TECHNICAL REPORT STANDARD PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/LA.11/451		

4. Title and Subtitle	5. Report Date
Determination of Coefficient of Thermal Expansion Effects on Louisiana's PCC Pavement Design	**December 2011**
	6. Performing Organization Code **LTRC Project Number: 07-2C** **State Project Number: 736-99-1450**
7. Author(s) **Hak-Chul Shin and Yoonseok Chung**	8. Performing Organization Report No.
9. Performing Organization Name and Address **Department of Civil and Environmental Engineering** **Louisiana State University** **Baton Rouge, LA 70803**	10. Work Unit No. 11. Contract or Grant No. **07-2C / 736-99-1450**
12. Sponsoring Agency Name and Address **Louisiana Department of Transportation and Development** **P.O. Box 94245** **Baton Rouge, LA 70804-9245**	13. Type of Report and Period Covered **Final Report, 2/01/07 – 4/31/09** 14. Sponsoring Agency Code

15. Supplementary Notes

Conducted in Cooperation with the U.S Department of Transportation, Federal Highway Administration

16. Abstract

With the development of the Mechanistic Empirical Pavement Design Guide (MEPDG) as a new pavement design tool, the coefficient of thermal expansion (CTE) is now considered a more important design parameter in estimating pavement performance including cracking, faulting, and international roughness index (IRI). This study was conducted to measure typical CTE values of Portland cement concrete (PCC) pavements having various aggregates used in Louisiana and to investigate the relationship between CTE and other critical variables such as aggregate types, age of concrete, dimension of specimen, amount of course aggregate in mixture, relative humidity, and concrete mechanical properties. AASHTO TP 60-00 was used for measuring concrete CTE and a recently new standard test method, AASHTO T 336-09, was adopted to replace the TP 60-00. A calibration factor was developed to convert the CTE values measured by AASHTO TP 60-00 to that of the new standard testing method. From the analysis of measured data, it was found that aggregate types, coarse aggregate proportion, and relative humidity have a significant influence on CTE. This finding was confirmed with a statistical analysis of variance (ANOVA). CTE tests and mechanical property tests were also performed at different ages to provide input data for Level 1 design of MEPDG. Based on the results of the MEPDG analysis, current maximum joint spacing [20 ft. (6.1 m)] in jointed plain concrete pavement (JPCP) can be adjusted to 15 or 18 ft. (4.6 or 5.5 m) when Kentucky limestone is used as a coarse aggregate.

17. Key Words	18. Distribution Statement
Pavement Design, CTE, MEPDG, Joint spacing	

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
not applicable		**98**	

Project Review Committee

Each research project will have an advisory committee appointed by the LTRC Director. The Project Review Committee is responsible for assisting the LTRC Administrator or Manager in the development of acceptable research problem statements, requests for proposals, review of research proposals, oversight of approved research projects, and implementation of findings.

LTRC appreciates the dedication of the following Project Review Committee Members in guiding this research study to fruition.

LTRC Manager

Tyson Rupnow, Ph.D., P.E.
Concrete Research Engineer

Members

Zhongjie "Doc" Zhang, Pavement & Geotechnical Research Administrator
Jeff Lambert, Pavement & Geotechnical Section
Philip Arena, FHWA
Chris Abadie, Materials Research Administrator
John Eggers, Construction Section
Scott Ashmore, James Construction Group
Mark Stinson, FHWA
Mike Ricca, Section 40

Directorate Implementation Sponsor

William H. Temple, P.E
DOTD chief engineer

Determination of Coefficient of Thermal Expansion Effects on Louisiana's PCC Pavement Design

by

Hak-Chul Shin
Yoonseok Chung

Department of Civil and Environmental Engineering
Louisiana State University
Baton Rouge, LA 70803

LTRC Project No. 07-2C
State Project No. 736-99-1450

conducted for

Louisiana Department of Transportation and Development
Louisiana Transportation Research Center

December 2011

ABSTRACT

With the development of the Mechanistic Empirical Pavement Design Guide (MEPDG) as a new pavement design tool, the coefficient of thermal expansion (CTE) is now considered a more important design parameter in estimating pavement performance including cracking, faulting, and international roughness index (IRI). This study was conducted to measure typical CTE values of Portland cement concrete (PCC) pavements having various aggregates used in Louisiana and to investigate the relationship between CTE and other critical variables such as aggregate types, age of concrete, dimension of specimen, amount of course aggregate in mixture, relative humidity, and concrete mechanical properties. AASHTO TP 60-00 was used for measuring concrete CTE and a recently new standard test method, AASHTO T 336-09, was adopted to replace the TP 60-00. A calibration factor was developed to convert the CTE values measured by AASHTO TP 60-00 to that of the new standard testing method. From the analysis of measured data, it was found that aggregate types, coarse aggregate proportion, and relative humidity have a significant influence on CTE. This finding was confirmed with a statistical analysis of variance (ANOVA). CTE tests and mechanical property tests were also performed at different ages to provide input data for Level 1 design of MEPDG. Based on the results of the MEPDG analysis, current maximum joint spacing [20 ft. (6.1 m)] in jointed plain concrete pavement (JPCP) can be adjusted to 15 or 18 ft. (4.6 or 5.5 m) when Kentucky limestone is used as a coarse aggregate.

ACKNOWLEDGEMENTS

The U.S Department of Transportation, Federal Highway Administration (FHWA), the Louisiana Department of Transportation and Development (LADOTD), and the Louisiana Transportation Research Center (LTRC) financially supported this research project.

The authors would like to acknowledge Randy Young, Matt Tircuit, and Keith Beard for their assistance in producing the CTE specimens and completing the hardened concrete testing.

IMPLEMENTATION STATEMENT

This study was conducted to measure the CTE of concrete specimen having various coarse aggregates widely used in pavement construction in Louisiana. This study provided the direct input data for MEPDG: (1) thermal properties (CTE, thermal conductivity, and heat capacity) of three different coarse aggregate concretes (Kentucky limestone, gravel, and Mexican limestone) and (2) concrete mechanical properties with ages (compressive strength, flexure strength, and modulus of elasticity) of each coarse aggregate concrete. The thermal properties with three different types of coarse aggregate were tabulated in Appendix B and can be used as input data in the MEPDG analysis to predict the performance of PCC pavement. Based on both the MEPDG analysis and the case study of other states, current maximum joint spacing [20 ft. (6.1 m)] can be adjusted to 15 or 18 ft. (4.6 or 5.5 m) joint spacing when Kentucky limestone is used as coarse aggregate. The findings of MEPDG analysis should be re-evaluated once the DARWin-ME software is available.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

The National Cooperative Highway Research Program (NCHRP) released the Mechanistic Empirical Pavement Design Guide in 2004. The MEPDG provides an advanced pavement design analysis to determine both the structural response and performance prediction within the design life of PCC pavement. The input parameters are categorized by three groups, traffic, climatic, and materials data, and each group consists of a hierarchical method that involves three levels. Level 1 provides the highest level of reliability and can be used for heavy traffic pavement design. Level 2 provides the intermediate level of reliability and is similar to the AASHTO pavement design guide. Level 3 provides the lowest level of reliability and can be applied to the relatively less significant pavement design. In order to obtain the highest reliability level of design, the concrete mechanical properties for 7, 14, 28, and 90 days are used for Level 1 input parameters in MEPDG.

The coefficient of thermal expansion has been widely considered as a fundamental property of PCC pavement but has never played an important role in the thickness design procedure for PCC pavement until recently. In the MEPDG developed through the NCHRP 1-37A project, the CTE became a direct input parameter that was closely related to the pavement performance [*1*]. Therefore, it was imperative to measure accurate CTE for PCC pavement to predict critical pavement distresses within the designed years. Mallela et al. found that increased CTE generally resulted in increasing cracking, joint faulting, and International Roughness Index (IRI) in jointed plain concrete pavements (JPCP) [*2*].

The current AASHTO test method (TP 60) measuring the CTE of hydraulic cement concrete was first published in 2000 and reconfirmed in 2006 [*3*]. AASHTO TP 60 is the only standard for measuring CTE, even though several researchers have suggested improvements for a more accurate CTE. AASHTO TP 60 is attached in Appendix A. The accuracy and repeatability of AASHTO TP 60 completely depend on the stability of displacement reading at both 50°F and 122°F (10˚C and 50˚C), but investigation found that both displacement readings are not stable. Won suggested a new regression model that used the relationship between temperature and displacement changes from 59˚F to 113˚F (15˚C to 45˚C) since the temperature gradient through the cylindrical specimen was not uniform between 50˚F and 59˚C (10˚C and 15˚C) and between 113˚F and 122˚F (45˚C and 50˚C) [*4*]. Both methods provided the same CTE mean values [4.45 με/°F (8.01 με/°C)], but the variation between the heating and cooling cycle of the proposed regression model [0.03 με/°F (0.06 με/°C)] was much less than that of AASHTO TP 60 [0.13 με/°F (0.24 με/°C)]. The conversion table of CTE unit is provided in Appendix B.

CTE for PCC pavement is generally influenced by (1) types and volumetric proportion of coarse aggregate in the mixture, (2) relative humidity in the mixture during the test, (3) strength parameters of the mixture, and (4) the age of the mixture [*2*], [*4*], and [*5*].

According to Mindess et al., the CTE of limestone and quartzite are 3.3 με/°F and 6.1 to 7.2 με/°F (6 με/°C and 11 to 13 με/°C), respectively [*5*]. The CTE of cement pastes ranges between 10 and 11.1 με/°F (18 and 20 με/°C). Due to the differences in the CTE of concrete ingredients, the proportion of coarse aggregate in concrete mixtures should be considered when the CTE is estimated. In a case of concrete having crushed limestone with siliceous sand, the CTE decreases steeply when the amount of crushed limestone increases. This is because the CTE of limestone is much smaller than that of cement paste [*5*]. However, with quartz gravel and siliceous sand, the concrete CTE increases slowly with the increase of the amount of quartz gravel. Typical CTE ranges for various aggregates and cement paste are presented in Table 1.

Mallela et al. tested 673 cores representing hundreds of pavement sections throughout the United States as part of the long term pavement performance (LTPP) program [*2*]. The general range of CTE values of PCC is between 5 and 7 με/°F (9 and 12.6 με/°C), and concrete made from igneous aggregates has CTE values around 5.2 με/°F or 5.3 με/°F (9.4 με/°C or 9.5 με/°C) and that made from sedimentary rock has a typical value of 6 με/°F (10.8 με/°C). The mean CTE value of the entire data set is 5.7 με/°C (10.3 με/°C).

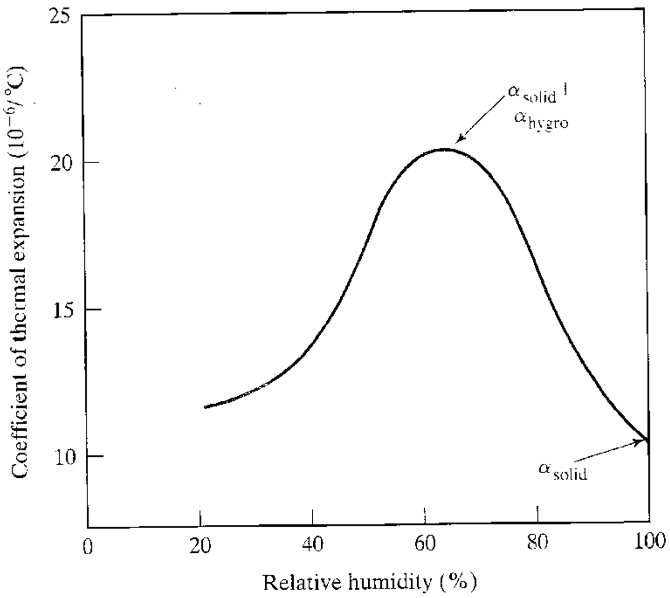
Alungbe et al. found that CTEs of three different aggregates (porous limestone, river gravel, and dense limestone) were significantly different from one another at ages 28 and 90 days [*6*]. The water/cement ratio (0.53, 0.45, and 0.33) and cement content (508 lb/yd^3, 564 lb/yd^3, and 752 lb/yd^3) did not statistically show significant effects on the CTE.

Kohler et al. observed that the difference in CTE of oven-dried specimen between the expansion and contraction was remarkable, and the difference was reduced during the first 10 to 15 hours [*7*]. The expansion CTE decreased considerably, while the contraction CTE stayed constant. This is because a rise in temperature decreases capillary tension and causes water to enter the gel pores. The intrusion of water in the gel pores causes swelling in addition to the normal thermal expansion, but no swelling is possible when the cement paste is dry or saturated due to the absence of capillary tension. Thus, the coefficient of thermal expansion in the two extreme cases is lower than that of partially saturated conditions.

Table 1
Typical CTE ranges for common components and concrete [*1*]

Material Type	Coefficient of Thermal Expansion, 10^{-6}/°F (10^{-6}/°C)	Concrete Coefficient of Thermal Expansion (made from this material), 10^{-6}/°F (10^{-6}/°C)
Aggregates		
Marble	2.2-3.9 (4.0-7.0)	2.3 (4.1)
Limestone	2.0-3.6 (3.6-6.5)	3.4-5.1 (6.1-9.2)
Granites & Gneisses	3.2-5.3 (5.8-9.5)	3.8-5.3 (6.8-9.5)
Syenites, Diorites, Andesite, Basalt, Gabbros, Diabase	3.0-4.5 (5.4-8.1)	4.4-5.3 (7.9-9.5)
Dolomites	3.9-5.5 (7.0-9.9)	5.1-6.4 (9.2-11.5)
Blast Furnace Slag	–	5.1-5.9 (9.2-10.6)
Sandstones	5.6-6.7 (10.1-12.1)	5.6-6.5 (10.1-11.7)
Quartz Sands & Gravels	5.5-7.1 (9.9-12.8)	6.0-8.7 (10.8-15.7)
Quartzite, Cherts	6.1-7.0 (11.0-12.6)	6.6-7.1 (11.9-12.8)
Cement Paste (saturated)		
w/c=0.4 to 0.6	10-11 (18.0-19.8)	–
Concrete Cores		
Cores from LTPP pavement sections, many of which were used in calibration	N/A	4.0×10^{-6}-5.5×10^{-6}-7.2×10^{-6} (7.2×10^{-6}-9.9×10^{-6}-13.0×10^{-6}) (Min – Mean – Max)

Mallela et al. emphasized that the CTE is remarkably sensitive to the relative humidity (RH) in the mixture during the test [*2*]. The CTE of concrete reaches its maximum value at 60 to 70 percent RH. The value at 100 percent RH is 20 to 25 percent less than the maximum value. However, the fully saturated condition is the most practical from a testing standpoint [*8*]. Figure 1 shows the variation of CTE with RH of concrete cement paste.



8

Figure 1
Variation of CTE with moisture content of cement paste [*5*]

Won evaluated the effect of concrete age on CTE and found that CTE values do not change with age of concrete up to 3 weeks [*4*]. However, Jahangirnejad et al. statistically investigated the impact of sample age with an aggregate geology and concluded that the magnitude of CTE at 28 days was significantly lower than that of CTE at 90 and 180 days for most aggregate types [*9*]. The difference of CTE between 28 days and 180 days varies from 0.08 to 0.52 με/°F (0.15 to 0.94 με/°C).

Hossain et al. studied the design strategy to alleviate the detrimental effect of higher CTE values and found that increasing PCC strength was one of the alternative methods [*10*]. Specifically, by increasing strength parameters of PCC pavement, the amount of cracking was reduced. Increased PCC slab thickness and increased dowel diameter eliminated slab cracking as well. Among all these alternatives, a 14-ft. widened lane was chosen as the most effective method since no additional cost is required. Mellela et al. also found that PCC flexural strength and PCC elastic modulus are critical inputs that interact with CTE [*2*].

Mallela et al. analyzed the effect of CTE on mean transverse joint faulting, percentage of slabs with transverse cracking, and IRI [*2*]. Three CTE values [4.5, 5.5, and 7.0 με/°F (8.1, 9.9, and 12.6 με/°C)], two transverse joint spacing [15 ft. and 20 ft. (4.6 m and 6.1 m)], and two PCC flexural strength (500 psi and 750 psi) were chosen for the analysis while all other parameters were kept constant. As CTE and transverse joint spacing increase, the mean transverse joint faulting also increases due to the higher curling deflection.

Increased CTE causes a high percentage of slabs with transverse cracking; for the longer slab length of 20 ft. (6.1 m), transverse cracking increases remarkably even in the increase at the smaller CTE values. Higher CTE generally results in increased IRI because of increased transverse joint faulting and transverse cracking.

Temperature and moisture gradients in PCC pavement are considered important factors that cause curling and warping stresses. As the variation of temperature and moisture throughout the slab thickness increases, the severe loss of support develops in PCC pavement. Thompson et al. observed that the temperature gradient along with slab depth was non-linear and showed large daily and seasonal variation as shown in Figure 2 and Figure 3 [*11*]. Jansen investigated the moisture gradient in the PCC pavement by using a computer model based on laboratory and field measurements [*12*]. Figure 4 shows that the top surface of pavement is at 50 percent saturation, while the bottom of pavement is at 100 percent saturation. The variance of moisture is only remarkable within the top 2 in. (50.8 mm) of PCC slab and the bottom of slab has more than 80 percent saturation.

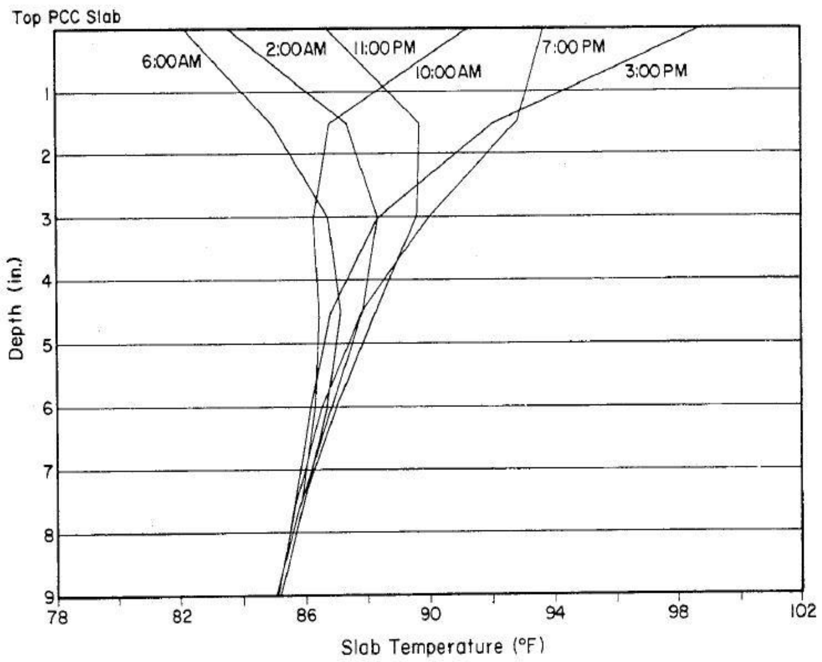


Figure 2
Temperature distribution in PCC slab (April, Urbana, IL) [*11*]

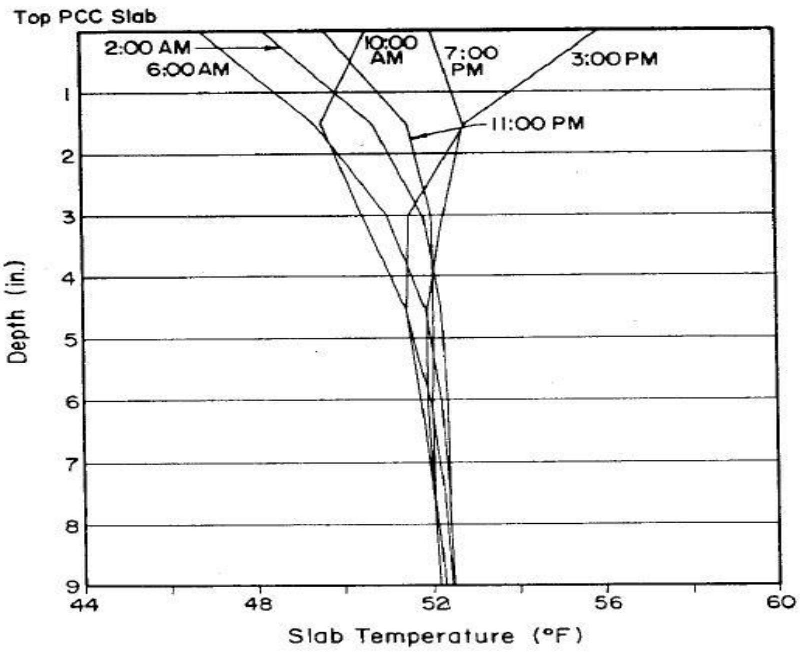


Figure 3
Temperature distribution in PCC slab (November, Urbana, IL) [*11*]

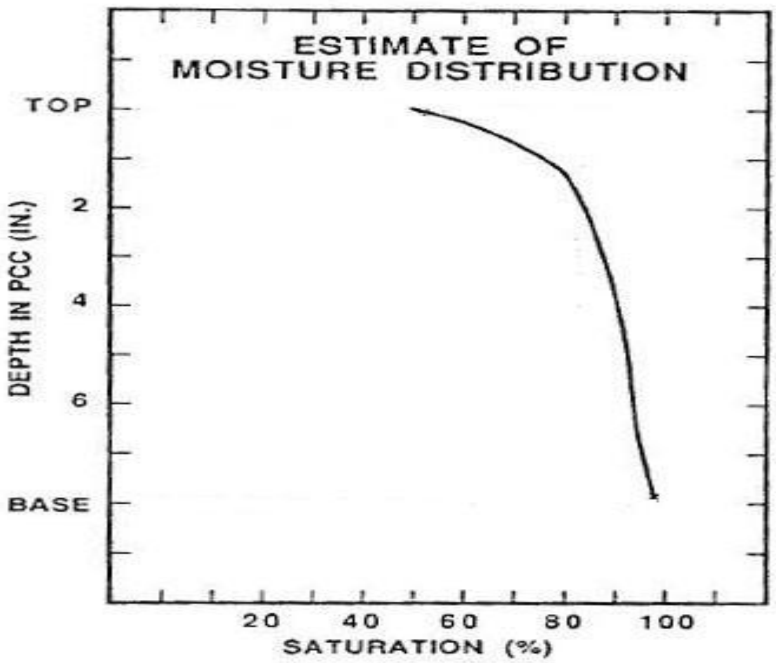


Figure 4
Estimate of moisture profile in PCC slab [*12*]

OBJECTIVE

The objectives of this research were to measure typical CTE values of concrete mixtures used for PCC pavement structures in accordance to the AASHTO TP 60-00; to investigate the relationship between CTE and other critical variables such as aggregate types, age of concrete, dimension of specimen, amount of course aggregate in mixture, relative humidity, and concrete mechanical properties; and to assist in the implementation of MEPDG for PCC pavement design in Louisiana. The recommendations for the coarse aggregate type in the mixture and maximum joint spacing in JPCP are provided based on the results of the MEPDG analysis. This study also calculated the curling stresses in the PCC pavement due to non-linear temperature and moisture gradients throughout the slab thickness. The second objectives were to re-measure the CTE of the concrete specimen in accordance to the recently adapted AASHTO T 336-09, and find calibration factors to convert the CTE values measured by AASHTO TP 60-00 without further measurements.

