the smoothness after construction. An initial IRI value of 63 in/mile has been used to develop the design tables.

8.1.5 Terminal Smoothness (IRI)

The Terminal Smoothness (IRI) is the smoothness condition of a road when it reaches a point where some type of rehabilitation or reconstruction is warranted. A value of 172 in/mile has been used.

8.1.6 Terminal Faulting

The Terminal Faulting is the mean differential elevation across joints in the wheel path where the condition of a road reaches a point where some type of rehabilitation or reconstruction is warranted. A value of 0.12 in is used in these design tables.

8.1.7 Terminal Cracking

The Terminal Cracking value is the percent of transverse slab cracking in the design lane where reconstruction would be warranted. A value of 15% has been used.

8.1.8 28-Day PCC Compressive Strength

A mean 28 day Portland Cement Concrete (PCC) compressive strength of 4000 psi is used to develop these design tables. The Pavement ME Design software uses this value to estimate the elastic modulus and modulus of rupture for the concrete.

8.1.9 Coefficient of Thermal Expansion (CTE) of Paving Concrete

CTE of paving concrete has a significant impact on design and performance of JPCP and the CTE of paving mixes being used in NM ranges from 3.7 to 5.9 με/°F so these design tables have been prepared for various CTE values i.e. 4.0, 4.5, 5.0, 5.5 and 6.0 με/°F.

8.1.10 Road Bed Soil Resilient Modulus (MR)

The Resilient Modulus (MR) is a measurement of the stiffness of the roadbed soil. Since rigid concrete pavement is considerably stiffer than flexible asphalt pavement, the rigid designs spread the vehicle loads over a wider area and are not very sensitive to the subgrade modulus. A value of 12,000 psi is used to develop the Design Tables. If the evaluation of a significantly different Design MR value for a specific project site is desired, the Pavement ME software can be run to see if it makes a difference in the concrete thickness.

8.1.11 Base Coarse, Resilient Modulus and Thickness

Sub-base MR value of 25,000 psi has been used for these design tables with a constant thickness of 6 in.

8.1.12 Joint Spacing

A standard JPCP transverse joint spacing of 15 ft is used.

8.1.13 Dowel Size

The dowel size will vary according to the pavement slab thickness. The details are given in Table 8.2.

Table 8.2. Dowel Size Details

JPCP Slab Thickness	Dowel Size (in)
Less than 8 in	1
8 in to 9.5 in	1.25
10 in and more	1.5

8.1.14 Design Lane Slab Width

A slab width of 12 ft for the design lane has been used to develop these design tables. Tied PCC shoulders were considered for the design simulations.

8.1.15 Design Period

The design period is taken as 20 years.

8.2 DESIGN THICKNESS BASED ON PAVEMENT ME DESIGN SOFTWARE

Simulations were conducted in Pavement ME design software version 2.3 and the results were analyzed to formulate the table and chart for design thickness of new JPCP. The required thicknesses for various traffic levels and different CTE values of paving concrete are shown in Table 8.3 to 8.8 for the 6 districts of New Mexico. The results are also presented in the form of thickness design charts as shown in Figure 8.1 to 8.6. These results show that for the same traffic volume the required JPCP thickness increases with increase in CTE. Also, with an increase in traffic volume the JPCP thickness would increase if CTE is kept constant. JPCP thickness also varies for different districts due to varying climatic conditions.

Table 8.3. Required Design Thickness for JPCP in NM District-1

AADTT	ESALs (E6)	CTE=4 με/°F	CTE=4.5 με/°F	CTE=5 με/°F	CTE=5.5 με/°F	CTE=6 με/°F
345	3	8	8	8	8.5	9
1150	10	8.5	9	9	9.5	10.5
3450	30	9	9.5	10	11	12.5
11500	100	10	10.5	11.5	13	13.5

