Thermal stress analysis of jointed plain concrete pavements containing fly ash and slag

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THERMAL STRESS ANALYSIS OF JOINTED PLAIN CONCRETE PAVEMENTS CONTAINING FLY ASH AND SLAG

A Dissertation

Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College in partial fulfillment of the requirements for the degree of Doctor of Philosophy

in

The Department of Civil Engineering

by

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ABSTRACT

With the current demand for Portland cement concrete (PCC) sustainability, supplementary cementitious materials (SCMs) are used in concrete mixtures. The SCMs positively impact the environmental and economic aspects of concrete mixtures and improve the mixture properties in both fresh and hardened concrete. In this research, one control and twenty-four ternary mixtures, with various combinations of fly ashes (Class C and F), slags (Grade 100 and 120), and Portland cement were fabricated. The thermal properties (coefficient of thermal expansion (CTE), thermal conductivity, and heat capacity) and mechanical properties of the selected ternary mixtures were measured at various ages.

Temperature gradients were measured using a concrete pavement (10-in. thick) to characterize daily and seasonal temperature variations through the slab thickness. The correlation between air temperature and surface temperature, as well as air temperature and temperature difference of the slab thickness, were established based on the measured temperature gradients in the concrete pavement. The enhanced integrated climatic model (EICM) analysis was conducted, using measured material properties and climatic conditions. A local calibration of EICM was performed by comparing EICM-predicted temperature gradients to field measurement. It was concluded that the surface temperature is suitable to accurately predict temperature gradients in EICM.

A thermal stress analysis of the ternary mixtures was conducted to calculate the critical tensile stress on the PCC pavements by means of the measured mechanical properties, nonlinear temperature gradients obtained from EICM, and CTE gradients throughout the slab thickness. The ratios of tensile stress-to-strength at the critical state of concrete pavements were estimated as well, in order to investigate the vulnerability of ternary mixtures to tensile stress. The ratio of
tensile stress-to-strength shows that all the ternary mixtures, inclusive of the replacement of 30 % slag with 20 % fly ashes, 30 % slag with 30 % fly ashes, and 50 % slag with 20 % fly ashes (both Class C and F), do not exceed 100 % tensile stress-to-strength ratio at all ages. These combinations may be considered as the limitation of ternary mixture replacement with slags and fly ashes.
CHAPTER 1 INTRODUCTION

1.1. General Introduction and Rationale of the Study

Portland cement concrete (PCC) pavements are usually exposed to repeated dynamic loads produced by heavy truck traffic, as well as deformations caused by various climatic conditions. One of the most important environmental factors for designing and predicting performance of PCC pavements is temperature. Temperature gradients throughout the slab thickness play a key role in calculating thermal stresses in PCC pavements, known as curling [Huang, 2004]. Due to the variable characteristics of curling stresses resulting from the daily and seasonal fluctuation of temperature gradients, an accurate measurement of temperature gradients at various climatic conditions is necessary to correctly predict the performance of PCC pavement. Westergaard [1926] and Bradbury [1938] studied the stress of concrete slab subjected to a linear temperature gradient and noted that the temperature stress alone may be as high as the stresses due to traffic loading under certain climatic conditions. Moisture is another environmental factor which should not be ignored in predicting PCC pavement deformation [Bradbury, 1938]. The surface of the PCC pavement dries when the intensity of the solar radiation or the air temperature rises during the day. The dryness on the surface of PCC pavement causes it to shrink, called warping. If there are any restraints against this shrinkage, a tensile stress will develop in the pavement [Shin and Lange, 2012]. Figure 1-1 illustrates the PCC pavement deformation caused by environmental effects. Figure 1-1b and Figure 1-1c show the pavement deformation caused by temperature variation throughout the slab thickness during night and day, respectively. Figure 1-1d shows the pavement deformation induced by moisture variation throughout the slab thickness.
Figure 1-1 PCC pavement deformations under temperature and moisture variations

The thermal properties have been widely considered as a fundamental property of PCC pavements but have not been considered in the thickness design procedure of PCC pavement. In the Mechanistic-Empirical Pavement Design Guide (MEPDG) developed through the National Cooperative Highway Research Program (NCHRP) 1-37A project, the thermal properties became a direct input parameter closely related to the pavement performance [ARA, 2004]. Therefore, it is essential to measure accurate thermal properties, such as coefficient of thermal expansion (CTE), thermal conductivity (TC), and heat capacity (HC) of PCC pavements, to predict critical pavement distresses within the design life. CTE is directly used to predict the
amount of thermal cracking and its effect is significant on PCC pavement performance. Thermal conductivity and heat capacity are used to calculate the temperature gradient throughout the slab thickness. Thermal conductivity and heat capacity are the main input parameters to estimate temperature and moisture gradients of PCC pavement in the enhanced integrated climatic model (EICM), in the built-in software in MEPDG.

Supplementary cementitious materials (SCMs) have been commonly used in modern concrete practice to achieve sustainability for PCC pavements. Environmental, economic, and social impacts are three main categories that are generally considered a sustainability footprint (Elkington, 1994). As an environmental impact, the usage of SCMs reduces the carbon dioxide (CO$_2$), thus contributing to the earth’s greenhouse-effect by reducing the production of Portland cement. Since SCMs are industrial by-products, use of the SCMs in concrete mixtures conserves natural resources and reduces landfill disposal. With respect to economic impact, the use of SCMs saves material cost by replacing the use of Portland cement, the most expensive ingredient in concrete mixtures. In addition, properly used SCMs enhance the durability of a concrete mixture, thus increasing the service life, while decreasing the life-cycle cost [Tikalsky et al., 2007]. In regarding to social impact, the use of SCMs ensures a high level of user comfort by providing same or better performance. Ternary-blended cement constitutes a blend of Portland cement, together with two of the following SCMs: fly ash, slag, or silica fume. Ternary-blended cement is used increasingly in the industry at present, thus American Society of Testing and Materials (ASTM) provides the specification for ternary-blended cement [ASTM C595].

This study measured the thermal and mechanical properties of selected ternary-blended concrete mixtures to investigate the characteristics of the ternary-blended mixture and to find the effect of nonlinear temperature gradients on the performance of ternary-blended concrete
pavements. The thermal stress analysis of ternary-blended concrete pavements containing fly ash and slag were conducted to calculate the critical tensile stress on the jointed plain concrete pavements (JPCP). Figure 1-2 presents the research outline.
1.2. Objectives of Study

The overall objectives of this study are to evaluate the thermal stresses of JPCP containing fly ash and slag and to account for the stresses in the design process. The specific objectives to achieve from this study are the following:

- To measure and characterize the thermal properties (CTE, TC, and HC), and mechanical properties of selected ternary-blended mixtures,
- To characterize the variations of daily and seasonal temperature gradients throughout the slab thickness under local climatic conditions,
- To calibrate the EICM and develop a correlation between air temperature and slab temperature difference by utilizing measured temperature gradients throughout the slab thickness in Louisiana, and
- To calculate thermal stresses of JPCP containing fly ash and slag, by using the measured thermal and mechanical properties in this study.

1.3. Scope

In mixture design, five different conventional concrete mixtures (various coarse aggregate types and coarse aggregate proportions) and twenty-four ternary-blended concrete mixtures (various proportions of Portland cement, two fly ashes, and two slags), were selected to characterize the thermal and mechanical properties of both conventional and ternary mixtures.

In experimental measurements, thermal properties including CTE, TC, and HC were measured at around 28 days. Mechanical properties of hardened concrete, inclusive of compressive strength, modulus of elasticity, and flexural strength were measured at various ages (7, 14, 28, and 90 days), because the strength development of ternary-blended concrete with age varies on the proportions of each fly ash and slag.
Pavement performance was predicted by using the mechanistic-empirical pavement design guide (MEPDG) within reasonable ranges of critical input data, such as CTE and joint spacing, in order to discover the sensitivity of the critical factors on pavement distresses in jointed plain concrete pavements.

Temperature gradients were measured using on-site pavement to characterize daily and seasonal temperature variations within the slab thickness.

### 1.4. Organization of the Dissertation

In Chapter 2, literature reviews present the thermal effect, sustainability, EICM, temperature measurements, thermal properties of PCC pavements, and thermal stress analysis.

In Chapter 3, the study presents the mixture design of both conventional and ternary-blended concrete mixtures, chemical compositions of materials, measuring apparatus, and testing results of thermal and mechanical properties.

Chapter 4 describes influential factors on CTE values for conventional concrete mixtures, and predicts pavement performance, using MEPDG for various CTEs and joint spacings. Based on the results of conventional concrete mixtures, the study characterizes CTE of ternary-blended concrete mixtures, using various proportions of fly ash and slag.

Chapter 5 measures temperature gradients by installing sensors at various depths of concrete pavement. The study established correlations between the air temperature and the surface temperature, as well as the surface temperature and the temperature difference of the slab thickness, based on the measured temperature gradients in the concrete pavement.

In Chapter 6, the study conducted the EICM analysis, using local material properties and climatic conditions. A local calibration of EICM was conducted by comparing temperature
The measured daily and seasonal temperature gradients in 10-in. thick concrete pavement were compared to the FEM analysis.

In Chapter 7, the study calculated thermal stresses of JPCP containing fly ash and slag analytically, using measured thermal and mechanical properties with the typical dimensions of JPCP in Louisiana.

In Chapter 8, this study presents the summary of findings, practical applications, and recommendations for future study.
CHAPTER 2 LITERATURE REVIEW

2.1. Thermal Effect in PCC Pavements

As the variation of temperature throughout the slab thickness increases, a loss of support occurs under the PCC pavements. At night, the temperature at the top surface of the slab is lower than that of the bottom, and therefore, the top tends to contract. The self-weight of the slab constrains the top surface from contraction, which produces tensile stresses at the top of the slab as illustrated in a Figure 2-1a [Huang, 2004]. Top-down transverse cracking can develop on the slab surface when heavy traffic loads are applied at the edge of the slab at the same time. During the day (Figure 2-1b), the temperature at the top surface of the slab is higher than that of the bottom. Similarly, the top tends to expand while the bottom tends to contract, but the weight of the slab restrains them, and tensile stresses are induced at the bottom of the slab [Huang, 2004]. Bottom-up transverse cracking can develop when heavy traffic loads are applied near the center of the slab simultaneously. In the case of moisture variation, tensile stresses are imposed on the top of the slabs, due to the shrinkage of the concrete surface by evaporation. The curling and warping stresses, combined with traffic loading, predominantly affect the performance of PCC pavements. In MEPDG, both top-down and bottom-up cracking are considered to develop the total transverse cracking in JPCP [ARA, 2004]. The total transverse cracking is calculated by combining a percentage of predicted top-down and bottom-up slab cracking and subtracting the probability of both occurring simultaneously.

Westergaard [1926] investigated the stresses and deflections caused by the variations of temperature and stated that the stresses due to temperature variations should be determined by two aspects: the first is the early life of the pavement, before the pavement obtains its full strength and opens to the traffic; the second is the later life of the pavement, after it gains full
strength and opens to traffic. In the first point of view, the stresses caused by the temperature variations are an independent event to initiate the cracks and should be considered by themselves. However, in the second point of view, the stresses induced by the temperature variations should be combined with the stresses due to traffic loads. Bradbury [1938] developed a simple chart to easily calculate curling stresses, applying the dimensions and radius of the relative stiffness of the slab, based on Westergaard’s analysis. Teller and Sutherland [1935] studied the effects of variations in temperature and moisture on the stress development of the concrete pavement slabs. The results demonstrate that the temperature distribution through the slab thickness is not a linear shape, but a highly nonlinear shape. The measured curling stresses under a restrained condition of the concrete slab due to temperature variations are equal in importance to those produced by the heaviest legal wheel loads. Hudson and Flanagan [1987] examined the damage due to environmental effects when compared to that caused by traffic loads. Fourteen pairs of trafficked and untrafficked pavement sections in Arizona, California, Maryland, Minnesota, and Texas were selected to cover a wide range of environmental conditions. The overall damage on
trafficked pavement sections was more severe than the damage on untrafficked pavement sections; the damage discrepancy of the two sections proved to be generally greater in harsh environments. This study concluded that the observed difference of damage was not only due to traffic load alone, but also due to the interaction between traffic loads and environmental effect. The environmental effect may initially cause a thermal crack, while repeated traffic loads may intensify that crack. Thus, the interaction of traffic loads and environmental effect contributes to the deterioration of PCC pavements.

Armaghani et al. [1987] analyzed pavement temperature and evaluated the vertical and horizontal displacements in a 12-ft. joint spacing pavement. Thermocouples were installed at five different depths in a 9-in. thick slab. Linear variable differential transformers (LVDTs) were installed to measure the vertical displacement of slab. Figure 2-2 shows the typical temperature differential between the top and the bottom of the slab, demonstrating that the temperature difference proves to be negative during nighttime and positive during daytime. The maximum negative and positive temperature differences occur at 6 a.m. and 1 p.m. and the zero temperature difference occurs at approximately 9 a.m. and 7 p.m. Figure 2-3 displays the vertical displacements at the corner, edge, and center of a slab with respect to time. The vertical slab displacements are closely related to the temperature difference of the top and the bottom of the slab, as shown in both Figure 2-2 and Figure 2-3. The maximum vertical displacement at various slab locations is coincident with the maximum negative and positive temperature differences. The corner and edge of the slab curl upward (an increase in negative displacement as shown in Figure 2-3) during the nighttime, and curl downward (a decrease in negative displacement as shown in Figure 2-3) during the daytime. Similarly, the center of the slab curls downward (an
increase in positive displacement as shown in Figure 2-3) during the nighttime, and curls upward (a decrease in positive displacement as shown in Figure 2-3) during the daytime.

Figure 2-2 Typical temperature variation for 2-days [Armaghani et al., 1987]

Figure 2-3 Vertical displacement of slab at the center, edge, and corner [Armaghani et al., 1987]
Jeong and Zollinger [2005] constructed a concrete slab to investigate environmental effects on the behavior of JPCP. Climatic data, slab temperature and moisture distribution, and slab displacement and strain were monitored to predict the environmental effect on the pavement behavior. According to the day-to-day trends of slab strain data, the slab showed a periodic tensile and compressive strain every 24 hours at both the top and bottom. The corner displacement, caused by the temperature and moisture differences between the top and bottom of the slab, induced a curl-up in the morning and a curl-down in the afternoon.

Siddique et al. [2005] simulated the deflection of a slab under various conditions of temperature differential, by applying a finite element analysis. The results show that deflection by a temperature differential of a positive 20 °F is not twice the deflection of a temperature differential of positive 10 °F, but about 75 % higher. A positive temperature differential means the temperature of the surface is higher than that of the bottom. Furthermore, the magnitude of deflection by the same positive and negative temperature differential is not the same value. A deflection caused by a positive temperature differential is higher than that of a negative temperature differential. The amount of up-ward deflection (negative temperature differential) is about 70 % and 84 % of the down-ward deflection (positive temperature differential) for 5 °F and 10 °F temperature differentials, respectively. Belshe et al. [2011] studied the thermal behaviors of a concrete pavement with and without an open-graded asphalt rubber friction course. Placing a thin open-graded friction course overlay to rigid pavement reduced the temperature changes at the top of the slab between day and night. This reduction of temperature occurs due to the aeration effect of an open-graded friction course when combined with traffic. Adding a thin open-graded friction course mitigates the temperature difference between the top and bottom of the pavement; thus, reducing the curling stresses caused by the temperature differentials.
2.2. Sustainable PCC Pavements

The manufacture of Portland cement impacts environmental aspects, as an energy-intensive process that generates large emissions of CO₂ (carbon dioxide). A CO₂ is a primary greenhouse gas due to its prevalent dominance compared to other gases, such as NOₓ (nitrous oxide) and CH₄ (methane) [Malhotra, 2006]. Specifically, one ton of CO₂ is released to the atmosphere in order to produce one ton of Portland cement clinker. This accounts for approximately seven percent of the global CO₂ emission [Mehta, 2002]. Most of CO₂ emission is generated from the process of Portland cement production [Jahren, 2003]. Therefore, a long-term policy to mitigate the environmental impact could not address the reduction of its consumption rates, but importantly, the replacement of Portland cement in concrete mixtures with supplementary cementitious materials (SCMs). This literature review focuses on a brief description of fly ash and slag, together with their contribution to the fresh and hardened concrete properties. The use of ternary-blended concrete mixture is also reviewed subsequently. Figure 2-4 shows the Portland cement and several SCMs.

![Portland cement, grade 120 slag, silica fume, and class C fly ash](image)

Figure 2-4 Portland cement, grade 120 slag, silica fume, and class C fly ash [Rupnow, 2012]
2.2.1. Fly Ash

Fly ash is a by-product from the combustion of pulverized coal and is widely used as a cementitious ingredient in hydraulic cement concrete. To understand the characteristics and the factors affecting the quality of fly ash, a typical power plant operational process such as that of burning coal and collecting fly ash must be considered. Once the pulverized coal passes through a high-temperature area in the furnace, the volatile matter and carbon are burned off. However, most of the mineral impurities such as clays, quartz, and feldspar melt at the high temperature [Mehta and Monteiro, 2005]. The various gases with noncombustible residue are quickly transported into the precipitator. The electrostatic precipitator generates a magnetic field and the fly ash particles are collected automatically [Cook, 1983]. When the magnetic field is broken periodically, the collected fly ash falls into the collecting hoppers. A typical pattern of gas flow through an electrostatic precipitator is presented in Figure 2-5.

![Figure 2-5 Electrostatic precipitator](ACI 232, 2003)
ASTM C 618 (2008) categorizes fly ash into two distinct classes, based on the coal source. Class F fly ash with pozzolanic properties is produced from anthracite or bituminous coals that are found east of the Mississippi River. Class C fly ash with pozzolanic and cementitious properties is produced from lignite or subbituminous coals found in western states, such as Wyoming and Montana [Mindess et al., 2003]. The amount of three constituents (SiO$_2$, Al$_2$O$_3$, and Fe$_2$O$_3$) should exceed 70 % in order to be classified as a class F fly ash, while the three constituents should exceed 50 % in order to be classified as a class C fly ash, shown in Table 2-1. Class C fly ash generally contains more than 20 % of lime (CaO); thus, the amount of SiO$_2$, Al$_2$O$_3$, and Fe$_2$O$_3$ may be significantly less than that of class F fly ash. The high-lime ashes (class C fly ash) contain predominantly glassy particles, thus contributing to the strength development of concrete; the calcium oxide is easily combined with silicates and aluminates [Cook, 1983]. The incomplete combustion of coal serves as the main contributor to the carbon content in fly ash [ACI 232, 2003]. The carbon content is measured by the loss of ignition (LOI), since the carbon concentration of fly ashes is well correlated to the LOI. Class C fly ashes generally present low LOI values (less than 1 %), but class F fly ashes carry various ranges of LOI values (up to 20 %).

Table 2-1 Chemical composition of fly ashes [ASTM C 618, 2008]

<table>
<thead>
<tr>
<th></th>
<th>Class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>F</td>
</tr>
<tr>
<td>Silicon dioxide (SiO$_2$) plus aluminum oxide (Al$_2$O$_3$) plus iron oxide (Fe$_2$O$_3$), min %</td>
<td>70.0</td>
</tr>
<tr>
<td>Sulfur trioxide (SO$_3$), max %</td>
<td>5.0</td>
</tr>
<tr>
<td>Moisture content, max %</td>
<td>3.0</td>
</tr>
<tr>
<td>Loss on ignition, max %</td>
<td>6.0 (12.0)*</td>
</tr>
</tbody>
</table>

* The use of class F pozzolan containing up to 12.0 % loss of ignition may be approved by the user if either acceptable performance records or laboratory test results are made available.
The predominant fraction of fly ash particles are composed of glassy spheres of solid or hollow shape. The specific gravity of fly ash is normally less than that of Portland cement: thus, when fly ash is used to substitute a certain amount of Portland cement by the same mass, the amount of paste will increase with the constant water-cementitious ratio. A concrete mixture containing fly ash influences the demand to lower water in order to maintain a constant workability, thus reducing the bleeding capacity and decreasing the evolution of the heat of hydration [Swamy and Laiw, 1995; Neville, 1996]. It is well known that a concrete with a high-volume fly ash displays a lower strength development than Portland cement concrete at early ages, due to the slow rate of hydration of fly ash [Swamy et al., 1983; Langley et al., 1992]. Malhotra et al. [2000] studied the long-term compressive strength development of high-strength concrete, incorporating SCMs for as much as ten years. A high-volume fly ash concrete mixture, 57 % of total cementitious materials was replaced by fly ash, was casted; then a compressive strength test was performed at several ages. Although the compressive strength of fly ash concrete demonstrates lower values than that of Portland cement concrete until 28 days, its compressive strength surpasses the compressive strength of Portland cement concrete after 91 days. The development of compressive strength between 28 days and 10 years of Portland cement concrete was 70 %, while that of fly ash concrete was 120 %. The fly ash concrete, although exhibiting lower strength development at early ages, usually presents a higher strength development at later ages, due to its continuous pozzolanic activities. Lane and Best [1982] investigated the fly ash concrete characteristics of the modulus of elasticity. Similar to the compressive strength of fly ash concrete, the modulus of elasticity of fly ash concrete is lower at early ages, and a little higher at a later time, as compared to Portland cement concrete without fly ash. Yet, the impact of fly ash on the modulus of elasticity is not as critical as the influence of fly
ash on the compressive strength. Khatri and Sirivivatnanon [2001] investigated and noted that the most effective replacement of fly ash for chloride resistance durability in aggressive and moderate environments was 40% and 30%, respectively. Due to its beneficial effects on the properties of fresh and hardened concrete, as well as a lower cost, fly ash is increasingly used as a supplementary material or a component of blended cement.

2.2.2. Ground Granulated Blast-Furnace Slag

Ground granulated blast-furnace slag (hereafter referred to as slag) is a by-product from metallurgical processes either from production of metals from ore or refinement of impure metals [Mindess et al., 2003]. Blast-furnace slag contains large amounts of lime, silica, and alumina, which are suitable compositions for use in concrete. The molten slag must be rapidly quenched as it leaves the blast-furnace, in order to enhance the hydraulic properties [ACI 233, 2003]. In the granulation process, the rapid quenching minimizes crystallization and produce a hydraulically active calcium aluminosilicate glass. The most common process for granulating slag for cementitious materials is quenching with water, but more efficient granulating process, which requires less water, uses high-pressure water jet as shown in Figure 2-6 [ACI 233, 2003].