SCOPE

Three aggregates widely used in Louisiana, Kentucky limestone, gravel, and Mexican limestone, were chosen for the coarse aggregate of concrete mixture, and CTE tests were performed to find the aggregate effects on CTE. CTE is also measured at various ages (3, 5, 7, 14, 28, 60, and 90 days); various coarse aggregate proportions (20, 64, 80 percent of coarse aggregates); and various relative humidities (between 30 and 100% RH) of specimens to verify the factor that has the most critical impact on CTE. After finding the relationship between CTE and other critical variables, the results of CTE and mechanical property tests were used to run the MEPDG analysis. The results of the MEPDG analysis were PCC pavement distresses such as mean joint faulting, transverse cracking, and terminal IRI. Appropriate coarse aggregate type and joint spacing in JPCP can be recommended by comparing the results of the MEPDG analysis to the specification. CTE tests of three coarse aggregate types (Kentucky limestone, gravel, and Mexican limestone) and two various coarse aggregate proportions (20 and 80 percent of coarse aggregates) were performed in accordance to AASHTO T 336-09 to calibrate the incorrect CTE values measured by AASHTO TP 60-00.

METHODOLOGY

Apparatuses

CTE

To measure the CTE of concrete, a HM-251 CTE measuring system manufactured by Gilson/Challenge technology was used. Figure 5 shows the apparatus.

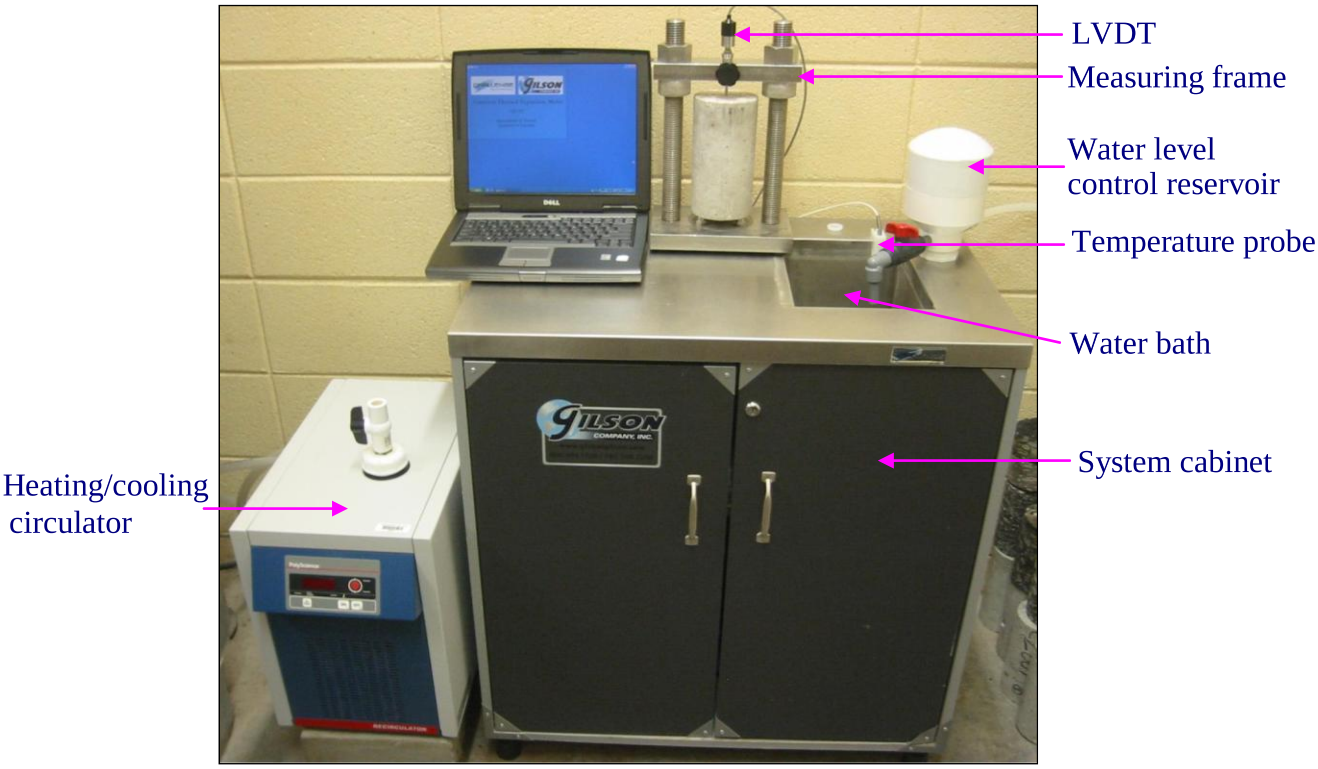


Figure 5
CTE measuring apparatus (HM-251)

The HM-251 strictly follows AASHTO TP 60 mentioned earlier. The HM-251 is divided into three parts: measuring frame, system cabinet with water bath, and a heating/cooling circulator. The measuring frame is designed for a typical cylindrical specimen, and its height can be adjusted depending on the specimen heights. A precise linear variable displacement transducer (LVDT) with a resolution of $0.122 * 10^{-8}$ in. ($3.1 * 10^{-8}$ mm) and total travel distance of 0.05 in. (1.27 mm) is installed on the top of the frame and measures the length change of concrete specimen automatically. The material of the measuring frame is A304 stainless steel, which is used to eliminate corrosion of the frame. A calibration bar [8 in. (203.2 mm)] made with the same material as the measuring frame is used to calibrate the length change of the frame itself. During the calibration process, the calibration factor of the stainless frame was measured and directly used for the calculation of concrete CTE. The water bath mounted in the system cabinet is of appropriate size to place the measuring frame. A temperature probe is installed inside the water bath to measure water temperature continually. The water level in the bath is maintained constant by a water level control reservoir to prevent the effect of evaporation during heating. The heating/ cooling circulator is separated from the water bath because its vibration can affect the measurement of the LVDT. The heating/cooling circulator is controlled by the HM-251 software to increase and decrease the water temperature between 50°F and 122°F (10°C and 50°C). When the temperature changes from 50°F and 122°F (10°C to 50°C), the expansion CTE is measured, while the contraction CTE is measured when it changes from 122°F to 50°F (50°C to 10°C). Schematic expansion and contraction graphs are presented in Figure 6 and Figure 7. The test procedures of MH-251 and raw data of CTE test results are described in Appendix C.

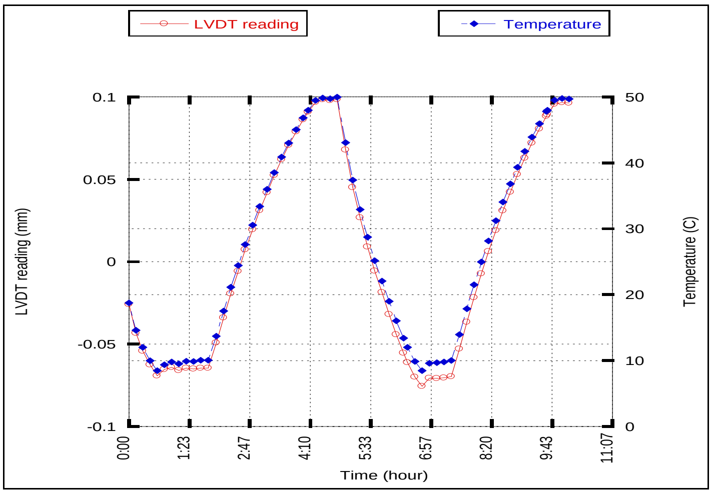


Figure 6
Time vs displacement and temperature plot

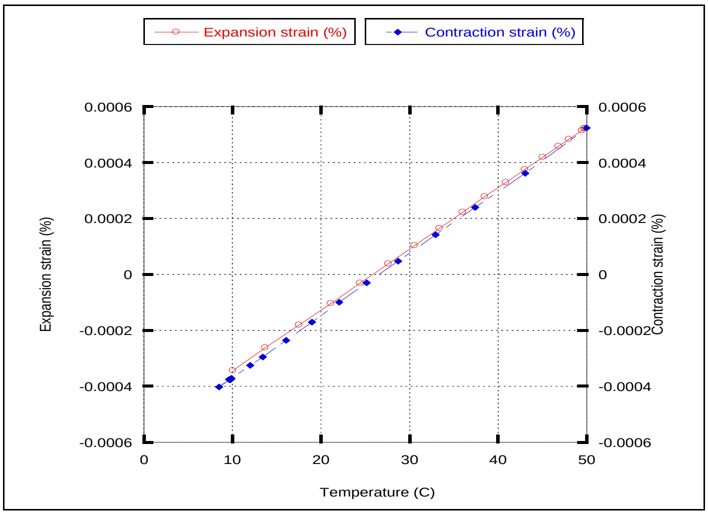


Figure 7
Temperature vs strain plot

The test is terminated when the difference between expansion CTE and contraction CTE is within 0.2 με/°F (0.3 με/°C), and the average value of two CTEs becomes a "representative" CTE. Otherwise, the software adjusts the temperature for another cycle and calculates the CTE. The CTE of concrete is calculated by the following equation:

$$\mathrm{CTE} = \left(\frac{\Delta L_a}{L_o}\right)/\Delta T \tag{1}$$

where,

ΔL_a = actual length change of specimen during temperature change,
L_o = initial length of specimen at room temperature, and
ΔT = measured temperature change (increase = positive, decrease = negative).

The fabricated concrete specimen is of cylindrical shape and its dimensions are 4 in. (101.6 mm) in diameter by 8 in. (203.2 mm) in height. The concrete specimen was ground to reduce the height to 7.5 in. (190 mm) to match the height of the calibration bar provided with the CTE device.

Thermal Conductivity and Heat Capacity

Quickline-30 manufactured by Anter Corporation is multi-functional equipment used for measuring surface temperatures, thermal conductivity, heat capacity, and thermal diffusivity. This method takes a few minutes to reach steady-state conditions. The factors influencing the measurement of the readings are quality of thermal contact between the probe and specimen, temperature differences between the surface specimen and room temperature, dimensions of the sample, and moisture content. Measurement range of thermal conductivity is 0.08-2W/m-K, and the precision is ±10 percent of the reading value. Measurement temperature is -40°F to 752°F (-40°C to 400°C), and it typically takes 16-20 minutes. Figure 8 shows the schematic figure of Quickline-30.

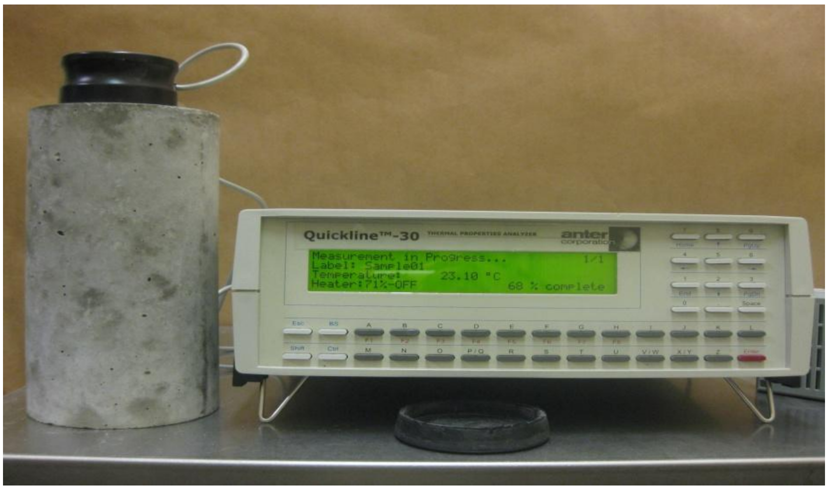


Figure 8
Experimental procedure for determining thermal conductivity and heat capacity

Concrete Mixture Design

To study the effects of different parameters on CTE, three different mixtures were designed as shown in Table 2. The three mixtures had different coarse aggregates: Kentucky limestone, river gravel, and Mexican limestone. These three aggregates were chosen because they are the most widely used in pavement construction in Louisiana. The mixtures were named with the coarse aggregate due to its dominant effects on CTE. A siliceous sand (A 133 TXI Dennis Mills) was used for fine aggregate for all of the mixtures. The percentile of coarse aggregate and fine aggregate were kept close to 64 percent and 36 percent, respectively. The same amount of type II Portland cement (Holcim) was used in all blends. A constant water-to-cement (w/c) ratio of 0.451 was used for the mixtures to minimize the effect of cement paste. Daravair 1440 and WRDA 35 were used as admixtures to provide desirable air content and workability. Fresh concrete properties were measured according to ASTM standards and are provided in Table 2. Hardened mechanical properties of the mixtures were measured at several ages to study aging effects and are presented in Table 3. Detailed discussions on the hardened mechanical properties of the concrete will follow in a later section.

Table 2
Concrete mixture designs

Mixtures	Unit	Kentucky Limestone	Gravel	Mexican Limestone
Holcim Type II (GP) Portland Cement	lbs/yd^3	475	475	475
Sand, A133 TXI Dennis Mills	lbs/yd^3	1171	1131	1149
Kentucky Limestone, AB29 Martin Marietta	lbs/yd^3	2104	–	–
Gravel, A133 TXI Dennis Mills	lbs/yd^3	–	2027	–
Mexican Limestone, AA36	lbs/yd^3	–	–	2071
% by volume Fine Aggregate	%	36.2	35.0	35.7
% by volume Coarse Aggregate	%	63.8	65.0	64.3
Water	lbs/yd^3	214	214	214
Water Cement Ratio	None	0.451	0.451	0.451
Admixture (Daravair 1400)	Dosage (oz/100ct)	0.50	0.50	0.50
Admixture (WRDA 35)	Dosage (oz/100ct)	3.50	6.40	20.00
ASTM C 1064 Air Temperature	°F	68.5	69.0	71.2
ASTM C 1064 Concrete Temperature	°F	72.0	73.5	74.6
ASTM C 143 Slump	Inches	0.25	1.50	1.25
ASTM C 231 Pressure Air Content	%	7.00	6.30	4.00
ASTM C 138 Unit Weight	lbs/ft^3	144.4	140.0	149.2
Specific gravity	None	2.69	2.53	2.62
Water absorption	%	1.0	2.2	3.5

Table 3
Mechanical properties of concrete mixtures

Coarse Aggregate	Mechanical property tests	7 days		14 days		28 days		90 days	
		Avg.	S Dev	Avg.	S Dev	Avg.	S Dev	Avg.	S Dev
Kentucky Limestone	Compressive strength (psi)	6,015	114.4	6,775	49.0	7,408	271.4	8,640	115.8
	Modulus of Elasticity (10^6psi)	5.883	0.378	5.866	0.621	6.466	0.375	6.750	0.132
	Poisson's ratio	0.23	0.03	0.27	0.03	0.26	0.02	0.26	0.02
	Flexural strength (psi)	678	112.4	925	48.1	811	12.7	809	84.9
	Splitting Tensile (psi)	497	–	528	–	456	–	594	–
Gravel	Compressive strength (psi)	3,782	72.8	4,363	101.8	4,900	172.4	6,004	376.4
	Modulus of Elasticity (10^6psi)	5.033	0.407	4.766	0.076	5.083	0.104	5.866	0.076
	Poisson's ratio	0.23	0.01	0.15	0.02	0.14	0.03	0.15	0.02
	Flexural strength (psi)	519	9.9	551	0.0	589	65.8	738	86.3
	Splitting Tensile (psi)	396	53.0	424	48.8	455	41.0	532	31.1
Mexican Limestone	Compressive strength (psi)	4,671	570.0	5,272	331.1	5,935	355.7	6,314	177.8
	Modulus of Elasticity (10^6psi)	4.150	0.086	4.600	0.050	4.550	0.086	4.633	0.076
	Poisson's ratio	0.19	0.01	0.23	0.03	0.26	0.01	0.22	0.04
	Flexural strength (psi)	559	29.0	652	26.2	686	27.6	710	106.1
	Splitting Tensile (psi)	394	54.5	423	14.1	425	2.1	433	1.4

* The average and standard deviation are based on three samples for compressive strength, modulus of elasticity and Poisson’s ratio, and two samples for flexure strength and splitting tensile test.

Effect of Aggregates Types on CTE

CTE is influenced by aggregate types in the mixture. In this research, three popular coarse aggregates used in Louisiana were chosen for practical purposes. Those aggregates were Kentucky limestone, river gravel, and Mexican limestone. Kentucky and Mexican limestone have a different origin, and Mexican limestone is more absorptive. CTE tests were performed at several ages to compare the variation of CTE depending on aggregate types.

Effect of Aging on CTE

To investigate the aging effect on CTE, cylindrical specimens were produced in the laboratory. To eliminate experimental variability, all specimens were produced from the same batch. The specimens were cured in a 100 percent moisture chamber until the time of testing. CTEs were measured at 3, 5, 7, 14, 28, 60, and 90 days for each concrete mixture and compared.

Effect of Dimension on CTE

To verify the scale effect on CTE, both cylindrical [4 in. (101.6 mm) diameter and 8 in. (203.2 mm) high] and prismatic [3 in. (76.2 mm) long, 3 in. (76.2 mm) wide, and 8 in. (203.2 mm) high] specimens for Kentucky limestone were fabricated at the same batch. The specimens were cured in a 100 percent moisture chamber until the time of testing. CTE tests were conducted at 7, 14, 28, and 60 days, and the CTE value of prismatic specimen was compared to the CTE value of cylindrical specimen.

Effect of Coarse Aggregate Proportion on CTE

The CTE is also influenced by the volume fraction of cement paste and aggregates since the CTEs of the ingredients are different. To verify it, two additional mixtures were fabricated. In the Kentucky limestone mixture, the volume of coarse aggregate was changed to 20 percent and 80 percent while keeping the total volume of aggregates constant. That means the volume of fine aggregate was changed to 80 percent and 20 percent, respectively. The 20 percent and 80 percent volume of coarse aggregate are rather extreme cases and were chosen to verify the relationship between the amount of coarse aggregates and CTE.

Effect of Relative Humidity on CTE

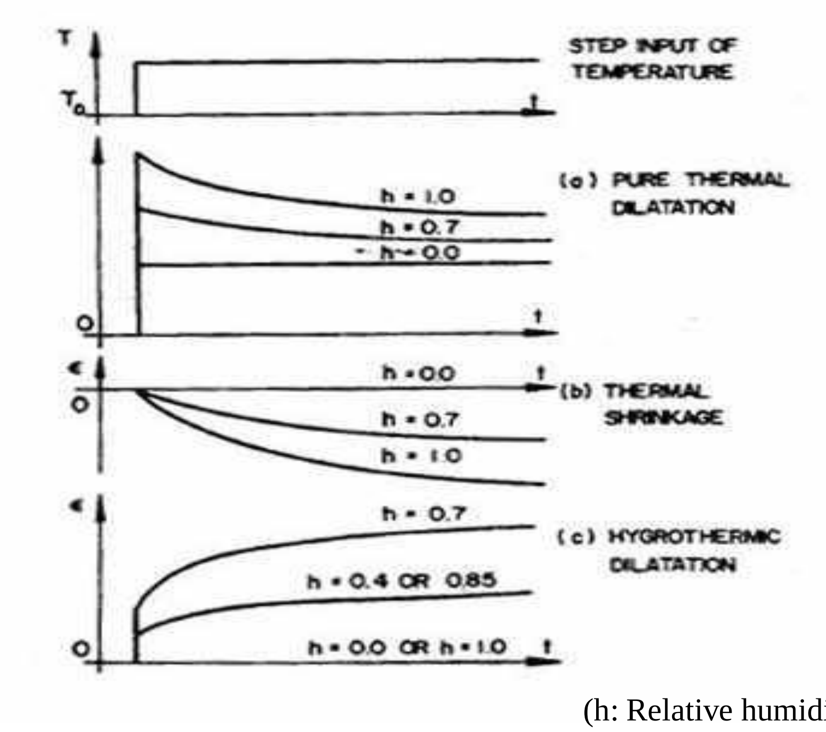
The CTE is commonly defined by a constant value in MEPDG, but it has been known that CTE varies depending on RH. The mechanism of moisture interaction is classified by three categories: (1) pure thermal dilation, (2) thermal shrinkage or swelling, and (3) relative humidity change [*13*].

1. Pure thermal dilation

   This is the dilation due to the CTE of each constituent material, such as solid particles, adsorbed water, and pore water. As temperature increases rapidly, immediate expansion of each constituent occurs and then a time dependent contraction occurs because excess pore pressure created by expansion of each constituent dissipates by moving to an empty space of pores. This phenomenon is also effective in a cooling process where immediate contraction occurs followed by a time dependent expansion. Figure 9(a) explains the pure thermal dilation with various RH where higher RH has larger amounts of both an immediate expansion and a time dependent contraction during the heating process.

2. Thermal shrinkage or swelling

   Pore water is categorized by two phases: (1) gel water is located in the interconnected spaces between the solid particles such as interlayer water and absorbed water in very small pores and (2) capillary water is free water, which induces capillary tension in partially saturated condition and its space is much larger than gel water. Increasing temperature cause the moisture to move from gel pores to capillary pores leading to shrinkage, while a cooling process drives the water from capillary pores to gel pores leading to expansion. The amount of shrinkage in the heating process increases as the RH increases since a thicker layer of gel water is prone to move easier than a thin layer of gel water as shown in Figure 9(b).



(h: Relative humidity)

Figure 9

Estimated typical response to a step input of temperature: (a) pure thermal dilation, (b) thermal shrinkage, and (c) hygrothermal dilation [*13*]

3. Relative humidity change

   Once the RH increases above 45 percent, capillary tension plays the most important role in shrinkage and dilation mechanism, while capillary tension doesn't exist below 45 percent RH due to the instability of meniscus [*13*]. Capillary tension is related to the curved capillary meniscus in the partially saturated porous materials, and the relationship is presented by using the Laplace equation:

$$P = \frac{2\gamma}{r} \tag{2}$$

where, γ is the surface tension of the poor fluid, and r is the average radius of meniscus curvature.

$$\frac{2\gamma}{r} = \frac{-\ln(RH)RT}{v'} \tag{3}$$

where, RH is the initial relative humidity, R is the universal gas constant, T is the temperature in Kelvin, and v' is the molar volume of water.

$$p = \frac{-\ln(RH)RT}{v'} \tag{4}$$

The increased temperature causes expansion of gel water, thus the radius of meniscus increases as well. The increased radius of meniscus leads to decreased surface tension and increased RH according to the Kelvin equation, which is a physicochemical equilibrium between the vapor and liquid phases in Equation (3) [*13*]. In Equation (2), the negative pressure acting on the pore system goes down when the surface tension decreases, thus decreased negative pressure on the pore system expels the solid particle away from each other as shown in Figure 10. The combination of both the Laplace and Kelvin equations provides the direct relationship between RH and pore fluid pressure in Equation (4).

