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Research Report

TEMPERATURE CONTROL REQUIREMENTS FOR THE CONSTRUCTION OF MASS CONCRETE MEMBERS

Submitted to

The Alabama Department of Transportation

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16. Abstract
Special precautions must be taken when constructing concrete elements designated as mass concrete. These precautions may be satisfied through the formation of a mass concrete specification. Temperature requirements must be established to mitigate thermal cracking and delayed ettringite formation (DEF), both of which cause premature deterioration of concrete. The temperature difference between the concrete core and outside edge must be limited to prevent excessively wide thermal cracks. To prevent DEF, the maximum concrete temperature must be limited to 160°F or 185°F, depending on the amount and type of supplementary cementing materials (SCMs) used. When designating a member as mass concrete, a minimum element dimension must be established to help ensure that the temperature limits are not exceeded. These temperature and size limits are investigated here in order to develop an ALDOT specification for mass concrete construction.

Temperature data from seven ALDOT bridge elements were collected using temperature sensors. Maximum temperatures and temperature differences were used to determine appropriate temperature limits for the ALDOT specification as well as validate the temperature predictions of the ConcreteWorks software used for the same elements. Once ConcreteWorks' accuracy was determined, 480 theoretical concrete placements were statistically analyzed to determine that the use of low coefficient of thermal expansion (CTE) concrete and SCMs play a major role in limiting maximum temperatures and temperature differences. A mass concrete specification was then developed, which designates mass concrete least dimensions, maximum temperature limits, and maximum temperature difference limits based on SCM use and concrete CTE.

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ABSTRACT

Special precautions must be taken when constructing concrete elements designated as mass concrete. These precautions may be satisfied through the formation of a mass concrete specification. Temperature requirements must be established to mitigate thermal cracking and delayed ettringite formation (DEF), both of which cause premature deterioration of concrete. The temperature difference between the concrete core and outside edge must be limited to prevent excessively wide thermal cracks. To prevent DEF, the maximum concrete temperature must be limited to 160°F or 185°F, depending on the amount and type of supplementary cementing materials (SCMs) used. When designating a member as mass concrete, a minimum element dimension must be established to help ensure that the temperature limits are not exceeded. These temperature and size limits are investigated here in order to develop an ALDOT specification for mass concrete construction.

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Chapter 1

Introduction

1.1 Background

Mass concrete construction began in the United States with the construction of concrete dams. The frequency of such projects increased significantly during the early 1900s with improving concrete placement capabilities (ACI 207.1R 2005). As project size increased, engineers began to observe significant cracking in large, newly-placed concrete elements. In 1930, an American Concrete Institute (ACI) Committee was formed to examine and solve the problems that had been discovered. At that time, the Hoover Dam in Nevada was in the early stages of planning. Because of the unprecedented size of the dam, extensive research was carried out to determine the factors that were causing cracking in these large placements.

The results for the Hoover Dam were the use of a low-heat cement, the use of embedded cooling pipes for the first time, and a new era of improved concrete dam construction (ACI 207.1R 2005). Over the years, as concrete technology has improved and structures have grown larger, mass concrete elements became a commonplace, including foundations for bridges and large buildings and many other bridge elements. As such, research and investigations into issues associated with the construction and performance of mass concrete have increased due to the demand for better performing structures.

ACI 301 (2016) defines mass concrete as any “volume of structural concrete in which a combination of dimensions of the member being cast, the boundary conditions, the characteristics of the concrete, and the ambient conditions can lead to undesirable thermal stresses, cracking, deleterious chemical reactions, or reduction in long-term strength as a result of elevated concrete temperature due to heat of hydration”.

Based on this definition, the size of a concrete element directly affects the potential for heat to become entrapped. The amount of heat generated within an element comes from the cementitious materials used that react exothermically when in contact with water. This buildup of heat can lead to thermal cracking, delayed ettringite formation (DEF), and other issues that develop over the life of the concrete.