![Pelletizing process for quenching slag](image)

Figure 2-6 Pelletizing process for quenching slag [Regourd, 1983]
Slags are subdivided into three grades: grade 80, 100, and 120, as determined by the slag activity index [ASTM C989, 2010]. The slag activity index represents the ratio of the average compressive strength of the slag-reference cement cubes (50 to 50 mass combinations of slag and Portland cement) to the average compressive strength of the reference cement cubes, multiplied by 100. The higher grade of slag implies increased levels of reactivity or faster strength development. The detailed slag activity index of each grade of slag (7 and 28 days) is presented in Table 2-2. The slag primarily consists of silicon, calcium, aluminum, magnesium, and oxygen (95%) [ACI 233, 2003]. Advanced blast-furnace slag technology provides very low variability in the compositions of slag from a single source.

Table 2-2 Slag activity index [ASTM C989, 2010]

<table>
<thead>
<tr>
<th>Designation</th>
<th>Relative strength* , min %</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>7 days</td>
<td></td>
<td>28 days</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Average</td>
<td>Individual</td>
<td>Average</td>
<td>Individual</td>
</tr>
<tr>
<td>Grade 80</td>
<td></td>
<td>-</td>
<td>-</td>
<td>75</td>
<td>70</td>
</tr>
<tr>
<td>Grade 100</td>
<td></td>
<td>75</td>
<td>70</td>
<td>95</td>
<td>90</td>
</tr>
<tr>
<td>Grade 120</td>
<td></td>
<td>95</td>
<td>90</td>
<td>115</td>
<td>110</td>
</tr>
</tbody>
</table>

(* Percentage of strength of a reference mortar made with pure cement)

A large amount of slag (20-70 % by mass) can be used for replacement of Portland cement since the chemical compositions of slag are closely related to that of Portland cement [Mehta and Monteiro, 2005]. This range of slag replacement proportions should be adjusted depending on the purpose of use, the curing temperature, the grade of slag, and the type of Portland cement. Several researchers studied that the workability of concrete containing slag was improved by a better particle dispersion and higher fluidity of the pastes and mortars [Fulton, 1974; Wood, 1981; Wu and Roy, 1982; Haque and Chulilung, 1990]. Kelham et al. [1995]
studied the development of compressive strength of cement pastes and mortars at various ages. The compressive strength increases after 7 days, when slag is added to the mixture. However, the compressive strength did not develop until 28 days when fly ash was added. Kashima et al. [1992] found that a replacement of slag in the concrete mixture reduces heat generation during the hydration process, and potential thermal cracking is reduced correspondingly. Hogan and Muesel [1981] investigated the influence of proportions of slag replacement on the strength and rate of strength gain at the age of 3, 7, and 28 days. Three levels of slag replacements were examined by mass (40, 50, and 65%) for both concrete compressive and flexural strength development. Compared to Portland cement concrete, concrete contains slags gains compressive more slowly until three days; however, the compressive strength of slag concrete overtakes the compressive strength of Portland cement concrete after seven to ten days. Similar trends were also shown in the flexural strength development. The optimum proportion of slag replacement for concrete strength development appears to be a 40% replacement. Klieger and Isberner [1967] found that the modulus of elasticity in concrete containing slag presents the same modulus of elasticity that of Portland cement concrete. Brook et al. [1992] also stated that water-cured concrete containing fly ash showed no improvement of elastic modulus compared to the Portland cement without fly ash at an early age.

2.2.3. Ternary Mixtures Containing Fly Ash and Slag

With the current demand for Portland cement concrete (PCC) sustainability, supplementary cementitious materials (SCMs) are often used in concrete mixtures to improve the mixture properties in both fresh and hardened concrete. Nehdi [2001] explained the two synergistic actions at work in ternary-blended cement concrete. First, various sizes and spatial distributions of cementitious particles provide a tight particle-packing density. The dense
particle-packing of the cementitious blend can improve the rheological properties and mechanical strength of concrete. Second, ternary-blended cement containing pozzlanic materials presents an extra chemical reaction, which in turn provides long-term strength development. Binary and ternary-blended cementitious combinations are encouraged to develop for both advanced technical performance specification and a reduction of CO₂ emission during the production of Portland cement [Mehta, 2002]. However, binary-blended cements is often underestimated by its shortcomings, such as a lower early-age strength, an increased use of admixtures, a need for extended moisture curing, and an increased cracking tendency, due to plastic shrinkage [Nehdi, 2001]. Ternary-blended cement can be used for optimizing the engineering properties of concrete by enabling each ingredient to compensate for any mutual shortcomings. ASTM provides the specification for ternary-blended cement. Approved in 2011, ASTM C595, as the standard specification for blended hydraulic cements, explains a designation for ternary-blended cement. Specifications for ternary-blended cement define a hydraulic cement, consisting of an intimate and uniform blend, produced either by blending Portland cement with 1) two different pozzolans, or 2) slag cement and a pozzolan, or 3) a combination of intergrinding and blending.

The ternary mixtures, including fly ash and slag, enhance mixture properties in both fresh and hardened concrete. Fly ash and slag are broadly used as a cementitious ingredient in hydraulic cement concrete for many reasons, including a reduction in temperature rise during initial hydration, improved resistance to sulfates, reduced expansion due to alkali-silica reactions (ASR), and lastly, a contribution to the durability and strength of hardened concrete [ACI 232, 2003; Mehta and Monteiro, 2005]. Berry [1980] investigated the compressive strength development of ternary mortars containing fly ash and slag. It was stated that no interaction
between fly ash and slag was observed when fly ash and slag were used together with Portland cement. Li and Zhao [2003] studied the influence of the combination of fly ash and slag on the strength development of concrete. The comparison of strength development was performed for an ordinary Portland cement concrete, a high-volume fly ash concrete (containing 40 % fly ash), and ternary-blended concrete (combination of 25 % fly ash and 15 % slag) at several ages. The compressive strengths of the ternary-blended concrete were similar to the compressive strengths of ordinary Portland cement concrete up to 28 days, while the compressive strengths of the high-volume fly ash remained lower than that of ordinary Portland cement concrete. The compressive strengths of ternary-blended concrete clearly exceeded the compressive strengths of ordinary Portland cement concrete after 56 days. Douglas and Pouskouleli [1991] found a statistical approach for the equations governing the strength development of ternary mixture containing fly ash and slag at different ages. The proposed governing equations were based on a minimum of seven experimental data points and verified by comparing the compressive strength values at several ages (1, 7, 28, and 90 days). Despite a few outliers, the proposed equations showed a good accuracy of 95 % or greater.

2.3. Enhanced Integrated Climatic Model (EICM)

The enhanced integrated climatic model (EICM) is a one-dimensional coupled heat and moisture flow model, adopted for use in the MEPDG [Heydinger, 2003]. In 1989, the integrated climatic model (ICM) was introduced, which combined three models: the climatic-material-structural model (CMS); the cold regions research and engineering laboratory (CRREL); and the infiltration and drainage model (ID). Figure 2-7 provides a summary of the major elements of the integrated climatic model.
The EICM was released after further improvement of ICM through the NCHRP 1-37A project [ARA, 2004]. The EICM requires the climatic data obtained from the weather station to
serve as input data, inclusive of temperature, wind speed, sunshine, precipitation, humidity, and the water table [Larson and Dempsey, 1997]. The climatic data can be found at the MEPDG website for an average of ten years or can be generated manually, using local climatic data [ARA, 2004]. The typical outputs of EICM are the following: hourly PCC temperature profiles for use in cracking and faulting models for JPCP and punch-out for continuously reinforced concrete pavement (CRCP); a freezing index for JPCP performance prediction; and lastly, monthly humidity values for use in JPCP and CRCP modeling of moisture profiles.

The most crucial factors involved in a heat transfer between pavement and atmosphere are meteorological parameters, shown in Figure 2-8. In addition, convection and radiation are important parameters for the heat transfer between pavement surface and air, while conduction impacts the heat transfer within pavement thicknesses [Dempsey and Thompson, 1970]. In order to predict the pavement temperature, the energy balance at the pavement surface used in the EICM model is described in Equation 2-1.

\[ Q_l - Q_r + Q_a - Q_e + Q_c + Q_h + Q_g = 0 \]  
(2-1)

where, \( Q_i \) is incoming short wave radiation; \( Q_r \) is reflected short wave radiation; \( Q_a \) is incoming long wave radiation; \( Q_e \) is outgoing long wave radiation; \( Q_c \) is convective heat transfer; \( Q_h \) is the effects of transpiration, condensation, evaporation, and sublimation; and \( Q_g \) is energy absorbed by the ground. The net radiation flux on the pavement surface is determined based on the short wave absorptivity of the pavement surface and extraterrestrial radiation [Heydinger, 2003; Johanneck and Khazanovich, 2010]. The short wave absorptivity, as an input parameter of EICM, is determined by the type of pavement. The extraterrestrial radiation relies on the latitude and solar declination. The convection of heat transfer between pavement and air is dependent upon the net radiation flux and wind speed, while heat convection is always transferred from high
temperature to low temperature between the pavement and the air. A percentage sunshine is essential in calculating the heat balance on the pavement surface; the percentage of sunshine is numerically represented by the cloud cover. Although precipitation data is not used to determine the surface heat flux condition, precipitation data is utilized to calculate the infiltration of moisture in the pavements. The relative humidity values are used to model moisture gradients in the pavements.

![Figure 2-8 Heat transfer between air and pavement as modeled by EICM [Dempsey and Thompson, 1970]]
By including EICM, MEPDG pays more attention to the climatic effect on design procedures than the 1993 American Association of State Highway and Transportation Officials (AASHTO) design guide [Heydinger, 2003]. This study finds that in the 1993 AASHTO design guide, only 4% of the guide addresses drainage and climatic effects, while most of the information is focused on the weighted average of the subgrade support [Connecticut DOT, 2006]. On the other hand, MEPDG uses 38% of its design guide in order to explain the effect of climatic condition on the design procedures, such as the climatic effect on material properties and the temperature influence on slab curling and warping. Some input parameters of EICM directly affect the performance of PCC pavements. For example, temperature and solar radiation impact the joint load transfer, freeze and thaw cycle, slab curling, and crack width. Wind speed is a critical factor for heat transfer at the pavement surface. Humidity directly influences moisture warping, dry shrinkage, and initial crack width. The EICM was designed and calibrated using nationwide climatic data so that local calibration processes based on regional climatic patterns and material properties are required to enhance the reliability of the outputs. Ahmed et al. [2005] evaluated the suitability of the EICM model to predict subsurface temperature conditions in New Jersey. The validation procedures were conducted by comparing the EICM prediction values with the field measurement values for each test section. Since specific climatic data from the weather station, such as wind speed and percentage sunshine, were not available for the test section, regional average values were used. According to a sensitivity analysis of the two unavailable parameters, the impact of wind speed on the predicted pavement temperature becomes more significant than that of the sunshine percent. The report was stated that EICM-predicted surface temperatures tend to be 5 to 10 °C greater than the field-measured temperature; the study noted that the discrepancy was improved below the 12-in. depth from the surface,
concluding that there exists no strong correlation between EICM-predicted and field-measured temperatures. Furthermore, an adjustment of the EICM model is required to effectively implement the EICM in New Jersey.

A temperature calibration of the EICM model was performed to utilize long-term pavement performance (LTPP) data in New Mexico [Zhang and Zhang, 2009]. Since the percentage of sunshine was not available from LTPP data for the test sites, assumptions were made based on the local historical data in order to obtain the percentage of sunshine for each site. The comparisons indicated that the measured surface temperature was always higher than the EICM-predicted surface temperature, due to a special soil condition in New Mexico. A correlation between the measured surface temperature and the EICM-predicted surface temperature is presented in Equation 2-2.

\[
\text{Surface Temp. (°F)} = 0.9193 \times [\text{EICM} - \text{Predicted Surface Temp. (°F)}] + 14.085 \quad (2-2)
\]

2.4. Temperature Measurements in PCC Pavements

Daily and seasonal temperature fluctuation in the concrete slab presents a main factor influencing the behavior of concrete pavements. The analytical solution for curling stress by Westergaard [1926] and Bradbury [1938] was developed, based on the assumption of a linear temperature gradient throughout the depth of the concrete slab. Teller and Sutherland [1935] reported that the actual temperature distributions through slab thickness were not uniform, yet were highly nonlinear, as presented in Figure 2-9. It was stated that the temperature distribution through the slab thickness is affected by not only the air temperature but also several other factors, such as the angle of incidence of the sun’s ray, the previous temperature conditions, temperature of the subgrade, the moisture condition, and the humidity of atmosphere.
Thomlinson [1940a and 1940b] first addressed the curling stresses caused by a nonlinear temperature gradient in a concrete slab. The stresses due to nonlinear temperature distribution were divided into three components, as shown in Figure 2-10. The three divided components are based on the classical plate-bending theory, that the cross-section of a plate must remain a plane after bending; thus, three components can only cause axial or bending stresses in the plate.

(a) Temperature gradients of 6- and 9-in. slab (February)

(b) Temperature gradients of 6- and 9-in. slab (July)

Figure 2-9 A typical daily and seasonal temperature variation in concrete pavement (Arlington road tests, Washington D.C.) [Teller and Sutherland, 1935]
The axial stress component (Figure 2-10a) is due to a uniform temperature change that causes an expansion or contraction of the slab. This type of stress in joint concrete pavement at a mature age is usually ignored. The linear bending stress component (Figure 2-10b) is caused by a linear temperature distribution. The nonlinear self-equilibrating internal stress component (Figure 2-10c) is obtained by subtracting a uniform and linear temperature component from the total temperature distribution.

![Stress components](image)

Figure 2-10 Stress components due to nonlinear temperature gradient [Hiller and Roesler, 2010]

Lang [1941] investigated the variations of temperature changes in a concrete pavement slab under a northern climatic condition. In Figure 2-11, the solid line represents the weekly maximum and minimum air temperatures; the broken line represents the weekly maximum and minimum temperatures in the center of the concrete slab. The maximum weekly variation of concrete temperature occurred during June with a value of 47 °F. The maximum annual variation of concrete temperature was 115 °F. The maximum concrete temperature was 108 °F in July when the corresponding air temperature was 96 °F, while the minimum concrete temperature was
-8 °F in February, with a corresponding air temperature of +7 °F. Lang compared the temperature distribution between an actual nonlinear gradient and a straight line relationship from top to bottom, for a 7-in. slab thickness. The maximum temperature difference between the straight line gradient and the actual nonlinear gradient was less than 2 °F at 2.5 and 4.5-in. below the slab surface. It was concluded that a temperature gradient can be considered to be a linear distribution for convenience, because the small discrepancy of the straight line from an actual nonlinear distribution is not critical, compared to many variables affecting the design process.

Figure 2-11 Max. and min. air and concrete temperature for 12 months (Minneapolis, MN) [Lang, 1941]

Choubane and Tia [1992] suggested that a simple quadratic equation can adequately capture the temperature distribution through the depth of the slab. The quadratic temperature equation can be expressed as a function of depth (z):
\[ T(z) = A + Bz + Cz^2 \]  \hspace{1cm} (2-3)

\[ A = T_{top}, \quad B = \frac{(4T_{mid} - 3T_{top} - T_{bot})}{h}, \quad C = \frac{2(T_{top} + T_{bot} - 2T_{mid})}{h^2} \]

Where, \( A, B, \) and \( C \) are regression coefficients based on the measured slab temperature, \( T \) is the temperature in Fahrenheit, and \( z \) is the slab depth. \( T_{top}, T_{mid}, \) and \( T_{bot} \) are the measured temperatures at the top, middle, and bottom of the slab, respectively, and \( h \) is the slab thickness.

Harik et al. [1994] proposed a technique to calculate stresses under nonlinear temperature distribution through the slab thickness by using a two-dimensional plate element. An equivalent linear temperature gradient was introduced to account for the nonlinear temperature distributions. The equivalent linear temperature gradients are equivalent to the nonlinear temperature gradients, in the sense that they yield the same axial force and moment. The study suggested that the stress caused by temperature loading can accelerate the deterioration of the pavement performance, when wheel loads are applied at the corner of the deformed pavement. The assumption of linear temperature gradients, as derived from a temperature difference between the top and bottom surfaces of a pavement, cannot accurately predict the stress; thus, nonlinearity of the temperature gradient through the slab thickness should be considered. Mohamed and Hansen [1996 and 1997] suggested a method to calculate a curling stress in a concrete pavement under both linear and nonlinear temperature gradients. According to the results of comparison, the curling stress induced by a linear temperature gradient not only underestimates the tensile stress at the top of the slab during the nighttime, but also overestimates the tensile stress at the bottom of the slab during daytime. This is because the linear temperature gradient causes an equal and opposite curling stress at the two surfaces. It is concluded that the temperature gradient (shape) rather than the temperature difference is more critical to an accurate prediction of curling stress in concrete.
pavements. Although the tensile stresses caused by temperature curling are insufficient to generate fatigue cracking, the tensile stress may be magnified when combined with wheel loading stresses. Zhang et al. [2003] concurred with the results from Mohamed and Hansen [1996 and 1997] that tensile stress due to linear temperature distribution at the top of the slab underestimates by 55.9 % during nighttime, while tensile stress due to linear temperature distribution at the bottom of the slab overestimates by 74.9 % during daytime in contrast to corresponding tensile stresses computed using nonlinear temperature distributions, respectively.

Many researchers [Khazanovich, 1994; Mohamed and Hansen, 1997; Zhang et al., 2003; Jeong and Zollinger, 2005; Belshe et al., 2011; Hiller and Roesler, 2010] used third-order polynomials to accurately predict the actual nonlinear temperature gradient in concrete pavements. Figure 2-12 shows the typical nonlinear temperature gradients of a concrete slab (9-in. thickness) during April and November in Illinois.

(a) Temperature gradients in April  
(b) Temperature gradients in November

Figure 2-12 Temperature gradients of 9-in. PCC slab in Illinois [Thompson et al., 1987]
2.5. Thermal Properties of PCC Pavements

The thermal properties, such as the coefficient of thermal expansion (CTE), thermal conductivity (TC), and heat capacity (HC), have been considered to be the fundamental factors that control the severity of curling of the PCC pavement, yet these thermal properties have never played an important role in the thickness design procedures for PCC pavement. In the MEPDG, the thermal properties became direct input parameters that were closely related to the pavement performance [ARA, 2004]. CTE is used to predict the amount of thermal cracking and its effect is significant on PCC pavement performance [Mallela et al., 2005]. The provisional AASHTO test method (TP 60) of measuring the CTE of hydraulic cement concrete was recently replaced by the AASHTO T 336-09 standard in order to correct the calibration constant used in calculating the CTE value [AASHTO TP60, 2005; AASHTO T336, 2009; Stephanos, 2009]. Thermal conductivity and heat capacity are used to calculate the temperature gradient throughout the pavement. Thermal conductivity and heat capacity present main input parameters in estimating the temperature and moisture gradients in PCC pavement for the enhanced integrated climatic model (EICM), the built-in software in MEPDG [Heydinger, 2003].

2.5.1. Coefficient of Thermal Expansion (CTE)

The same as other construction materials, Portland cement concrete carries a positive coefficient of thermal expansion, but the value varies depending on the composition of mixtures and its moisture conditions at the time of the temperature change [Neville, 1996]. The CTE of aggregate directly influences the CTE value of concrete, because a higher CTE of the aggregate generally causes a higher CTE value of concrete, but CTE values depend on the aggregate proportions in the mixture. If the difference of CTE between coarse aggregate and hydraulic cement paste becomes too large, a breakage of the bond between coarse aggregate particles and
the surrounding paste may cause due to the differential movements [Neville, 1996]. According to Mindess et al. [2003], the CTE of limestone and gravels are 2.0 to 3.6 με/°F and 5.5 to 7.1 με/°F (3.6 to 6.5 με/°C and 9.9 to 12.8 με/°C), respectively. The CTE of cement pastes ranges between 10 and 11 με/°F (18 and 19.8 με/°C), while the CTE of cement paste is much higher than that of coarse aggregates. Typical CTE ranges for various aggregates and cement paste are presented in Table 2-3.

Table 2-3 Typical CTE ranges for concrete constituents [Mindess et al., 2003]

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Coefficient of Thermal Expansion, 10^-6/°C (10^-6/°F)</th>
<th>Concrete Coefficient of Thermal Expansion (made from this material), 10^-6/°C (10^-6/°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Aggregates</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Marble</td>
<td>4.0 - 7.0 (2.2 - 3.9)</td>
<td>4.1 (2.3)</td>
</tr>
<tr>
<td>Limestone</td>
<td>3.6 - 6.5 (2.0 - 3.6)</td>
<td>6.1 - 9.2 (3.4 - 5.1)</td>
</tr>
<tr>
<td>Granites &amp; Gneisses</td>
<td>5.8 - 9.5 (3.2 - 5.3)</td>
<td>6.8 - 9.5 (3.8 - 5.3)</td>
</tr>
<tr>
<td>Syenites, Diorites, Andesite, Basalt, Gabbros, Diabase</td>
<td>5.4 - 8.1 (3.0 - 4.5)</td>
<td>7.9 - 9.5 (4.4 - 5.3)</td>
</tr>
<tr>
<td>Dolomites</td>
<td>7.0 - 9.9 (3.9 - 5.5)</td>
<td>9.2 - 11.5 (5.1 - 6.4)</td>
</tr>
<tr>
<td>Blast Furnace Slag</td>
<td>–</td>
<td>9.2 - 10.6 (5.1 - 5.9)</td>
</tr>
<tr>
<td>Sandstones</td>
<td>10.1 - 12.1 (5.6 - 6.7)</td>
<td>10.1 - 11.7 (5.6 - 6.5)</td>
</tr>
<tr>
<td>Quartz Sands &amp; Gravels</td>
<td>9.9 - 12.8 (5.5 - 7.1)</td>
<td>10.8 - 15.7 (6.0 - 8.7)</td>
</tr>
<tr>
<td>Quartzite, Cherts</td>
<td>11.0 - 12.6 (6.1 - 7.0)</td>
<td>11.9 - 12.8 (6.6 - 7.1)</td>
</tr>
<tr>
<td><strong>Cement Paste (saturated)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>w/c=0.4 to 0.6</td>
<td>18.0 - 19.8 (10 - 11)</td>
<td>–</td>
</tr>
<tr>
<td><strong>Concrete Cores</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cores from LTPP pavement sections, many of which were used in calibration</td>
<td>N/A</td>
<td>7.2 x 10^{-6} - 9.9 x 10^{-6} - 13.0 x 10^{-6} (Min – Mean – Max)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4.0 x 10^{-6} - 5.5 x 10^{-6} - 7.2 x 10^{-6} (Max – Mean – Min)</td>
</tr>
</tbody>
</table>
Due to the variances 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 where concrete contains crushed limestone with siliceous sand, the CTE decreases steeply when the amount of crushed limestone increases as shown in Figure 2-13. This is because the CTE of limestone is much smaller than that of cement paste. However, with quartz gravel and siliceous sand, the concrete CTE increases slowly with the increase of the amount of quartz gravel.