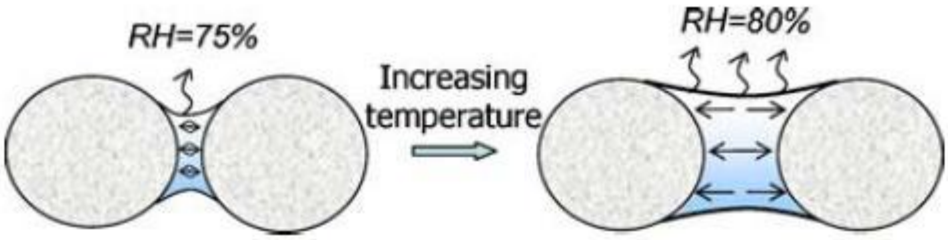


Figure 10
Dilation of solid particles caused by capillary relaxation with increasing temperature [*13*]

The combination of those three components is summarized in Figure 11. Both long-term and immediate thermal dilation due to increasing temperature show the maximum value at 70 percent RH, while dried and saturated conditions show the minimum values in long-term thermal dilation due to the absence of capillary meniscus. Thus, it has a good agreement with Grasley that the primary shrinkage and dilation mechanism is regarded as capillary tension when RH is greater than 45 percent [*14*].

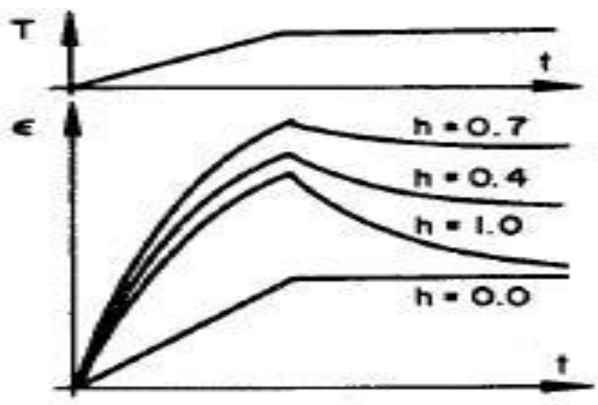


Figure 11
Combination of three components of thermal dilation for various RH [*13*]

AASHTO TP 60 clearly states that the specimen shall be conditioned by submersion in saturated limewater at 73±4°F (23±2°C) for no less than 48 hours until two successive weightings of the surface-dried sample at intervals of 24 hours show an increase in weight of less than 0.5 percent [*3*]. As mentioned earlier, the saturated condition was chosen from a practical testing point of view. In reality, PCC pavements are neither a dry nor saturated condition. Janssen found that the moisture condition at the top 2 in. (50.8 mm) of PCC pavement changes significantly [*12*]. A nonlinear gradient of moisture may cause a non-uniform CTE in PCC pavements, and a synergy effect with nonlinear temperature gradient can result in significant curling and joint problems. Therefore, it is necessary to measure the CTE corresponding to changing relative humidity inside the specimen to better understand pavement performance under changing temperatures and moisture conditions. RH was measured using the Rapid RH (ASTM F2170-02) device manufactured by Wagner Electronics as shown in Figure 12. It consists of a smart sensor probe and an RH reader. First, a hole [0.75 in. (19.1 mm) diameter and 1.75 in. (44.5 mm) deep] was drilled at the top surface of the cylindrical specimen (Mexican limestone specimen). A smart sensor probe was installed in the hole. The RH reader was inserted inside of the smart sensor and both temperature and relative humidity of the concrete specimen were measured immediately.

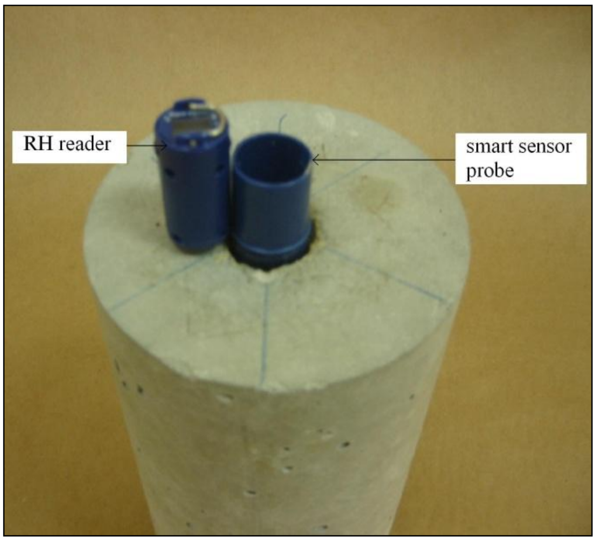


Figure 12
Relative humidity measuring device (rapid RH)

Effect of Concrete Mechanical Properties on CTE

Several mechanical property tests were conducted at 7, 14, 28, and 90 days. The mechanical property tests include compressive strength (ASTM C39), modulus of elasticity (ASTM C469), Poisson’s ratio (ASTM C469), flexure strength (ASTM C78), and splitting tensile test (ASTM C496). These properties were compared with the CTE value at the same age to discover any relationships. The mechanical properties are basic input data for the MEPDG and will be used as Level 1 input data for PCC pavement design. Based on the measured thermal input parameters, such as coefficient of thermal expansion, thermal conductivity, and heat capacity, the distresses in PCC pavement will be predicted and recommendations will be given. The mechanical properties with various ages were presented from Figure 13 through Figure 16. The data of outliers, flexural test at 14 days and splitting tensile test at 28 days of Kentucky limestone, were removed from the analysis.

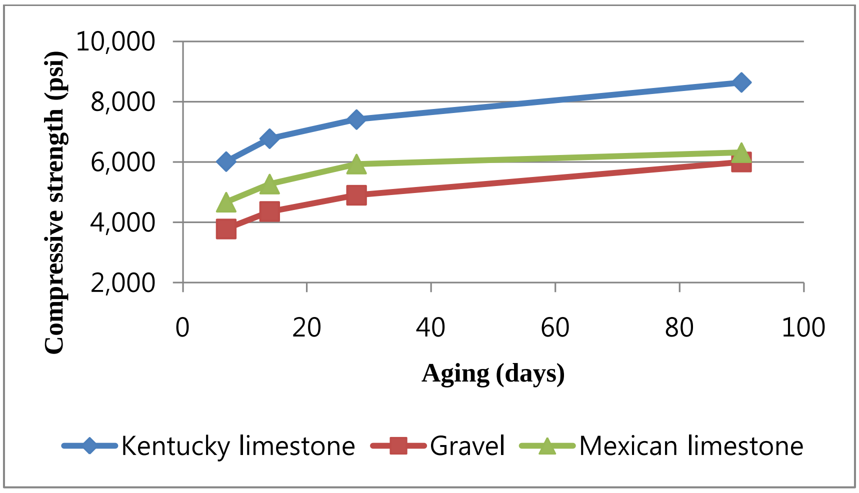


Figure 13
Compressive strength with concrete ages

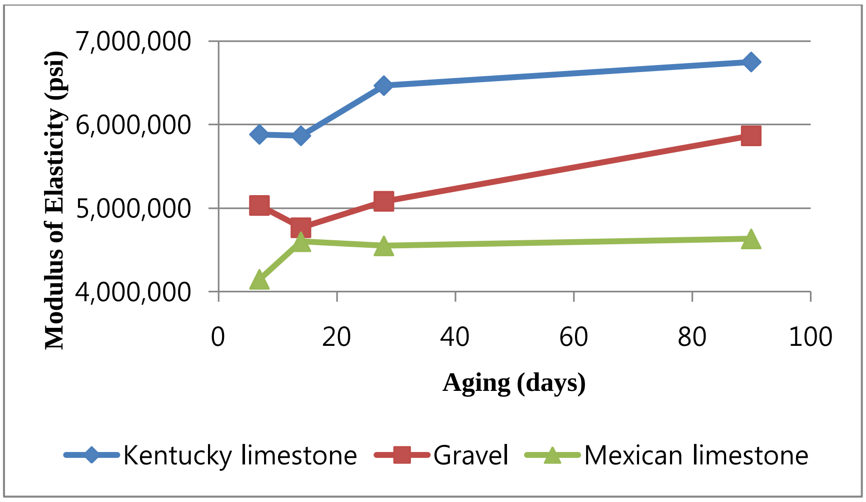


Figure 14
Modulus of elasticity with various concrete ages

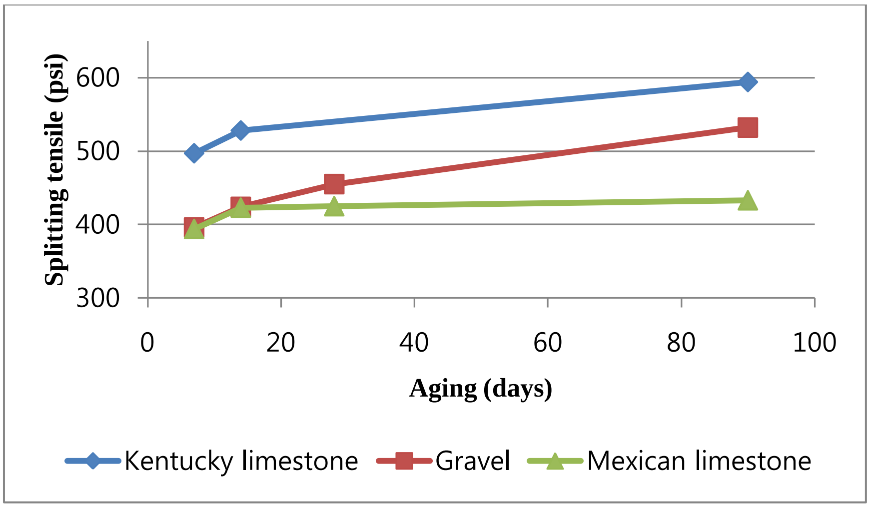


Figure 15
Splitting tensile with various concrete ages (outlier of Kentucky limestone at 28 days)

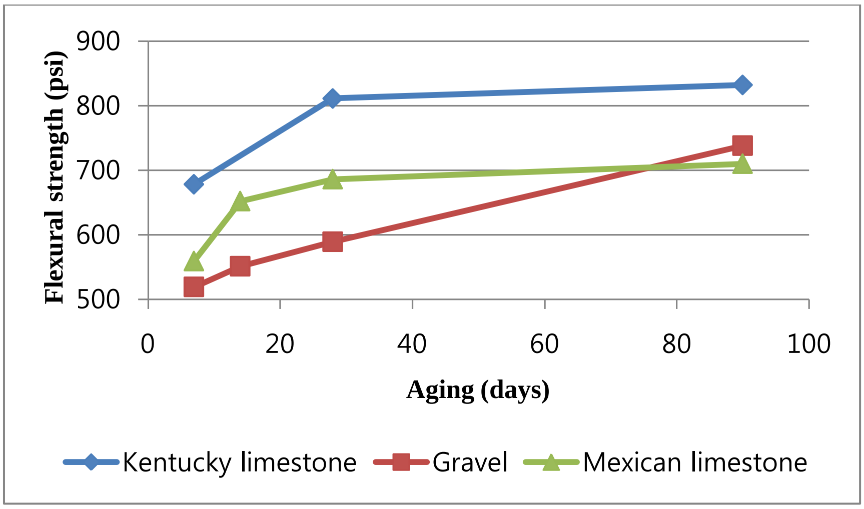


Figure 16
Flexural strength with various concrete ages (outlier of Kentucky limestone at 14 days)

Correction Factor (CF) for the Overestimated CTE

The FHWA issued a memorandum on December 2009 to take an action on the "Concrete Coefficient of Thermal Expansion Input for MEPDG" [*15*]. According to the memorandum, FHWA has identified a problem with the American Association of State Highway Transportation Official (AASHTO) TP 60-00 provisional test method used to measure the CTE for concrete. The CTE value of the reference specimen (304 stainless steel) for determining a calibration factor to account for expansion of the measuring apparatus was based on the literature values [9.6×10^{-6}/°F (17.3×10^{-6}/°C)], instead of the actual CTE value based on the temperature range specified 50°F to 122°F (10 to 50°C) in AASHTO TP 60-00 [*3*]. The use of the incorrect CTE value for the reference specimen has also resulted in a higher CTE value for the specimen being tested according to TP 60-00. In order to fix this problem, AASHTO adopted T 336-09 test standard to replace the TP 60-00 provisional test method [*16*].

There is a need to re-measure the CTE value according to AASHTO T 336-09 since the measured CTE value used the incorrect CTE value of the reference specimen. Therefore, this study was revised to measure the correct CTE of the concrete specimens in accordance to AASHTO T 336-09, and find calibration factors to convert the CTE values measured by AASHTO TP 60-00 without further measurements.

FHWA sent three reference specimens made of 304 stainless steel (304 SS) to the Precision Measurements and Instruments Corporation (PMIC) and Thermophysical Properties Research Laboratory, Inc. (TPRL) in order to determine their CTE values according to ASTM E228-06, standard test method for linear thermal expansion of solid materials with a push-rod dilatometer. ASTM E 228-06 is a widely accepted test method in the materials and aerospace industry to measure the CTE of metals. The CTE test results from the two independent laboratories are presented in Table 4.

Table 4
CTE of the reference materials from two independent laboratories

Specimens	PMIC Average CTE (10^{-6} / °C) (10 to 50 °C)	TPRL Average CTE (10^{-6} / °C) (10 to 50 °C)
304 stainless steel-Gilson reference specimen	16.2	N/A
304 stainless steel-Pine reference specimen	15.9	15.6
304 stainless steel-FHWA reference specimen	15.8	15.8

Although all three reference specimens (Gilson, Pine, and FHWA) are made of 304 SS, the range of the CTE values from two independent laboratories is between 8.7 and 9.0×10^{-6}/ °F (15.6 and 16.2 $\times10^{-6}$/ °C). AASHTO T 336-09 states that an ISO9001 or equivalent laboratory should determine the CTE of the reference specimen according to ASTM E 228-06 or ASTM E 289-04 within the temperature range of 50°F to 122°F (10 to 50°C). The CTE value of 9.0×10^{-6}/°F (16.2×10^{-6}/°C) was chosen for the CTE value of the reference specimen in this study since HM-251 was manufactured by Gilson.

AASHTO T 336-09 specifies that the reference material sample should be of the same nominal dimensions as the test samples so that no adjustment of the frame and/or the

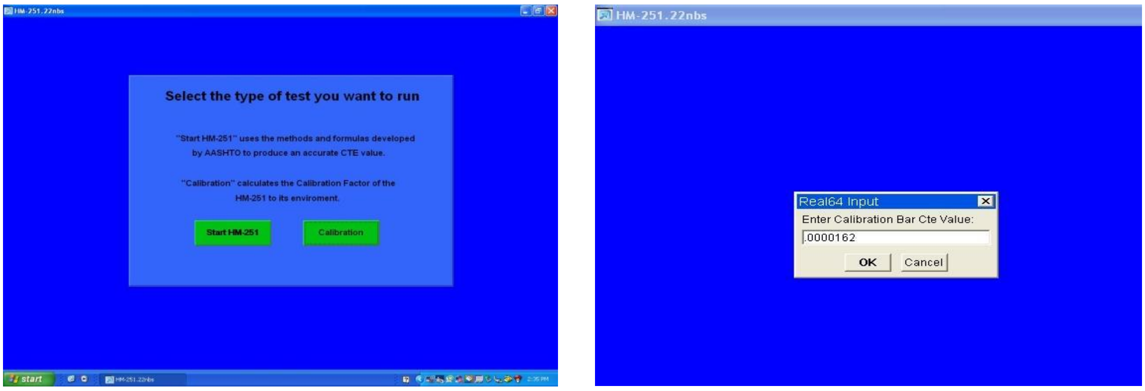
LVDT is necessary between calibration and testing. Thus, a full size reference specimen [4 in. (101.6 mm) diameter and 7 in. (177.8 mm) height] made of 304 SS [Figure 17 (b)] was purchased to calibrate the testing frame and correctly measure the concrete CTE.

(a) Old reference specimen (b) New reference specimen

Figure 17

Old and new reference specimen

The HM-251 manufactured by Gilson/Challenge technology was developed to measure CTE in accordance to AASHTO TP 60-00. The length change of the measuring frame was determined by testing a reference specimen of the known CTE value. A 304 SS was used as a reference specimen in HM-251 and the erroneous literature value [9.6×10^{-6}/°F (17.3×10^{-6}/°C)] was built in as a known CTE of the reference specimen. Thus, the Gilson/Challenge technology provided an upgraded software that allows the user to input the CTE value of the reference specimen to overcome this problem. Figure 18 shows the overview of the upgraded HM-251 software. When the calibration testing is running, click the calibration button in Figure 18 (a) and a prompted window will show up as shown in Figure 18 (b). The CTE value of reference specimen can be typed in the box. After completion of the calibration process, the typed CTE value of the reference specimen will be effective in the subsequent CTE testing.



(a) Calibration window **(b) A prompted window to enter CTE value of reference specimen**

Figure 18
Overview of upgraded HM-251 software

Five different mixtures were used for the previous chapters in this study. The first three mixtures had different coarse aggregates: Kentucky limestone (limestone from three rivers rock quarry in Kentucky); river gravel (TXI, Dennis mills); and Mexican limestone (limestone from Tampico, Mexico), all at 64 percent of coarse aggregate rate. The last two mixtures had different coarse aggregate proportions: 20 percent and 80 percent of Kentucky limestone as a coarse aggregate. The mixtures were named with the coarse aggregate type and proportion due to its dominant effects on CTE.

DISCUSSION OF RESULTS

In order to analyze test results more efficiently, an ANOVA was utilized in this research. Statistical analyses (ANOVA) of each variable were performed on the corresponding CTE, and the overall ANOVA results are summarized in Table 5.

Table 5
Summary of ANOVA results

Variables	DF	F-value	P > F	Significance
Aggregate types (KL, G, ML)	2	2852.56	< .0001	Yes
Mixture age	6	0.46	0.8195	No
Dimension of specimen	1	24.72	0.0025	Yes
Coarse aggregate proportion (57.8, 46.1, 14.5% of KL)	2	419.74	< .0001	Yes
Relative humidity (Average CTE)	5	0.78	0.5976	No
Relative humidity (expansion CTE)	6	4.40	0.0366	Yes
Concrete mechanical properties	3	0.06	0.9794	No

(KL: Kentucky Limestone, G: Gravel, ML: Mexican Limestone)

The P-value means the probability of error of the statement. A small P-value for a variable indicated that the variable has a significant effect. If the P-value of the variable is equal to or less than alpha (α), the variable is regarded as having a significant effect on measuring parameters. Alpha is a probability error level and 0.05 was used in the analysis. It should be emphasized that a statistical significance does not necessarily imply a practical significance or vice versa.

Effect of Aggregates Types on CTE

The average CTE for Kentucky limestone, gravel, and Mexican limestone concrete were 4.964 με/°F, 7.144 με/°F, and 4.900 με/°F (8.935 με/°C, 12.860 με/°C, and 8.820 με/°C), respectively. Figure 19 shows the CTE with different aggregate types at different ages.

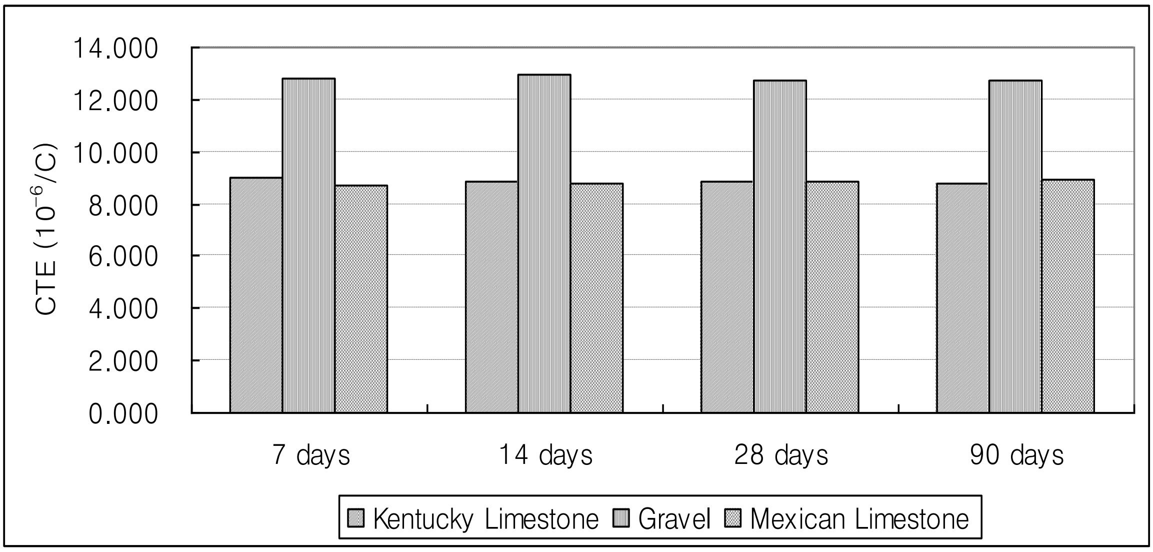


Figure 19
CTE with different aggregate types

Although Kentucky and Mexican limestone came from different sources, the average CTE values were very close. The CTE of gravel was a much higher value than the CTEs of the two limestones. It was confirmed that the effect of aggregate types have a significant influence on the CTE through ANOVA analysis (Table 5). In other words, the CTE of gravel was significantly higher than the others, but there was no significant difference between Kentucky and Mexican limestone. Higher CTE means the higher probability of pavement distresses during the design life if other conditions remain the same. From the observation, Kentucky and Mexican limestone are more desirable from a design point of view in order to minimize any thermal deformations and damages.

Effect of Aging on CTE

To investigate the effect of aging, the CTE was measured at 3, 5, 7, 14, 28, 60, and 90 days for each aggregate. Figure 20 shows the CTE at several different ages of the mixtures. The graphs showed that it fluctuated within 0.2 με/°F (0.3 με/°C) and there were no increasing or decreasing tendencies up to 90 days. It is also verified by the statistical analysis (ANOVA) that there was no significance difference due to age. The results agreed with previous findings that the effect of aging of concrete has little effect on the CTE [*4*]. However, this finding does not correspond to another research result that states the CTE at 28 days was significantly lower than CTE at 90 and 180 days [*9*].