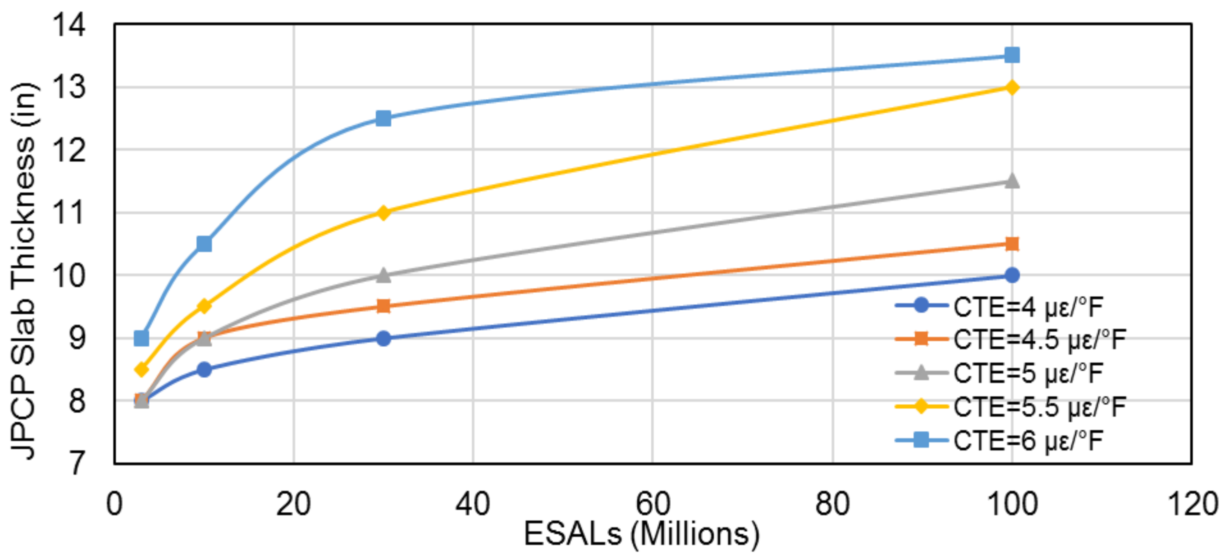

Figure 8.1. Design Chart for Required Design Thickness for NM District-1

Table 8.4. Required Design Thickness for JPCP in NM District-2

AADTT	ESALs (E6)	CTE=4 με/°F	CTE=4.5 με/°F	CTE=5 με/°F	CTE=5.5 με/°F	CTE=6 με/°F
345	3	8	8.5	8.5	9	9
1150	10	8.5	9	9	10	10.5
3450	30	9.5	9.5	10	11	12.5
11500	100	10	10.5	11.5	13	13.5