![Figure 2-13 Effect of aggregate content on thermal expansion of concrete [Zoldners, 1971]](image)

Mallela et al. [2005] tested 673 cores representing hundreds of pavement sections as part of the long term pavement performance (LTPP) program throughout the United States. The general range of CTE values of PCC lies between 5 and 7 με/°F (9 and 12.6 με/°C), and concrete made from igneous aggregates shows CTE values around 5.2 με/°F or 5.3 με/°F (9.4 με/°C or 9.5 με/°C) and the aggregates made from sedimentary rock have 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. [1992] 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. However, the water/cement ratio (0.53, 0.45, and 0.33) and cement content (508 lb/yd³, 564 lb/yd³, and 752 lb/yd³) did not statistically display significant effects on the CTE. Kohler et al. [2006] observed that the difference in the CTE of oven-dried specimens between the expansion and contraction were remarkable, and that difference was reduced during the first 10 to 15 hours. The expansion of CTE decreased considerably, while the contraction of CTE stayed constant. This is due to rise in temperature that 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 [Neville, 1996]. Simon and Dallaire [2002] emphasized that the CTE is remarkably sensitive to the relative humidity (RH) in the mixture during the test. The CTE of concrete reaches a maximum value at 60 to 70 % RH. The value at 100 % RH is 20 to 25 % less than the maximum value. However, from a testing standpoint, the fully saturated condition is the most practical [Simon and Dallaire, 2002]. Figure 2-14 shows the variation of CTE with RH of concrete cement paste.

The CTE is commonly defined by constant value, but it has been known that CTE varies, depending on the 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 [Bazant, 1970].
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, followed by time dependent contraction, because excess pore pressure, created by the expansion of each constituent, dissipates by moving to an empty pore space. This phenomenon is also effective in a cooling process, where an immediate contraction occurs, followed by a time dependent expansion. Figure 2-15(a) explains the pure thermal dilation with various RHs where the 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 a partially-saturated condition, with a space much larger than gel water.
An increasing temperature causes 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 more easily than a thin layer of gel water, as shown in Figure 2-15(b).

![Figure 2-15 An estimated typical response to a step input of temperature: (a) pure thermal dilation, (b) thermal shrinkage, and (c) hygrothermal dilation [Bazant, 1970]](image)

3. Relative humidity change

Once the RH increases over 45 %, capillary tension plays the most important role in shrinkage and dilation mechanism, yet capillary tension doesn’t exist below 45 % RH
due to the instability of meniscus [Bazant, 1970]. 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:

\[ P = \frac{2\gamma}{r} \]  

(2-4)

where, \( \gamma \) is the surface tension of the poor fluid, and \( r \) is the average radius of meniscus curvature.

\[ \frac{2\gamma}{r} = \frac{-\ln(RH)RT}{v'} \]  

(2-5)

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.

\[ P = \frac{-\ln(RH)RT}{v'} \]  

(2-6)

The increased temperature causes expansion of gel water, thus the radius of meniscus increases as well. The increased radius of meniscus leads to a decreased surface tension and an increased RH, according to the Kelvin equation, which is a physicochemical equilibrium between the vapor and liquid phases in Equation (2-5) [Bazant, 1970]. In Equation (2-4), the negative pressure acting on the pore system decreases as the surface tension decreases, thus allowing the decreased negative pressure on the pore system to expel the solid particles away from one another, as shown in Figure 2-16. The combination of both the Laplace and Kelvin equations provides the direct relationship between RH and pore fluid pressure in Equation (2-6).

The combination of these three components is summarized in Figure 2-17. Both long-term and immediate thermal dilation due to an increasing temperature show a maximum value at 70 % RH, while dried and saturated conditions show the minimum values in long-term
thermal dilation, due to the absence of capillary meniscus. Thus, the result displays a good agreement with Grasley and Lang [2007], in that the primary shrinkage and dilation mechanism is regarded as capillary tension, when RH is greater than 45%.

![Figure 2-16 Dilation of solid particles caused by capillary relaxation as response to an increasing temperature [Bazant, 1970]](image)

Figure 2-16 Dilation of solid particles caused by capillary relaxation as response to an increasing temperature [Bazant, 1970]

![Figure 2-17 Combination of three components of thermal dilation for various RH [Bazant, 1970]](image)

Figure 2-17 Combination of three components of thermal dilation for various RH [Bazant, 1970]

Won [2005] evaluated the effect of concrete age on CTE and found that CTE values do not change with the age of concrete for up to 3 weeks. However, Jahangirnejad et al. [2008] 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. The difference of CTE between 28 days and 180 days varies from 0.08 to 0.52 $\mu e/\degree F$ (0.15 to 0.94 $\mu e/\degree C$). Hossain et al. [2006] studied the design strategy to alleviate the detrimental effect of higher CTE values and found that increasing PCC strength provided an alternative method. Specifically, by increasing the strength parameters of PCC pavement, the amount of cracking was reduced. An increased PCC slab thickness as an increased dowel diameter eliminated slab cracking as well. Among all these alternatives, a 14-ft. widened lane was chosen for the most effective method, since no additional cost was required. Mallela et al. [2005] analyzed the effect of CTE on mean transverse joint faulting, a percentage of slabs with transverse cracking, and the international roughness index (IRI). Three CTE values of 4.5, 5.5, and 7.0 $\mu e/\degree F$ (8.1, 9.9, and 12.6 $\mu e/\degree C$), two transverse joint spacing of 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. An increased CTE causes a high percentage of transverse cracking; transverse cracking increases remarkably for the longer slab length of 20-ft. (6.1 m), even including the smaller CTE values. A higher CTE generally results in increased IRI, due to increased transverse joint faulting and transverse cracking.

2.5.2. Thermal Conductivity (TC) and Heat Capacity (HC)

Thermal conductivity and heat capacity are pertinent to the temperature profiles throughout the slab thickness and thermal cracking of PCC pavements. Thermal conductivity is defined by the ratio of a heat flux to a temperature gradient (Btu per hour per square ft. when temperature difference is 1 °F per ft.) [Neville, 1996]. In other words, thermal conductivity
represents a uniform heat flow through a unit thickness of material between two faces of unit area that are subjected to a unit temperature difference [Mindess et al., 2003]. A typical thermal conductivity value listed in Table 2-4 shows that basalt has a low thermal conductivity while quartzite has a high thermal conductivity. The thermal conductivity of water is less than that of cement paste, thus a lower w/c ratio of cement paste has a higher thermal conductivity. Therefore, the thermal conductivity of water and air are remarkably lower than that of solid materials, so that the degree of saturation and density of specimen have a great impact on the thermal conductivity values of concrete [Mindess et al., 2003]. Kim et al. [2003] studied the seven parameters that mostly affect thermal conductivity and found that the aggregate volume fraction and moisture condition of both cement paste and concrete specimens are the most influential factors on thermal conductivity values. The temperature effect on thermal conductivity is diminutive at room temperature, but becomes complex at a high temperature [Neville, 1996]. Thermal conductivity increases slowly as the temperature increases up to 50 to 60 °C, abruptly decreasing up to 120 °C, due to a loss of water by high temperature.

Heat capacity is defined as the ability of a given volume of a substance to store internal energy, while undergoing a given temperature change (Btu. per lb. per 1 °F) [Rohsenow et al., 1998]. Heat capacity is little affected by the type of aggregate, since heat capacity is not sensitive to mineralogical characteristics [Mindess et al., 2003]. The heat capacity of concrete increases when the moisture contents and temperature increases, but the heat capacity will decrease when a density of concrete increases [Whiting et al., 1978]. A specific heat value of a material can be converted into the volume heat capacity (VHC) by multiplying the density of the material [Morabito, 1989]. Typical heat capacity values are listed in Table 2-4.
Table 2-4 Typical TC and HC ranges for concrete constituents [Mindess et al., 2003]

<table>
<thead>
<tr>
<th>Material Type</th>
<th>Thermal conductivity, W/m·K (Btu/ft·h·°F)</th>
<th>Heat capacity, J/Kg·°C (Btu/lb·°F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregates</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Granite</td>
<td>3.1 (1.8)</td>
<td>800 (0.19)</td>
</tr>
<tr>
<td>Basalt</td>
<td>1.4 (0.8)</td>
<td>840 (0.20)</td>
</tr>
<tr>
<td>Limestone</td>
<td>3.1 (1.8)</td>
<td>–</td>
</tr>
<tr>
<td>Dolomite</td>
<td>3.6 (2.1)</td>
<td>–</td>
</tr>
<tr>
<td>Sandstone</td>
<td>3.9 (2.3)</td>
<td>–</td>
</tr>
<tr>
<td>Quartzite</td>
<td>4.3 (2.5)</td>
<td>–</td>
</tr>
<tr>
<td>Marble</td>
<td>2.7 (1.6)</td>
<td>–</td>
</tr>
<tr>
<td>Cement Paste (saturated)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>w/c= 0.4</td>
<td>1.3 (0.75)</td>
<td>–</td>
</tr>
<tr>
<td>w/c= 0.5</td>
<td>1.2 (0.7)</td>
<td>–</td>
</tr>
<tr>
<td>w/c= 0.6</td>
<td>1.0 (0.6)</td>
<td>1600 (0.38)</td>
</tr>
<tr>
<td>Concrete</td>
<td>1.5 - 3.5 (0.9 - 2.0)</td>
<td>840 - 1170 (0.2 - 0.28)</td>
</tr>
<tr>
<td>Water</td>
<td>0.5 (0.3)</td>
<td>4200 (1)</td>
</tr>
<tr>
<td>Air</td>
<td>0.03 (0.02)</td>
<td>1050 (0.25)</td>
</tr>
<tr>
<td>Steel</td>
<td>120 (70)</td>
<td>460 (0.11)</td>
</tr>
</tbody>
</table>

2.6. Analytical Solution of Thermal Stress

2.6.1. Elastic Plate Theory

The general solution for a slab subjected to a linear temperature gradient was presented by Westergaard [1926] based on the bending of plates with uniform thickness as shown in Figure 2-18.
Figure 2-18 A finite slab subjected to a temperature gradient [Mohamed and Hansen, 1996]

The right-hand side of the equations explains the total curvature of the slab. The first part of the equation represents the moment curvature in x and y-directions, and the second part of the equation represents the curvature caused by the linear temperature gradients.

\[-\frac{\partial^2 \omega}{\partial x^2} = \frac{12}{E h^3} (M_x - v M_y) + \frac{\alpha \Delta T(z)}{h}\]  \hspace{1cm} (2-7)

\[-\frac{\partial^2 \omega}{\partial y^2} = \frac{12}{E h^3} (M_y - v M_x) + \frac{\alpha \Delta T(z)}{h}\]  \hspace{1cm} (2-8)

where
\(w\) = the displacement in z-direction
\(h\) = the slab thickness
\(E\) = the modulus of elasticity
\(v\) = Poisson’s ratio
\(\alpha\) = the coefficient of thermal expansion
2.6.2. Residual Stress due to Internal Restraint

According to Mohamed and Hansen [1996], the nonlinear stress distribution within a concrete slab subjected to a nonlinear temperature gradient can be analyzed by the summation of residual stresses and curvature stresses. Residual stresses due to the internal restraint were developed to satisfy the continuity requirements. Residual stresses are independent of the external restraints of the slab and boundary conditions. The residual stresses equation is as follows:

\[
\sigma_{\text{residual}} = \frac{E_c}{(1-\nu)} \left[ -\varepsilon(z) + \frac{12M^*}{h^3}(z) + \frac{N^*}{h} \right]
\]  

(2.9)

\(N^*\) and \(M^*\) are constants which are dependent on the temperature distribution and obtained by the following integration:

\[
N^* = \int_{-\frac{h}{2}}^{\frac{h}{2}} \varepsilon(z) dz = \int_{-\frac{h}{2}}^{\frac{h}{2}} \alpha T(z) \, dz
\]  

(2-10)

\[
M^* = \int_{-\frac{h}{2}}^{\frac{h}{2}} \varepsilon(z) \alpha T(z) \, dz
\]  

(2-11)

2.6.3. Curvature Stress due to External Restraint

The curvature stresses are induced when the slab is restrained externally by self-weight and subgrade reaction. Westergaard-Bradbury equations are commonly used to calculate stresses in the slab, due to the linear temperature gradient [Huang, 2004]. The equations are based on elasticity and assumed reaction stresses, which are proportional to the deflection of the slab. For a finite slab with length \((L_x)\) in the \(x\)-direction and with width \((L_y)\) in the \(y\)-direction, the stresses in the \(x\) and \(y\) directions can be expressed as
\[ \sigma_x = \frac{\alpha \Delta T(z) E_c}{2(1 - v^2)} (C_x + v C_y) \]  \hspace{1cm} (2-12)

\[ \sigma_y = \frac{\alpha \Delta T(z) E_c}{2(1 - v^2)} (C_y + v C_x) \]  \hspace{1cm} (2-13)

where
\( \alpha \) = the coefficient of thermal expansion,
\( E_c \) = the modulus of elasticity,
\( v \) = Poisson’s ratio, and
\( C_x \) and \( C_y \) = correction factors in x and y directions.

The correction factors can be obtained from Figure 2-19 developed by Bradbury [1938].

\( C_x \) depends on \( L_x/l \) and \( C_y \) depends on \( L_y/l \). The radius of relative stiffness (\( l \)) is a measurement of the stiffness of the slab relative to the stiffness of the subgrade (\( k \)), and is defined as Equation (2-14).

\[ l = \frac{Eh^3}{4 \sqrt{12(1 - v^2)k}} \]  \hspace{1cm} (2-14)

An equivalent linear temperature gradient between the top and the bottom of the slab that produces the same curvature as Westergaard solution is obtained by Equation 2-15. The equivalent linear temperature gradient that is a basic parameter to calculate curvature of the slab is directly obtained by the determining constant \( M^* \).

\[ \Delta T_{\text{equivalent}} = -\frac{12M^*}{ah^2} \]  \hspace{1cm} (2-15)
2.7. Summary

Literature related to the subject of this study was reviewed. The study introduced the thermal effect in PCC pavement by explaining the mechanism of both top-down and bottom-up cracking. The study reviewed the production procedures and characteristics of fly ash and slag and examined their synergistic effects in ternary-blended cement concrete. The historical background and heat transfer models of EICM were presented, as well as a listing of climatic input data and the subsequent output of EICM in the JPCP. After stating the main thermal properties (CTE, TH, and HC) of concrete pavements, the study presented typical ranges of thermal properties with various aggregate types and described characteristics of nonlinear temperature gradients providing daily and seasonal temperature variations in concrete pavement in several states. Finally, the study reviewed the fundamental analytical solution of thermal stress in JPCP, using mechanical and thermal properties.
CHAPTER 3 MIXTURE DESIGNS AND EXPERIMENTAL TESTS

3.1 Conventional Concrete Mixtures

To study the effects of various parameters on CTE, five different conventional concrete mixtures were fabricated as shown in Table 3-1. 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 with 64% of coarse aggregate; the last two mixtures had different coarse aggregate proportions: 20% and 80% 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. A siliceous sand (TXI, Dennis Mills) was used as a fine aggregate for all of the mixtures. 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, except for KL (20%), due to the poor workability. The extremely large amount of fine aggregates caused poor workability due to an increase in the surface area of the aggregates. Although the w/c ratio was increased to 0.547, the workability presented the minimum to mix the ingredients. 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 3-1.

3.2 Ternary Concrete Mixtures

Twenty-five concrete mixtures, including one control mixture and twenty-four ternary mixtures with various combinations of fly ashes (Classes C and F), slags (Grades 100 and 120 ground granulated blast furnace slag (GGBFS)), and Portland cement (Type I), were fabricated. The mixtures were named by the proportions of different cementitious materials. TI stands for
Table 3-1 Conventional concrete mixture designs

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>Unit</th>
<th>KL (20%)</th>
<th>KL (64%)</th>
<th>KL (80%)</th>
<th>G (64%)</th>
<th>ML (64%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Holcim Type II (GP) Portland Cement</td>
<td>lbs/yd³</td>
<td>475</td>
<td>475</td>
<td>475</td>
<td>475</td>
<td>475</td>
</tr>
<tr>
<td>Sand, A133 TXI Dennis Mills</td>
<td>lbs/yd³</td>
<td>2551</td>
<td>1171</td>
<td>637</td>
<td>1131</td>
<td>1149</td>
</tr>
<tr>
<td>Kentucky Limestone, AB29 Martin Marietta</td>
<td>lbs/yd³</td>
<td>654</td>
<td>2104</td>
<td>2612</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Gravel, A133 TXI Dennis Mills</td>
<td>lbs/yd³</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>2027</td>
<td>–</td>
</tr>
<tr>
<td>Mexican Limestone, AA36</td>
<td>lbs/yd³</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>2071</td>
</tr>
<tr>
<td>% by volume Fine Aggregate</td>
<td>%</td>
<td>80</td>
<td>36.2</td>
<td>20</td>
<td>35.2</td>
<td>35.7</td>
</tr>
<tr>
<td>% by volume Coarse Aggregate</td>
<td>%</td>
<td>20</td>
<td>63.8</td>
<td>80</td>
<td>64.8</td>
<td>64.3</td>
</tr>
<tr>
<td>Water</td>
<td>lbs/yd³</td>
<td>260</td>
<td>214</td>
<td>214</td>
<td>214</td>
<td>214</td>
</tr>
<tr>
<td>Water Cement Ratio</td>
<td></td>
<td>0.547</td>
<td>0.451</td>
<td>0.451</td>
<td>0.451</td>
<td>0.451</td>
</tr>
<tr>
<td>Admixture (Daravair 1400)</td>
<td>Dosage, oz/100ct</td>
<td>0.5</td>
<td>0.50</td>
<td>0.5</td>
<td>0.50</td>
<td>0.50</td>
</tr>
<tr>
<td>Admixture (WRDA 35)</td>
<td>Dosage, oz/100ct</td>
<td>10</td>
<td>3.50</td>
<td>2.0</td>
<td>6.40</td>
<td>20.00</td>
</tr>
<tr>
<td>ASTM C 1064 Air Temperature</td>
<td>°F</td>
<td>75.3</td>
<td>68.5</td>
<td>70.8</td>
<td>69.0</td>
<td>71.2</td>
</tr>
<tr>
<td>ASTM C 1064 Concrete Temperature</td>
<td>°F</td>
<td>78.0</td>
<td>72.0</td>
<td>70.9</td>
<td>73.5</td>
<td>74.6</td>
</tr>
<tr>
<td>ASTM C 143 Slump</td>
<td>Inches</td>
<td>0</td>
<td>0.25</td>
<td>0.25</td>
<td>1.50</td>
<td>1.25</td>
</tr>
<tr>
<td>ASTM C 231 Pressure Air Content</td>
<td>%</td>
<td>3.5</td>
<td>7.0</td>
<td>3.6</td>
<td>6.30</td>
<td>4.0</td>
</tr>
<tr>
<td>ASTM C 138 Unit Weight</td>
<td>lbs/ft³</td>
<td>143.2</td>
<td>144.4</td>
<td>148</td>
<td>140.0</td>
<td>149.2</td>
</tr>
</tbody>
</table>

Type I Portland cement, S for slag, and FA for fly ash (C for class C fly ash and F for class F fly ash), respectively. The six ternary mixtures presented in Table 3-2 can be separated into two groups. The first three ternary mixtures have 30% slag-replacements with the proportion of
Portland cement, decreasing from 50 % to 30 %, and the next three ternary mixtures have 50 % slag-replacement with the accompanying proportion of Portland cement, decreasing from 30 % to 10 %. The six ternary mixtures shown in Table 3-2 were repeated four times with combinations of two fly ashes and two slags. The combinations of the mixtures are Class C fly ash with Grade 100 slag, Class C fly ash with Grade 120 slag, Class F fly ash with Grade 100 slag, and Class F fly ash with Grade 120 slag. The complete sets of mixture design, fresh concrete, and hardened concrete properties for ternary mixtures are presented in Appendix A.

The concrete mixtures conform to LADOTD standard specifications for roads and bridges [2006] and are comprised as follows: cement content is 500 lb/yd³; coarse aggregate is limestone; 60 /40 ratio of coarse to fine aggregate; coarse aggregate gradation is grade P; sand fraction is adjusted to keep constant volume; and air entrainment and water reducers are used to achieve air content and slump within LA DOTD specifications. A Kentucky limestone (Three Rivers Rock Quarry in Kentucky) and siliceous sand (A 133 TXI Dennis Mills) were used for coarse and fine aggregates for all the mixtures. A constant water-to-cementitious (w/cm) ratio of 0.45 was used for the mixtures, in order to minimize the effect of cement paste. Daravair 1440 and WRDA 35 were used as admixtures to provide desirable air content and workability.

Table 3-2 Ternary concrete mixture designs

<table>
<thead>
<tr>
<th>Mixtures</th>
<th>100TI</th>
<th>50TI30S20FA</th>
<th>40TI30S30FA</th>
<th>30TI30S40FA</th>
<th>30TI50S20FA</th>
<th>20TI50S30FA</th>
<th>10TI50S40FA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type I Portland cement (% by weight)</td>
<td>100</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>30</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>Slags (% by weight)</td>
<td>0</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Fly ashes (% by weight)</td>
<td>0</td>
<td>20</td>
<td>30</td>
<td>40</td>
<td>20</td>
<td>30</td>
<td>40</td>
</tr>
</tbody>
</table>

TI is Type I Portland cement; S is slags (grade 100 and 120 GGBFS); FA is fly ashes (class C and F fly ash)
3.3 Chemical Compositions of Ingredients

Figure 3-1 and Table 3-3 show the chemical compositions of various coarse aggregate types analyzed using scanning electron microscope-energy dispersive spectroscopy (SEM-EDS). The EDS is an analytical technique to characterize the chemical composition of a sample used in concert with the SEM [St. John et al., 1998]. Two representative types of gravels were investigated, since river gravel includes various kinds of gravels. Gravel 1 is mainly composed of calcium (Ca) and Oxygen (O), while Gravel 2 dominantly consists of silicon (Si). Kentucky and Mexican limestone have similar chemical compositions, which are mostly composed of calcium (Ca) and Oxygen (O), and Silicon (Si), as shown in Figure 3-1.