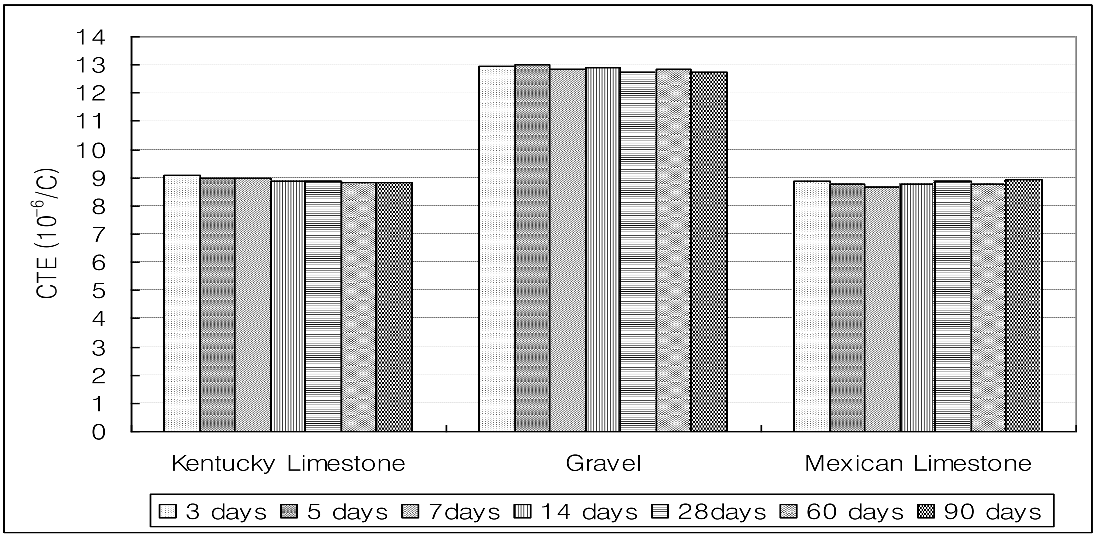


Figure 20
CTE at several different ages of mixture

Effect of Dimension on CTE

The CTE of both cylindrical [4 in. × 7.5 in. (101.6 mm × 190.0 mm)] and prismatic [3 in. × 3 in. × 7.5 in. (76.2 mm × 76.2 mm × 190 mm)] for Kentucky limestone specimens showed the similar trend that had a peak value at 7 days and decreased gradually as shown in Figure 21. The differences of the CTE in both specimens were between 0.118 με/°F and 0.185 με/°F (0.212 με/°C and 0.333 με/°C) with each age and it has a significant difference statistically. These two specimens had the same height, but a different projection area. Since 4-in. (101.6-mm) diameter cylindrical specimen is specified in AASHTO TP 60, other shapes of specimen having different projection area should be avoided.

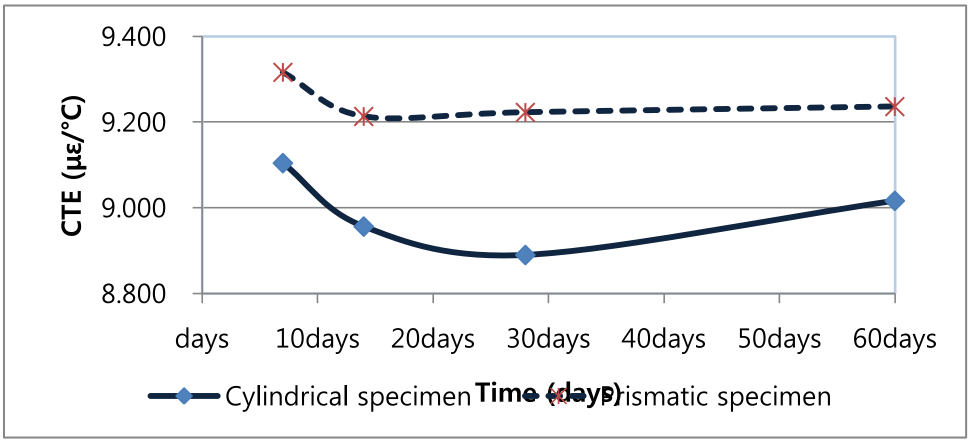


Figure 21
CTE of cylindrical and prismatic specimen

Effect of Coarse Aggregate Proportion on CTE

Gravel has a much higher CTE than limestone (Table 1). The CTE of limestone is 3.3 με/°F (6 με/°C) and gravel is between 6.1 and 7.2 με/°F (11 and 13 με/°C) [*5*]. Zoldners showed the effect of aggregate content on the thermal expansion of concrete as shown in Figure 22 [*17*]. The CTE varies depending on the types and proportion of aggregates because the volume of aggregates occupies more than 70 percent of concrete volume. Thus, the CTE of aggregates, especially coarse aggregates, predominantly control the CTE of concrete. Figure 22 explains the CTE variation depending on types of aggregates for both coarse aggregates (quartz gravel and crushed limestone) and fine aggregates (siliceous sand and crushed limestone) and proportion of coarse aggregates from 0 to 100 percent. The best combination to reduce the CTE of concrete is crushed limestone as coarse aggregate and limestone sand as fine aggregate. To study the effect of aggregate contents, Kentucky limestone and siliceous sand were chosen for coarse and fine aggregates and two additional mixtures were produced. From the Kentucky limestone mixture, the volume of coarse aggregate was changed to 20 percent and 80 percent while keeping the total volume of aggregate contents constant. That means the volume of fine aggregates was changed to 80 percent and 20 percent, respectively. Those 20, 64, and 80 percent of relative coarse aggregate volume in total aggregate can be converted into 14.5 percent, 46.1 percent, and 57.8 percent of coarse aggregate volume in concrete mixture. The results of measured CTE are shown in Figure 23.

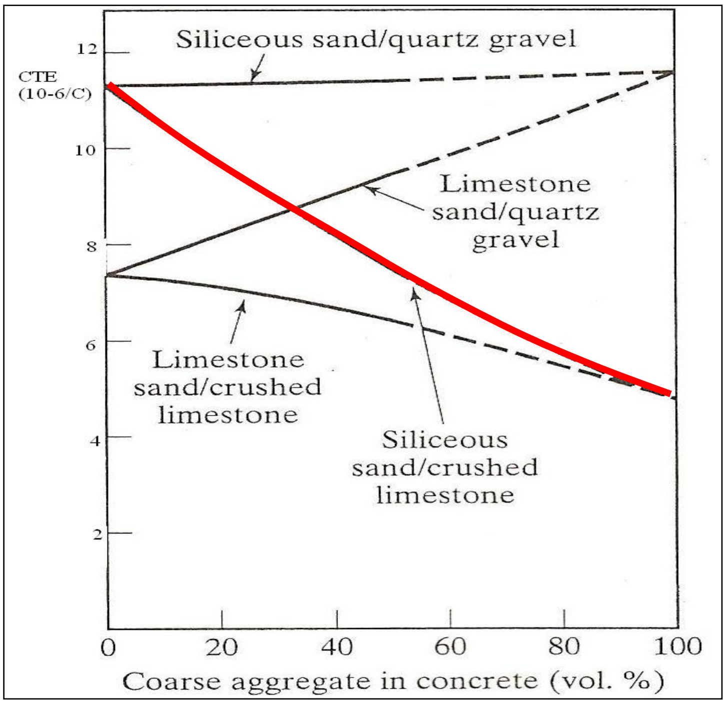


Figure 22

Effect of aggregate content on the thermal expansion of concrete [*5*]

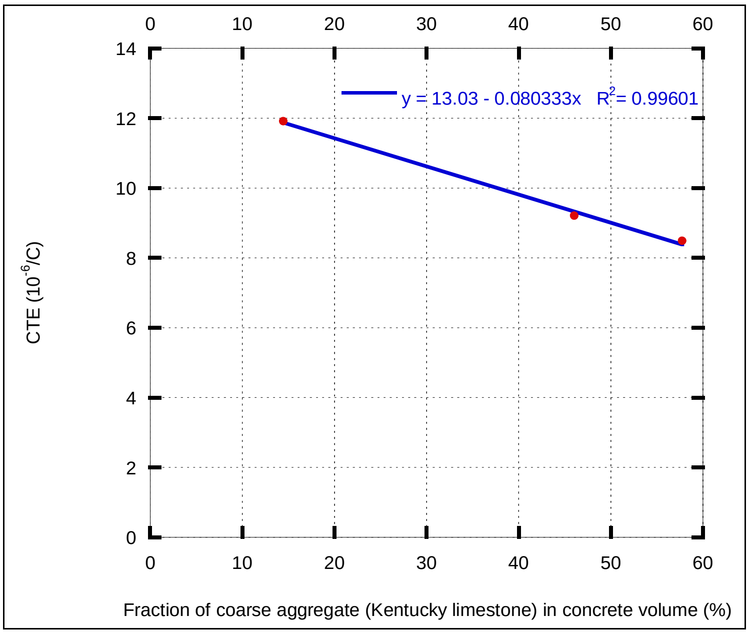


Figure 23
CTE vs proportion of coarse aggregate (Kentucky limestone) in concrete mixture

From Figure 22, the CTE starts at 6.3 με/°F (11.3 με/°C), which corresponds to 0 percent of coarse aggregate (crushed limestone). The CTE decreased steeply until it reached 2.8 με/°F (5 με/°C) when coarse aggregate (crushed limestone) is 100 percent. Similarly, in Figure 23, CTE decreased from 6.6 με/°F to 4.7 με/°F (11.9 με/°C to 8.4 με/°C) at 14.5 percent and 57.8 percent of coarse aggregate (Kentucky limestone) in concrete, respectively. These data are fit into a linear curve with 0.996 of R^2 value. The relation between the CTE and the proportion of coarse aggregate (Kentucky limestone) in a specified concrete mixture is as following:

$$Y = -0.080X + 13.03 \tag{5}$$

where,

X= Proportion of Kentucky limestone in concrete mixture (%), and

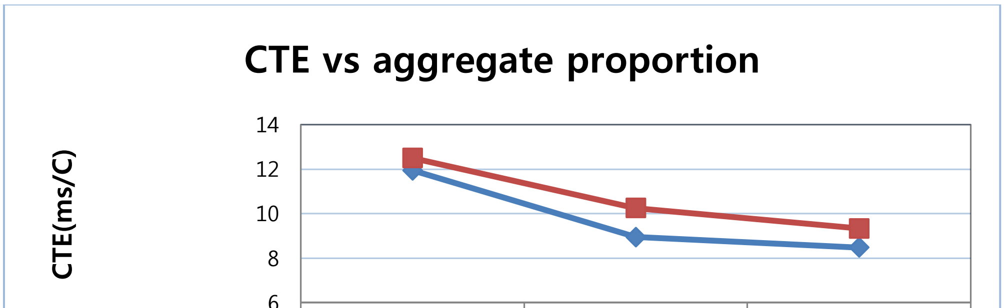
Y= CTE corresponding to the proportional Kentucky limestone (με/°C).

A statistical analysis (ANOVA) also showed that the effect of coarse aggregate proportion has a significant impact on CTE results (Table 5). CTE tests with a reasonable range of coarse aggregate contents are needed to verify this result.

Concrete is comprised of two-phase material with coarse aggregate particles embedded in a matrix of cement mortar. Hansen proposed the model to predict the modulus of elasticity for composite concrete [*18*]. Hansen considered a two-phase material consisting of spherical particles evenly distributed in a continuous matrix. The equation of the model is as following.

$$E = \left[\frac{(1-V_{agg})E_{matrix}+(1+V_{agg})E_{agg}}{(1+V_{agg})E_{matrix}+(1-V_{agg})E_{agg}}\right] E_{matix} \tag{6}$$

By replacing the modulus of elasticity with CTE in Equation (6), CTEs of composite concrete were calculated for each 20, 64, and 80 percent of relative coarse aggregate volume in total aggregate. Figure 24 shows that the comparison between the calculated CTE by Hansen's model and the measured CTE by HM-251. The calculated CTE generally had a higher CTE than the measured CTE, and the percentage of difference varied from 4.7 percent to 14.7 percent depending on the proportion of coarse aggregate.



	20% Coarse agg	64% coarse agg	80% coarse agg
Measured CTE	11.94	8.94	8.48
Calculated CTE	12.50	10.25	9.33

Figure 24
CTE calculation by Hansen's model

Effect of Relative Humidity on CTE

According to AASHTO TP 60, the specimens are conditioned in a saturated surface dry (SSD) state and submerged under water until the end of the test. In order to simulate real concrete pavement conditions with various RH, the specimens were tested at different relative humidity levels. The specimens were first placed in an oven at 140°F (60°C) for 24 hours. Using the mounted sensor probe and reader, the internal relative humidity (RH) was measured. Then the specimens were placed in a 100 percent moisture room until reaching the target relative humidity. The specimens were tested for CTE as soon as the target RH was reached. Due to the test requiring samples to be submerged underwater, the RH increased during the test. The change of RH under the water was measured as illustrated in Figure 25. The particular specimens illustrated that they started at 31 percent of relative humidity and passed 41 percent of relative humidity after 8 hours and abruptly increased to 96 percent after 15 hours. Since most CTE tests were done in 8 hours, the average change in relative humidity for 8 hours was used in further analysis.

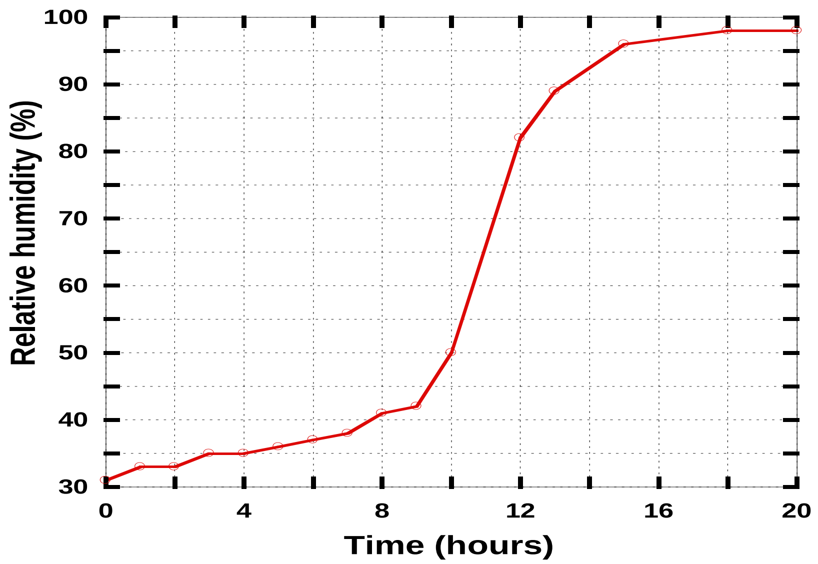


Figure 25
Change of relative humidity in water

As shown in Figure 26, the representative CTE increases gradually as the relative humidity increases to 86 percent RH. Then the CTE decreases until reaching 100 percent RH. Although the graph showed a peak CTE value between 80 percent and 90 percent RH, the variation was statistically too small to have a significant difference. In Figure 27, however, the effect of relative humidity has a significant difference on the expansion CTE by statistical analysis (ANOVA results in Table 3). Hockman and Kessler found that the length change on a heating cycle has a higher CTE than that on a cooling cycle, particularly in granite and marble [*19*] [*20*]. The heating and cooling procedures cause complex stress and slippage among mineral crystals. The permanent deformation caused by heating procedure is because the crystal fails to return its original volume due to temperature change. This permanent deformation creates a different thermal coefficient between expansion and contraction. According to Mitchell and Meyers, moisture content may cause the variation of CTE of neat cement paste by as much as 100 percent [*21*] [*22*]. The minimum value was observed in both oven-dry and saturated conditions, and the maximum value was observed at 65 percent to 70 percent RH for up to 6 months old and at 45 percent to 50 percent RH after several years. In Figure 27, the expansion and contraction CTE were obtained from the first cycle of the CTE test. Once the number of cycles increased, the difference between the expansion and contraction CTE became smaller and finally became a similar value.

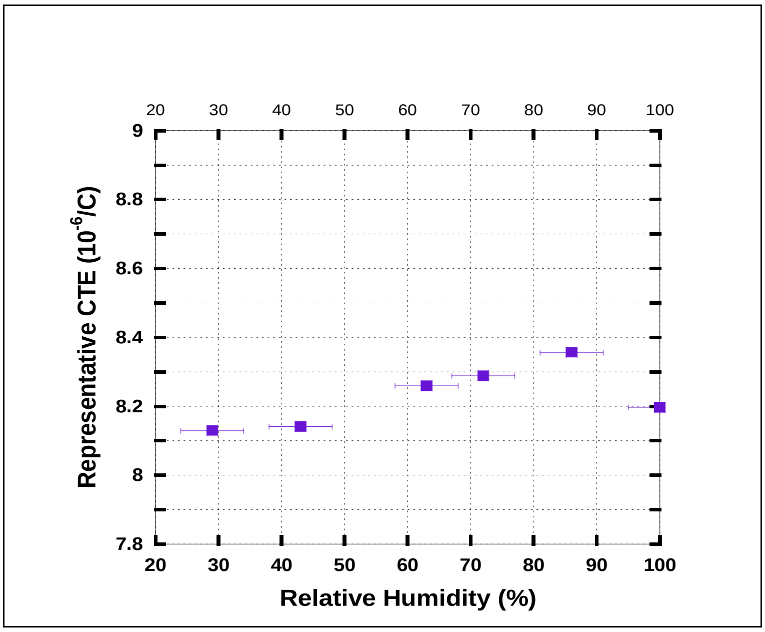


Figure 26
Representative CTE vs RH

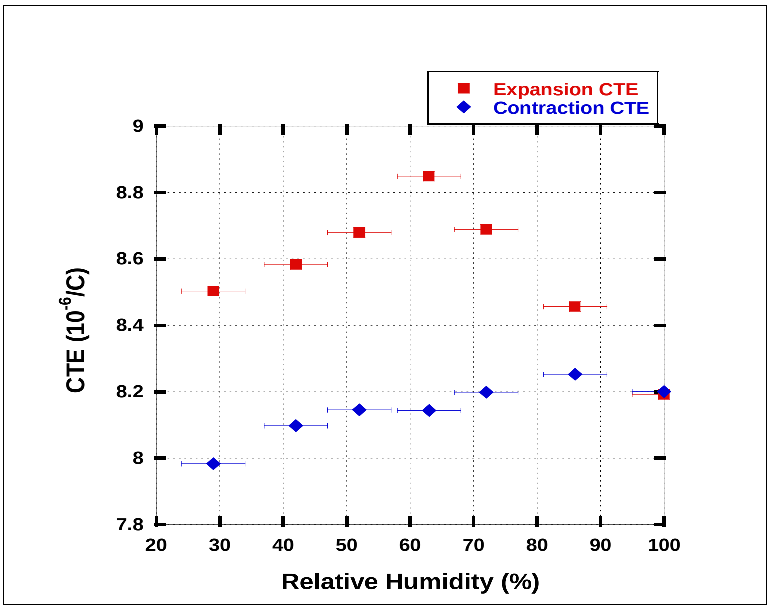


Figure 27
Expansion and contraction CTE vs RH

The relative humidity is directly related to the permeability of a concrete mixture, thus a rapid chloride permeability test (ASTM C1202) was performed to estimate the permeability of the concrete mixture. The specimen was cut 4 in. (101.6 mm) in diameter and 2 in. (50.8 mm) in height and the side of the cylindrical specimen was coated with epoxy. The specimen was put in the vacuum chamber to soak in water for 18 hours. One side (-) of the cell was filled with a 3 percent NaCl solution, while the other side (+) of the cell was filled with 0.3 normal NaOH solution. Then a 60-volt potential was applied for 6 hours. After 6 hours, the specimen was removed and the amount of coulombs that passed through the specimen was measured. The chloride permeability was classified by five categories depending on the amount of passed coulombs as shown in Table 6. The Mexican limestone and gravel specimen fell into moderate chloride permeability since the amount of charge passed was 2067 and 2226, respectively, but Kentucky limestone specimen fell into low chloride permeability because the amount of charge passed was 1936. Although they were categorized at different levels, the passed coulombs were very close to 2000. This test is not accurate enough to define the concrete permeability level precisely and should be used for only comparison purposes. To reduce the chloride permeability, material modification using fly ash, slag, or silica fume is considered an appropriate method. Figure 28 shows the apparatus of the rapid chloride permeability test.

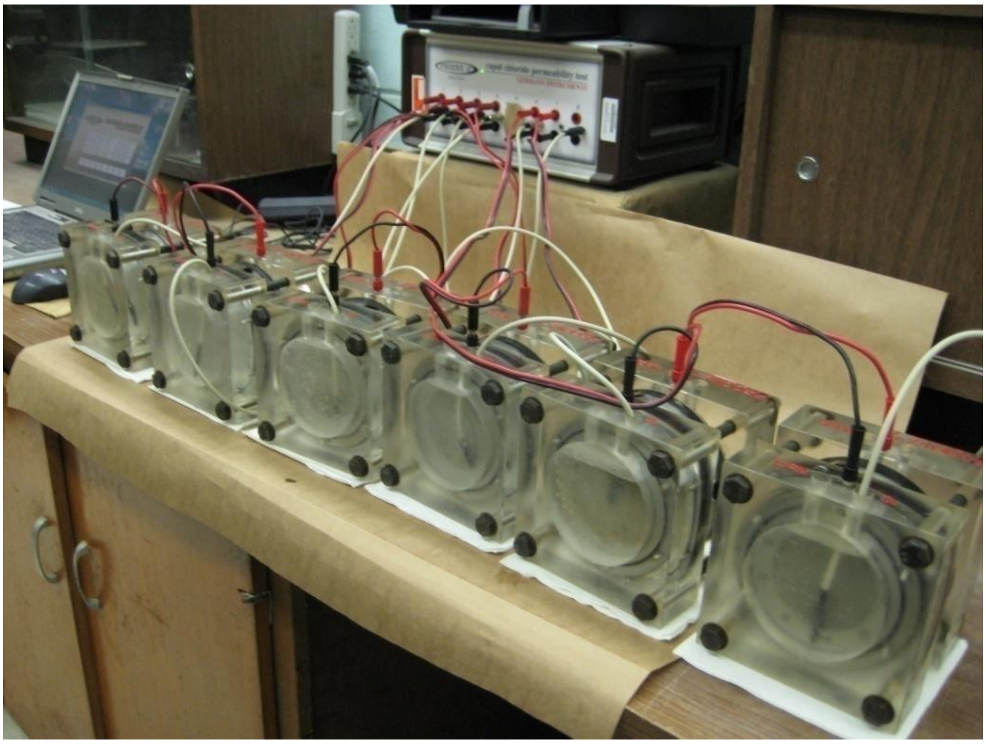


Figure 28
Rapid chloride permeability test

Table 6
Chloride permeability based on charge passed

Charge passed (coulombs)	Chloride permeability	Typical of
➢ 4000	High	High W/C ratio (> 0.6) Conventional PCC
2000 – 4000	Moderate	Moderate W/C ratio (0.40-0.50) Conventional PCC
1000 – 2000	Low	Low W/C ratio (< 0.4) Conventional PCC
100 – 1000	Very low	Latex-modified concrete or Internally-sealed concrete
< 100	Negligible	Polymer-impregnated concrete, Polymer concrete

Effect of Relative Humidity on Thermal Conductivity

Moisture content in the specimen dramatically changes the thermal conductivity of the concrete. Five samples of different aggregate contents and different compositions were selected and placed in the 50 percent humidity room and stabilized to room temperature of 73.4°F (23°C) for one day. The specimen was then weighed and the thermal conductivity of the specimen was measured. The specimen was placed in a water bath for 24 hours and then weight and thermal conductivity were measured. The specimen was placed in the water bath again and saturated until the specimen was in a fully saturated state and the test was repeated. These specimens were dried further in the oven at 116.6 °F (47°C) for about 24 hours in the same manner. Figure 29 shows that the moisture change has a linearly proportional relationship with weight change.