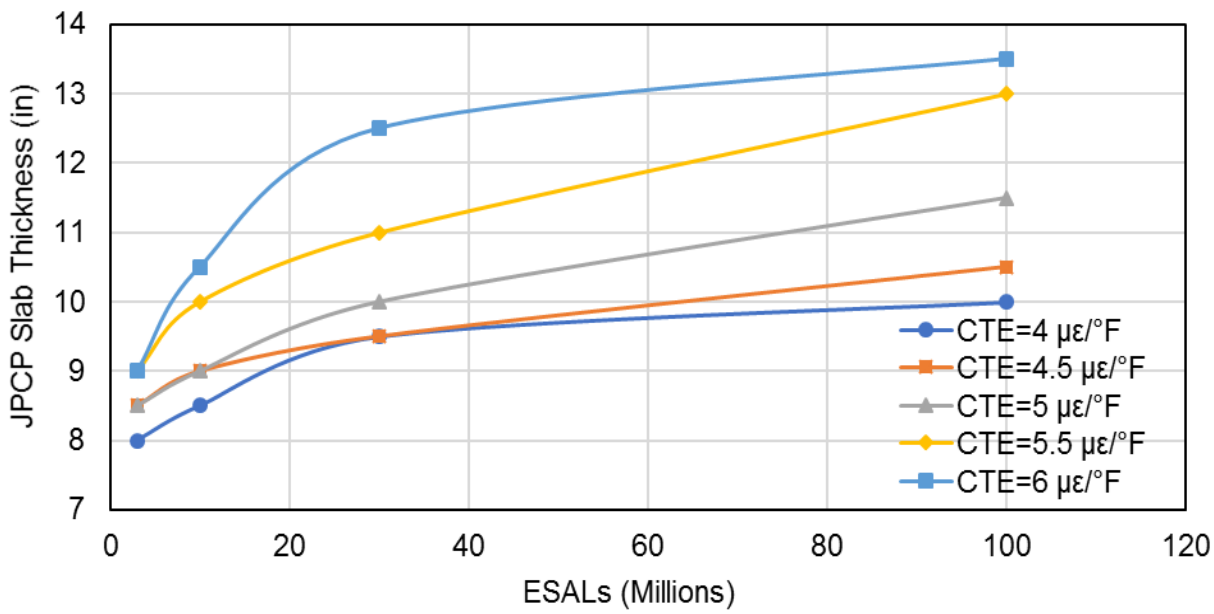

Figure 8.2. Design Chart for Required Design Thickness for NM District-2

Table 8.5. Required Design Thickness for JPCP in NM District-3

AADT T	ESALs (E6)	CTE=4 με/°F	CTE=4.5 με/°F	CTE=5 με/°F	CTE=5.5 με/°F	CTE=6 με/°F
575	5	8	8	8	8	8.5
1150	10	8	8	8.5	8.5	9
3450	30	9	9	9	9	10
11500	100	9	9.5	10	10.5	11.5

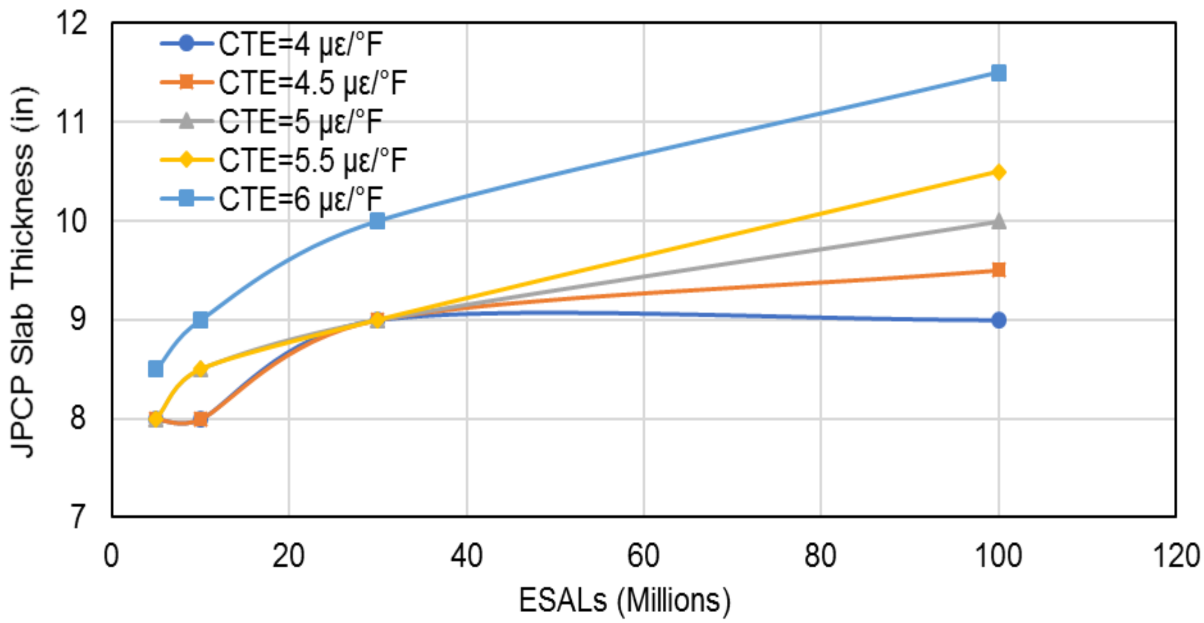

Figure 8.3. Design Chart for Required Design Thickness for NM District-3

Table 8.6. Required Design Thickness for JPCP in NM District-4

AADT T	ESALs (E6)	CTE=4 με/°F	CTE=4.5 με/°F	CTE=5 με/°F	CTE=5.5 με/°F	CTE=6 με/°F
345	3	8	8	8.5	8.5	9
1150	10	8.5	9	9	9.5	10.5
3450	30	9.5	9.5	10	10.5	12
11500	100	10	10.5	11.5	12.5	13