(a) Gravel 1 (light color)  
(b) Gravel 2 (dark color)  
(c) Kentucky limestone  
(d) Mexican limestone

Figure 3-1 A comparison of the chemical compositions of coarse aggregates
Table 3-3 The chemical compositions of coarse aggregates

<table>
<thead>
<tr>
<th>Elements</th>
<th>Gravel 1</th>
<th>Gravel 2</th>
<th>Kentucky limestone</th>
<th>Mexican limestone</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>16.41</td>
<td>10.44</td>
<td>15.17</td>
<td>18.05</td>
</tr>
<tr>
<td>O</td>
<td>40.61</td>
<td>37.69</td>
<td>38.26</td>
<td>35.60</td>
</tr>
<tr>
<td>Na</td>
<td>0.39</td>
<td>0.32</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Mg</td>
<td>4.26</td>
<td>-</td>
<td>3.34</td>
<td>2.91</td>
</tr>
<tr>
<td>Al</td>
<td>1.18</td>
<td>1.14</td>
<td>1.06</td>
<td>1.31</td>
</tr>
<tr>
<td>Si</td>
<td>3.70</td>
<td>49.82</td>
<td>3.69</td>
<td>4.94</td>
</tr>
<tr>
<td>Ca</td>
<td>33.44</td>
<td>0.58</td>
<td>38.49</td>
<td>37.19</td>
</tr>
</tbody>
</table>

In this research, two different types of fly ashes (Classes C and F) and slags (Grades 100 and 120) were used. The Class C fly ash was obtained from Headwaters Inc., Westlake, Louisiana; the Class F fly ash was obtained from Headwaters Inc., Tatum, Texas; the Grade 100 slag was obtained from Holcim Inc., Theodore, Alabama; and the Grade 120 slag was obtained from BuzziUnicem, New Orleans, Louisiana. Since SCMs exhibit a significant variation in chemical and physical properties among the sources, the X-ray fluorescence (XRF) analyses were performed. The results are presented in Table 3-4. ASTM 618 [2008] specifies the limitations of chemical requirements for fly ashes. The minimum percentage of summation of silicon dioxide (SiO₂), aluminum oxide (Al₂O₃), and iron oxide is 50 % for Class C fly ash and 70 % for Class F fly ash. The maximum percentage of sulfur trioxide (SO₃) and loss of ignition for both fly ashes are 5 % and 6 %, respectively. The amount of carbon in fly ash can vary over a wide range. Thus, ASTM 618 [2008] limits the use of fly ash in concrete to a 6 % loss on ignition (LOI). Generally, a high loss on ignition means a presence of pre-hydration and carbonation due to improper or prolonged storage [Kosmatka et al., 2005]. The high amount of carbon is not necessarily due to NOx control, since a higher amount of coal burning can result in
an incomplete burning. ASTM 989 [2010] specifies the limitations of chemical requirements for slag. Nevertheless, the maximum percentage of sulfate reported as SO₃ should be less than 4 %. All the chemical compositions of fly ashes and slags in Table 3-4 satisfy the aforementioned requirements.

Table 3-4 Chemical compositions of SCMs

<table>
<thead>
<tr>
<th>Oxide</th>
<th>Type I Portland Cement</th>
<th>Class C Fly Ash</th>
<th>Class F Fly Ash</th>
<th>Grade 100 GGBFS</th>
<th>Grade 120 GGBFS</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>20.24</td>
<td>35.04</td>
<td>60.74</td>
<td>38.59</td>
<td>34.77</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>4.45</td>
<td>19.30</td>
<td>19.41</td>
<td>7.61</td>
<td>10.73</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>3.47</td>
<td>5.32</td>
<td>7.93</td>
<td>0.76</td>
<td>0.56</td>
</tr>
<tr>
<td>CaO</td>
<td>63.28</td>
<td>24.98</td>
<td>5.33</td>
<td>38.61</td>
<td>40.52</td>
</tr>
<tr>
<td>MgO</td>
<td>3.82</td>
<td>5.48</td>
<td>1.84</td>
<td>13.00</td>
<td>11.99</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.22</td>
<td>1.95</td>
<td>0.77</td>
<td>0.25</td>
<td>0.29</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.44</td>
<td>0.46</td>
<td>1.19</td>
<td>0.38</td>
<td>0.38</td>
</tr>
<tr>
<td>TiO₂</td>
<td>0.28</td>
<td>1.36</td>
<td>1.01</td>
<td>0.36</td>
<td>0.60</td>
</tr>
<tr>
<td>SO₃</td>
<td>2.62</td>
<td>2.81</td>
<td>0.37</td>
<td>0.38</td>
<td>0.41</td>
</tr>
<tr>
<td>LOI</td>
<td>1.10</td>
<td>0.60</td>
<td>0.60</td>
<td>0.20</td>
<td>0.20</td>
</tr>
</tbody>
</table>

3.4 Measuring Apparatus

3.4.1. HM-251

An HM-251 system manufactured by Gilson/Challenge technology was used to measure the CTE of concrete mixtures, as shown in Figure 3-2. The HM-251 strictly follows AASHTO T 336 [2009] and is divided into three parts, a) a measuring frame, b) a system cabinet with water bath, and c) a heating/cooling circulator. The measuring frame is designed for a typical
cylindrical specimen, and the height can be adjusted, depending on specimen heights. A precise linear variable displacement transducer (LVDT) with a resolution of \(3.1 \times 10^{-8}\) mm and a total travel distance of 1.27 mm is installed on the top of the frame and automatically measures the length change of a concrete specimen [Gilson, 2006].

The material of the measuring frame is A304 stainless steel, which is used to eliminate any corrosion of the frame. A calibration bar 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 is measured and directly used for the calculation of concrete CTE. The water bath mounted in the system cabinet is of an appropriate size for placement of the measuring frame. A temperature probe is installed inside the water bath to measure water temperature continuously. The water level in the bath is maintained constantly by

Figure 3-2 HM-251
a water level control reservoir, in order 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 10 °C and 50 °C. When the temperature changes from 10 °C to 50 °C, the expansion of CTE is measured, while the contraction of CTE is measured after it changes from 50 °C to 10 °C. The test is terminated when the difference between the expansion of CTE and the contraction of CTE is within 0.3 μe/°C, and the average value of the two CTEs becomes a “representative CTE” [Gilson, 2006]. Otherwise, the software adjusts the temperature for another cycle until the difference of two CTEs becomes tolerable.

Figure 3-3 shows the change of both temperature and LVDT reading with respect to time; the trend of LVDT reading is quietly coincident with that of temperature change. The detailed procedures to measure CTE using the HM-251 system are presented in Appendix B.

The relationship between temperature change and strain is presented in Figure 3-4. By definition of the CTE, the slope of the graph represents the CTE value of the concrete. The CTE of concrete is calculated by the following equation:

\[
CTE = \left( \frac{\Delta L_a}{L_o} \right) / \Delta T
\]

(3-1)

where \( \Delta L_a \) is the actual length change of the specimen during temperature change, \( L_o \) is the initial length of specimen at room temperature, and \( \Delta T \) is the measured temperature change (increase = positive, decrease = negative).
Figure 3-3 Time vs temperature and displacement plot

Figure 3-4 Temperature vs strain plot
3.4.2. Quickline-30

Quickline-30, manufactured by Anter Corporation, is multi-functional equipment used for measuring surface temperatures, thermal conductivity, heat capacity, and thermal diffusivity [Anter, 2004]. In Quickline-30, ASTM D 5930-01, as the standard test method for thermal conductivity of plastics by means of a transient line-source technique, is used for a surface probe method; ASTM D 5334-05, as the standard test method for determination of thermal conductivity of soil and soft rock by thermal needle probe procedure, is used for the needle probe method [Kodide, 2010]. A line-source technique, the surface probe method, was used to measure thermal conductivity and heat capacity in this study. The method takes a few minutes to reach a steady-state condition. The factors influencing the measurement of the readings are a) the quality of a thermal contact between the probe and specimen; b) the temperature differences between the surface specimen and room temperature; c) the dimensions of the sample; and d) the moisture content [Kodide, 2010]. The measurement range of thermal conductivity is 0.08-2W/m-K, and the precision is ±10 % of reading value. The measurement temperature is -40 °F to 752 °F, which typically takes 16 to 20 minutes [Anter, 2004]. Figure 3-5 shows the schematic figure of Quickline-30.

![Quickline-30 experimental setup](image)

Figure 3-5 Experimental set up of Quickline-30 [Kodide, 2010]
3.4.3. Rapid RH

AASHTO TP 336-09 [2009] 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 %. As mentioned earlier, the saturated condition was chosen from a practical testing point of view. In reality, PCC pavements present neither a dry nor a saturated condition. Janssen [1987] found that the moisture condition at the top 2-in. of PCC pavement changes significantly. A nonlinear gradient of moisture may cause a non-uniform CTE in PCC pavements, and a synergy effect with a nonlinear temperature gradient can result in significant curling and joint problems. Therefore, it is necessary to measure the CTE as corresponding to a changing relative humidity inside the specimen in order to better understand pavement performance under changing temperatures and moisture conditions. RH was measured using the Rapid RH (ASTM F2170) device manufactured by Wagner Electronics, as shown in Figure 3-6. The device 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. A smart sensor probe then was installed in the hole. The RH reader was inserted inside of the smart sensor, and both the temperature and relative humidity of the concrete specimen were measured immediately.

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 60 °C for 24 hours. Using the mounted RH sensor probe and reader, the internal RH was measured. Then the specimen was placed in a 100 % moisture room until it reached the target RH. The study tested the specimen for CTE as soon as the target RH was reached. Due to the test
requiring the sample to be submerged underwater, the RH increased during the test. The change of RH under water was measured as illustrated in Figure 3-7. This particular specimen illustrates that it started at 31 % RH and passed 41 % RH after 8 hours, abruptly increasing to 96 % after 15 hours. Since most CTE tests were completed in 8 hours, the average change in relative humidity for 8 hours was used for further analysis.

Figure 3-6 Relative humidity measuring device (Rapid RH)

Figure 3-7 Change of relative humidity in water
3.4.4. Rapid Chloride Permeability Test

The relative humidity is directly related to the permeability of concrete mixture; thus, a rapid chloride permeability test (ASTM C1202) was performed to estimate the permeability of the concrete mixture, as shown in Figure 3-8. The specimen was cut by 4-in. (100 mm) in diameter and 2-in. (50 mm) in height, and the side of the cylindrical specimen was coated with epoxy. The specimen then was placed in the vacuum chamber to soak in water for eighteen hours. One side (-) of the cell was filled with a 3 % NaCl solution, while the other side (+) of the cell was filled with a 0.3 normal NaOH solution. Then a 60-volt potential was applied for six hours. After six hours, the specimen was removed and the amount of coulombs that passed through the specimen was measured. The chloride permeability was classified in five categories, depending on the amount of passed coulomb, as shown in Table 3-5. However, the test is not accurate enough to define the concrete permeability level precisely, and thus should be used only for comparison purposes.
Table 3-5 Chloride permeability based on charge passed

<table>
<thead>
<tr>
<th>Charge passed (coulombs)</th>
<th>Chloride permeability</th>
<th>Typical of</th>
</tr>
</thead>
<tbody>
<tr>
<td>4000 &gt;</td>
<td>High</td>
<td>High W/C ratio (&gt; 0.6), Conventional PCC</td>
</tr>
<tr>
<td>2000 – 4000</td>
<td>Moderate</td>
<td>Moderate W/C ratio (0.40-0.50), Conventional PCC</td>
</tr>
<tr>
<td>1000 – 2000</td>
<td>Low</td>
<td>Low W/C ratio (&lt; 0.4), Conventional PCC</td>
</tr>
<tr>
<td>100 – 1000</td>
<td>Very low</td>
<td>Latex-modified concrete or Internally-sealed concrete</td>
</tr>
<tr>
<td>&lt; 100</td>
<td>Negligible</td>
<td>Polymer-impregnated concrete, Polymer concrete</td>
</tr>
</tbody>
</table>

3.4.5. SEM and XPS

The scanning electron microscope (SEM) is extensively used for the microstructure analysis of cement paste and concrete. Since concrete is non-conducting material, a thin conductive coating should be given to prevent charge build-up on the surface of the sample [St. John et al., 1998]. In this study, the specimen was coated with platinum to get better image resolution. X-ray photoelectron spectroscope (XPS) is a high performance surface analysis system. XPS provides a surface composition and chemical state of elements at the surface of the sample. Figure 3-9 shows SEM and XPS used for microstructure analysis in this study.

![FEI Quanta 3D FEG Dual Beam SEM/FIB](image1)

![Kratos AXIS 165 XPS/SAM](image2)

Figure 3-9 SEM and XPS
3.5 Results of Experimental Tests

3.5.1. Conventional Concrete Mixtures

The mechanical properties of the conventional concrete mixtures (compressive strength, modulus of elasticity, flexural strength, and splitting tensile strength) were measured at several ages (7, 14, 28, and 90 days) in order to study aging effects. To eliminate experimental variability, all specimens were produced from the same batch. The specimens were cured in a 100 % moisture chamber until the time of testing. After curing, the compressive strength and the static modulus of elasticity were tested in accordance to ASTM C 39 and ASTM C 469 using three 4-in. by 8-in. cylindrical specimens at each age. The flexural strength was measured by the two beams (6-in. by 6-in. by 20-in.) at each age using the method of ASTM C 78. The two cylindrical specimens, 6-in. in diameter and 12-in. in height, were used for splitting tensile strength test at each age following ASTM C 496. Table 3-6 shows the test results of the mechanical properties for conventional concrete mixtures. The average and standard deviation are based on three samples for compressive strength, modulus of elasticity, and Poisson’s ratio, as well as two samples for flexural strength and splitting tensile test.

Development of compressive strength and modulus of elasticity at various ages (7, 14, 28, and 90 days) are presented in Figure 3-10 and Figure 3-11. Kentucky limestone concrete has the greatest compressive strength followed by Mexican limestone and gravel concrete with all ages as shown in Figure 3-10. The differences of hardness and surface textures of coarse aggregates can explain the variance of compressive strengths [Buch and Frabizzio, 2000]. Gravel aggregate constitutes a round shape with a smooth surface texture. The bond between cement paste and aggregate becomes relatively weak due to the characteristics of the gravel, thus cracks possibly may propagate around the aggregate, rather than through the aggregate. However, the crushed
Table 3-6 Mechanical properties of conventional concrete mixtures

<table>
<thead>
<tr>
<th>Coarse Aggregate</th>
<th>Mechanical property tests</th>
<th>7 days</th>
<th>14 days</th>
<th>28 days</th>
<th>90 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kentucky Limestone</td>
<td>Compressive strength (psi)</td>
<td>6,015</td>
<td>114.4</td>
<td>6,775</td>
<td>49.0</td>
</tr>
<tr>
<td></td>
<td>Modulus of Elasticity ((10^6\text{psi}))</td>
<td>5.883</td>
<td>0.378</td>
<td>5.866</td>
<td>0.621</td>
</tr>
<tr>
<td></td>
<td>Poisson's ratio</td>
<td>0.23</td>
<td>0.03</td>
<td>0.27</td>
<td>0.03</td>
</tr>
<tr>
<td></td>
<td>Flexural strength (psi)</td>
<td>678</td>
<td>112.4</td>
<td>925</td>
<td>48.1</td>
</tr>
<tr>
<td></td>
<td>Splitting Tensile (psi)</td>
<td>497</td>
<td>–</td>
<td>528</td>
<td>–</td>
</tr>
<tr>
<td>Gravel</td>
<td>Compressive strength (psi)</td>
<td>3,782</td>
<td>72.8</td>
<td>4,363</td>
<td>101.8</td>
</tr>
<tr>
<td></td>
<td>Modulus of Elasticity ((10^6\text{psi}))</td>
<td>5.033</td>
<td>0.407</td>
<td>4.766</td>
<td>0.076</td>
</tr>
<tr>
<td></td>
<td>Poisson's ratio</td>
<td>0.23</td>
<td>0.01</td>
<td>0.15</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>Flexural strength (psi)</td>
<td>519</td>
<td>9.9</td>
<td>551</td>
<td>0.0</td>
</tr>
<tr>
<td></td>
<td>Splitting Tensile (psi)</td>
<td>396</td>
<td>53.0</td>
<td>424</td>
<td>48.8</td>
</tr>
<tr>
<td>Mexican Limestone</td>
<td>Compressive strength (psi)</td>
<td>4,671</td>
<td>570.0</td>
<td>5,272</td>
<td>331.1</td>
</tr>
<tr>
<td></td>
<td>Modulus of Elasticity ((10^6\text{psi}))</td>
<td>4.150</td>
<td>0.086</td>
<td>4.600</td>
<td>0.050</td>
</tr>
<tr>
<td></td>
<td>Poisson's ratio</td>
<td>0.19</td>
<td>0.01</td>
<td>0.23</td>
<td>0.03</td>
</tr>
<tr>
<td></td>
<td>Flexural strength (psi)</td>
<td>559</td>
<td>29.0</td>
<td>652</td>
<td>26.2</td>
</tr>
<tr>
<td></td>
<td>Splitting Tensile (psi)</td>
<td>394</td>
<td>54.5</td>
<td>423</td>
<td>14.1</td>
</tr>
</tbody>
</table>

(Note: Standard deviations for splitting tensile strength of Kentucky limestone concrete were not presented since only one specimen was tested at each age.)

limestone has an angular shape with a rough surface texture; therefore, a strong bond between cement paste and aggregate can develop. The cracks are more likely to propagate through the crushed limestone, because the bond between cement paste and aggregate is relatively stronger than the strength of aggregate. The compressive strength of Mexican limestone concrete is
smaller than that of Kentucky limestone concrete at all ages, because Mexican limestone has an
higher water absorption, and thus includes a large amount of dust that deteriorates the
workability of concrete mixtures. Figure 3-11 shows that the elastic modulus of Kentucky
limestone concrete presents the greatest value, followed by gravel and Mexican limestone
concrete with all ages. Thermal conductivity and heat capacity of conventional mixtures are presented in Appendix C.

3.5.2. Ternary Concrete Mixtures

The mechanical properties of ternary mixtures (compressive strength, modulus of elasticity, and flexural strength) and a rapid chloride permeability test were measured at several ages to characterize the strength development with aging. Aforementioned sample preparations and ASTM standards in the conventional mixtures were also used to perform experimental tests for ternary mixtures. The development of a compressive strength for ternary concrete mixtures at various ages (7, 14, 28, 56, and 90 days) is presented in both Figure 3-12 and Figure 3-13. Figure 3-12 illustrates the strength development of a combination of Grade 100 slag with Class C and F fly ash, while Figure 3-13 illustrates the strength development combination of Grade 120 slag with Class C and F fly ash. The compressive strengths of ternary concrete mixtures at early ages are mostly smaller than that of conventional concrete mixtures. The development of

![Figure 3-12 Compressive strength development of ternary mixtures containing Grade 100 slag with Class C and F fly ash](image_url)
Figure 3-13 Compressive strength development of ternary mixtures containing Grade 120 slag with Class C and F fly ash

Compressive strength at mature ages varies depending on the types and proportions of fly ash and slag. The error bar represents the coefficient of variation of the compressive strength.

Four samples of cement pastes (100TI, 50TI-30G120S-20C, 10TI-50G120S-20F, and 50TI-30G120S-20C) were chosen to investigate the strength variations based on the microscopic examination of the surface texture and chemical composition analysis. The results obtained from SEM analysis for both plain cement paste (100TI) and cement paste containing fly ash and slag (50TI-30G120S-20C) are presented in Figure 3-14. By examining the SEM images, it was noticed that a great quantity of amorphous substance is formed when fly ash and slag were incorporated in the cement paste as shown in Figure 3-14b. X-ray photoelectron spectroscope (XPS) analysis revealed that the primary elements of the amorphous substance are Ca and Si. Calcium silicate hydrates (C-S-H) are most likely the reaction product of SiO$_2$, CaO, and Ca(OH)$_2$ from fly ash and slag and hydration product of C$_3$S and C$_2$S [Tan and Pu, 1998]. C-S-H
is the main binding phases in cement paste system and beneficial for the strength of concrete [Richardson, 1999]. Additional SEM and XPS data are provided in Appendix D.

Figure 3-14 SEM images of plain cement paste and cement paste containing fly ash and slag

(a) 100TI  (b) 50TI-30G120S-20C

Figure 3-14 and Figure 3-16 show the normalized compressive strength for ternary combinations of Grade 100 and 120 slag with Class C fly ash at various ages. The normalized compressive strength provides a convenient qualitative comparison for variations of compressive strength with ages. In each concrete mixture, the compressive strength at each age was divided by the compressive strength of the control mixtures (100TI) at 28 days. In Figure 3-15, all the ternary mixtures have a smaller compressive strength than that of the control mixture at 7 and 14 days. Yet, some ternary mixtures have a greater compressive strength at 56 and 90 days. In Figure 3-16, the compressive strength of some ternary mixtures exceeds the compressive strength of the control mixture (100TI) at 7 and 14 days. Compressive strength of all the ternary mixtures surpasses the compressive strength of the control mixture after 28 days,
Figure 3-15 A comparison of normalized compressive strength of ternary mixtures containing Grade 100 slag with Class C fly ash

Figure 3-16 A comparison of normalized compressive strength of ternary mixtures containing Grade 120 slag with Class C fly ash

except for an extreme case of combination (100TI-50G120S-40C). Thus, use of Grade 120 slag with Class C fly ash is more favorable than use of Grade 100 slag with Class C fly ash, with respect to the strength development in both early and mature ages. The normalized compressive
strengths of other ternary combinations, such as the combinations of Grade 100 and 120 slag with Class F fly ash are presented in Appendix A.

A modulus of elasticity values used in concrete design can be estimated by the empirical relationship with compressive strength and density of concrete [Mehta and Monteiro, 2005]. The ACI building code 318 [2005] and the CEB-FIP [1993] model code provides an empirical relationship between the modulus of elasticity and compressive strength, as shown in Figure 3-17. In the ACI building code 318, the equation for normal weight concrete of 145 lb/ft³ is

\[ E_c = 57,000 \sqrt{f_c'} \]  

(3-2)

Where, \( E_c \) is a modulus of elasticity (psi) and \( f_c' \) is a compressive strength (psi); it should be noted that the ACI equation is effective only up to a compressive strength of 6000 psi. The equation in the CEB-FIP model [1993] code for normal weight concrete is

\[ E_c = 2.15 \times 10^4 \left( \frac{f_{cm}}{10} \right)^{1/3} \]  

(3-3)

Where, \( E_c \) is a modulus of elasticity (MPa), and \( f_{cm} \) is a compressive strength of concrete at an age of 28 days (MPa). The moduli of elasticity of ternary concrete mixtures corresponding to the compressive strengths at each age are plotted in Figure 3-17. The ACI equation generally provides a conservative lower boundary for all the ternary mixtures at various ages. The CEB-FIP model displays a good agreement with measured values of ternary mixtures at all ages.