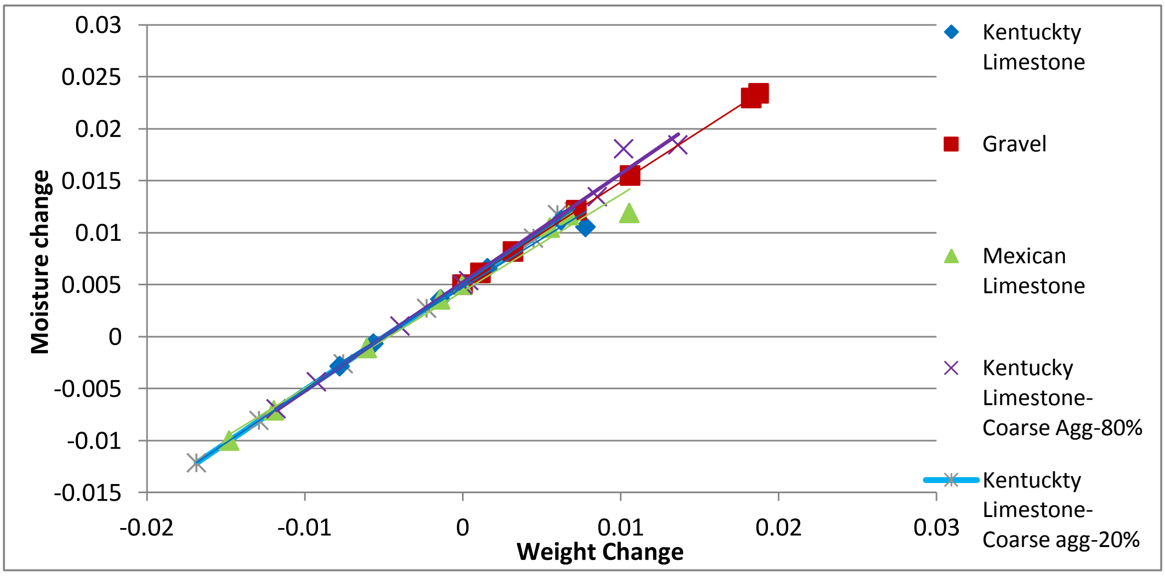


Figure 29
Relationship between moisture content and weight change

Figure 30 shows the relationship between change of thermal conductivity and weight changes in the specimen. The origin of the graph is the normal state where a standard temperature of 73.4°F (23°C) and a constant relative humidity of 50 percent were maintained. The percentile change in weight and thermal conductivity were plotted to find a trend of the thermal conductivity with respect to water content. The trend lines and regression analyses curves indicate that there is a linear relationship between the increase of thermal conductivity and weight change of the specimen. As the moisture state of specimen changes from dried state to saturated state, there is a change in the thermal conductivity value. From the results, it can be inferred that thermal conductivity of specimen increases with increase in water content. Since water is denser than air, air voids in the concrete specimen were replaced by the water, making the specimen more dense and increasing thermal conductivity.

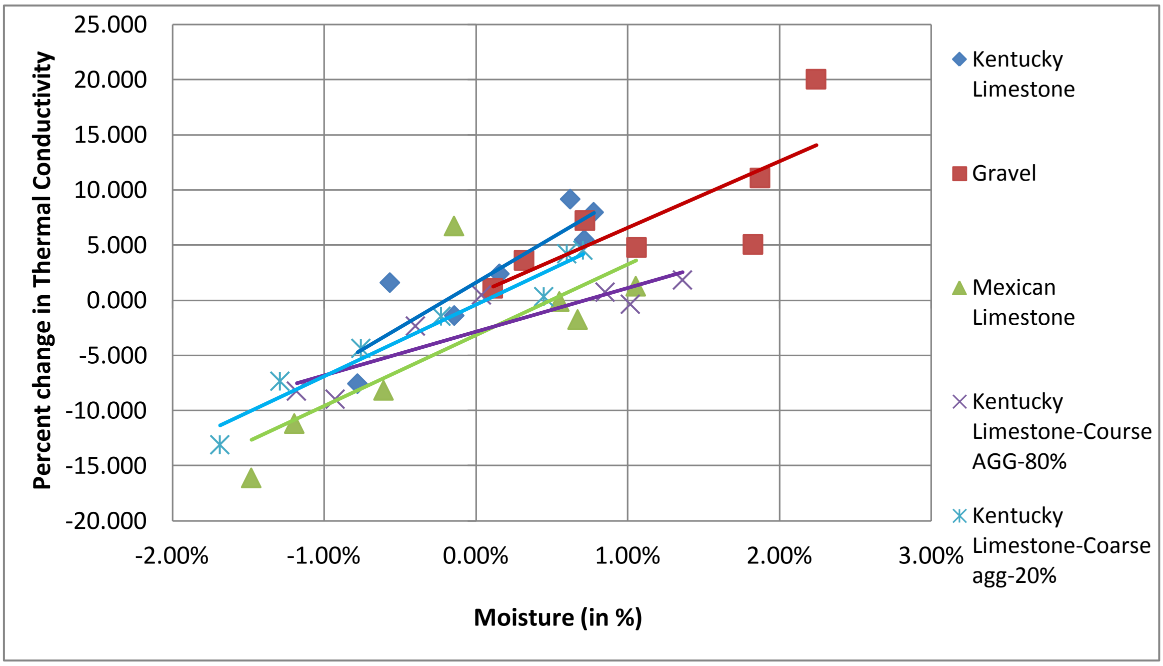


Figure 30
Relationship between thermal conductivity and weight change

Effect of Concrete Mechanical Properties on CTE

The mechanical properties measured at 7, 14, 28, and 90 days are direct input data for Level 1, 2, and 3 designs in the MEPDG software. The detailed information is shown in Table 3 and summarized input data are presented in the Appendix. The relationship between mechanical properties and the CTE of concrete specimens was investigated to easily predict CTE of concrete specimens with various aggregates. Through the statistical analysis (ANOVA), there was no significant effect of mechanical properties on CTE of concrete.

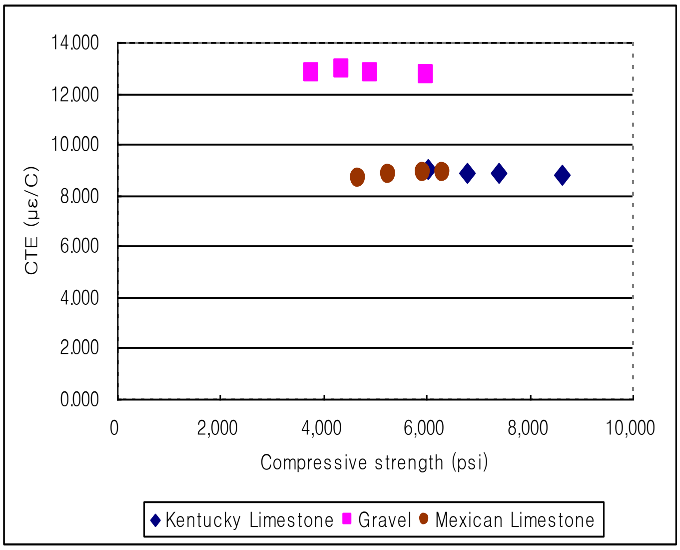


Figure 31
CTE vs compressive strength

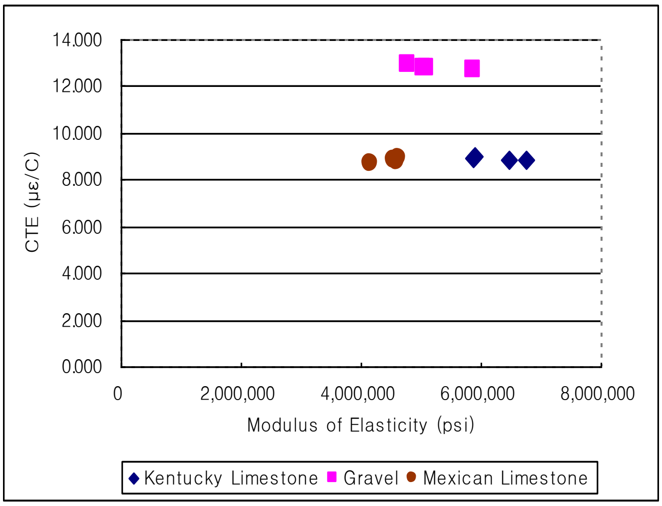


Figure 32
CTE vs modulus of elasticity

In Figure 31, the compressive strengths of Kentucky limestone, Mexican limestone, and gravel concrete have the highest, intermediate, and lowest values, respectively. Modulus of elasticity of Kentucky limestone, gravel, and Mexican limestone concrete have the highest, intermediate, and lowest values as shown in Figure 32. This is due to the characteristics of aggregates. Although gravel has a hard structure, the failure surface during compressive strength test goes around the aggregate surfaces because of the interfacial transition zone's weakness. Considering the results in Figure 31, Kentucky limestone is considered as a desirable aggregate for PCC pavements since it has a lower CTE and has higher mechanical properties than others.

Other Results

To verify the effect of water/cement ratio on CTE values, another mixture with a water/cement ratio of 0.6 was fabricated and the CTE test was performed. The CTE of higher water/cement ratio mixture was compared with that of fixed water/cement ratio (0.451) mixture. It was found that change of water/cement ratio did not affect CTE values, so the water/cement ratio did not have a significant effect on CTE values.

Seven percent silica fume was added to the mixture with a fixed water/cement ratio (0.451) to test the effect of supplementary cementing materials (SCM) on CTE. Silica fume consists of very small particles and usually provides concrete with high strength parameters and lower porosity. The concrete mixture with silica fume resulted in a higher CTE than the concrete mixture without silica fume. The difference was between 0.3 and 0.4 με/°F (0.6 and 0.7 με/°C) with various ages. The significant impact of silica fume on the CTE was also confirmed by statistical analysis (ANOVA).

Correction Factor (CF) for the Overestimated CTE

Table 7 shows the variation of CTE values with various coarse aggregate types and proportions as measured with AASHTO TP 60-00 and AASHTO T 336-09. Three replicated samples for coarse aggregate types [KL(64%), G(64%), and ML(64%)] and duplicated samples for coarse aggregate proportions [KL(20%) and KL(80%)] were used. All the CTE values with AASHTO T 336-09 are lower than with AASHTO TP 60-00. Correction factor (CF) was introduced to correlate the measured CTE values with TP 60-00 and T336-09. The range of correction factors for all five mixtures is between 0.91 and 0.96.

$$CTE_{T\ 336-09} = CF \times CTE_{TP\ 60-00} \tag{7}$$

Table 7
CTE values comparison between AASHTO TP 60-00 and AASHTO T 336-09

Specimen	CTE ($\times 10^{-6}$ /°C)		CTE difference	Correction factor (CF)
	AASHTO TP 60-00	AASHTO T 336-09		
KL (64%)	8.935	8.137	0.798	0.91
G (64%)	12.744	12.184	0.550	0.96
ML (64%)	8.729	7.900	0.829	0.91
KL (20%)	12.057	11.232	0.825	0.93
KL (80%)	8.463	7.678	0.785	0.91

Figure 33 shows the comparison of correction factors at various coarse aggregate types and proportions. The correction factor of gravel (G 64%) has the largest values while the correction factor of Mexican limestone (ML 64%) has the smallest value.

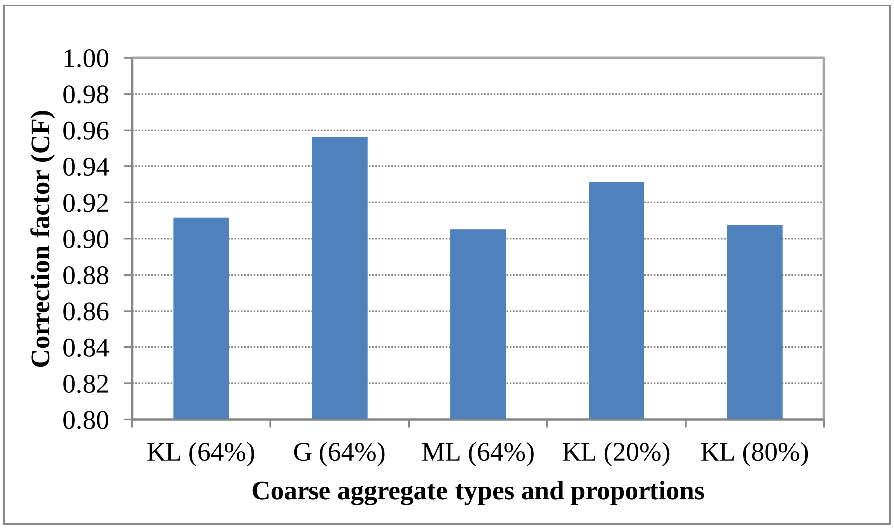


Figure 33

Comparison of CFs at various aggregate types and proportions

An analysis of variance (ANOVA) was utilized to validate the impact of both coarse aggregate type and proportion on the correction factor of CTE. The overall ANOVA results are summarized in Table 8. The P-value means the probability of error of the statement. A small P-value for a variable indicates that the variable has a significant effect. If the P-value of the variable is equal to or less than alpha (α), the variable is regarded as having a significant effect on measuring parameters. Alpha is a probability error level and 0.05 was used in the analysis.

Table 8

ANOVA results of correction factor

Variables	DF	F-value	P > F	Significance
Coarse aggregate type [KL(64%), G(64%), ML(64%)]	2	48.40	0.0002	Yes
Coarse aggregate proportion [KL(20%), KL(64%), KL(80%)]	2	3.48	0.1655	No

The statistically significance indicates that the null hypothesis (all group means are the same) is rejected. Once the null hypothesis is rejected in the results of ANOVA, it implies that at least one pair of group means are unequal. In order to determine specifically which of the means are different from one another, further analyses called multiple comparisons procedure are necessary. Tukey's procedure was conducted as a multiple comparison since it allows for all possible pair-wise tests [*23*].

The ANOVA analysis shows that the effect of coarse aggregate type has a significant impact on correction factor of CTE. Specifically, the correction factor of G(64%) was statistically different from both KL(64%) and ML(64%), and the correction factors of KL(64%) and ML(64%) were not statistically different each other. Therefore, the correction factor for both Kentucky and Mexican limestone concretes can be determined by 0.91 and for gravel concrete can be determined by 0.96 when two extreme cases [KL(20%) and KL(80%)] are excluded. The correction factors calculated in Table 7 can be used to produce correct CTE values for duplicated specimen without further measurement.

MEPDG Analysis (version 1.0)

To predict the impact of the CTE on the performance of concrete pavement, an analysis was conducted using the MEPDG (version 1.0) and CTE values measured according to the AASHTO TP-60. MEPDG provides concrete pavement distresses such as mean joint faulting, transverse cracking, and terminal IRI. MEPDG generally has three levels of design that are related to the reliability level of design. Level 1 design requires all the material properties through laboratory and field testing to obtain the highest accuracy. Level 2 design provides intermediate accuracy and the results are similar to the AASHTO pavement design guide. The input data can be collected from an agency database. Level 3 design produces the lowest accuracy and the input data are typically default values or historical data. This study is targeted to the Level 1 design, so the thermal properties (the coefficient of thermal expansion, thermal conductivity, and heat capacity) and concrete mechanical properties were tested at each mixture and designated ages. Both thermal and mechanical properties used in the analysis are summarized in Appendix B.

MEPDG requires many inputs to perform successful JPCP design, thus the input data were determined for a JPCP project on US 61, West Feliciana Parish, LA. The overview of the input window in MEPDG software is presented in Appendix D.

- Design life: 20 years
- Slab thickness: 10 in. (254 mm)
- Traffic: 1379 average annual daily truck traffic (AADTT)
- PCC flexural strength and modulus of elasticity: Kentucky limestone and gravel at 7, 14, 28, and 90 days
- Transverse joint spacing: 15, 18, and 20 ft. (4.6, 5.5, and 6.1 m)
- PCC CTE, thermal conductivity, and heat capacity: Kentucky limestone (4.96 με/°F, 1.451 BTU/h·ft·F, 0.282 BTU/lb·F) and gravel (7.14 με/°F, 1.601 BTU/h·ft·F, 0.273 BTU/lb·F)

- Layers: JPCP [10 in. (254 mm)], crushed stone [4 in. (101.6 mm)], soil cement [6 in. (152.4 mm)], and cement treated 6% [(8 in.(203.2 mm)]
- Climate: interpolated among New Orleans, Baton Rouge, and Lafayette.

Mean joint faulting increases linearly as joint spacing increases for both Kentucky limestone and gravel in Figure 34. It is clear that mean joint faulting is higher for longer joint spacing and higher CTE because of severe curling deflection in longer joint spacing. Neither Kentucky limestone nor gravel exceeds the specification for all joint spacing. The effect of CTE and joint spacing on the transverse cracking is presented in Figure 35. Gravel has a higher percentage of transverse cracking than Kentucky limestone in all joint spacing due to the higher CTE and lower strength parameters. Even with 15 ft. (4.6 m) joint spacing, the percent of transverse cracking of gravel exceeds the specification because of the high CTE of gravel. It is obvious that transverse cracking is very sensitive to the CTE value. The percentage of transverse cracking of Kentucky limestone increases slightly from joint spacing 15 to 18 ft. (4.6 to 5.5 m), but it increases dramatically from joint spacing 18 to 20 ft. (5.5 to 6.1 m). Only 15 and 18 ft. (4.6 and 5.5 m) joint spacing of Kentucky limestone satisfies specified limits. The terminal IRI also shows similar trends to previous results since smoothness is directly connected to joint faulting and transverse cracking. Increased CTE and longer joint spacing causes increased IRI as shown in Figure 36. The Kentucky limestone for all joint spacing and gravel for 15 ft. (4.6 m) joint spacing meet the specification. The specified limits of target distresses in MEPDG are 0.12 in. (3.0 mm), 15 percent, and 172 in/mile for mean joint faulting, transverse cracking, and terminal IRI, respectively. Based on the results of MEPDG analysis, 15 and 18 ft. (4.6 and 5.5 m) joint spacing for Kentucky limestone satisfy the specification of all three PCC pavement distresses. Thus, 20 ft. (6.1 m) joint spacing used in Louisiana should be shortened to 15 or 18 ft. (4.6 or 5.5 m) joint spacing when using Kentucky limestone as a coarse aggregate. The reliability summary of MEPDG results for Kentucky limestone with 18 ft. (5.5 m) joint spacing is presented in Appendix E.

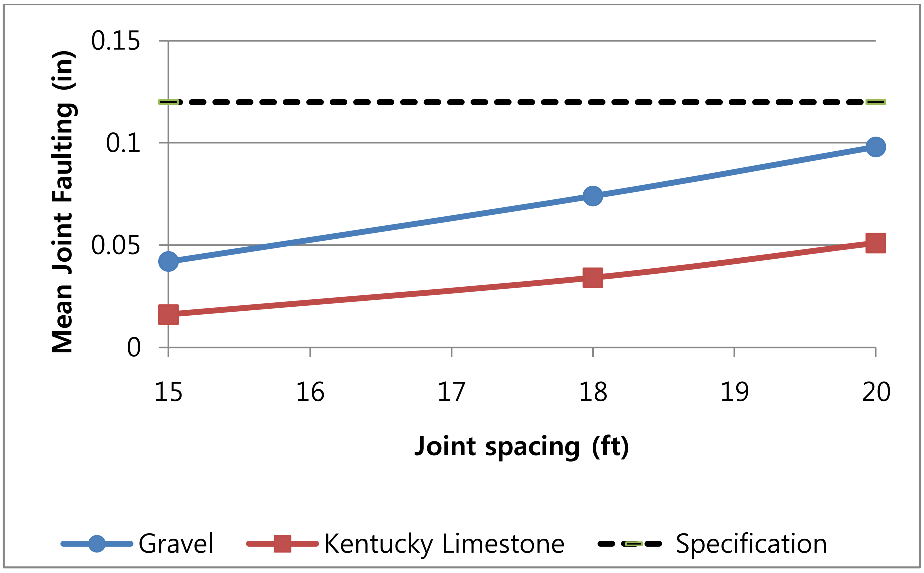


Figure 34
Mean joint faulting vs joint spacing

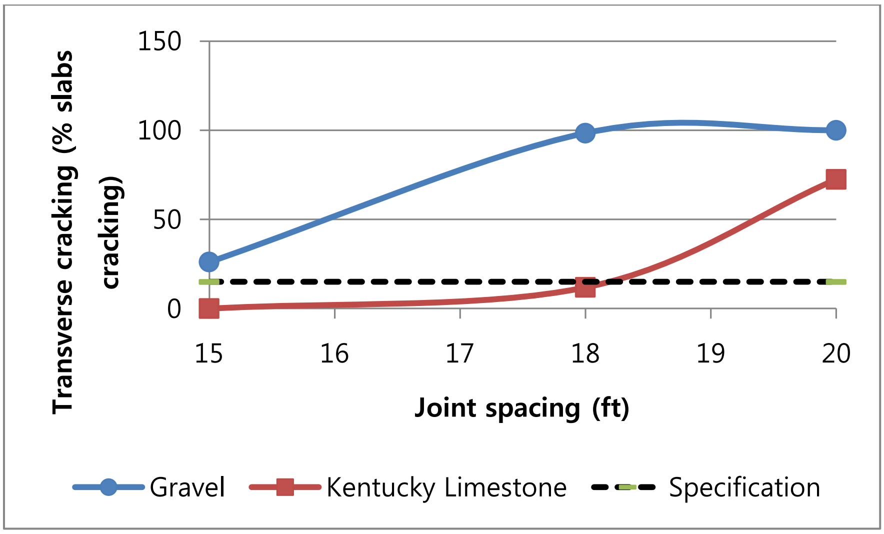


Figure 35
Transverse cracking vs joint spacing

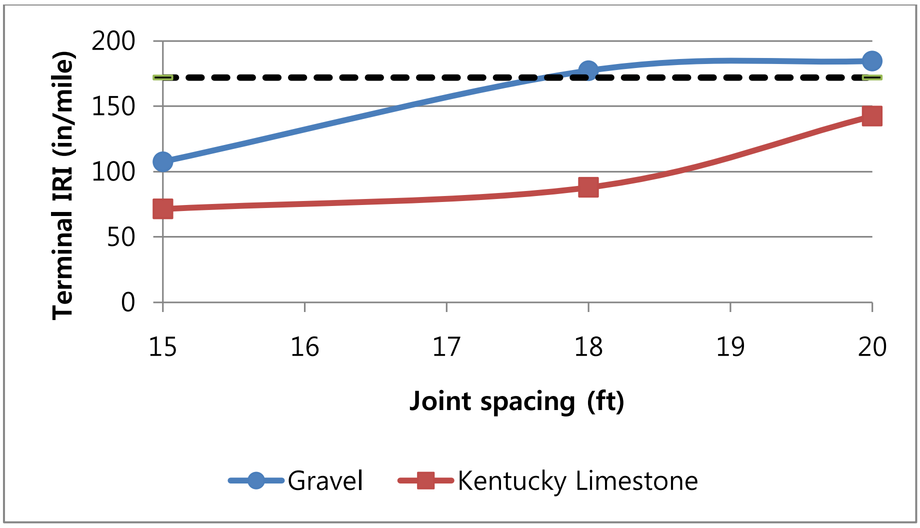


Figure 36
Terminal IRI vs joint spacing

A memorandum issued by the Federal Highway Administration (FHWA) extended the caution for state highway agencies (SHAs) to use the CTE value as an input for the AASHTO Mechanistic-Empirical Pavement Design Guide, interim edition [*15*]. As many researchers identified, the CTE value is a sensitive input for the MEPDG and will affect concrete pavement design. The interim edition of MEPDG was calibrated with the LTPP database with an incorrect CTE value measured according to AASHTO TP 60-00. The corrected CTE value using the correct calibration factor provided in AASHTO T 336-09 can result in a significant bias in the predicted distresses when comparing to the measured values in the LTPP database. The FHWA adjusted the LTPP database of the CTE value based on the changes described above. A research proposal has been approved for NCHRP 20-07 funding to recalibrate the concrete model in the MEPDG to account for the change in CTE values. According to FHWA, the following recommendations are provided to implement the MEPDG for rigid pavement design: (1) the recommendation on the joint spacing in concrete pavements in Louisiana should be further reevaluated once a DARWin-ME is published, and (2) do not interchange the CTE values from AASHTO TP 60-00 and AASHTO T 336-09 with different versions of the software. Changing the concrete CTE input in the MEPDG without recalibrating the models will negatively impact the resulting design.