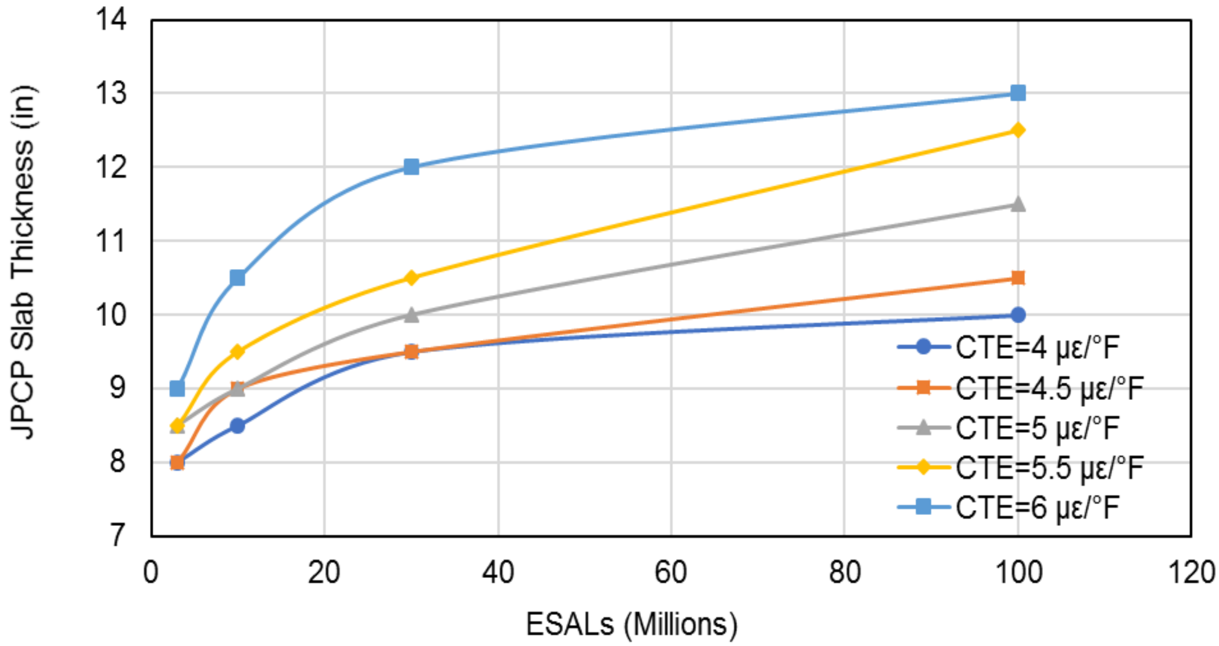

Figure 8.4. Design Chart for Required Design Thickness for NM District-4

Table 8.7. Required Design Thickness for JPCP in NM District-5

AADT T	ESALs (E6)	CTE=4 με/°F	CTE=4.5 με/°F	CTE=5 με/°F	CTE=5.5 με/°F	CTE=6 με/°F
345	3	8	8	8.5	8.5	9
1150	10	8.5	8.5	9	9.5	10.5
3450	30	9	9.5	10	11	12
11500	100	10	10.5	11.5	13	13.5