The tensile strength of concrete is usually predicted by the flexural strength test in pavement design. The flexural strength test, called the modulus of the rupture test, tends to over-predict the tensile strength by about 50 %, because the stress-strain relationship throughout the cross-section of a beam is simplified to a linear [Mindess et al., 2003]. In addition, the whole volume of cross-section in the direct tension test resists tensile stress. Only the partial volume,
especially the bottom part of a beam of cross-section resists tensile stress in the flexural strength test [Mehta and Monteiro, 2005]. However, flexural strength is still beneficial, because most concrete beams tend to failed due to bending rather than axial tension. Figure 3-18 shows the relationship between flexural strength and compressive strength. ACI building codes 318 [2005] and 363 [2005] provide a relationship of normal weight concrete for both regular and high-strength concrete as follow:

\[ f_r = 7.5\sqrt{f'_c} \] (ACI 318) \hspace{1cm} (3-4)

\[ f_r = 11.7\sqrt{f'_c} \] (ACI 363) \hspace{1cm} (3-5)

Where, \( f_r \) is a flexural strength (psi) and \( f'_c \) is a compressive strength (psi). The flexural strength values of ternary concrete are close to the equation of ACI 363. Similar to the moduli of elasticity, ACI 318 equation provides a conservative, lower boundary to flexural strength with all ranges of compressive strength, except for extremely low compressive strengths.
A low permeability of concrete mixture remains a critical factor for the durability of concrete including resistance to sulfate attack and alkali silica reaction. Figure 3-19 shows that ternary mixtures have a dramatic reduction of permeability compared to control mixtures regardless of any combination of slag and fly ash. All the ternary mixtures fall into a very low chloride permeability category.
3.6 Summary

Five conventional concrete mixtures with different coarse aggregate types and proportions, as well as twenty-four ternary mixtures with various combinations of fly ashes, slags, and Portland cement were cast. Mechanical properties such as compressive strength, modules of elasticity, and flexural strength of both conventional and ternary concrete mixtures were performed at several ages in order to characterize the strength development of each mixture.

In conventional concrete mixtures, Kentucky and Mexican limestone concrete shows a greater compressive strength than gravel concrete with all ages, due to the differences in hardness and surface textures of coarse aggregates. Kentucky limestone concrete has the greatest compressive strength, followed by Mexican limestone and gravel concrete to all ages. For the modulus of elasticity, Kentucky limestone concrete has the greatest value, followed by gravel and Mexican limestone concrete to all ages.

In ternary concrete mixtures, the compressive strengths of ternary mixtures at early ages (7 and 14 days) are usually smaller than that of conventional concrete mixtures; thus the strength development at mature ages varies, based on the types and proportions of fly ash and slag. Ternary mixtures, when combined with Grade 120 slag and Class C fly ash, generally show a higher compressive strength than that with Grade 100 slag and Class C fly ash in both early and mature ages. The compressive strength of ternary mixtures combined with Grade 120 slag and Class C fly ash surpasses the compressive strength of the control mixture after 28 days, except for in extreme cases of combination (100TI-50G120S-40C). In a relationship between compressive strength and a modulus of elasticity, the ACI equation generally provides a conservative lower boundary of all the ternary mixtures at various ages, while the CEB-FIP model has a good agreement with the measured value of ternary mixtures at all ages. In regard to
the relationship between compressive strength and flexural strength, the measured values of ternary mixtures are close to the ACI 363 equation, whereas the ACI 318 equation provides a conservative low boundary limit with all ranges of compressive strength, except for extremely low compressive strengths. All the ternary mixtures present a dramatic reduction of permeability compared to control mixtures, and thereby fall into a very low category of the chloride permeability test.
CHAPTER 4 CHARACTERIZATION OF CTE FOR CONVENTIONAL AND TERNARY-BLENDED MIXTURES

4.1 CTE of Conventional Concrete Mixtures

4.1.1 Specimen Preparation

For practical purposes, this study chose three popular coarse aggregates (KL (64 %), M (64 %), and ML (64 %)) used in Louisiana. The specimens were cured in a 100 % moisture chamber until the time of testing. CTEs were measured at 3, 5, 7, 14, 28, 60, 90, and 360 days for each concrete mixture to compare the variation of CTE depending on aggregate types and aging. Two additional mixtures were fabricated (KL (20 %) and KL (80 %)). In the Kentucky limestone mixture, the volume of coarse aggregate was changed to 20 % and 80 %, while keeping the total volume of aggregates constant. That means the volume of fine aggregate was changed to 80 % and 20 %, respectively. Those 20, 64, and 80 % of relative coarse aggregate volume in total aggregate can be converted into 14.5 %, 46.1 %, and 57.8 % of coarse aggregate volume in a concrete mixture. The 20 % and 80 % volume of coarse aggregate as extreme cases were chosen to find a relationship between the amount of coarse aggregate and CTE.

4.1.2 Statistical Analysis Method (ANOVA)

An analysis of variance (ANOVA) was utilized to validate the impact of variables on the CTE. This study employed a factorial experiment to investigate both aggregate types and mixture ages, thus aggregate types were tested over all mixture ages. Levels of each factor are three levels (KL (64 %), M (64 %), and ML (64 %)) for aggregate types and eight levels (3, 5, 7, 14, 28, 60, 90, and 360 days) for mixture ages. Duplicate samples were tested for each combination.

of aggregate types and mixture ages and both factors were considered to be fixed effects. One-way ANOVA was employed to investigate coarse aggregate proportion and relative humidity, respectively. Coarse aggregate proportion was tested for KL (20 %), KL (64 %), and KL (80 %), and relative humidity was tested for ML (64 %). A Coarse aggregate proportion was tested with three levels (14.5, 46.1, and 57.8 % of the volume in a concrete mixture), and relative humidity was tested with seven levels (between 30 and 100 % RH). Three replicate samples for coarse aggregate proportion and duplicate samples for relative humidity were tested, respectively. The required assumptions including 1) residuals (deviations) are normally distributed; 2) observations are independent; and 3) variances are homogeneous, were checked before the analyses.

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 analysis called multiple comparisons procedure were used. Tukey’s procedure was conducted as a multiple comparison since it allows for all possible pairwise tests [Freund and Wilson, 2003; Devore, 2004]. The Pr-value indicates the probability of error of the statement. If a Pr-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 analyses. It should be emphasized that a statistical significance does not necessarily imply a practical significance. The overall ANOVA results are summarized in Table 4-1.

### 4.1.3 Influential Factors on CTE

The average CTEs for KL (64 %), G (64 %), and ML (64 %) concrete were 9.19 με/°C (5.11 με/°F), 12.86 με/°C (7.14 με/°F), and 8.82 με/°C (4.90 με/°F), respectively. Figure 4-1
shows the CTE with various aggregate types at different ages. Although Kentucky and Mexican limestone came from separate sources, the average CTE values were very close. The CTE of the

Table 4-1 Summary of ANOVA results for conventional mixtures

<table>
<thead>
<tr>
<th>Variables</th>
<th>F-value</th>
<th>Pr &gt; F</th>
<th>Significance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggregate types (KL, G, ML)</td>
<td>3601.97</td>
<td>&lt;.0001</td>
<td>Yes</td>
</tr>
<tr>
<td>Mixture age</td>
<td>0.95</td>
<td>0.4876</td>
<td>No</td>
</tr>
<tr>
<td>Aggregate types x Mixture age</td>
<td>0.74</td>
<td>0.7156</td>
<td>No</td>
</tr>
<tr>
<td>Coarse aggregate proportion (14.5, 46.1, 57.8 % of KL)</td>
<td>417.16</td>
<td>&lt;.0001</td>
<td>Yes</td>
</tr>
<tr>
<td>Relative humidity (Representative CTE)</td>
<td>0.81</td>
<td>0.5834</td>
<td>No</td>
</tr>
<tr>
<td>Relative humidity (expansion CTE)</td>
<td>6.80</td>
<td>0.0117</td>
<td>Yes</td>
</tr>
</tbody>
</table>

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

gravel showed 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 4-1). Increased CTE leads to an increased probability of pavement distresses during the design life, if other conditions remain the same. The observation indicated that Kentucky and Mexican limestone are more desirable from a design point of view in order to minimize any thermal deformations and damages. To investigate the effects of mixture age, CTE was measured at 3, 5, 7, 14, 28, 60, 90, and 360 days for each aggregate. Figure 4-1 shows the CTE at several different ages of the mixtures. The graph shows that CTE fluctuated within 0.3 με/°C (0.2 με/°F), with no increasing or decreasing tendencies up to 360 days. It is also verified by the statistical analysis (ANOVA) that there was no significance difference in CTEs due to age.

Quartz has a much higher CTE than limestone (Table 2-3). The CTE of limestone is between 3.6 and 6.5 με/°C (2.0 and 3.6 με/°F), and quartz sand is between 9.9 and 12.8 με/°C (5.5 and 7.1 με/°F). Zoldners [1971] showed the effect of aggregate content on the CTE of concrete, as shown in Figure 4-2. The CTE varies depending on the types and proportion of
aggregates, because the volume of aggregates occupies more than 70 % of the concrete volume. Thus, the CTEs of aggregates, especially coarse aggregates, predominantly control the CTE of concrete. To study the effect of aggregate contents, the study chose Kentucky limestone and siliceous sand for coarse and fine aggregates and two additional mixtures were produced. The results of measured CTE are shown in Figure 4-3.

From Figure 4-2, the CTE starts at 11.3 \( \mu \varepsilon / ^\circ C \), which corresponds to 0 % of coarse aggregate (crushed limestone). The CTE decreases steeply until it reaches 5 \( \mu \varepsilon / ^\circ C \), when coarse aggregate (crushed limestone) is 100 %. Similarly, in Figure 4-3, CTE decreases from 11.9 \( \mu \varepsilon / ^\circ C \) to 8.4 \( \mu \varepsilon / ^\circ C \) at 14.5 % and 57.8 % of coarse aggregate (Kentucky limestone) in concrete, respectively. These data 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 follows:
Figure 4-2 Effect of aggregate content on the CTE of concrete [Zoldners, 1971]

Figure 4-3 CTE vs proportion of coarse aggregate (KL) in concrete mixture
\[ Y = -0.0803X + 13.03 \]  \hspace{1cm} (4-1)

where, \( X \) is the proportion of Kentucky limestone in concrete mixture (\%) and \( Y \) is CTE corresponding to the proportional Kentucky limestone (\( \mu e/\circ C \)). A statistical analysis (ANOVA) indicated that the effect of coarse aggregate proportion has a significant impact on CTE results (Table 4-1). Specifically, the three CTEs of each proportion (14.5 %, 46.1 %, and 57.8 %) were statistically different from one another.

As shown in Figure 4-4, the representative CTE increases gradually, as the relative humidity increases up to 86 % RH. Then the CTE decreases until it reaches 100 % RH. Although the graph shows a peak CTE value between 80 % and 90 % RH, the variation was statistically too small to have a significant difference. However, the effect of relative humidity had a significant difference on the expansion of CTE with regard to the results of statistical analysis (ANOVA results in Table 4-1). Specifically, CTE at 52 %, 63 %, and 72 % RH was significantly different from CTE at 100 % RH, as shown in Figure 4-5. According to Mitchell [1953] and Meyers [1951], moisture content may cause the CTE of neat cement paste to vary by as much as 100 %. The minimum value was observed in both oven-dry and saturated conditions, and the maximum value was observed at 65 % to 70 % RH for up to 6 months, and at 45 % to 50 % RH after several years [Mitchell, 1953].

A representative CTE means that the average value between expansion and contraction of the CTE is used for practical purposes. Although expansion and contraction represent independent events; yet each of these has the ability to generate a slab cracking on the PCC pavement. The expansion and contraction of CTE are separately presented in Figure 4-5. The expansion of CTE was obtained by the first expansion cycle, and the contraction of CTE was
obtained by the first contraction cycle. As expected, the expansion of CTE shows more variation than the contraction of CTE as the relative humidity changes; both CTE values are coincident at 100 % RH. It was found that CTE at 63 % RH is 8 % higher than CTE at 100 % RH. Kessler
[1948] and Hockman and Kessler [1950] found that the length change on a heating cycle has a higher CTE than that on a cooling cycle, particularly in granite and marble. The heating and cooling procedures tend to cause complex stresses and slippage among mineral crystals. The permanent deformation caused by the heating procedure is because the crystal fails to return its original volume due to temperature change. This permanent deformation creates different thermal coefficients in the actions of expansion and contraction. The expansion of CTE presents a more fluctuating trend with various relative humidities. This relatively high CTE at 63 % RH, compared to that at 100 % RH, can cause tensile cracking on the surface of concrete pavement, due to the fact that the top surface of concrete pavement normally has a lower relative humidity, compared to the bottom of concrete pavement.

4.1.4 CTE Prediction Equation

An aggregate type and coarse aggregate proportion were included in the equation to predict CTE. Concrete is comprised of two-phase materials, with aggregate particles embedded in a matrix of cement mortar. Hansen [1965] developed a model to predict the modulus of elasticity for a two-phase composite concrete consisting of spherical particles evenly distributed in a continuous matrix.

\[
\alpha = \left[ \frac{(1 - V_{agg})\alpha_{\text{matrix}} + (1 + V_{agg})\alpha_{agg}}{(1 + V_{agg})\alpha_{\text{matrix}} + (1 - V_{agg})\alpha_{agg}} \right] \alpha_{\text{matrix}}
\]  

(4-2)

In the equation (4-2), the elasticity modulus was replaced by the CTE (\(\alpha\)) and a volume of aggregate was obtained by means of the weighted average volume of both coarse and fine aggregates. The CTE of composite concrete was calculated for KL (20 %), KL (64 %), KL (80 %), G (64 %), and ML (64 %) using the CTE of each aggregate and cement paste. The CTE
values of aggregates and cement paste were selected by the average value of the range in Table 2-3. Figure 4-6 shows a comparison between the predicted CTE by the model and the measured CTE. The predicted CTE generally had a good agreement with the measured CTE, with the percentage difference between the two CTEs varying from 1.5 % to 8.1 %.

![Figure 4-6 Predicted CTE vs. measured CTE](image)

4.2 CTE of Ternary Concrete Mixtures

4.2.1 CTE of Ternary Mixtures

Figure 4-7 shows the CTE values of the control and twenty-four ternary mixtures. As shown in the figure, all the CTE values of ternary mixtures were larger than that of the control mixture, except for a ternary mixture (30TI-30G100S-40C). Since the replacements of SCMs were made by the mass basis, the volumes of replaced SCMs are different from that of Portland cement due to various specific gravities. Fly ash and slag have lower specific gravities than Portland cement: thus, volume of cementitious paste is higher, while aggregate volume is lower than control mixtures. As shown in Table 2-3, the paste tends to have the highest CTE value.
among other constituents in the mixtures. The largest CTE value of a ternary mixture (50TI-30G120S-40F) was 10.2×10^{-6}/°C (5.7×10^{-6}/°F), 10% larger than that of the control mixture. The CTE of ternary mixtures decreases gradually, as the proportions of fly ashes (Class C and F) increase from 20% to 40%. That result means that the proportions of Type I Portland cement decrease from 50% to 30% for a 30% slag-replacement, and decrease from 30% to 10% for a 50% slag-replacement. Overall, the combination of Class F fly ash with Grade 120 slag results in higher CTE values than the combination of Class C fly ash with Grade 100 slag.

![Figure 4-7 CTE values of a control and ternary mixtures](image)

**4.2.2 Statistical Analysis Method (ANOVA)**

An analysis of variance (ANOVA) was employed to validate the influence of SCMs on the CTE values. ANOVA is a statistical method used to compare the group means among each group (Freund and Wilson, 2003). Twenty-four ternary mixtures can be divided into four groups with respect to the combinations of two fly ashes and slags, distinguished by the different colors in Figure 4-7. Furthermore, those four groups can be divided into two separated subgroups by the
proportions of slag, such as 30 % and 50 % slag-replacement. Therefore, eight groups were generated, based on the types of fly ashes and slags, together with the replacement ratio of slag. The mean values of these eight groups, with each group consisting of three ternary mixtures, were compared to that of a control mixture. One-way ANOVA was utilized to investigate the significance and duplicated samples of each ternary mixture were tested. The required assumptions were checked in the same manner as conventional concrete mixture; then a multiple comparison (Tukey’s procedure) was conducted to determine particularly which means differed from the mean of control mixture. Alpha (α) is a probability error level, fixed by 0.05 in the analyses. The results of the ANOVA analyses are presented in Table 4-2. As mentioned earlier, twenty-four ternary mixtures were categorized into eight groups. Each group includes three ternary mixtures. The group name is determined by the types and proportions of slag, as well as the types of fly ash. For example, 30G100S-C means that a ternary mixtures group contains 30% of Grade 100 slag and Class C fly ash. Mean values of each group were shown in the first column of the ANOVA analyses. The letters in the Tukey grouping indicate that the same letter is not significantly different within the 95 % confidence level. For example, there are three letters in the Tukey grouping: A, B, and C. Since the letter of control group is C, any group that does not include letter C in the Tukey grouping has a significantly different mean value, compared to the mean value of the control mixture. Three groups of ternary mixtures (50G100SF, 30G120S-F, and 50G120S-F) show a statistical significance. The denominator of those three groups is a ternary mixture that contains Class F fly ash. The dominant chemical composition of Class F fly ash is silicon dioxide (SiO₂), which contributes to high CTE values (9.9-12.8×10⁻⁶/°C) in quartz sand and gravel. All of the four extreme combinations by replacing 50 % slag and 40 % fly ash did not satisfy the strength requirements at 28 days (4000 psi).
### Table 4-2 Results of ANOVA analysis for ternary mixtures

<table>
<thead>
<tr>
<th>Grouping of SCM combinations</th>
<th>Strength at 28 days (psi)</th>
<th>ANOVA analyses</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Compressive strength</td>
<td>Mean value</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(10^6/°C)</td>
</tr>
<tr>
<td>100TI 100TI</td>
<td>5,860 Pass</td>
<td>9.311</td>
</tr>
<tr>
<td>30G100S-C</td>
<td>50TI-30G100S-20C 4,830 Pass</td>
<td>9.403</td>
</tr>
<tr>
<td></td>
<td>40TI-30G100S-30C 5,460 Pass</td>
<td></td>
</tr>
<tr>
<td></td>
<td>30TI-30G100S-40C 4,280 Pass</td>
<td></td>
</tr>
<tr>
<td>50G100S-C</td>
<td>30TI-50G100S-20C 6,430 Pass</td>
<td>9.518</td>
</tr>
<tr>
<td></td>
<td>20TI-50G100S-30C 5,550 Pass</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10TI-50G100S-40C 2,710 N.G</td>
<td></td>
</tr>
<tr>
<td>30G100S-F</td>
<td>50TI-30G100S-20F 6,070 Pass</td>
<td>9.607</td>
</tr>
<tr>
<td></td>
<td>40TI-30G100S-30F 5,500 Pass</td>
<td></td>
</tr>
<tr>
<td></td>
<td>30TI-30G100S-40F 4,490 Pass</td>
<td></td>
</tr>
<tr>
<td>50G100S-F</td>
<td>30TI-50G100S-20F 5,740 Pass</td>
<td>9.904</td>
</tr>
<tr>
<td></td>
<td>20TI-50G100S-30F 4,790 Pass</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10TI-50G100S-40F 3,000 N.G</td>
<td></td>
</tr>
<tr>
<td>30G120S-C</td>
<td>50TI-30G120S-20C 8,580 Pass</td>
<td>9.768</td>
</tr>
<tr>
<td></td>
<td>40TI-30G120S-30C 7,300 Pass</td>
<td></td>
</tr>
<tr>
<td></td>
<td>30TI-30G120S-40C 7,930 Pass</td>
<td></td>
</tr>
<tr>
<td>50G120S-C</td>
<td>30TI-50G120S-20C 7,680 Pass</td>
<td>9.787</td>
</tr>
<tr>
<td></td>
<td>20TI-50G120S-30C 6,780 Pass</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10TI-50G120S-40C 2,650 N.G</td>
<td></td>
</tr>
<tr>
<td>30G120S-F</td>
<td>50TI-30G120S-20F 6,830 Pass</td>
<td>10.025</td>
</tr>
<tr>
<td></td>
<td>40TI-30G120S-30F 6,000 Pass</td>
<td></td>
</tr>
<tr>
<td></td>
<td>30TI-30G120S-40F 4,830 Pass</td>
<td></td>
</tr>
<tr>
<td>50G120S-F</td>
<td>30TI-50G120S-20F 6,310 Pass</td>
<td>9.950</td>
</tr>
<tr>
<td></td>
<td>20TI-50G120S-30F 4,820 Pass</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10TI-50G120S-40F 3,530 N.G</td>
<td></td>
</tr>
</tbody>
</table>
4.2.3 CTE Prediction Equation

Figure 4-8 shows a trend of normalized CTE values, where the CTEs of ternary mixtures were divided by the CTE of control mixture, with an increase of fly ash proportions in a 30% slag-replacement. Regardless of the combination of SCMs, the CTE values decrease gradually, as proportions of fly ashes increase. The largest normalized CTE values occur with a combination of Class F fly ash with Grade 120 slag, while the smallest normalized CTE values occur with a combination of Class C fly ash with Grade 100 slag. The prediction equations and R² values of CTE for 30% slag-replacement ternary mixtures are presented in Table 4-3.

![Figure 4-8 CTE prediction of 30% slag-replacement](image)

Figure 4-8 CTE prediction of 30% slag-replacement

Figure 4-9 shows a trend of normalized CTE values, with regard to increased fly ashes proportions in a 50% slag-replacement. The ternary combinations, including Class C fly ash, display a similar trend to a 30% slag-replacement in Figure 4-8. The normalized CTE values of the ternary combination, inclusive of Class F fly ash, decrease sharply as the proportions of fly ash increase. The prediction equations of CTE for eight groups of ternary mixtures are presented in Table 4-3. The X values represent the proportions of both fly ashes (20 to 40%), and the
range of $R^2$ values are between 0.841 and 0.998. These equations should be used with caution, when the proportions and sources of SCMs differ from Table 4-3.

![Figure 4-9 CTE prediction of 50% slag-replacement](image)

Table 4-3 CTE prediction equations of ternary mixtures

<table>
<thead>
<tr>
<th>Grouping of SCM combinations</th>
<th>Equations</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>30G100S-C</td>
<td>CTE = (-0.0016X + 1.0571) × 9.311</td>
<td>0.890</td>
</tr>
<tr>
<td>30G100S-F</td>
<td>CTE = (-0.0022X + 1.0992) × 9.311</td>
<td>0.879</td>
</tr>
<tr>
<td>30G120S-C</td>
<td>CTE = (-0.0027X + 1.1307) × 9.311</td>
<td>0.868</td>
</tr>
<tr>
<td>30G120S-F</td>
<td>CTE = (-0.0023X + 1.1453) × 9.311</td>
<td>0.998</td>
</tr>
<tr>
<td>50G100S-C</td>
<td>CTE = (-0.0009X + 1.0503) × 9.311</td>
<td>0.920</td>
</tr>
<tr>
<td>50G100S-F</td>
<td>CTE = (-0.003X + 1.1547) × 9.311</td>
<td>0.841</td>
</tr>
<tr>
<td>50G120S-C</td>
<td>CTE = (-0.0008X + 1.0753) × 9.311</td>
<td>0.957</td>
</tr>
<tr>
<td>50G120S-F</td>
<td>CTE = (-0.0042X + 1.1841) × 9.311</td>
<td>0.978</td>
</tr>
</tbody>
</table>
Figure 4-10 shows a comparison between the predicted CTE by the equation in Table 4-3, and measured CTE values. The predicted CTE shows a good agreement with the measured CTE, since the $R^2$ value is 0.918 and the maximum percentage difference between the two CTEs is 1.3%.