Stress Analysis Caused by Non-linear Temperature and Moisture Gradient

Temperature and moisture variation through slab thickness cause curling and warping deformation in PCC pavement as shown in Figure 37. The curling and warping stresses, which are combined with traffic load, predominantly affect the performance of PCC pavement.

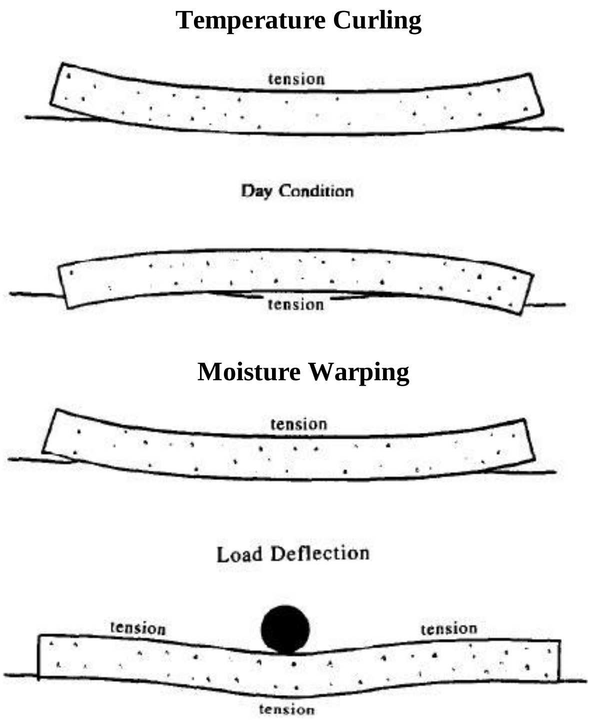


Figure 37
PCC pavement deformation under temperature, moisture, and loading [*24*]

Westgaard and Bradbury solved the stress of concrete slab subjected to a linear temperature gradient that the stresses due to temperature gradient may be as high as the stresses due to traffic load [*25*], [*26*]. However, the actual distribution of both temperature and moisture content gradient through the slab thickness is non-linear [*11*], [*12*]. By assuming a typical non-linear temperature and moisture gradient with an 8 percent increase of CTE at 63 percent RH comparing to saturated condition obtained from this study, the tensile stress at the top surface of PCC pavement increases by 12 percent in the morning as shown in Figure 38. When the moisture gradient with a 20 percent increase of CTE cited from Simon is compared to a saturated condition, the tensile stress at the top surface of PCC pavement remarkably increases by 29 percent in the morning as presented in Figure 38 [*8*]. Those increases of tensile stress at the top surface of PCC pavement in the morning can dramatically change the reliability prediction of PCC pavement in MEPDG.

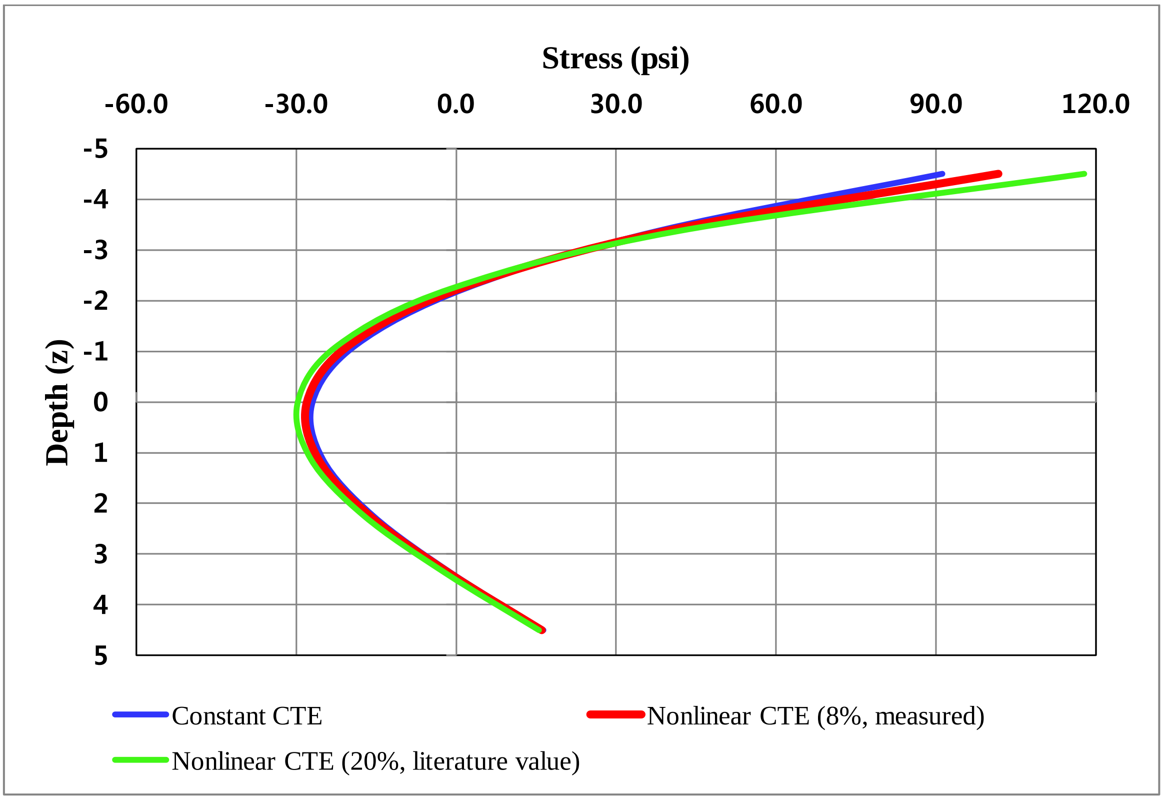


Figure 38
Stress distribution through the slab thickness

CONCLUSIONS

With the development of MEPDG as a new pavement design tool, thermal properties of PCC pavements, especially the CTE, are now considered more important design parameters in estimating pavement performance including cracking, faulting, and IRI. This study was developed to measure typical CTE values of the concrete mixture used for PCC pavement structures with various aggregates used in Louisiana and to investigate the relationship between the CTE and other critical variables such as aggregate types, age of concrete, dimension of specimen, amount of coarse aggregate, relative humidity, and concrete mechanical properties. Based on the measured CTE and MEPDG analysis, the following conclusions were drawn:

- The results of the MEPDG (version 1.0) analysis show that joint spacing of 15 and 18 ft. (4.6 and 5.5 m) for Kentucky limestone are in compliance with the reliability of slab cracking, faulting, and IRI. Thus, the current maximum transverse joint spacing [20 ft. (6.1 m)] in Louisiana should be adjusted by coarse aggregate types following the results of the MEPDG analysis. This result should be reevaluated with DARWin-ME using the CTE values measured according to AASHTO T 336-09.
- CTE for Kentucky limestone, gravel, and Mexican limestone concrete were 4.964 $\mu\varepsilon/°F$, 7.144 $\mu\varepsilon/°F$, and 4.900 $\mu\varepsilon/°F$ (8.935 $\mu\varepsilon/°C$, 12.860 $\mu\varepsilon/°C$, and 8.820 $\mu\varepsilon/°C$), respectively. Aggregate types have a statistically significant impact on CTE.
- Measured CTEs at various ages (3, 5, 7, 14, 28, 60, 90 days) fluctuated within 0.2 $\mu\varepsilon/°F$ (0.3 $\mu\varepsilon/°C$), and the age of concrete was statistically found to have no significant effect on CTE.
- The CTE between cylindrical and prismatic specimen has a statistically significant difference, thus 4-in. (101.6-mm) nominal diameter cylinders should be used for measuring CTE to follow AASHTO TP 60.
- As the amount of coarse aggregate (Kentucky limestone) increases with decreasing fine aggregate (siliceous sand), the measured CTE decreases accordingly. Statistical analysis (ANOVA) showed that the CTE of concrete is significantly influenced by the amount of coarse aggregate. The impact of coarse aggregate percentile on CTE should be considered at the mixture design and construction stage to minimize thermal distresses in PCC pavement.
- Relative humidity changes the CTE of concrete. Representative CTE shows the peak point around 85 percent humidity, but it did not have statistical significance. However, expansion CTE shows a clear peak point around 60 percent of relative humidity and has a statistical significance. Concrete mechanical properties are

essential to the MEPDG software, but its relationship with concrete CTE was not found.

- With an 8 percent increase of CTE at 63 percent RH compared to a saturated condition under the same non-linear temperature gradient, tensile stress at the top surface of PCC pavement increases by 12 percent in early morning. Higher CTE value causes high tensile stress at the top surface of PCC pavement.
- The CTE values according to AASHTO TP 60-00 can be corrected to comply with the AASHTO T 336-09 with the correction factor of 0.91 for limestone concretes (both Kentucky and Mexican limestone) and 0.96 for gravel concrete.

RECOMMENDATIONS

Kentucky and Mexican limestone have a higher compressive strength and lower CTE value compared to gravel. Although both limestones satisfy the required compressive strength at 28 days (4000 psi), Kentucky limestone is considered as a better coarse aggregate in concrete mixture which is susceptible to thermal distresses in the PCC pavement due to its higher compressive strength and low water absorption.

Three joint spacings [15-, 18-, 20-ft. (4.6-, 5.5-, 6.1-m)] joint spacing] and two coarse aggregates (Kentucky limestone, and gravel) were analyzed in MEPDG. Only 15- and 18-ft. (4.6- and 5.5-m) joint spacing of Kentucky limestone satisfied the specification for all distress types. Even with the shortest joint spacing [15 ft. (4.6 m)], gravel exceeded the specification for transverse cracking, so transverse cracking is the most sensitive distress to the CTE. According to an AASHTO Research and Communication (RAC) survey, most states have joint spacing of JPCP less than 20 ft. (6.1 m), for example, 15 ft. (4.6 m) for Arkansas, between 13- to 17-ft. (4.0- to 5.2-m) for Arizona, 15 ft. (4.6 m) for Georgia, and 15 ft. (4.6 m) for Missouri. Considering both the MEPDG analysis and the case study of other states, current maximum joint spacing [20 ft. (6.1 m)] can be adjusted to 15- or 18- ft. (4.6- or 5.5-m) joint spacing when Kentucky limestone is used as coarse aggregate.

The recommendation on the joint spacing in concrete pavements in Louisiana should be further reevaluated once a DARWin-ME is published. The CTE values measured from AASHTO TP 60-00 and AASHTO T 336-09 should not be interchanged with different versions of the software. The CTE values measured according to AASHTO TP 60-00 should be used for only the MEPDG version 1.xx software. The CTE values measured according to AASHTO TP 60-00 can be corrected to comply with the AASHTO T 336-09 using the correction factor developed in this study, and corrected CTE values can be used for the DARWin-ME software that is currently not available. Changing the concrete CTE input in the MEPDG without recalibrating the models will negatively impact the resulting design.

A future study can be focused on the nonlinear stress effect on curling and failure in PCC pavement. Measurement of RH gradients and temperature through slab thickness are necessary to understand curling behavior of PCC pavements. With the measured nonlinear RH gradient in the slab thickness, CTE gradients in the slab thickness can be predicted. The nonlinear stress analysis in the PCC pavements can be performed using the nonlinear temperature and CTE gradients, and the results should be verified with the measured curling behavior in the pavements. When there is a significant impact of nonlinear temperature and CTE gradients, implementation of MEPDG is recommended.

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

AADTT	Average Annual Daily Truck Traffic
AASHTO	American Association of State Highway and Transportation Officials
ANOVA	Analysis of Variance
CF	Correction Factor
CTE	Coefficient of Thermal Expansion
FHWA	Federal Highway Administration
IRI	International Roughness Index
JPCP	Jointed Plain Concrete Pavements
LADOTD	Louisiana Department of Transportation and Development
LTPP	Long Term Pavement Performance
LTRC	Louisiana Transportation Research Center
LVDT	Linear Variable Displacement Transducer
MEPDG	Mechanistic Empirical Pavement Design Guide
NCHRP	National Cooperative Highway Research Program
PCC	Portland cement concrete
PMIC	Precision Measurements and Instruments Corporation
RAC	Research and Communication
RH	Relative Humidity
SCM	Supplementary Cementing Materials
SHA	State Highway Agencies
SSD	Saturated Surface Dry
TPRL	Thermophysical Properties Research Laboratory
WLCR	Water Level Control Reservoir

REFERENCES

1. ARA, Inc., ERES Consultants Division, *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures*, NCHRP Project 1-37A, Final Report, Applied Research Association, Inc., ERES Consultants Division, 2004.

2. Mallela, J.; Abbas, A.; Harman, T.; Chetana, R.; Liu, R.; and Darter, M. *Measurement and Significance of the Coefficient of Thermal Expansion of Concrete in Rigid Pavement Design*, *TRR: Journal of the Transportation Research Board* No. 1919, Washington D.C., 2005.

3. AASHTO, *Standard Test Method for the Coefficient of Thermal Expansion of Hydraulic Cement Concrete*, TP60-00, 2000.

4. Won, M. *Improvements of Testing Procedures for Concrete Coefficient of Thermal Expansion*, TRR: *Journal of the Transportation Research Board* No. 1919, Washington D.C., 2005.

5. Mindess, S.; and Young, J.F.; and Darwin, D. *Concrete*, 2nd ed. Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 2003.

6. Alungbe, G.D.; Tia, M.; and Bloomquist, D.G. *Effects of Aggregate, Water/Cement Ratio, and Curing on the Coefficient of Linear Thermal Expansion of Concrete*, TRR 1335, 1992, pp. 44-51.

7. Kohler E.; Alvarado, R.F.; and Jones, D. *Measurement and Variability of Coefficient of Thermal Expansion for Concrete Pavements*, Proceeding of the 86th Transportation Research Board Annual Meeting, Washington D.C., 2006.

8. Simon, M.J., and Dallaire, M.P. *Taking Concrete to the Next Level*, Public Roads, Vol. 66, No. 1, 2002. http://www.tfhrc.gov/pubrds/02jul/01.htm. Accessed April 10, 2009.

9. Jahangirnejad, S.; Buch, N.; Kravchendo, A. *A Laboratory Investigation of the Effects of Aggregate Geology and Sample Age on the Coefficient of Thermal Expansion of Portland Cement Concrete*, Proceeding of the 87th Transportation Research Board Annual Meeting, Washington D.C., 2008.

10. Hossain, M.; Khanum, T.; Tanesi, J.; Scheiber, G.; and Montney, R. *The PCC Coefficient of Thermal Expansion Input for the Mechanistic-Empirical Pavement Design Guide*, Proceeding of the 85th Transportation Research Board Annual Meeting, Washington D.C., 2006.

11. Thompson, M. R., Dempsey, B. J., Hill, H., and Vogel, J. *Characterizing Temperature Effects for Pavement Analysis and Design*, Transportation Research Record, 1121, 1987, pp. 14-22.

12. Janssen, D.J. *Moisture in Portland Cement Concrete*, Transportation Research Record 1211, 1987, pp. 40-44.

13. Bazant, Z.P. *Delayed Thermal Dilations of Cement Paste and Concrete Due to Mass Transport*, Nuclear Engineering and Design, Vol. 14, 1970, pp. 308-318.

14. Grasley, Z.C. and Lange, D.A. *Thermal Dilation and Internal Relative Humidity of Hardened Cement Paste*, Materials and Structures, Vol. 40, 2007, pp. 311-317.

15. FHWA, *Concrete Coefficient of Thermal Expansion Input for MEPDG*, Memorandum, Federal Highway Administration, Washington D.C., Dec. 2009.

16. AASHTO, *Standard Test Method for the Coefficient of Thermal Expansion of Hydraulic Cement Concrete*, T 336-09, 2009.

17. Zoldners, N.G. *In Temperature and Concrete*, SP-25, American Concrete Institute, Detroit, MI., 1971, pp. 1-31.

18. Hansen, T.C. *Influence of Aggregate and Voids on Modulus of Elasticity of Concrete, Cement Mortar, and Cement Paste*, *ACI Journal*, Vol. 62, No. 2, Feb. 1965, pp. 193-216.

19. Hockman, A. and Kessler, D.W. *Thermal and Moisture Expansion Studies of Some Domestic Granite*, National Bureau of Standards, RP2087, 1950.

20. Kessler, D.W. *Growth of Marble as a Basis for Durability Tests*, Technical News Bulletin, National Bureau of Standards, March, 1948, Vol. 32, pp. 28-29.

21. Mitchell, L.J. *Thermal Expansion Tests on Aggregates*, Neat Cements, and Concrete, Proceeding, ASTM, V. 53, 1953, pp. 963-976.

22. Meyers, S.L. *How Temperature and Moisture Changes May Affect Durability of Concrete*, Rock Products, August, 1951, Vol. 54, No. 8, pp. 153-157.

23. Devore, J., *Probability and Statistics for Engineering and the Science*, 6^{th} ed., Thompson Learning, Inc., Belmont, CA, 2004.

24. Mohamed, A.R. and Hansen, W. *Prediction of Stress in Concrete Pavement Subjected to Non-linear Gradients*, Cement and Concrete Composites, Vol. 18, 1996, pp. 381-387.

25. Westergaard, H.M. *Computation of Stress in Concrete Roads*, In process of the Fifth Annual Meeting, Vol. 5, Part 1, Highway Research Board, pp. 90-112, 1926.

26. Bradbury, R.D. *Reinforced Concrete Pavement*, Wire Reinforcement Institute, Washington D.C., 1947.

APPENDIX

APPENDIX A

AASHTO TP-60

Standard Method of Test for

Coefficient of Thermal Expansion of Hydraulic Cement Concrete

AASHTO Designation: TP 60-00 (2006)[1]

1. SCOPE

1.1. This test method covers determination of the coefficient of thermal expansion (CTE) of hydraulic cement concrete cores or cylinders. Since it is known that the degree of saturation of concrete influences its measured coefficient of thermal expansion, the moisture condition of the concrete specimens must be controlled. For this test procedure, the specimens must be in a saturated condition.

1.2. The values stated in SI units shall be regarded as the standard.

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:

- R 39, Making and Curing Concrete Test Specimens in the Laboratory
- T 23, Making and Curing Concrete Test Specimens in the Field
- T 24, Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

3. SUMMARY OF TEST METHOD

3.1. 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 (previously determined), and the CTE is then calculated by dividing the corrected length change by the temperature change and then the specimen length, as described in the section on calculations.

4. SIGNIFICANCE AND USE

4.1. Measurement of the CTE permits assessment of the potential for length/volume change of concrete due to a uniform temperature change, and the potential deformation of a concrete structure due to a temperature gradient through the concrete. As an example, for pavement slabs on grade, uniform temperature change will affect the openings at joints, and a temperature gradient through the thickness of these same slabs will produce curling of the slabs. Using the results of this test, better estimates of slab movement and stress development due to temperature change can be obtained.

5. APPARATUS

5.1. *Concrete Saw*—Capable of sawing the ends of a cylindrical specimen perpendicular to the axis and parallel to each other.

5.2. *Balance*—A scale or balance having a capacity of 20 kg (44 lb), and accurate to 0.1 percent over its range.

5.3. *Caliper*—Comparator or other suitable device to measure the specimen length to the nearest 0.1 mm (0.004 in.).

5.4. *Water Bath*—A controlled-temperature water bath with a temperature range of 10 to 50°C (50 to 122°F), capable of controlling the temperature to 0.1°C (0.2°F).

5.5. *Support Frame*—A rigid support frame for the specimen to be used during length change measurement. The frame should be designed to have minimal influence on the length change measurements obtained during the test, and support the specimen such that the specimen is allowed to freely adjust to any change in temperature. A suitable support frame is described in detail in Appendix X1.

5.6. *Temperature Measuring Devices*—Four submersible temperature measuring devices with a resolution of 0.1°C (0.2°F) and accurate to 0.2°C (0.4°F).

5.7. *Gauge*—A submersible LVDT gauge head with excitation source and digital readout, with a minimum resolution of 0.00025 mm (0.00001 in.), and a range suitable for the test (for ease in setting up the apparatus, a range of ±3 mm (0.1 in.) has been found practical) (Note 1).

Note 1—Linear variable differential transformers (LVDTs) with the appropriate associated electronic actuating and indicating apparatus appear to give the best results with respect to stability, sensitivity, and reliability. Multichannel recording of outputs has been found to be practical and efficient. As an alternate, a data logger can be used to excite the LVDT and record the LVDT and both temperature and time outputs. The data can be stored directly in a personal computer for graphing of test results.

5.8. *Micrometer*—A micrometer or other suitable device for calibrating the LVDT over the range to be used in the test and with a minimum resolution of 0.00025 mm (0.00001 in.).