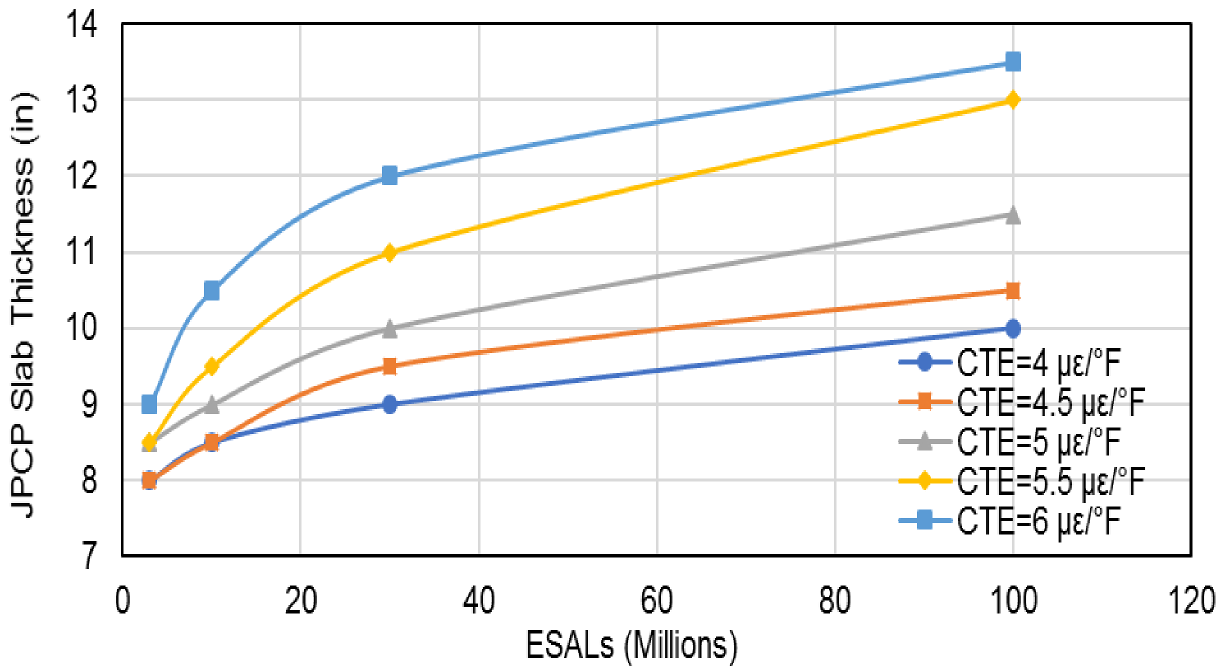

Figure 8.5. Design Chart for Required Design Thickness for NM District-5

Table 8.8. Required Design Thickness for JPCP in NM District-6

AADTT	ESALs (E6)	CTE=4 με/°F	CTE=4.5 με/°F	CTE=5 με/°F	CTE=5.5 με/°F	CTE=6 με/°F
345	3	8	8.5	9	9	9.5
1150	10	9	9.5	9.5	10	10.5
3450	30	9.5	10	10.5	11.5	12.5
11500	100	10	10.5	11.5	13	14