![Figure 4-10 Measured CTE vs. predicted CTE](image)

4.3 Prediction of Pavement Distresses using MEPDG

To predict the impact of the CTE on the performance of concrete pavement, an analysis was conducted for 20 years period according to the Louisiana state policy. MEPDG provides concrete pavement distresses such as mean joint faulting, transverse cracking, and terminal IRI. This study is targeted to the Level 1 design, so the thermal properties (CTE, TC, and HC) and concrete mechanical properties were tested at each mixture and designated ages. 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, shown in Table 4-4.
Table 4-4 Input parameters for MEPDG analysis

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Input data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design life (years)</td>
<td>20</td>
</tr>
<tr>
<td>Traffic (AADTT)</td>
<td>1397</td>
</tr>
<tr>
<td>PCC layers (in.)</td>
<td>JPCP (10)</td>
</tr>
<tr>
<td></td>
<td>Crushed stone (4)</td>
</tr>
<tr>
<td></td>
<td>Soil cement (6);</td>
</tr>
<tr>
<td></td>
<td>Cement treated (8)</td>
</tr>
<tr>
<td>Transverse joint spacing (ft.)</td>
<td>15/ 18/ 20</td>
</tr>
<tr>
<td>Dowel diameter and spacing (in)</td>
<td>1.25/ 12</td>
</tr>
<tr>
<td>Thermal properties of</td>
<td>CTE (4.96 με/°F)</td>
</tr>
<tr>
<td>Kentucky limestone concrete</td>
<td>thermal conductivity(1.45 BTU/h·ft·F)</td>
</tr>
<tr>
<td></td>
<td>and heat capacity(0.28 BTU/lb·F)</td>
</tr>
<tr>
<td>Thermal properties of</td>
<td>CTE (7.14 με/°F)</td>
</tr>
<tr>
<td>Gravel concrete</td>
<td>thermal conductivity(1.60 BTU/h·ft·F)</td>
</tr>
<tr>
<td></td>
<td>and heat capacity(0.27 BTU/lb·F)</td>
</tr>
<tr>
<td>Climatic data</td>
<td>Interpolated among New Orleans, Baton Rouge, and Lafayette</td>
</tr>
</tbody>
</table>

Mean joint faulting increases linearly as joint spacing increases for both Kentucky limestone and gravel in Figure 4-11. 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 4-12. 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. 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., but it increases dramatically from joint spacing 18- to 20-ft. Only 15-and 18-ft. 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 4-13. The Kentucky limestone for all joint spacing and gravel for 15-ft. joint spacing meet the specification. The specified limits of target distresses in MEPDG are 0.12-in., 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. joint spacing for Kentucky limestone satisfy the specification of all three PCC pavement distresses.
Measuring accurate CTE is the crucial issue to predict PCC pavement performance during the design life, but sometimes a typical value available from the literature or default value for the region is used without caution. The discrepancy between an accurate CTE and an assumed CTE will cause erroneous prediction of PCC pavement performance. To evaluate the sensitivity of the CTE on the pavement performance in JPCP, the measured CTE value of gravel (12.86 με/°C) was considered as an accurate CTE. The assumed CTE values were created by increasing or decreasing up to 25 % from the accurate value. The three different joint spacings (15-, 18-, and 20-ft.) were included in the analysis, because joint spacing is a critical factor in JPCP design. Figure 4-14 shows the mean joint faulting at various joint spacings, as well as CTE prediction error. Mean joint faulting is slightly more sensitive to a positive CTE prediction error; the relationship is nonlinear. Overestimated CTE (positive prediction error) has a steeper slope than an underestimated CTE (negative prediction error), meaning that overestimation tends to be more sensitive than underestimation in mean joint faulting with all joint spacings. A 10 %
positive bias leads to about 38 \% overestimation, while a 10 \% negative bias leads to about 30 \% underestimation in mean joint faulting in 15-ft. joint spacing.

![Figure 4-14 Mean joint faulting depending on various CTE and joint spacing](image)

Figure 4-14 Mean joint faulting depending on various CTE and joint spacing

Figure 4-15 illustrates the effect of various joint spacings, as well as a CTE prediction error on percent slab cracking. While a positive CTE prediction error is more sensitive to shorter joint spacing (15-ft.), a negative CTE prediction error is more sensitive to longer joint spacing (18- and 20-ft.). In the case of 15-ft. joint spacing, a 10 \% positive bias leads to about 188 \% overestimation, while a 10 \% negative bias leads to about 75 \% underestimation in percent slab cracking. The percent slab cracking is the most sensitive to not only the CTE value, but also joint spacing among other pavement distresses, because the percent slab cracking of longer joint spacings (18- and 20-ft.) reaches almost 100 \%, even in the 0 \% prediction error, due to the high CTE value of gravel (12.86 \( \mu \varepsilon /{°C} \)). Moreover, an overestimated CTE can cause an excessively conservative pavement design, while an underestimated CTE will induce unreasonable pavement design that is vulnerable to pavement distresses. Thus, an accurate prediction of the CTE value is
important for safe and economic pavement design. The result of terminal IRI shows a similar trend with a combination of the two previous results.

![Figure 4-15 Percent Slab cracking depending on various CTE and joint spacing](image)

**Figure 4-15** Percent Slab cracking depending on various CTE and joint spacing

### 4.4 Correction Factor for Overestimated CTE

A memorandum was issued by the Federal Highway Administration (FHWA) [Stephanos, 2009] to take action on the concrete coefficient of thermal expansion input for MEPDG. According to the memorandum, FHWA has identified a problem with the 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 the expansion of the measuring apparatus was based on the literature values ($17.3 \times 10^{-6}/^\circ C$) in AASHTO TP 60-00, rather than the actual CTE value, based on the temperature range specified (10 to 50 °C). The use of the incorrect CTE value for the reference specimen 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 [2009] test standard to replace the TP 60-00 [2005] provisional test method. The memorandum cautioned State Highway Agencies (SHAs) to use the CTE value as an input to the
AASHTO mechanistic empirical pavement design guide, interim edition. Many researchers identified the CTE value as a sensitive input for the MEPDG that could adversely affect the concrete pavement design. The interim edition of MEPDG was calibrated with the long-term pavement performance (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 predicted distresses, when compared to the measured values in the LTPP database. As a result, the FHWA adjusted the LTPP database of the CTE value, based on the changes described above.

4.4.1 Upgrading of HM-251 Software

The length change of the measuring frame was determined by testing a reference specimen of known CTE value. A 304 stainless steel (304 SS) was used as the reference specimen in HM-251, and the erroneous literature value (17.3×10^{-6} / °C) was built in as a known CTE for the reference specimen. The Gilson/Challenge technology provides an upgraded software that allows the user to input the CTE value of the reference specimen to overcome this problem. Figure 4-16 shows an overview of the upgraded HM-251 software. As the calibration

![Calibration window](image1)

(a) Calibration window

![Prompted window](image2)

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

Figure 4-16 Overview of upgraded HM-251 software.
test runs, click the calibration button in Figure 4-16(a); a prompt window will appear, as shown in Figure 4-16(b). The CTE value of the reference specimen then may be typed into the box. After completion of the calibration process, the typed CTE value of the reference specimen is effective in subsequent CTE testing.

4.4.2 Reference Specimen

FHWA sent three reference specimens made of 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 [2006], the standard test method for linear thermal expansion of solid materials using a push-rod dilatometer. In the materials and aerospace industries, ASTM E 228-06 is a widely accepted test method to measure the CTE of metals. The CTE test results from the two independent laboratories presented in Table 4-5.

Table 4-5 CTE of the reference materials from two independent laboratories

<table>
<thead>
<tr>
<th>Specimens</th>
<th>PMIC Average CTE ($10^{-6}/°C$) (10 to 50 °C)</th>
<th>TPRL Average CTE ($10^{-6}/°C$) (10 to 50 °C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>304 stainless steel-Gilson specimen</td>
<td>16.2</td>
<td>N/A</td>
</tr>
<tr>
<td>304 stainless steel-Pine specimen</td>
<td>15.9</td>
<td>15.6</td>
</tr>
<tr>
<td>304 stainless steel-FHWA specimen</td>
<td>15.8</td>
<td>15.8</td>
</tr>
</tbody>
</table>

Although all three reference specimens (Gilson, Pine, and FHWA) are made of 304 SS, the range of the CTE values from the two independent laboratories ranges between 15.6 and $16.2 \times 10^{-6}/°C$. AASHTO T 336-09 [2009] 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 10 to 50 °C. The CTE value of $16.2 \times 10^{-6}/\degree \text{C}$ was chosen for the CTE value of the reference specimen in this study, since HM-251 was manufactured by Gilson.

AASHTO T 336-09 [2009] further 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 (with a 4-in. diameter and a 7-in. height), made of 304 SS in Figure 4-17(b), was used to calibrate the length change of the testing frame and correctly measure the concrete CTE.

![Figure 4-17 Old and new reference specimens](image)

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

4.4.3 Test Results of Conventional Mixtures

Table 4-6 indicates 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 those with AASHTO TP 60-00. This result
means that the old CTE values, measured with AASHTO TP 60-00, were overestimated. A correction factor (CF) was introduced to correlate the measured CTE values with TP 60-00 and T336-09; Equation (4-3) was used to calculate the correction factor shown in Table 4-6. The range of correction factors for all five mixtures is between 0.91 and 0.96. The correction factor of Gravel (G 64%) has the largest value, while the correction factor of Mexican Limestone (ML 64%) has the smallest value.

\[ CTE_{T336-09} = CF \times CTE_{TP60-00} \]  

(4-3)

Table 4-6 CTE comparisons of two methods for conventional mixtures

<table>
<thead>
<tr>
<th>Specimen</th>
<th>CTE (×10⁻⁶/°C)</th>
<th>CTE difference</th>
<th>Correction factor (CF)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AASHTO TP 60-00</td>
<td>AASHTO T 336-09</td>
<td></td>
</tr>
<tr>
<td>KL (64%)</td>
<td>8.935</td>
<td>8.137</td>
<td>0.798</td>
</tr>
<tr>
<td>G (64%)</td>
<td>12.744</td>
<td>12.184</td>
<td>0.550</td>
</tr>
<tr>
<td>ML (64%)</td>
<td>8.729</td>
<td>7.900</td>
<td>0.829</td>
</tr>
<tr>
<td>KL (20%)</td>
<td>12.057</td>
<td>11.232</td>
<td>0.825</td>
</tr>
<tr>
<td>KL (80%)</td>
<td>8.463</td>
<td>7.678</td>
<td>0.785</td>
</tr>
</tbody>
</table>

An analysis of variance (ANOVA) was utilized to validate the impact of both coarse aggregate types and proportions on the correction factor of CTE. The overall ANOVA results are summarized in Table 4-7. The P-value means the probability of error of the statement. If the P-value of the variable is equal to or less than alpha (\(\alpha\)), the variable is considered to have a significant effect on measuring parameters. Alpha is a probability error level; 0.05 was used in the analysis. The rejection of the null hypothesis in the results of ANOVA implies that at least
one pair of the group means is unequal. In order to determine specifically which of the means are
different from one another, Tukey’s procedure was conducted as a multiple comparison, since the
procedure allows for all possible pair-wise tests [Freund and Wilson, 2003]. The ANOVA
analysis shows that the coarse aggregate type has a significant impact on the correction factor of
the 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 from each other. Therefore, the correction factor for both Kentucky and
Mexican limestone concretes can be determined by 0.91, and for gravel concrete, the factor can
be determined by 0.96 when the two extreme cases (KL (20 %) and KL (80 %)) are excluded.
The correction factors calculated in Table 4-6 can be used to produce correct CTE values for
duplicated specimens with no further measurement.

Table 4-7 Summary of ANOVA results for correction factors

<table>
<thead>
<tr>
<th>Variables</th>
<th>DF</th>
<th>F-value</th>
<th>P &gt; F</th>
<th>Significance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse aggregate type (KL (64%), G (64%), ML (64%))</td>
<td>2</td>
<td>48.40</td>
<td>0.0002</td>
<td>Yes</td>
</tr>
<tr>
<td>Coarse aggregate proportion (KL (20%), KL (64%), KL (80%))</td>
<td>2</td>
<td>3.48</td>
<td>0.1655</td>
<td>No</td>
</tr>
</tbody>
</table>

4.4.4 Test Results of Ternary Mixtures

Table 4-8 shows the variation of CTE values with twelve ternary mixtures as measured
with both AASHTO TP 60-00 and AASHTO T 336-09. Duplicated samples were used for the
CTE measurement of each ternary mixture. All of the CTE values measured with AASHTO TP
60-00 are larger than the values measured with AASHTO T 336-09. Two ternary mixtures
(50P30S20C and 30P30S40C) show the largest correction factor of 0.95, while one ternary mixture (10P50S40F) shows the smallest correction factor of 0.90.

Table 4-8 CTE comparisons of two methods for ternary mixtures

<table>
<thead>
<tr>
<th>Specimen</th>
<th>CTE (×10⁻⁶/°C)</th>
<th>CTE difference</th>
<th>Correction factor (CF)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>AASHTO TP 60-00</td>
<td>AASHTO T 336-09</td>
<td></td>
</tr>
<tr>
<td>50P30S20F</td>
<td>10.717</td>
<td>9.760</td>
<td>0.957</td>
</tr>
<tr>
<td>40P30S30F</td>
<td>10.479</td>
<td>9.485</td>
<td>0.994</td>
</tr>
<tr>
<td>30P30S40F</td>
<td>10.204</td>
<td>9.353</td>
<td>0.851</td>
</tr>
<tr>
<td>30P50S20F</td>
<td>11.039</td>
<td>10.253</td>
<td>0.786</td>
</tr>
<tr>
<td>20P50S30F</td>
<td>10.983</td>
<td>9.997</td>
<td>0.986</td>
</tr>
<tr>
<td>10P50S40F</td>
<td>10.564</td>
<td>9.539</td>
<td>1.025</td>
</tr>
<tr>
<td>50P30S20C</td>
<td>10.061</td>
<td>9.596</td>
<td>0.465</td>
</tr>
<tr>
<td>40P30S30C</td>
<td>9.923</td>
<td>9.255</td>
<td>0.668</td>
</tr>
<tr>
<td>30P30S40C</td>
<td>9.605</td>
<td>9.149</td>
<td>0.456</td>
</tr>
<tr>
<td>30P50S20C</td>
<td>10.516</td>
<td>9.522</td>
<td>0.994</td>
</tr>
<tr>
<td>20P50S30C</td>
<td>10.474</td>
<td>9.390</td>
<td>1.084</td>
</tr>
<tr>
<td>10P50S40C</td>
<td>9.965</td>
<td>9.273</td>
<td>0.692</td>
</tr>
</tbody>
</table>

4.4.5 Implementation of the Correction Factor

Table 4-6 and Table 4-8 present the correction factors for both conventional and ternary mixtures. There is limited knowledge toward understanding the reason for the various correction factors’ dependence upon various types and proportions of mixture ingredients, but the reasonable boundaries of correction factors may be determined from the seventeen mixtures. Figure 4-18 shows the frequencies of each value of the correction factor. The highest frequency
occurs at the correction factor of 0.91, but there is no trend shown in this graph. However, all the correction factors fall into the boundary between 0.90 and 0.96. A correction factor of 0.90 indicates that the CTE value measured by AASHTO T 336-09 is 10% smaller than the CTE value measured by AASHTO TP 60-00. The 10% overestimation of CTE value in a 15-ft. joint spacing results in an over-predicted joint faulting by 38% and percent slab cracking by 188%, respectively [Chung and Shin, 2011]. The CTE values, according to AASHTO TP 60-00, can be cautiously corrected, to comply with the AASHTO T 336-09, by using the correction factor developed in this study. The corrected CTE values can be used with the DARWin-ME software. If the CTE values according to AASHTO T 336-09 are used in conjunction with MEPDG version 1.xx, the thickness design of concrete pavement can be underestimated by 0.5 to 1.0 inches [Stephanos, 2009].

![Figure 4-18 Frequency of correction factors for all mixtures](image.png)

4.5 Summary

The CTEs of conventional concrete specimens having various aggregate types, ages, amounts of coarse aggregate and relative humidities were measured. A statistical analysis (ANOVA) of the measured results showed that the CTE of concrete was significantly affected by
aggregate types, the amount of coarse aggregate, and relative humidity. The maximum value of expansion CTE was observed at 63 % RH, and the value was 8 % higher than the value at 100 % RH. A sensitivity analysis using MEPDG showed that an overestimation of the CTE by 10 % from the accurate value in a joint spacing of 15-ft., a mean joint faulting and a percent slab cracking could cause excessive prediction of 38 % and 188 %, respectively. Over-predicted distresses can make a pavement design too conservative. Therefore, an accurate measurement of CTE value is imperative in order to achieve an optimal procedure in PCC pavements design.

The CTE tests of ternary-blended concrete specimens were performed to characterize ternary mixtures with various combinations of fly ashes and slags. All the CTE values of ternary mixtures were larger than that of the control mixture, except for a ternary mixture (30TI-30G100S-40C). The volumes of replaced SCMs are higher than that of Portland cement because fly ash and slag have lower specific gravities than Portland cement. The increased volume of cementitious paste induces higher CTE value of concrete since the paste tends to have the highest CTE value among other constituents in the mixture. The largest CTE value of a ternary mixture (50TI-30G120S-40F) was $10.2 \times 10^{-6}/^\circ\text{C}$, found to be 10% larger than that of the control mixture.

From the results of ANOVA analyses, the mean CTE values of three ternary groups (50G100S-F, 30G120S-F, and 50G120S-F) were significantly different than that of the control mixture in the 95 % confidence level. The prediction equations of CTE for eight groups of ternary mixture were introduced, finding that the predicted CTE has a good agreement with the measured CTE because the $R^2$ value is 0.918 and the maximum percentage difference between the two CTEs is 1.3 %.
This study explored two different CTE test methods and measured CTE values for both conventional and ternary concrete mixtures in accordance with AASHTO T 336-09. The goal was to develop correction factors to calibrate the incorrect CTE values measured by AASHTO TP 60-00. The range of correction factors for conventional mixtures was between 0.91 and 0.96. After statistical analyses, the correction factor for limestone concrete was determined to be 0.91; for gravel concrete, the correction factor was determined to be 0.96, with the exception of two extremely coarse aggregate proportions. The range of correction factors for ternary mixtures was between 0.90 and 0.95, depending on the types and proportions of cementitious materials. The boundaries of correction factors for both conventional and ternary concrete mixtures were restricted between 0.90 and 0.96, and the highest frequency of correction factor occurred at 0.91.
CHAPTER 5 VARIATIONS OF TEMPERATURE GRADIENTS IN PCC PAVEMENTS

5.1 Temperature Measurement in PCC Pavement

5.1.1 Installation of Temperature Sensors

In order to monitor the daily and seasonal variations of temperature profiles, temperature sensors were installed at various depths of the existing PCC pavement. The pavement section was located on Northline Road, West Baton Rouge, Louisiana. The typical section of the JPCP at Northline Road is illustrated in Figure 5-1. The subgrade soil is categorized as CH (fat clay) with respect to United Soil Classification System (USCS), as well as and A-7-6 (clayey soil) with respect to the AASHTO [Rasoulian et al., 2006]. A base course layer (10-in. thick crushed limestone) was placed on the compacted roadbed, and a 10-in. thick Portland cement concrete surface layer was placed on top of the base course using slip form paving method.

The temperature sensor manufactured by the Transtec group has 0.75-in. diameter and 0.25-in. thickness. The size of the sensor is small enough to install them within the limited thickness of PCC pavements. Theses sensors were originally developed for measuring maturity of
placed Portland cement concrete structures. The sensors are covered by rubber materials in order to protect from moisture. The temperature sensors were installed at various depths (0-, 2.5-, 5.0-, 7.5- and 10-in.) from the pavement surface, as shown in Figure 5-2. The sensors were set to measure temperature at twenty-minute intervals. The slab temperatures at various depths were measured from May 24 to November 29, 2011. Interruption of continuous data collection was experienced because of replacement of sensors and technical difficulties. The detailed procedures of sensor installation were illustrated in Appendix E. Because of limited storage capacity, the temperature should be collected every 28 days to obtain continuous data. The pocket PC, provided by the manufacturer, was used to collect data by connecting the cable to wire of temperature sensors. The measurement range of the temperature sensor is 14 to 185 °F, and the precision is ±2 °F of reading value.

Figure 5-2 Sensor locations in PCC pavement
5.1.2 Temperature Profiles

Figure 5-3 shows the temperature profiles of both the top and bottom of the pavement for a span of 49 days between June and July. The temperature fluctuation at the top of the concrete pavement was between 74.3 °F and 140 °F, while that at the bottom of the concrete pavement was between 84.2 °F and 104.9 °F. This demonstrates that the temperature variation of the pavement surface (65.7 °F) is much larger than that of the pavement bottom (20.7°F). The unusual temperature data observed between 27 and 29 days was a low air temperature due to rain. The temperature profiles of the pavement between October and November are presented in Appendix F.

Figure 5-3 Temperature profiles of the top and bottom of the concrete pavement (June and July)

Figure 5-4 shows a typical daily temperature variation at each sensor in the PCC pavements in June and November, respectively. In Figure 5-4, several points that characterize the daily temperature variations should be noted. As a direct effect of sunshine and air temperature, the surface temperature has the highest values during the day, with the lowest values at night. The surface temperature reaches its peak value at around 2 p.m., and the lowest value occurs at around
6 a.m. The peak values of each depth gradually delay in a descent from the surface: the peak values at 2.5-in. and 5-in. depth occur at 4 p.m.; and, the peak values at 7.5-in. and 10-in. depth occur at 5 p.m. All the measured temperatures at every depth in the PCC pavement become close together at both 9 a.m. and 5 p.m. The slope of temperature profiles in June is steeper than that of temperature profiles in November. The maximum surface temperature reaches 129.2 °F and 91.4 °F in June and November, respectively.