6. TEST SPECIMENS

6.1. Test specimens shall consist of drilled 100-mm (4-in.) nominal diameter cores sampled from the concrete structure being evaluated, or 100-mm (4-in.) nominal diameter cylinders. Cores shall be obtained in accordance with T 24. Cylinders shall be cast in accordance with T 23 or R 39. The specimens shall be sawed perpendicular to the axis at a suitable length. A length of 180 ± 2 mm (7.0 ± 0.1 in.) has been found acceptable. The standard reference material used for calibration (see Appendix X2) shall be the same length as the test specimen so that the frame does not have to be adjusted between calibration and testing. The sawed ends shall be flat and parallel.

7. PROCEDURE

7.1. *Specimen Conditioning*—The specimens shall be conditioned by submersion in saturated limewater at 23 ± 2°C (73 ± 4°F) for not less than 48 hours and until two successive weighings of the surface-dried sample at intervals of 24 hours show an increase in weight of less than 0.5 percent. A surface-dried sample is obtained by removing the surface moisture with a towel.

7.2. *Testing Procedure*:

7.2.1. Place the measuring apparatus, with LVDT attached, in the water bath and fill the bath with cold tap water. Place the four temperature sensors in the bath at locations that will provide an average temperature for the bath as a whole. To avoid any sticking at the points of contact with the specimen, put a very thin film of silicon grease on the end of the support buttons and LVDT tip.

7.2.2. Remove the specimen from the saturation tank and measure its length at room temperature to the nearest 0.1 mm (0.004 in.). After measuring the length, place the specimen in the measuring apparatus located in the controlled-temperature bath, making sure that the lower end of the specimen is firmly seated against the support buttons, and that the LVDT tip is seated against the upper end of the specimen (Note 2).

Note 2—The desired range of travel is the linear range of the LVDT over which it has been calibrated. The LVDT travel during the test should remain well within this range to insure accurate results.

7.2.3. Set the temperature of the water bath to 10 ± 1°C (50 ± 2°F). When the bath reaches this temperature, allow the bath to remain at this temperature until thermal equilibrium of the specimen has been reached, as indicated by consistent readings of the LVDT to the nearest 0.00025 mm (0.00001 in.) taken every 10 minutes over a one-half hour time period. Also at this time, check that the specimen is firmly seated against the support buttons, as confirmed by the LVDT reading.

7.2.4. Record the temperature readings from the four sensors to the nearest 0.1°C (0.2°F). Record the LVDT reading to the nearest 0.00025 mm (0.00001 in.). These are the initial readings.

7.2.5. Set the temperature of the water bath to 50 ± 1°C (122 ± 2°F). Once the bath has reached 50 ± 1°C (122 ± 2°F), allow the bath to remain at this temperature until thermal equilibrium of the specimen has been reached, as indicated by consistent readings of the LVDT to the nearest 0.00025 mm (0.00001 in.) taken every 10 minutes over a one-half hour time period.

7.2.6. Record the temperature readings from the four sensors to the nearest 0.1°C (0.2°F). Record the LVDT reading to the nearest 0.00025 mm (0.00001 in.). These are the second readings.

7.2.7. Set the temperature of the water bath to 10 ± 1°C (50 ± 2°F). When the bath reaches this temperature, allow the bath to remain at this temperature until thermal equilibrium of the specimen has been reached, as indicated by consistent readings of the LVDT to the nearest 0.00025 mm (0.00001 in.) taken every 10 minutes over a one-half hour time period.

7.2.8. Record the temperature readings from the four sensors to the nearest 0.1°C (0.2°F). Record the LVDT reading to the nearest 0.00025 mm (0.00001 in.). These are the final readings.

8. CALCULATIONS

8.1. *Coefficient of Thermal Expansion*—Calculate the CTE of one expansion or contraction test segment of a concrete specimen as follows (reported in micro strains/°C):

$$CTE = (\Delta L_a / L_o) / \Delta T \qquad (1)$$

where:

ΔL_a = actual length change of specimen during temperature change, mm, (see Equation 2);

L_o = measured length of specimen at room temperature, mm; and

ΔT = measured temperature change (average of the four sensors), °C (increase = positive, decrease = negative).

$$\Delta L_a = \Delta L_m + \Delta L_f \qquad (2)$$

where:

ΔL_m = measured length change of specimen during temperature change, mm (increase = positive, decrease = negative); and

ΔL_f = length change of the measuring apparatus during temperature change, mm. (See Equation 3.)

$$\Delta L_f = C_f \times L_o \times \Delta T \qquad (3)$$

where:

C_f = correction factor accounting for the change in length of the measurement apparatus with temperature, in.$^{-6}$/in./°C. (See Appendix X2.)

8.2. For the expansion test segment, the initial and second readings are used in the calculations. For the contraction test segment, the second and final readings are used in the calculations.

8.3. The test result is the average of the two CTE values obtained from the two test segments provided the two values are within 0.3 micro strain/°C (0.5 micro strain/°F) of each other. If the two values are not within 0.3 micro strain/°C (0.5 micro strain/°F) of each other, one or more additional test segments are completed until two successive test segments yield CTE values within 0.3 micro strain/°C (0.5 micro strain/°F) of each other. The test result is the average of these two CTE values (Note 3).

$$CTE = (CTE_1 + CTE_2)/2 \qquad (4)$$

Note 3—Differences in successive CTEs greater than the required value sometimes occur during the first few cycles of temperature change due to minor misalignment, or lack of proper initial seating of the specimen. This is usually self-correcting during the first few temperature cycles. However, it does point out the importance of carefully positioning the specimen at the start of the test.

9. REPORT

9.1. *The report shall include the following information:*

9.1.1. Identification number;

9.1.2. Specimen type, description, and source;

9.1.3. Specimen dimensions, including length and diameter;

9.1.4. Mixture proportions and aggregate type, if available;

9.1.5. All temperature and length measurements collected during the test;

9.1.6. All calculated values, including CTE data and the final CTE value;

9.1.7. The frame's correction factor, C_f, as well as the reference material used and its thermal coefficient;

9.1.8. Date of test;

9.1.9. Place of test;

9.1.10. Technician conducting test; and

9.1.11. Any other pertinent information.

10. PRECISION AND BIAS

10.1. *Precision*—No precision has been established for this test method.

10.2. *Bias*—No bias can be established because no reference material is available for this test.

APPENDIXES

(Nonmandatory Information)

X1. SPECIMEN MEASURING APPARATUS

X1.1. The measuring apparatus consists of two primary components: a frame and a length change measuring device.

X1.2. *Frame*:

X1.2.1. Figure X1.1 shows a schematic of a suitable measuring frame. Any specimen measuring frame should be constructed with the following features in mind:

X1.2.2. Because the frame will be submerged in water throughout the test, components should be made of a noncorroding material. In so far as possible, the portions of the frame, which directly affect measurement over a change in temperature, should be constructed of invar and protected from corrosion as necessary.

X1.2.3. The frame may be designed to be adjustable to accommodate different sample lengths; however, calibrations will be required after each adjustment.

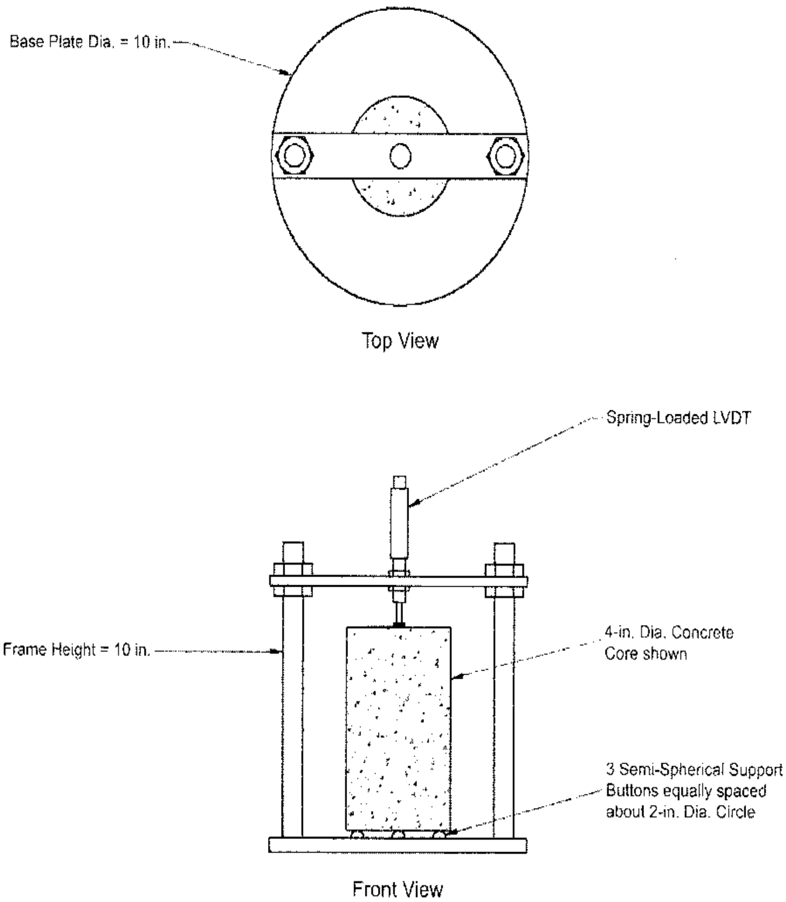


Figure X1.1—Schematic of a Measuring Frame

X1.3. *Length Change Measurement Devices*:

X1.3.1. The sample length change may be measured using any suitable apparatus that can be submerged in water, has sufficient resolution, and gives reproducible results. Some apparatus use a submersible spring-loaded LVDT gauge head for length change measurement.

X1.3.2. Appropriate signal conditioning equipment will be required if an LVDT or other electronic transducer is used for length change measurements. A voltmeter or a computer and data acquisition software may also be required if the signal conditioning equipment does not have a digital readout. The LVDT will require calibration using a micrometer to relate the digital readout output (which may be in volts or arbitrary units) to actual length changes.

X1.3.3. The contact tip (at the point of contact between the measuring device and the specimen) must be attached to the length change measuring device with a suitable adhesive to prevent loosening during a test.

X2. REFERENCE TEST FOR DETERMINATION OF CORRECTION FACTOR

X2.1. The test procedure described in Section 7.2 is used to determine a correction factor to account for expansion of the measuring apparatus during the test. A specimen with a known coefficient of thermal expansion is used. The specimen should be composed of a material which is essentially linearly elastic, noncorroding, non-oxidizing, and nonmagnetic, and should have a thermal coefficient as close as possible to that of concrete (304 stainless steel, which has a CTE of 17.3×10^{-6}/°C, is a suitable material). The reference material sample should also be of the same nominal dimensions as the test samples, so that no adjustment of the frame and/or the LVDT is necessary between calibration and testing.

X2.2. *Calculation of the Correction Factor:*

X2.3. Assuming that the length change of the apparatus varies linearly with temperature, the correction factor C_f is defined as:

$$C_f = \Delta L_f / L_{cs} / \Delta T \qquad (X2.1)$$

where:

ΔL_f = length change of the measuring apparatus during temperature change, mm (see Equation X2.2);

L_{cs} = measured length of calibration specimen at room temperature, mm; and

ΔT = measured temperature change, °C (increase = positive, decrease = negative).

$$\Delta L_f = \Delta L_a - \Delta L_m \qquad (X2.2)$$

where:

ΔL_a = actual length change of calibration specimen during temperature change, mm (see Equation X2.3); and

ΔL_m = measured length change of calibration specimen during temperature change, mm (increase = positive, decrease = negative).

$$\Delta L_a = L_{cs} \times \alpha_c \times \Delta T \qquad (X2.3)$$

where:

α_c = CTE of calibration specimen, °C (known).

Note X1—It is recommended that at least three calibration tests be performed, and that the average of the correction factors calculated for each test be used for calculations on actual concrete tests.

[1] Approved in January 2000, this standard was first published in 2000 and reconfirmed in 2006.

APPENDIX B

Thermal and Mechanical Properties in MEPDG

Table 9
Thermal properties

Coarse aggregate	Thermal properties		
	CTE (με/°F)	Thermal conductivity (BTU/h·ft·F)	Heat capacity (BTU/lb·F)
Kentucky limestone	4.96	1.451	0.282
Gravel	7.14	1.601	0.273
Mexican limestone	4.90	1.219	0.279

Thermal properties are only valid when the concrete mixture design is similar to this study (refer to Table 2)

Table 10
CTE unit conversion chart

(με/°F)	(με/°C)
4.0	7.2
4.5	8.1
5.0	9.0
5.5	9.9
6.0	10.8
6.5	11.7
7.0	12.6
7.5	13.5
8.0	14.4
8.5	15.3
9.0	16.2
9.5	17.1
10	18

$(10^{-6}/°\mathrm{C} \times \frac{5}{9} = 10^{-6}/°\mathrm{F})$

Table 11
Mechanical properties

Coarse aggregate	Design level	Mechanical properties	Ages			
			7 days	14 days	28 days	90 days
Kentucky limestone	Level 1	Modulus of elasticity (10^6 psi)	5.883	5.866	6.466	6.750
		Flexural strength (psi)	678	925	811	809
	Level 2	Compressive strength (psi)	6015	6775	7408	8640
	Level 3	Compressive strength (psi)	–	–	7408	–
		Flexural strength (psi)	–	–	811	–
Gravel	Level 1	Modulus of elasticity (10^6 psi)	5.033	4.766	5.083	5.866
		Flexural strength (psi)	519	551	589	738
	Level 2	Compressive strength (psi)	3782	4363	4900	6004
	Level 3	Compressive strength (psi)	–	–	4900	–
		Flexural strength (psi)	–	–	589	–
Mexican limestone	Level 1	Modulus of elasticity (10^6 psi)	4.150	4.600	4.550	4.633
		Flexural strength (psi)	559	652	686	710
	Level 2	Compressive strength (psi)	4671	5272	5935	6314
	Level 3	Compressive strength (psi)	–	–	5935	–
		Flexural strength (psi)	–	–	686	–

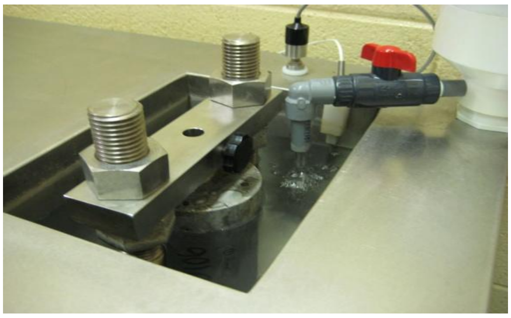
Mechanical properties are only valid when the concrete mixture design is similar to this study (refer to Table 2)

APPENDIX C

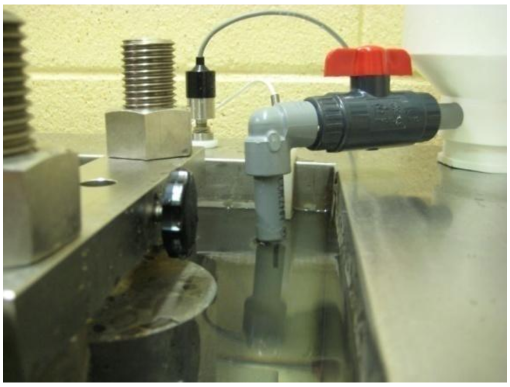
CTE Test Procedure (HM-251) and Raw Data from CTE Test Results

1. Calibration should be done by using a calibration rod provided by manufacturer to obtain calibration factor when it is first installed or has been moved to a new location.
2. Measure the dimensions of sample and record in millimeters.
3. Properly condition the specimen according to AASHTO TP-60.
4. Center the specimen on the standoffs using the engraved circle on the bottom of the frame.

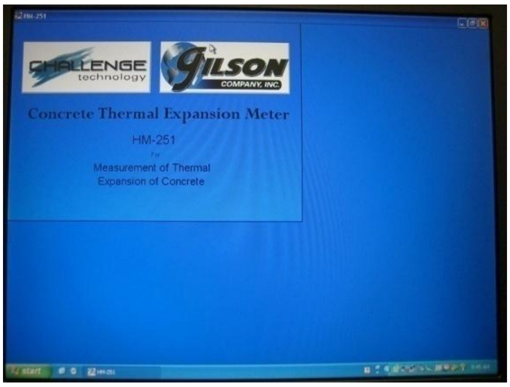
5. Place the frame with specimen into the water bath, and fill the water bath to 1 in. below from the specimen surface.

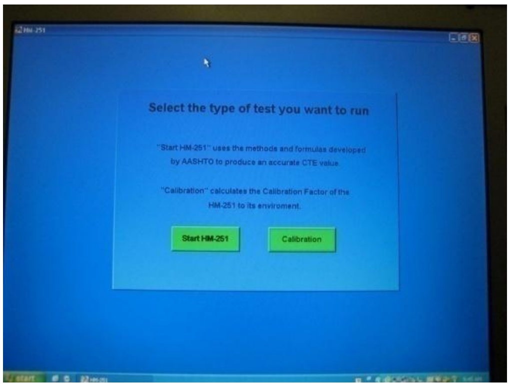


6. Fill the water level control reservoir (WLCR) by opening both valves. Pour water into the WLCR until water appears through the opposite valve. Close both valves. Place the WLCR in the back right corner of the cabinet with the 90° elbow in the water bath and open the value. This must be full and open so that it supplies the water consistently to prevent the water loss due to the evaporation.

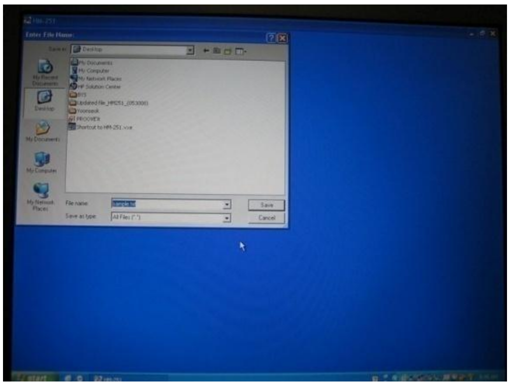
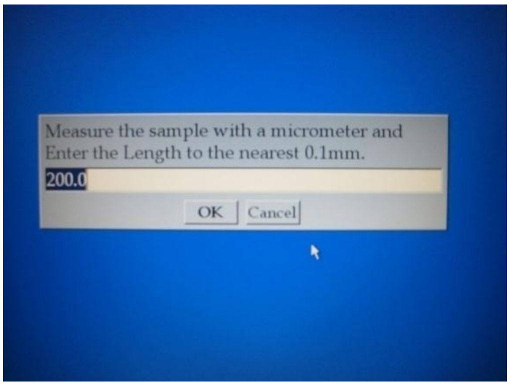


7. Turn on the HM-251 power switch, which is on the back side of heating/cooling circulator.
8. Open HM-251 software, press “Enter” at the initial screen and select “Start HM-251.”

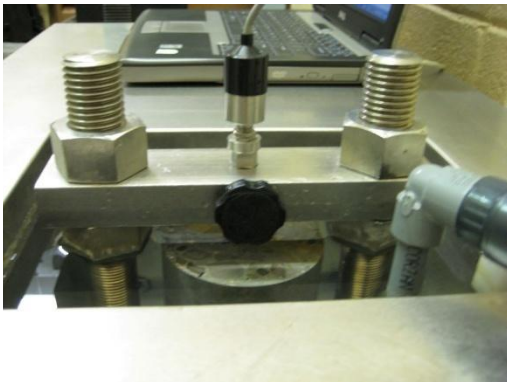




9. Enter file name and specimen length in millimeters consecutively. Heights range between 150 mm and 250 mm.

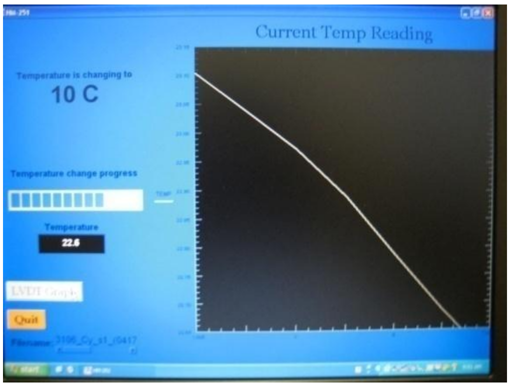



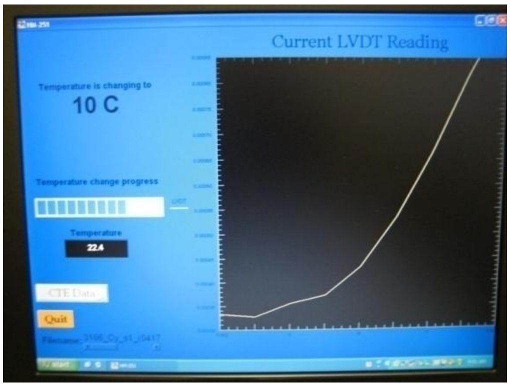
10. Insert LVDT in the top of the measuring frame with the flat side to the set-knob.
11. Manually raise and lower the LVDT in the direction of the screen arrow until the check mark appears. Once the check mark shows up on the screen, tight on the set-knob to hold the LVDT.



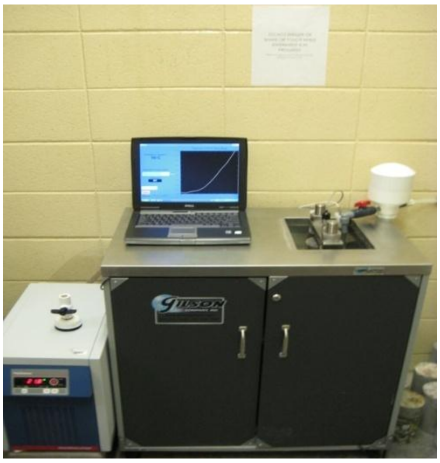

12. Press “Done” and the test will begin.

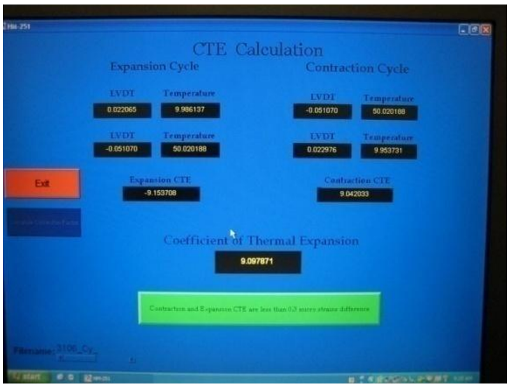
13. The temperature and LVDT graphs with time can be seen during the testing.





14. Testing is complete when the difference between the expansion CTE and contraction CTE is between 0.3 microstrains/C. The average CTE is then displayed.





15. When test is complete, select “Exit.”
16. After testing is complete, remove the LVDT and place it in the holder. Remove the specimen and turn off the CTE power switch.
17. Trouble shooting:
    - E-FL message appears on the LCD of heating/cooling unit.
      There is an air lock in the reservoir, so slowly open the reservoir vent valve until water reaches the top and close.
    - Temperature range is not between 10˚C and 50˚C.
      Press F3 at the initial start-up screen. Fix HI-L to 55˚C, and AFS to 48˚C. This adjustment will bring the temperature range back to between 10˚C and 50˚C.