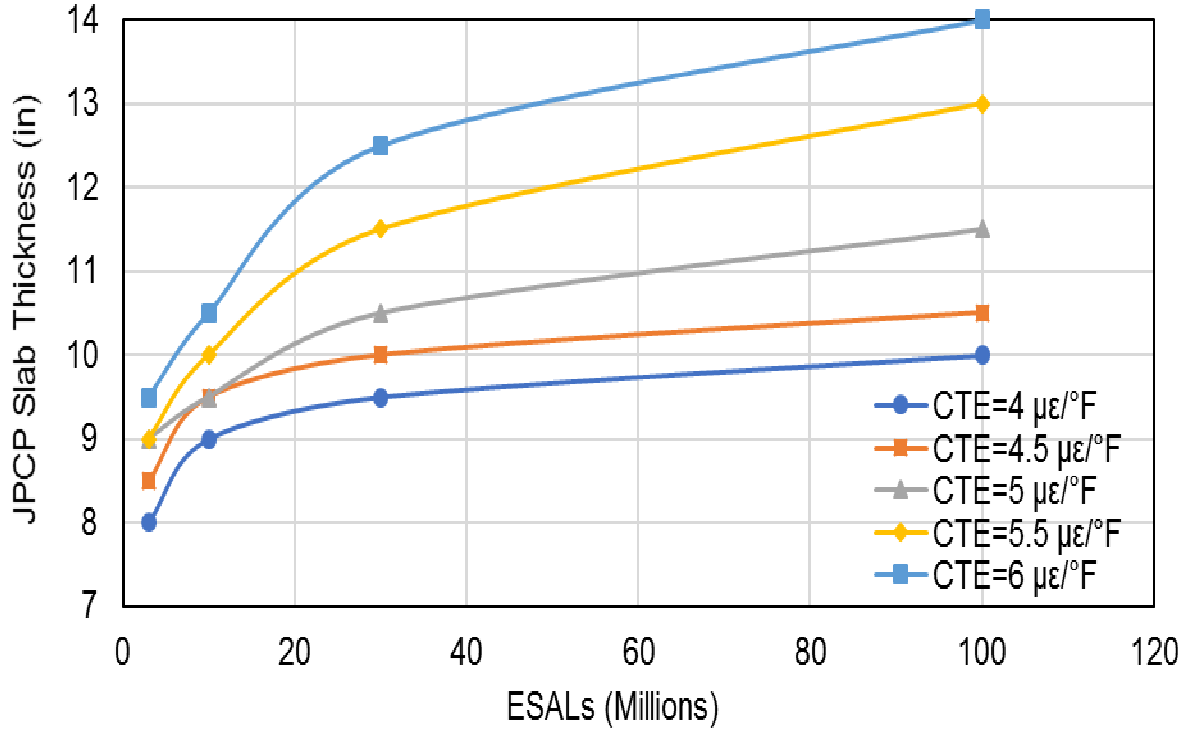

Figure 8.6. Design Chart for Required Design Thickness for NM District-6

CHAPTER 9

CONCLUSIONS

The focus of this study was on characterization of concrete paving mixes being used in various districts of New Mexico to generate level-1 material inputs to be used for rigid pavement design by using Pavement ME Design. Seven paving mixes prepared with different coarse aggregates were tested for mechanical and thermal properties and a database was generated for the design of rigid pavements. Design simulations were conducted in Pavement ME Design to evaluate the impact of various material inputs on the predicted performance of JPCP and CRCP. Following conclusions can be drawn from this research:

1. The interconversion models developed in this study gives better results for elastic modulus and MOR for New Mexico paving mixes in comparison to the Pavement ME default models.
2. The power model for interconversion of compressive strength into elastic modulus is the best fit to the experimental data of the tested paving mixes. The standard error of estimate (SEE) for the proposed model was 0.72 E6 psi in comparison to the ME default model having SEE of 0.99 E6 psi.
3. The proposed MOR power model with SEE of 48.8 psi works better than the ME default model with SEE of 103.5 psi for these paving mixes.
4. Coefficient of thermal expansion (CTE) of New Mexico paving mixes varies over a broad range i.e. 3.7 to 5.9 με/°F. The effect of coarse aggregate mineralogy on the concrete CTE was also highlighted with the test data. Concrete with limestone aggregate had the lowest CTE as compared to the CTE of concrete with other types of coarse aggregate including Basalt, Dolomite, Granite, and Quartzite.
5. There is significant variation in transverse cracking and joint faulting between the JPCP design simulations conducted with the tested CTE values and the default CTE values. The primary indication is that higher CTE value leads to higher distresses. It became evident that the JPCP must be designed with the accurately tested CTE value for the paving mix to be used so that the designed pavement can last for the entire service life. The ME default CTE data will not produce an accurate design for NMDOT paving mixes.
6. A significant difference was observed between the performance values of JPCP design simulations conducted with level-1 inputs and level-3 inputs. The most significant impacted parameter was transverse cracking with a change of 39 to 54% between level-1 and level-3 designs. With this, it is again highlighted that JPCP design shall be conducted with lab tested/level-1 material inputs.
7. The transverse joint spacing of JPCP was found to have a significant impact on transverse cracking and joint faulting and the adverse effects of high CTE concrete can be minimized by reducing the joint spacing.

8. The comparison of CRCP distresses was conducted by determining the percent increase in punch out and IRI for various ranges of CTE increase and It became evident that the impact of CTE increase on punch out is the most significant with percent increase of 30 to 40% while the IRI is affected with a percent increase of 3 to 14%. It is also pertinent to mention that as the CTE value increases the percent increase in the distress increases which means that higher CTE values are detrimental for the CRCP performance.
9. The simulation results were analyzed to compare the percentage change in performance parameters including IRI and punch outs with level-1 and level-3 inputs. The results show that the %age change in IRI ranges from 1 to 43.8% while the percentage change in punch outs ranges from 1 to 95.8%. It indicates that the impact of material input levels is more significant on the CRCP punch outs then the pavement roughness. The CRCP designed with level-3 material inputs may be under-designed or over-designed and to obtain accurate CRCP design, level-1 material inputs should be used.

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