![Temperature Profiles](image)

(a) Daily temperature profiles in June

(b) Daily temperature profiles in November

Figure 5-4 Typical daily temperature profiles at various depths in PCC pavement
Figure 5-5 shows the variations of temperature gradients in the PCC pavement in June and November. The temperature variation at the slab surface is 44.1 °F and 37.8 °F, while the temperature variation at the bottom of slab is 7.8 and 8.1 °F in June and November, respectively. In June, the temperature gradient changes remarkably up to 3-in. depth from the surface at 12 p.m. and 2 p.m., and the temperature gradient appears very stable throughout the slab thickness at 9 a.m. as shown in Figure 5-5a.

(a) Daily temperature gradients in June

(b) Daily temperature gradients in November

Figure 5-5 Daily temperature gradients of 10-in. thick PCC pavement
5.2 Prediction Model of Pavement Temperature in Louisiana

5.2.1 Correlation between Air and Surface Temperature

The measured temperature at the pavement surface was found to be much higher than the air temperature collected from the weather station. One reason is that air temperature is normally measured at 4-ft. above the ground; the thermometer is protected from direct sunshine and precipitation by the instrument shelter [Finklin and Fischer, 1990]. On the other hand, the surface temperature sensor was exposed to direct sunshine and precipitation to measure a realistic surface temperature on the pavement. Thus, any temperature discrepancy between air and pavement surface temperature is caused by different procedures of measurements.

The typical temperature difference between air and pavement surface temperatures in both June and November is presented in Figure 5-6. The pavement surface temperature was measured in PCC pavement as described in the previous section. The air temperature was obtained from the weather station at Baton Rouge Airport. The temperature difference increases remarkably during the daytime, and its peak value of 33.1 °F appears at 2 p.m. in June, while 26.6 °F appears at 1 p.m.

![Figure 5-6 A typical temperature difference between surface and air temperature](image-url)
in November, respectively. The surface temperature remains higher than the air temperature during nighttime and the minimum difference appears at 8 a.m. (5.9 °F) in June.

Figure 5-7 shows the correlation between pavement surface and air temperature for different seasons. The correlation between two temperatures is not a linear but a second-order polynomial. The Equations 5-1 and 5-2 can be used to easily predict surface temperature of

![Figure 5-7 Correlation between surface and air temperatures](image)

(a) From May to July

(b) From September to November
the pavement using the air temperature obtained from the weather station. The temperature unit of the following formulas is in °F. R² values of each equation are 0.795 and 0.804, respectively.

From May to July,

\[
\text{Surface temp.} = 0.055 \times (\text{Air temp.})^2 - 6.976 \times (\text{Air temp.}) + 297.44
\]  

\( (5-1) \)

From September to November,

\[
\text{Surface temp.} = 0.019 \times (\text{Air temp.})^2 - 1.319 \times (\text{Air temp.}) + 84.18
\]  

\( (5-2) \)

5.2.2 Correlation between Surface and Slab Temperature Difference

The slab temperature difference plays an important role in calculating the curling stress in PCC pavements. The slab temperature difference is defined as the temperature difference between the top and bottom of the slab. The increased slab temperature difference induces a high possibility of cracking in concrete pavements. The typical trend of slab temperature difference in June and November is presented in Figure 5-8. The maximum slab temperature difference of 32.4 °F in June

![Figure 5-8 A typical temperature difference between the top and bottom of the slab](image-url)
and 19.9 °F in November occurs at 2 p.m.; the maximum value should be used to find the most vulnerable case to crack. The slab temperature difference shows negative values at night, which means that the surface temperature is lower than the bottom temperature.

Figure 5-9 illustrates the relation between the surface temperature and the slab temperature difference from May to November. From the relation, a second-order polynomial equation was developed as shown in Equation 5-3. The slab temperature difference increases proportionally, as the surface temperature increases. The temperature unit of the formula is in °F and R² value is 0.784. Using Equation 5-1 through 5-3, the slab temperature difference of the pavement can be conveniently predicted based on air temperature obtained from the weather station. The detailed correlations between surface temperature and slab temperature differences at various time frames were presented in Appendix F.

\[
\text{Slab temp. difference} = 0.007 \times (\text{Surface temp.})^2 - 0.601 \times \text{Surface temp.} - 5.25 \quad (5-3)
\]

![Figure 5-9 Correlation between surface temperature and slab temperature difference (May to November)](image-url)
5.3 Conclusions

Temperature profiles, one of the most important environmental factors for designing and predicting performance in PCC pavement, were measured by installing sensors at various depths in the field. The following observations are drawn from the results of this study:

- The peak value of the surface temperature occurs at 2 p.m. and the lowest value occurs at 6 a.m. All the measured temperatures at each depth are equal at 9 a.m. and 5 p.m.

- The temperature profiles fluctuate at the top surface and remain mostly stable at the bottom of the slab during the day. The temperature variation at the slab surface is 44.1 °F and 37.8 °F, while the temperature variation at the bottom of the slab is 7.8 °F and 8.1 °F in June and November, respectively.

- The surface temperature measured in the field is much higher than the air temperature collected from the weather station, and a maximum difference of 33.1 °F was observed at 2 p.m. The correlations between surface and air temperature were presented by second-order polynomial equations in order to calculate surface temperature from the available air temperature.

- The slab temperature difference (the top surface temperature minus the bottom surface temperature of the slab) carries the maximum values of 32.4 °F at 2 p.m., whereas the slab temperature difference shows negative values at night. A second-order polynomial relationship between the surface temperature and the slab temperature difference was presented.
CHAPTER 6 LOCAL CALIBRATION OF EICM IN LOUISIANA

6.1 Climatic Input Data in EICM

Since the EICM was designed and calibrated using nationwide climatic data, requirements mandate the local calibration processes based on regional climatic patterns and material properties in order to enhance the reliability of the outputs. To predict temperature gradients in the PCC pavement, an analysis was conducted using EICM version 3.4. The cross-section of JPCP consists of 10-in. PCC pavement, followed by 10-in. crushed limestone base and subgrade as presented in Figure 5-2. Using local materials, thermal properties of PCC pavement were measured: thermal conductivity (1.43 BTU/hr-ft-°F), heat capacity (0.19 BTU/lb-°F), and unit weight (147.4 pcf). Since the MEPDG website provides a climatic data set spanning the period from July 1996 to February 2006 (10 years), the weather station data from both Baton Rouge Airport and Louisiana Agriclimatic Information System (LAIS) were used to create climatic inputs during the same period of field measurements. Two weather stations are marked as white squares and the field location of PCC pavement is marked as a white circle in Figure 6-1.

Figure 6-1 Locations of field site and weather stations
All the climatic data are directly transferred from the weather station, except the percentage of sunshine and water table. The percentage of sunshine drawn from the weather station data are described as follows: clear, scattered clouds, partly clouds, mostly clouds, overcast. These weather conditions are converted into the five discrete numbers (100, 75, 50, 25, and 0 %) to directly utilize as input data in EICM. The water table was obtained from the USGS website (USGS, 2011). Table 6-1 shows an example of the climatic input data for 24-hour intervals in EICM.

Table 6-1 Climatic input data in EICM (24-hour intervals)

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<th>Temperature (°F)</th>
<th>Windspeed (mph)</th>
<th>Sunshine (%)</th>
<th>Precipitation (in)</th>
<th>Humidity (%)</th>
<th>Water table (ft)</th>
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6.2 Calibration of EICM Using Field-Measured Temperature in Louisiana

The EICM analysis was performed using the air temperature obtained from the weather station and the results at different times are shown in Figure 6-2. The EICM results using air temperature (EICM with Air Temp.) do not match well with measured temperature gradients in field, though the temperature gradients show a similar trend. The temperature between EICM-predicted and field-measured displays a large discrepancy during all time frames. The largest gap between the two temperatures occurs on the surface of the pavement at 2 p.m., and the value is 33.1 °F. After an observation of EICM results, it was noticed that the EICM-predicted temperature at the surface is usually similar to the air temperature value used as an input data in

![Figure 6-2 Comparison of temperature gradients between EICM-predictions and field-measurement at various times](image-url)
this analysis. As an effort to better calculate pavement temperature in EICM, air temperature from the weather station was replaced by the field-measured surface temperature. By using the measured surface temperature as input data in EICM, the temperature gradients in pavement shift to the right side, as shown in Figure 6-2. The temperature gradients, calculated using surface temperature as input data in EICM, have better agreement with the field-measured temperature. Therefore, surface temperature is suitable for the input data in EICM to better predict temperature gradients in the PCC pavement. Equation (5-1) and (5-2), presented in the previous chapter, can be used to predict the surface temperature for the measured air temperature at weather station.

6.3 Numerical Analysis of Thermal Changes in PCC Pavement

Finite element analysis was employed to calculate the temperature gradients of the PCC pavement. The finite element model illustrated in Figure 6-3 was developed to simulate the pavement section used for field-temperature measurement. The nonlinear finite element package, LS-DYNA (Hallquist, 1998), was used to compute temperature gradients through the slab thickness. Multiple 8-noded quadratic elements throughout the slab thickness were used to accurately model nonlinear temperature gradients. In this analysis, it was assumed that the slab temperature changes only through its depth. This means that the slab temperature is the same for all nodes with the same depth. All four sides have an insulated boundary condition, and the thickness of subgrade was set to 36-in. in order to get enough depth to avoid the interference by the boundary condition. After a sensitivity analysis of mesh size effect, the mesh size was decided by 4-in.×4-in.×2-in. for all layers. A three-layer slab was modeled to simulate the variation of temperature gradients in pavement slab. The 10-in. slab is 20-ft. long and 13-ft. wide.
Figure 6-3 Finite element model of the concrete pavement layers

Measured material properties, presented in earlier chapter, were used for the analysis; a density of 147.4 lb/ft$^3$, a thermal conductivity of 1.43 BTU/ft-h-°F, and a heat capacity of 0.19 BTU/ft$^3$-°F. The slab was founded on a 10-in. thick crushed limestone base and a 3-ft. thick subgrade. The slab was meshed by 39×60×5 and had 11,700 solid elements.

Figure 6-4a shows selected node points of the finite element model, and Figure 6-4b presents temperature profiles of each point for two weeks. The selected six nodes represent the 2-in. intervals from the surface to the bottom of the PCC pavement. The temperature loads are applied on the surface of the pavement using measured surface temperature every hour; the heat transfers through the slab thickness accordingly. The temperature gradients, used for the results of finite element analysis in this study, were collected after 7 days, because no initial temperature condition was set for the analysis.
Figure 6-4 Results of LS-DYNA analysis for the temperature in June

Figure 6-5 shows the three temperature gradients: a predicted temperature in EICM using surface temperature (EICM with Surface Temp.), a field-measured temperature (Measured), and a calculated temperature from finite element analysis (FEM), using surface temperature. All three results share the same temperature value on the surface, because a measured surface temperature was used for the boundary condition for all of them. Finite element analysis (FEM) tends to underestimate the temperature gradients in the morning; however, FEM tends to have a
good agreement with the temperature gradients of field-measurement in the afternoon. Temperature gradients calculated form a finite element analysis present a similar nonlinear trend of field-measured temperature gradients at both 2 p.m. and 6 p.m.

![Graph](image)

(a) 6 A.M.  
(b) 9 A.M.  
(c) 2 P.M.  
(d) 6 P.M.

Figure 6-5 Comparison of temperature gradients between EICM-prediction, field-measurement, and FEM results at various times

### 6.4 Conclusions

An EICM analysis was conducted using local climatic conditions and material properties data to obtain EICM-predicted temperature profiles. Local calibration of EICM was carried out by comparing the field-measured temperature with the EICM-predicted temperature profiles. The following observations are drawn from the results of this study:
• The temperature gradients using air temperature as an input data of EICM have a large discrepancy with the field-measured temperature gradients.

• The surface temperature is suitable for the input data in EICM to better predict a temperature gradient in PCC pavements. The temperature gradients using surface temperature as input data make the temperature gradients shift to the right side; thus, a better agreement between the EICM-predicted and field-measured temperature gradients is established.

• A finite element analysis tends to underestimate the temperature gradients in the morning; however, such an analysis tends to have a good agreement with the temperature gradients of field-measurement in the afternoon. Temperature gradients calculated from finite element analysis present a similar nonlinear trend of field-measured temperature gradients during the times of 2 p.m. and 6 p.m.
CHAPTER 7 THERMAL STRESS ANALYSIS OF JOINTED PLAIN CONCRETE PAVEMENTS

7.1 Temperature and CTE Gradients

The temperature gradient for a year (2011) was generated using EICM, based on the thermal properties of control mixture (100P), to determine the most critical seasonal and daily conditions. Figure 7-1 shows typical seasonal and daily temperature gradients for the 10-in. slabs. The representative temperature gradient for every two months at a certain time was chosen through the results of EICM analysis, as shown in Figure 7-1a. The largest temperature difference between the top and bottom of a 10-in. slab occurred in July. Thus, the daily temperature gradients were generated at five different time frames at the same day. The temperature at the top of the slab is lower than the bottom at 6 a.m. and the temperature at the top of slab peaks at 2 p.m. as shown in Figure 7-1b.

![Image of temperature gradients](image)

(a) Seasonal temperature gradients  
(b) Daily temperature gradients in July

Figure 7-1 Seasonal and daily temperature gradients calculated from EICM

Using the daily temperature gradients in July (Figure 7-1b), daily temperature model was developed by equating the bottom temperature to zero. Figure 7-2 shows the modeling of

temperature gradients used for calculating curling stresses. The most critical state occurs at 2 p.m. when the temperature difference between the top and the bottom of the slab is at the maximum value. The critical temperature gradient is expressed by the third-order polynomial with respect to the function of depth \( z \) as shown in Equation (7-1). A, B, C, and D are constants of the polynomial equation and are generated for each mixture. The constant values of each mixture were presented in Table 7-1. The zero reference of depth \( z \) is in the middle of the slab, and a downward direction is a positive, while the upward direction is a negative.

\[
T(z) = Az^3 + Bz^2 + Cz + D
\]  

(7-1)

Figure 7-2 A modeling of temperature gradients using third-order polynomial

Janssen [1987] investigated the moisture gradient in PCC pavement, using a computer model based on laboratory and field measurements. Figure 7-3 shows that the top surface of the pavement is saturated at 50 %, while the bottom of the pavement is saturated at 100 %. The variation of moisture is only remarkable within the top 2-in. of the PCC slab, and the bottom part of the slab has more than 80 % degree of saturation. The abrupt moisture change in concrete pavement does not penetrate more than the top 2-in. due to the low permeability of concrete.
Table 7-1 Stress analysis results and curve fitting constants

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<th>40TI 30G100S 30C</th>
<th>30TI 50G100S 20C</th>
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<th>50TI 30G100S 40C</th>
<th>40TI 50G100S 30F</th>
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<th>20TI 50G100S 20F</th>
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Chung and Shin [2011] investigated the effect of relative humidity on CTE and found that the maximum CTE value at a partially saturated condition (63 % RH) is 8 % higher than the CTE value at 100 % saturated condition. Since the CTE values of ternary mixtures in this research were measured at 100 % saturated condition, the measured CTE value corresponds to the CTE at the bottom of the slab. The maximum CTE value, which corresponds to the top of the slab (50 % degree of saturation) is 8 % greater than the measured value at a saturated condition. CTE ($\alpha$) gradient through the slab thickness resembles the moisture gradient shown in Figure 7-3. The CTE gradient is expressed by a third-order polynomial due to being the same reason for the nonlinear temperature gradient. The polynomial equation constants (E, F, G, and H) were decided as follows and were used for further stress analysis:

$$\alpha(z) = Ez^3 + Fz^2 + Gz + H$$  \hspace{1cm} (7-2)

($E = -0.0004, F = 0.0013, G = 0.00005, H = 1.0133$)
In the thermal stress analysis of this study, the strains through the thickness of pavements were further detailed with the multiplication of nonlinear temperature gradients (Equation 7-1) and nonlinear CTE gradients (Equation 7-2), as shown in Equation 7-3:

\[ \epsilon(z) = \alpha(z)T(z) \]  

(7-3)

7.2 Thermal Stress of Ternary-Blended PCC Pavements

A stress analysis was performed on a typical Louisiana PCC pavement (20-ft. long, 13-ft. width, and 10-in. thick) located in the Northline Road, West Baton Rouge, Louisiana. The details of the pavement section are presented in Figure 5-1. Measured mechanical properties (compressive strength, modulus of elasticity and Poisson’s ratio) and thermal properties (CTE, thermal conductivity, and heat capacity), presented in Appendix A and C, were used for the analysis. Temperature gradients were calculated in EICM using weather data obtained from a weather station in both Baton Rouge Airport and LAIS. Critical temperatures gradients obtained using EICM and CTE gradients through the slab thickness were used for the stress analysis. The modeling of temperature data used for calculating curling stresses occurred at 2 p.m. in July because the temperature difference between the top and the bottom of the slab was the maximum value. The calculated critical tensile stresses were compared with the tensile strength of each ternary mixture at various ages. Curve-fitting constants for temperature gradients through the slab thickness, curvature stresses, residual stresses, and total stress at the top and the bottom of each ternary mixture are presented in Table 7-1. The ternary combinations with Grade 120 slag were excluded in the thermal stress analysis.

Figure 7-4a shows some notable stress distributions of ternary mixtures at 28 days. The compressive stresses occurred at the top, and tensile stresses occurred at the bottom because the critical temperature gradients were obtained during daytime (2 p.m.). Although the stresses were
calculated through the depth of pavement based on the nonlinear temperature and CTE gradients, only tensile stresses at the bottom of the slab were considered critical because PCC concrete pavement is vulnerable to tensile stress. Figure 7-4b shows the tensile stress at the bottom of pavements to be made of ternary mixtures. Two ternary mixtures (40P30S30F and 30P50S20F) have 18.9 % and 13.9 % larger tensile stresses than the control mixture (100P) at 28 days, respectively. The combination of the higher CTE value with lower thermal conductivity values of the two ternary mixtures causes larger tensile stress than the control mixture. In contrast, a ternary mixture (30P30S40C) has a 13.5 % lower tensile stress than the control mixture (100P) due to its lower CTE and higher thermal conductivity value. The drying shrinkage was not considered in this study since tensile stresses were calculated at the bottom of the slab, where the slab is almost 100 % saturated condition and the effect of drying shrinkage would be minimized.

7.3 Tensile Stress-to-Strength Ratio of Ternary-Blended PCC Pavements

The ratio of tensile stress-to-strength at various ages (14, 28, and 90 days) is presented in Figure 7-5 and Figure 7-6. As each mixture has various ranges of tensile stresses, the mixture
also has a variety of tensile strength at different ages. The tensile strength is assumed to be 10 % of compressive strength [Mehta, 2002]. Figure 7-5 shows the tensile stress-to-strength ratio for 30 % slag-replacement at various ages. The 30 % slag-replacement ternary mixtures show that the ratio of tensile stress-to-strength decreases slightly at 90 days as the replacement of class C fly ash increases from 20 % to 40 % due to a gradual increase of tensile strength up to 90 days.

Figure 7-5 Tensile stress-to-strength ratio of 30 % slag-replacement ternary mixtures

Figure 7-6 Tensile stress-to-strength ratio of 50 % slag-replacement ternary mixtures
All the ternary mixtures with the replacement of 30 % slag with 20 % fly ashes and 30 % slag with 30 % fly ashes (both Class C and F) do not exceed 100 % tensile stress-to-strength ratio at all ages. Figure 7-6 shows the tensile stress-to-strength ratio for 50 % slag-replacement at various ages. The 50 % slag-replacement ternary mixtures clearly show that the ratio of tensile stress-to-strength increases as fly ash (both Class C and F) replacement increases. A ternary mixture (10P50S40C) has 310 % of tensile stress-to-strength ratio at 14 days, due to its small tensile strength development at early-age. All the ternary mixtures with the replacement of 50 % slag with 20 % fly ashes (both Class C and F) do not exceed 100 % tensile stress-to-strength ratio at all ages. The use of ternary mixture improves the mixture properties in both fresh and hardened concrete, but limited amounts of fly ash and slag should be specified in order to avoid early-age cracking; thus, these combinations can be considered as a limitation of ternary mixture replacement with slag and fly ashes. This observation should be verified with field monitoring of temperature and moisture gradients with strain measurements.

7.4 Conclusions

The thermal stress analyses of ternary mixtures were performed to calculate the critical tensile stress on the PCC pavements, using nonlinear temperature and CTE gradients obtained from this study. On the basis of findings from this study, the following conclusions are drawn:

- The modeling of critical temperature gradient used for calculating curling stresses occurred at 2 p.m. in July because the temperature difference between the top and bottom of the slab was the maximum value.
- Two ternary mixtures (40P30S30F and 30P50S20F) have 18.9 and 13.9 % larger tensile stresses than the control mixture (100P) at 28 days, respectively. The combination of
higher CTE value with lower thermal conductivity of the two ternary mixtures causes larger tensile stresses than with the control mixture. A ternary mixture (30P30S40C) has 13.5 % smaller tensile stress than the control mixture (100P), due to its lower CTE value and higher thermal conductivity value.

- The ternary mixtures with the replacement of 30 % slag with 20 % fly ashes, 30 % slag with 30 % fly ashes, and 50 % slag with 20 % fly ashes (both Class C and F) do not exceed 100% tensile stress-to-strength ratio at all ages. These combinations can be considered as the limitation of ternary mixture replacement with slag and fly ashes.
CHAPTER 8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

In this study, one control and twenty-four ternary mixtures with various combinations of fly ashes (Class C and F), slags (Grade 100 and 120), and Portland cement were cast. The thermal properties (CTE, TC, and HC) and mechanical properties of the selected ternary mixtures were measured at various ages. Temperature gradients were measured using a concrete pavement (10-in. thick) to characterize daily and seasonal temperature variations through the slab thickness. The relationships between air temperature and surface temperature, and surface temperature and temperature difference of the slab thickness were established, based on the measured temperature gradients in the concrete pavement. Local calibration of EICM was performed by comparing EICM-predicted temperature gradients to field measurement. A thermal stress analysis of the ternary mixtures was conducted to calculate the critical tensile stress on the PCC pavements, using the measured mechanical properties, nonlinear temperature gradients obtained from EICM, and CTE gradients throughout the slab thickness. The ratios of tensile stress-to-strength at the critical state of concrete pavements were estimated to investigate the vulnerability of ternary mixtures to tensile stress. The following conclusions were made, based on the results of this study:

- In a relationship between compressive strength and a modulus of elasticity, the ACI equation generally provides a conservative lower boundary of all the ternary mixtures at various ages, while the CEB-FIP model has a good agreement with the measured value of ternary mixtures at all ages.
- In regard to the relationship between compressive strength and flexural strength, the measured values of ternary mixtures are close to the ACI 363 equation, whereas the ACI
318 equation provides a conservatively low boundary limit with all ranges of compressive strength, except for extremely low compressive strengths.