Raw Data from CTE Test Result

"Gilson Company,Inc."

HM-251 Concrete Thermal Expansion

Date: 22/Apr/2009 9:29:51 AM

Sample Length: 190.1

Cf: 1.874794590570295E-005

timer	Time	La (actual)	Displ	Lm (measured)	temp
9:41:19	0	0.021774308	-0.021774308	0.007236356	17.93415392
9:51:19	10	0.040183209	-0.040183209	0.013073105	14.40659864
9:59:25	20	0.052710368	-0.052710368	0.017020491	11.99924451
10:09:25	30	0.065428004	-0.065428004	0.021137756	9.586110859
10:19:25	40	0.0704736	-0.0704736	0.022855211	8.652284804
10:29:25	50	0.063590564	-0.063590564	0.020363698	9.884479625
10:39:25	60	0.06460988	-0.06460988	0.021093528	9.803254265
10:49:25	70	0.063781653	-0.063781653	0.020902955	9.982170273
10:59:25	80	0.063566918	-0.063566918	0.020902685	10.04234572
11:09:25	90	0.063928575	-0.063928575	0.021092852	9.994228495

Stable reading at 10C CTE=unknown Lm= 0.02109285182930963

La= 0.06392857458018066 temp= 9.994228494792642

11:20:52	100	0.04349608	-0.04349608	0.014177534	13.78694347
11:30:52	110	0.023874978	-0.023874978	0.00768726	17.47125465
11:40:52	120	0.005781714	-0.005781714	0.001727123	20.87562608
11:50:52	130	-0.011020707	0.011020707	-0.003943562	23.99902231
12:00:52	140	-0.026848116	0.026848116	-0.009394932	26.9103814
12:10:52	150	-0.041737183	0.041737183	-0.014532329	29.64655217
12:20:52	160	-0.056201198	0.056201198	-0.019608356	32.28067896
12:30:52	170	-0.069617284	0.069617284	-0.024185342	34.76079622
12:40:52	180	-0.0822717	0.0822717	-0.02853411	37.09123545
12:50:52	190	-0.094116185	0.094116185	-0.032678356	39.25180544
13:00:52	200	-0.105122412	0.105122412	-0.036486575	41.2714575
13:10:52	210	-0.115289733	0.115289733	-0.039989589	43.14136094
13:20:52	220	-0.124723853	0.124723853	-0.043242603	44.87568568
13:30:52	230	-0.133373198	0.133373198	-0.046253151	46.45784643
13:40:52	240	-0.141280255	0.141280255	-0.049011507	47.90249387
13:41:36	250	-0.141822137	0.141822137	-0.049180685	48.00706908
13:51:36	260	-0.148234664	0.148234664	-0.050994522	49.29739152
14:01:36	270	-0.152284447	0.152284447	-0.052299317	50.06759347
14:11:36	280	-0.150885196	0.150885196	-0.051756002	49.82743091
14:21:36	290	-0.150592597	0.150592597	-0.051721647	49.75497135

14:31:36	300	-0.152574547	0.152574547	-0.052546696	50.07958038
14:41:36	310	-0.152138014	0.152138014	-0.052268073	50.03527311
14:51:36	320	-0.151400325	0.151400325	-0.052140664	49.86403788
15:01:36	330	-0.152310827	0.152310827	-0.052493406	50.02053687
15:11:36	340	-0.152700579	0.152700579	-0.052677893	50.07813123
15:21:36	350	-0.152428859	0.152428859	-0.05254602	50.03889221
15:31:36	360	-0.152229071	0.152229071	-0.052453371	50.0088307

Stable reading at 50C Expansion CTE= -9.085255848917289 Lm= -0.05245337120443637

La= -0.1522290711512687 temp= 50.00883069929005

15:43:05	370	-0.117345347	0.117345347	-0.039474403	43.8626883
15:53:05	380	-0.085662187	0.085662187	-0.028937039	37.9294987
16:03:05	390	-0.058403787	0.058403787	-0.019956583	32.80098466
16:13:05	400	-0.034292291	0.034292291	-0.011914384	28.29218387
16:23:05	410	-0.012738323	0.012738323	-0.004714682	24.26459467
16:33:05	420	0.006723569	-0.006723569	0.001859471	20.64849044
16:43:05	430	0.024580478	-0.024580478	0.007829986	17.31334866
16:53:05	440	0.040802453	-0.040802453	0.01328613	14.29261977
17:01:39	450	0.053004631	-0.053004631	0.017297897	11.99451464
17:11:39	460	0.065648762	-0.065648762	0.021631163	9.662611726
17:21:39	470	0.070120225	-0.070120225	0.023123554	8.826729633
17:31:39	480	0.064907031	-0.064907031	0.021263407	9.767543801
17:41:39	490	0.064851694	-0.064851694	0.021527828	9.857263104
17:51:39	500	0.064876355	-0.064876355	0.021714749	9.902790818
18:01:39	510	0.064496717	-0.064496717	0.021620883	9.982974029
18:11:39	520	0.064809153	-0.064809153	0.021827281	9.953221083
18:21:39	530	0.064872165	-0.064872165	0.021883276	9.951252245
18:31:39	540	0.064952803	-0.064952803	0.021956989	9.949309418
18:41:39	550	0.06503581	-0.06503581	0.022047474	9.951407676

Stable reading at 10C Contraction CTE= 8.976530458528165 Lm= 0.02204747413268409

La= 0.0650443032958288 temp= 9.949024479028841

Expansion CTE: -9.085255848917289

Contraction CTE: 8.976530458528165

CTE: 9.030893153722726

APPENDIX D

Overview of Inputs in MEPDG Software

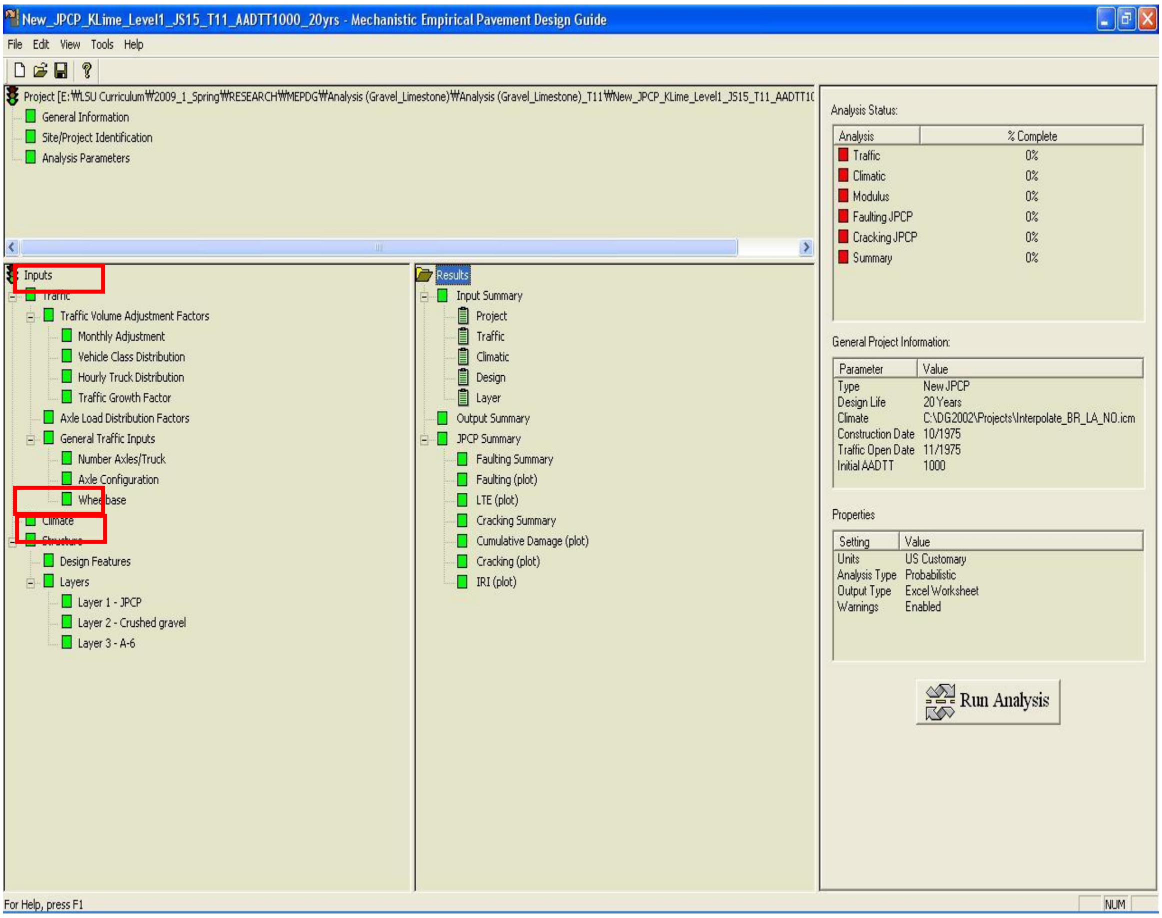


Figure 39
Overview of software

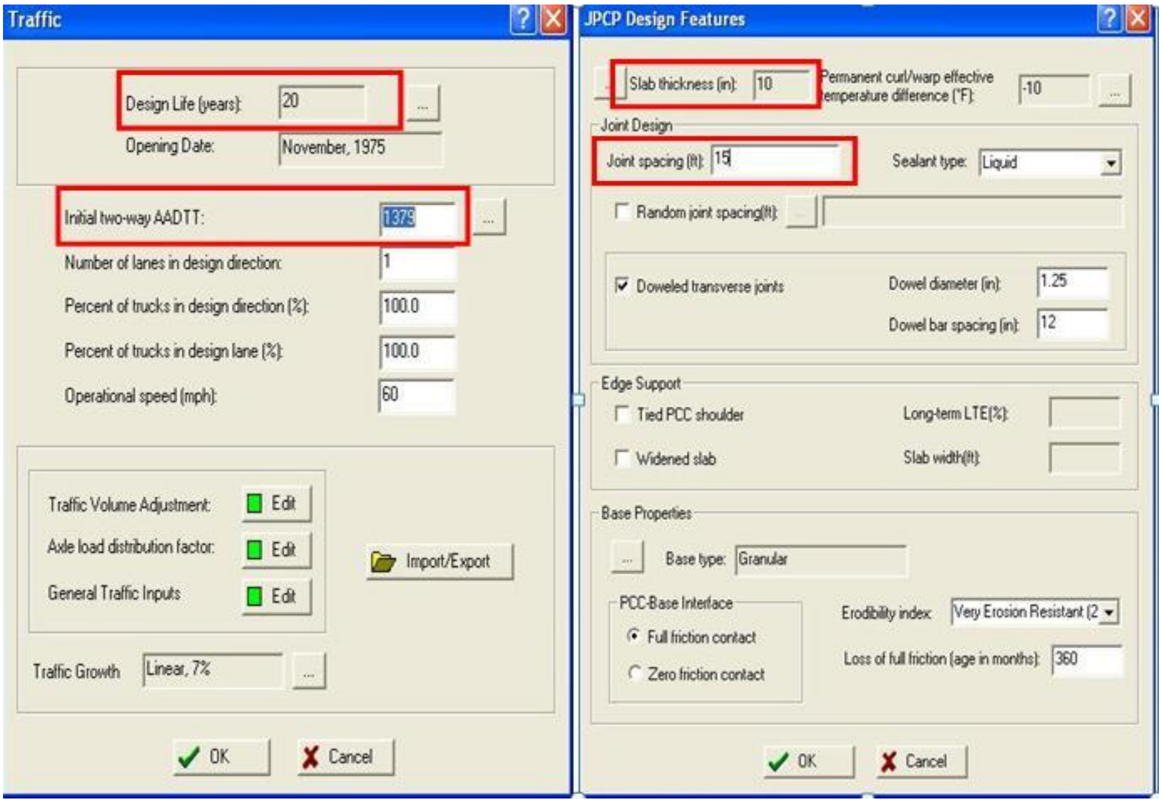


Figure 40
Design life, AADTT, slab thickness, and joint spacing

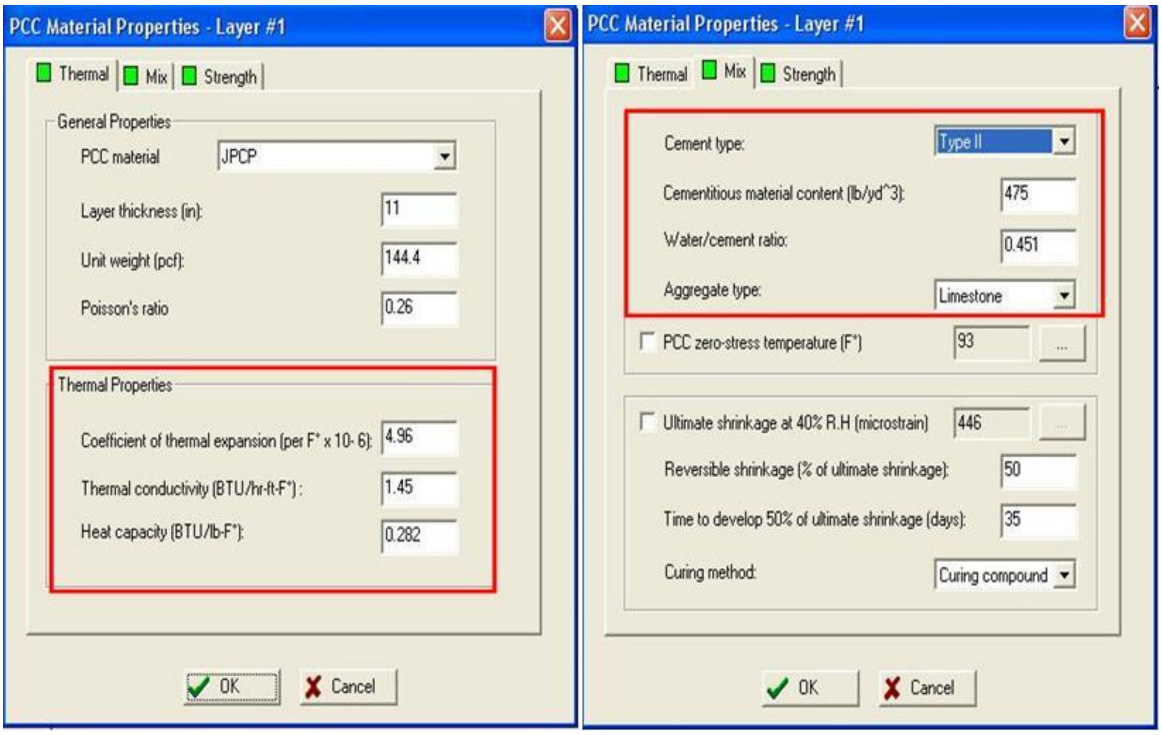


Figure 41
Thermal properties and mixture properties

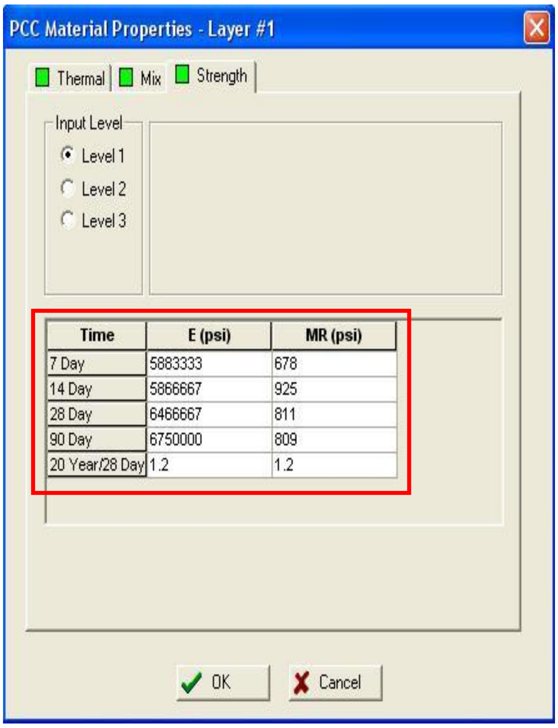


Figure 42
Strength properties

APPENDIX E

Reliability Summary of MEPDG Result (Kentucky limestone, 18 ft. joint spacing)

Table 12
Reliability summary of MEPDG result

Performance Criteria	Distress Target	Reliability Target	Distress Predicted	Reliability Predicted	Acceptable
Terminal IRI (in/mile)	172	90	88	99.7	***Pass***
Transverse Cracking (% slabs cracked)	15	90	12	64.53	***Fail***
Mean Joint Faulting (in)	0.12	90	0.034	99.8	***Pass***

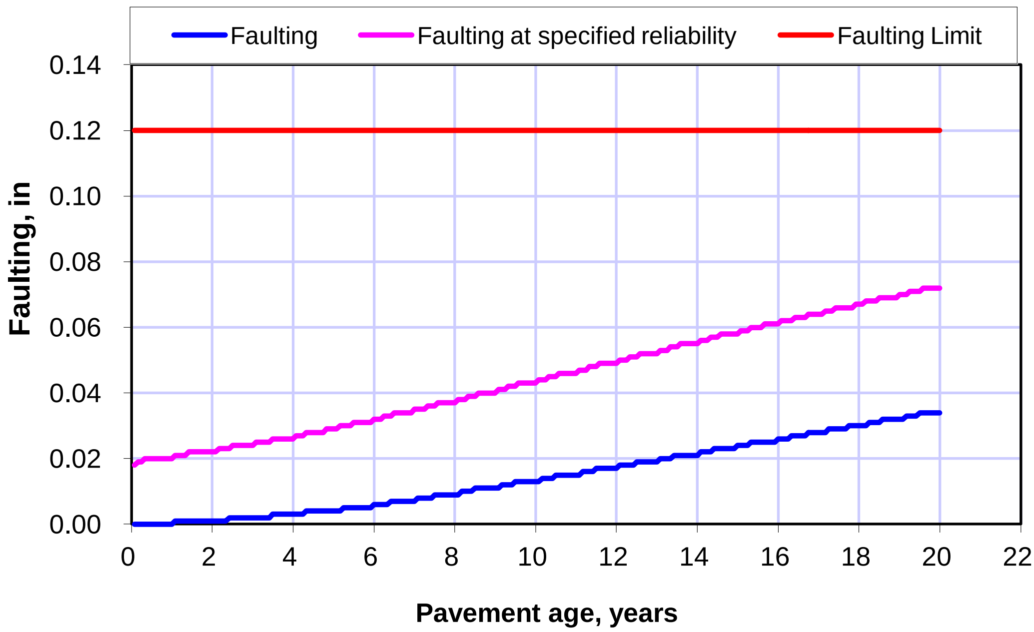


Figure 43
Reliability summary of MEPDG result, faulting

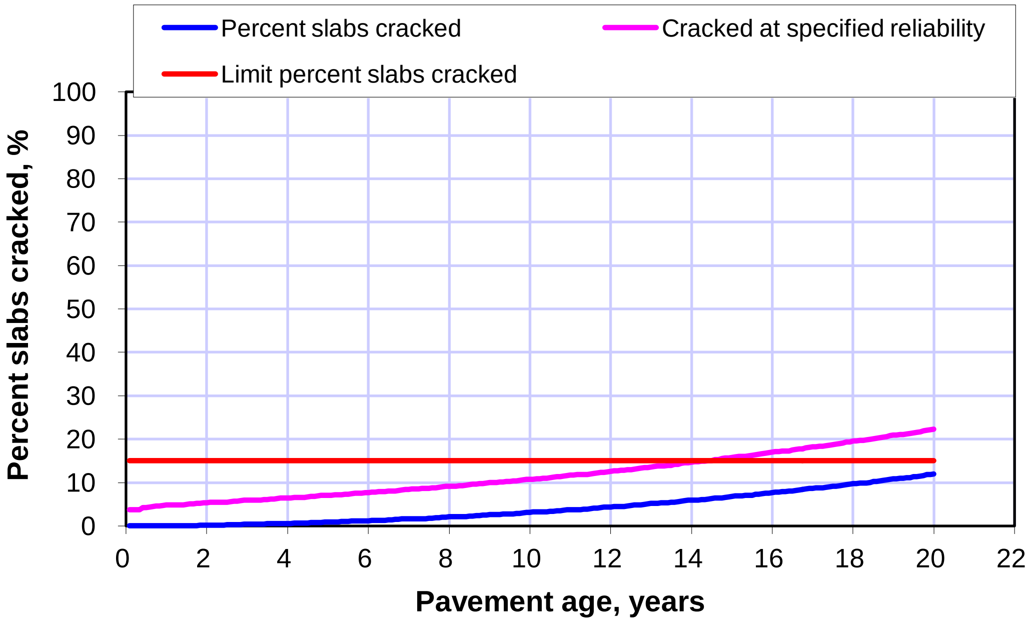


Figure 44
Reliability summary of MEPDG result, slab cracking

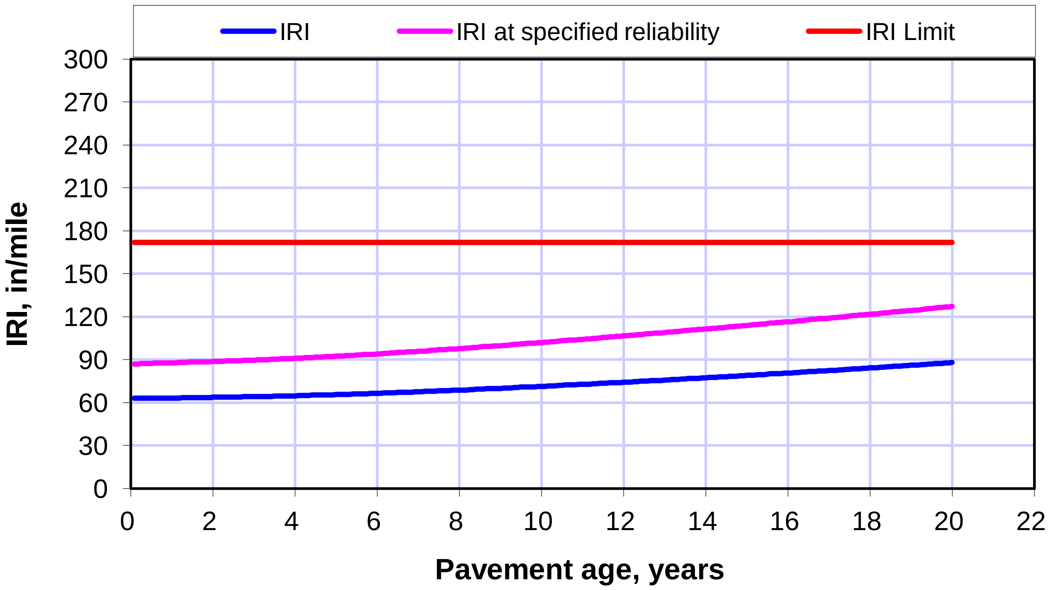


Figure 45
Reliability summary of MEPDG result, IRI