- The maximum value of expansion CTE was observed at a partially saturated condition (63 % RH), and the value was 8 % higher than the value at a saturated condition (100 % RH)
- A sensitivity analysis of CTE using MEPDG showed that CTE value is too sensitive to correctly predict the performance of PCC pavements. Therefore, either local calibration of distress model of MEPDG or accurate measurement of CTE value is imperative to optimize the design procedure of PCC pavements.
- All the CTE values of ternary mixtures were larger than that of the control mixture, except for a ternary mixture. The volumes of replaced SCMs are higher than that of Portland cement because fly ash and slag have lower specific gravities than Portland cement. The increased volume of cementitious paste induces higher CTE value of concrete since the paste tends to have the highest CTE value among other constituents in the mixture.
- The range of correction factors for conventional mixtures was between 0.91 and 0.96. After statistical analyses, the correction factor for limestone concrete was determined to be 0.91; for gravel concrete, the correction factor was determined to be 0.96. The range of correction factors for ternary mixtures was between 0.90 and 0.95, depending on the types and proportions of cementitious materials.
- The correlations between the surface and the air temperature, as well as the surface temperature and the slab temperature difference were presented by second-order polynomial equations in order to easily predict surface temperature or temperature difference of PCC pavement based on the air temperature.
- The surface temperature is suitable for the input data in EICM to accurately predict
temperature gradient in PCC pavements. The temperature gradients, using surface temperature as input data, make the temperature gradients shift to the right side; thus, a better agreement between the EICM-predicted and field-measured temperature gradients has been accomplished.

- Most of the ternary mixtures have higher tensile stresses than the control mixture at the bottom of the slab due to the combination of a higher CTE and lower thermal conductivity. The largest tensile stress of ternary mixtures was 18.9% higher than that of the control mixture at 28 days.

- The ternary mixtures with the replacement of 30% slag with 20% fly ashes, 30% slag with 30% fly ashes, and 50% slag with 20% fly ashes (both Class C and F) do not exceed 100% tensile stress-to-strength ratio at all ages. These combinations can be considered as the limitation of ternary mixture replacement with slag and fly ashes.

### 8.2 Practical Applications

- Based on the thermal stress analysis results, the ternary mixtures with the replacement of 30% slag with 20% fly ashes, 30% slag with 30% fly ashes, and 50% slag with 20% fly ashes (both class C and F) do not exceed 100% tensile stress-to-strength ratio all ages. These results have a good agreement with the recommendations from LADOTD: (1) the maximum Portland cement replacement should not exceed 70%, and (2) the fly ash content cannot be greater than the slag content for ternary combinations containing fly ash and slag. [Rupnow, 2012].

- Three joint spacings (15-, 18-, 20-ft. joint spacing) and two coarse aggregates (Kentucky limestone and gravel) were analyzed in MEPDG. Only 15 and 18-ft. joint spacing of Kentucky limestone satisfied the specification of all the distresses. Even in the shortest joint
spacing (15-ft.), gravel exceeded the specification of transverse cracking; thus, transverse 
 cracking is the most sensitive distress to the CTE. According to the AASHTO Research and 
 Communication (RAC) survey, most states have joint spacing of JPCP at less than 20-ft.; for 
 example, 15-ft. for Arkansas, between 13- to 17-ft. for Arizona, 15-ft. for Georgia, and 15-ft. 
 for Missouri. Considering both the MEPDG analysis and the case study of other states, 
 current maximum joint spacing (20-ft.) can be adjusted to 15- or 18-ft. 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. However, changing the concrete CTE input in the MEPDG without 
 recalibrating the models will negatively impact the resulting design.

8.3 Recommendations for Future Research

There are many aspects of future research to improve and validate the results of thermal 
 stress analysis in this study. A few recommendations for the enhancement of the study are 
 addressed as follows:

- A reliable temperature measuring system is necessary to continuously collect temperature 
 gradients through the slab thickness for over a year. The stable reading of temperature 
 gradients will provide the characteristics of daily and seasonal temperature gradients within
the slab; thus, providing an accurate prediction of the thermal stresses in PCC pavements.

- The results of the thermal stress analysis in this study should be compared to the results of the DARWin-ME software, because DARWin-ME furnishes stresses at specific points in the PCC pavement.

- Thermal stress analysis of 10-in. thick PCC pavement was performed based on the field-measured temperature gradients in this study. Further analysis with thicker PCC pavements (14-in. or 16-in. thick) should be performed, using measured temperature gradients in the field. Increased slab thickness usually causes increments of temperature difference between the top and the bottom of the slab. Subsequently, thermal stresses might develop.
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Rasoulian, M., Titi, H., and Martinez, M. “Evaluation of Narrow Transverse Contraction Joints in Jointed Plain Concrete Pavements,” LADOTD/FHWA/06-411, Louisiana Transportation Research Center, 2006.


Zoldners, N. G. “Temperature and Concrete,” SP-25, American Concrete Institute, Detroit, MI, pp. 1-31, 1971.
APPENDIX A. MIXTURE DESIGN, AND FRESH AND HARDENED CONCRETE PROPERTIES FOR TERNARY MIXTURES

Table A-1 Mixture design for the ternary mixtures containing Grade 100 slag with Class C and F fly ash

| Mixture ID | 100TI | 50TI | 30G100S 20C | 50TI | 30G100S 30C | 40TI | 30G100S 40C | 30TI | 50G100S 20C | 30TI | 50G100S 30C | 20TI | 50G100S 40C | 10TI | 50G100S 20F | 50TI | 30G100S 30F | 40TI | 30G100S 40F | 30TI | 50G100S 20F | 20TI | 50G100S 30F | 10TI | 50G100S 40F |
|------------|-------|------|-------------|------|-------------|------|-------------|------|-------------|------|-------------|------|-------------|------|-------------|------|-------------|------|-------------|------|-------------|------|-------------|
| Type I Portland cement (%)       | 100   | 50   | 40          | 30   | 30          | 20   | 10          | 50   | 40          | 30   | 30          | 20   | 10          | 50   | 40          | 30   | 30          | 20   | 10          | 50   | 40          |
| Grade 100 slag (%)               | –     | 30   | 30          | 30   | 50          | 50   | 50          | 30   | 30          | 30   | 50          | 50   | 50          | –     | –           | –     | –           | –     | –           | –     | –           |
| Grade 120 slag (%)               | –     | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           |
| Class C fly ash (%)              | –     | 20   | 30          | 40   | 20          | 30   | 40          | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           |
| Class F fly ash (%)              | –     | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           | –    | –           |
| Slump (in.)                      | 2.25  | 5    | 3.25        | 6.75 | 4.25        | 3    | 3.5         | 2.75 | 5.25        | 6.0  | 0.0         | 0.5  | 0.75        |
| Air content (%)                  | 4.5   | 5.2  | 4.7         | 4.3  | 3.5         | 3.9  | 4.0         | 3.9  | 3.85        | 5.8  | 4.3         | 4.0  | 2.8         |
| Unit weight (lb/ft³)             | 147.4 | 143.4| 144.4       | 147.0| 146.4       | 145.6| 147.0       | 147.6| 148.0       | 147.4| 148.8       | 149.2| 147.4       |

(Note: TI is Type I Portland cement, C is class C fly ash, F is class F fly ash, G100S is grade 100 slag, and G120S is grade 120 slag)
Table A-2 Mixture design for the ternary mixtures containing Grade 120 slag with Class C and F fly ash

<table>
<thead>
<tr>
<th>Mixture ID</th>
<th>50TI 30G120S 20C</th>
<th>40TI 30G120S 30C</th>
<th>30TI 30G120S 40C</th>
<th>30TI 50G120S 20C</th>
<th>20TI 50G120S 30C</th>
<th>10TI 50G120S 40C</th>
<th>50TI 30G120S 20F</th>
<th>40TI 30G120S 30F</th>
<th>30TI 50G120S 40F</th>
<th>30TI 50G120S 20F</th>
<th>20TI 50G120S 30F</th>
<th>10TI 50G120S 40F</th>
</tr>
</thead>
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<td>Type I Portland cement (%)</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>30</td>
<td>20</td>
<td>10</td>
<td>50</td>
<td>40</td>
<td>30</td>
<td>30</td>
<td>20</td>
<td>10</td>
</tr>
<tr>
<td>Grade 100 slag (%)</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Grade 120 slag (%)</td>
<td>30</td>
<td>30</td>
<td>30</td>
<td>50</td>
<td>50</td>
<td>50</td>
<td>30</td>
<td>30</td>
<td>30</td>
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<td>50</td>
</tr>
<tr>
<td>Class C fly ash (%)</td>
<td>20</td>
<td>30</td>
<td>40</td>
<td>20</td>
<td>30</td>
<td>40</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Class F fly ash (%)</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>–</td>
<td>20</td>
<td>30</td>
<td>40</td>
<td>20</td>
<td>30</td>
<td>40</td>
</tr>
<tr>
<td>Slump (in.)</td>
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<td>1.5</td>
<td>1.0</td>
<td>2.0</td>
<td>3.0</td>
<td>4.0</td>
<td>2.5</td>
<td>1.5</td>
<td>3.25</td>
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<td>7.5</td>
</tr>
<tr>
<td>Air content (%)</td>
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<td>3.3</td>
<td>3.4</td>
<td>3.6</td>
<td>3.5</td>
<td>2.9</td>
<td>3.6</td>
<td>2.9</td>
<td>4.0</td>
<td>3.7</td>
<td>4.4</td>
<td>3.4</td>
</tr>
<tr>
<td>Unit weight (lb/ft³)</td>
<td>149.2</td>
<td>148.8</td>
<td>149.2</td>
<td>148.4</td>
<td>146.8</td>
<td>148.8</td>
<td>147.8</td>
<td>147.6</td>
<td>146.6</td>
<td>147.8</td>
<td>145.6</td>
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</tr>
</tbody>
</table>

(Note: TI is Type I Portland cement, C is class C fly ash, F is class F fly ash, G100S is grade 100 slag, and G120S is grade 120 slag)
Table A-3 Hardened mechanical properties for the ternary mixtures containing Grade 100 slag with Class C and F fly ash [Rupnow, 2012]

<table>
<thead>
<tr>
<th>Mixture ID</th>
<th>100TI</th>
<th>50TI 30G100S 20C</th>
<th>40TI 30G100S 30C</th>
<th>30TI 30G100S 40C</th>
<th>30TI 50G100S 20C</th>
<th>30TI 50G100S 40C</th>
<th>20TI 50G100S 20F</th>
<th>10TI 30G100S 40F</th>
<th>50TI 30G100S 30F</th>
<th>30TI 30G100S 40F</th>
<th>30TI 50G100S 20F</th>
<th>20TI 50G100S 30F</th>
<th>10TI 50G100S 40F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (psi)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 days</td>
<td>5,440</td>
<td>2,560</td>
<td>3,020</td>
<td>1,820</td>
<td>2,640</td>
<td>1,390</td>
<td>490</td>
<td>3,440</td>
<td>2,830</td>
<td>2,350</td>
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</tr>
<tr>
<td>14 days</td>
<td>5,800</td>
<td>3,610</td>
<td>4,660</td>
<td>2,690</td>
<td>4,470</td>
<td>3,160</td>
<td>880</td>
<td>5,320</td>
<td>4,780</td>
<td>3,780</td>
<td>5,220</td>
<td>4,180</td>
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<tr>
<td>28 days</td>
<td>5,860</td>
<td>4,830</td>
<td>5,460</td>
<td>4,280</td>
<td>6,430</td>
<td>5,550</td>
<td>2,710</td>
<td>6,070</td>
<td>5,500</td>
<td>4,490</td>
<td>5,740</td>
<td>4,790</td>
<td>3,000</td>
</tr>
<tr>
<td>56 days</td>
<td>6,410</td>
<td>5,450</td>
<td>5,990</td>
<td>6,470</td>
<td>8,090</td>
<td>7,250</td>
<td>4,160</td>
<td>7,070</td>
<td>6,220</td>
<td>4,920</td>
<td>6,080</td>
<td>5,110</td>
<td>3,200</td>
</tr>
<tr>
<td>90 days</td>
<td>6,700</td>
<td>5,620</td>
<td>6,490</td>
<td>7,540</td>
<td>8,420</td>
<td>8,540</td>
<td>4,530</td>
<td>6,470</td>
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<td>Modulus of elasticity × 10^3 (psi)</td>
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<td></td>
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</tr>
<tr>
<td>7 days</td>
<td>4,895</td>
<td>3,900</td>
<td>4,075</td>
<td>3,425</td>
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<td>3,950</td>
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<td>4,600</td>
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<td>5,250</td>
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<td>5,750</td>
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<td>5,450</td>
<td>5,050</td>
<td>5,325</td>
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<td>Flexural strength (psi)</td>
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<tr>
<td>7 days</td>
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<td>578</td>
<td>604</td>
<td>474</td>
<td>556</td>
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<td>696</td>
<td>590</td>
<td>512</td>
<td>692</td>
<td>683</td>
<td>547</td>
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<td>14 days</td>
<td>798</td>
<td>753</td>
<td>738</td>
<td>590</td>
<td>741</td>
<td>614</td>
<td>264</td>
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<td>759</td>
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<td>820</td>
<td>661</td>
</tr>
<tr>
<td>28 days</td>
<td>913</td>
<td>888</td>
<td>817</td>
<td>685</td>
<td>1,067</td>
<td>897</td>
<td>598</td>
<td>957</td>
<td>879</td>
<td>836</td>
<td>1,031</td>
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<tr>
<td>90 days</td>
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<td>962</td>
<td>874</td>
<td>1,214</td>
<td>954</td>
<td>646</td>
<td>888</td>
<td>878</td>
<td>936</td>
<td>978</td>
<td>1,072</td>
<td>790</td>
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<tr>
<td>Rapid chloride permeability (Coulombs)</td>
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<td>475</td>
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<td>292</td>
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<td>320</td>
<td>348</td>
<td>205</td>
<td>141</td>
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</table>
Table A-4 Hardened mechanical properties for the ternary mixtures containing Grade 120 slag with Class C and F fly ash [Rupnow, 2012]

<table>
<thead>
<tr>
<th>Mixture ID</th>
<th>50TI 30G120S 20C</th>
<th>40TI 30G120S 30C</th>
<th>30TI 50G120S 40C</th>
<th>30TI 50G120S 20C</th>
<th>20TI 50G120S 30C</th>
<th>10TI 50G120S 20F</th>
<th>50TI 30G120S 20F</th>
<th>40TI 30G120S 30F</th>
<th>30TI 50G120S 20F</th>
<th>30TI 50G120S 20F</th>
<th>20TI 50G120S 30F</th>
<th>10TI 50G120S 40F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (psi)</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 days</td>
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<td>4,430</td>
<td>4,900</td>
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<td>100</td>
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<td>3,970</td>
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<td>7,260</td>
<td>5,880</td>
<td>6,690</td>
<td>6,360</td>
<td>5,600</td>
<td>530</td>
<td>5,680</td>
<td>5,400</td>
<td>4,050</td>
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<td>3,990</td>
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<td>8,580</td>
<td>7,300</td>
<td>7,930</td>
<td>7,680</td>
<td>6,780</td>
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<td>6,830</td>
<td>6,000</td>
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<tr>
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<td>4,475</td>
<td>4,525</td>
<td>4,375</td>
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<td>4,450</td>
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<td>4,925</td>
<td>4,500</td>
<td>4,650</td>
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<td>4,425</td>
<td>2,150</td>
<td>4,950</td>
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<td>5,075</td>
<td>5,200</td>
<td>5,575</td>
<td>4,900</td>
<td>4,275</td>
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<td>5,325</td>
<td>4,800</td>
<td>5,125</td>
<td>4,825</td>
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<tr>
<td>90 days</td>
<td>5,650</td>
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<td>5,800</td>
<td>5,800</td>
<td>5,625</td>
<td>5,325</td>
<td>5,875</td>
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<td>5,625</td>
<td>5,675</td>
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<td>4,775</td>
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<tr>
<td>7 days</td>
<td>732</td>
<td>858</td>
<td>751</td>
<td>873</td>
<td>628</td>
<td>54</td>
<td>717</td>
<td>736</td>
<td>578</td>
<td>687</td>
<td>611</td>
<td>477</td>
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<td>14 days</td>
<td>902</td>
<td>811</td>
<td>817</td>
<td>1,127</td>
<td>860</td>
<td>245</td>
<td>905</td>
<td>844</td>
<td>755</td>
<td>859</td>
<td>757</td>
<td>558</td>
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<tr>
<td>28 days</td>
<td>1,089</td>
<td>982</td>
<td>944</td>
<td>1,250</td>
<td>853</td>
<td>528</td>
<td>1,015</td>
<td>914</td>
<td>833</td>
<td>1,019</td>
<td>843</td>
<td>588</td>
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<tr>
<td>90 days</td>
<td>1,011</td>
<td>973</td>
<td>980</td>
<td>1,159</td>
<td>1,151</td>
<td>665</td>
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<td>1,110</td>
<td>972</td>
<td>905</td>
<td>868</td>
<td>674</td>
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<td>Rapid chloride permeability (Coulombs)</td>
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<td></td>
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<td></td>
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<td></td>
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<tr>
<td>56 days</td>
<td>765</td>
<td>451</td>
<td>401</td>
<td>356</td>
<td>374</td>
<td>726</td>
<td>609</td>
<td>533</td>
<td>453</td>
<td>388</td>
<td>253</td>
<td>236</td>
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Figure A-1 A comparison of normalized compressive strength of ternary mixtures containing Grade 100 slag with Class F fly ash

Figure A-2 A comparison of normalized compressive strength of ternary mixtures containing Grade 120 slag with Class F fly ash
APPENDIX B. CTE TEST PROCEDURES (HM-251)

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 valve. 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.
APPENDIX C. THERMAL CONDUCTIVITY AND HEAT CAPACITY OF CONVENTIONAL AND TERNARY MIXTURES

Table C-1 Thermal conductivity and heat capacity of conventional mixtures [Kodide, 2010]

<table>
<thead>
<tr>
<th>Thermal properties</th>
<th>Unit</th>
<th>KL (20%)</th>
<th>KL (64%)</th>
<th>KL (80%)</th>
<th>G (64%)</th>
<th>ML (64%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thermal conductivity</td>
<td>BTU/ft·h·°F</td>
<td>1.87</td>
<td>1.60</td>
<td>1.61</td>
<td>2.00</td>
<td>1.29</td>
</tr>
<tr>
<td>Heat capacity</td>
<td>BTU/ft³·°F</td>
<td>0.20</td>
<td>0.21</td>
<td>0.19</td>
<td>0.17</td>
<td>0.19</td>
</tr>
</tbody>
</table>

Table C-2 Thermal conductivity and heat capacity of ternary mixtures [Kodide, 2010]

| Mixture ID | 100TI | 50TI | 40TI | 30TI | 20TI | 10TI | 50TI | 40TI | 30TI | 20TI | 10TI | 50TI | 40TI | 30TI | 20TI | 10TI |
|------------|-------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|------|
|            | SSD   | SSD  | SSD  | SSD  | SSD  | SSD  | SSD  | SSD  | SSD  | SSD  | SSD  | SSD  | SSD  | SSD  | SSD  | SSD  | SSD  |
| Thermal conductivity (BTU/ft·h·°F) | | | | | | | | | | | | | | | | | |
| SSD        | 1.43  | 1.33 | 1.37 | 1.28 | 1.42 | 1.23 | 1.27 | 1.38 | 1.34 | 1.42 | 1.37 | 1.31 | 1.30 | | | |
| Heat capacity (BTU/ft³·°F) | | | | | | | | | | | | | | | | | |
| SSD       | 0.19  | 0.17 | 0.17 | 0.16 | 0.16 | 0.17 | 0.15 | 0.17 | 0.18 | 0.18 | 0.17 | 0.16 | 0.17 | | | | |

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APPENDIX D. MICROSTRUCTURE ANALYSIS OF PLAIN CEMENT PASTE AND CEMENT PASTE CONTAINING FLY ASH AND SLAG

Figure D-1 SEM images of plain cement paste and cement paste containing fly ash and slag

(a) 100TI  
(b) 50TI-30G100S-20F  
(c) 10TI-50G100S-40F  
(d) 50TI-30G120S-20C
Figure D-2 XPS spectra of plain cement paste and cement paste containing fly ash and slag

* Binding energy is the energy required to release an electron from its atomic or molecular orbital and values are reported as an electron volt (eV). Thus, the binding energies are used to identify the elements to which the peaks correspond.
* Intensity is measured in counts per unit time (such as counts per second). Comparing the areas under the peaks, intensity gives relative percentages of the elements detected in the sample.
APPENDIX E. INSTALLATION PROCEDURES OF TEMPERATURE SENSORS (iButton) IN PCC PAVEMENT

(a) iButton
(b) Set up the sensor at several depths
(c) Drilling a hole
(d) Insert the sensor in the hole
(e) Fill the hole with mortar
(d) Completion of sensor installation

Figure E-1 Installation procedures of temperature sensors in PCC pavements
APPENDIX F. TEMPERATURE MEASUREMENT IN THE FIELD

Figure F-1 Temperature profiles of the top and bottom of the pavement (Oct. and Nov.)
Figure F-2 Correlation between surface temperature and slab temperature difference at various times
Figure F-3 Comparison of temperature variation between field-measurement and FEM-calculated (June)
Figure F-4 FEM-Calculated 24-hour temperature variation at various pavement depths

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<th>Surface</th>
<th>2 in. depth</th>
<th>4 in. depth</th>
<th>6 in. depth</th>
<th>8 in. depth</th>
<th>10 in. depth</th>
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<td>Temperature (°F)</td>
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<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Time (hours)</td>
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<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
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APPENDIX G. LETTERS OF PERMISSION

Figure G-1 Permission letter from American Society of Civil Engineers
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<td>Characterization of the coefficient of thermal expansion and its effect on the performance of Portland cement concrete pavements</td>
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<td>Yoonseok Chung et al.</td>
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Figure G-2 Permission letter from Canadian Journal of Civil Engineering
VITA

Yoonseok Chung was born in June 1973 in Seoul, South Korea. He received a Bachelor of Science in Civil Engineering in February 1999 and a Master of Science in Civil Engineering in February 2001 from Kyunghee University, South Korea, respectively. He worked as a geotechnical engineer at Korea Construction Management Corporation for two years in Seoul, South Korea. He graduated from Purdue University in West Lafayette, Indiana, with a Master of Science in Civil Engineering in May 2007. He then began a doctoral program in civil engineering at Louisiana State University in Baton Rouge, Louisiana. He expects to receive the degree of Doctor of Philosophy in Civil Engineering in August 2012.