Regarding mass concrete designation, the standard metric of definition is the least dimension of the element. This least dimension is suggestive of the amount of heat the interior concrete will retain during hydration. The greater the least dimension, the more heat will be retained due to the larger amount of interior concrete. Compare a slab with dimensions of 40 ft × 40 ft × 1 ft to a foundation with measurements of 20 ft × 10 ft × 8 ft. Both of these elements have a total volume of 1600 ft^3; however, only the foundation would be considered mass concrete due to its greater least dimension. While there is no strict definition of what threshold least dimension value constitutes mass concrete, ACI 301 (2016) recommends that any structural concrete member with a least dimension of 4 ft or greater be considered mass concrete. Most states and concrete organizations designate a size threshold of 4 or 5 feet, as will be discussed in Chapter 3.

The heat of hydration associated with the larger volume of interior concrete is the primary concern when dealing with massive elements. Entrapped heat leads to two major thermal distresses within the element. First, the build-up of heat causes an element's core temperature to rise dramatically at early ages. Research has shown that DEF may occur in moist concrete that at early ages experienced temperatures greater than or equal to 158°F (Taylor et al. 2001). DEF is an internal sulfate attack that results from high concrete temperatures at early ages, resulting in later expansion within the paste that can lead to cracking and premature deterioration of the concrete (Taylor et al. 2001).

Second, due to the faster rate of heat dissipation at the outer edge of the element, a temperature difference is created between the hotter core and the cooler edge. This temperature difference leads to stresses across the cross section, which may lead to thermal cracking. A temperature difference above 35°F is the most widely used temperature difference limit before thermal cracking becomes a potential risk (Bamforth 2007; ACI 301 2016). Figure 1-1 is an example of thermal cracking in a Texas highway bridge column.

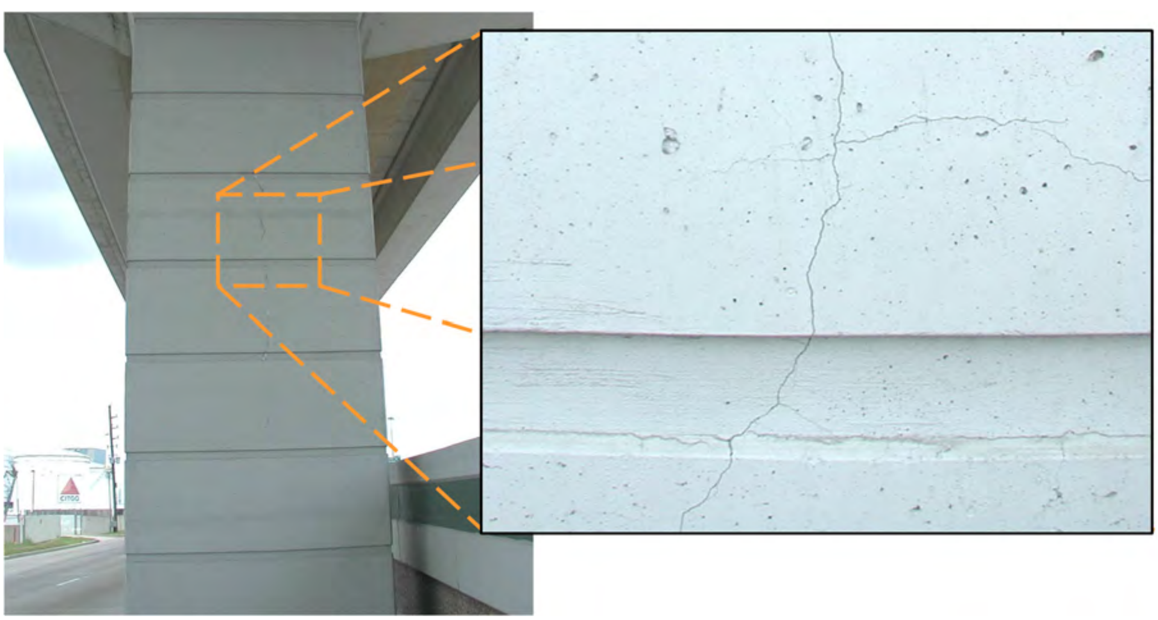


Figure 1-1: Thermal cracking of a bridge column in Texas (Photo Courtesy of Dr. J.C. Liu)

To mitigate these types of distresses, mass concrete construction specifications are being developed by concrete organizations and state DOTs. These specifications include recommendations for good construction practices, temperature control strategies, and concrete temperature limits. They are designed to be easy to understand and implement in the field, all with the goal of minimizing the risks of cracking and occurrence of DEF. The current Alabama Department of Transportation (ALDOT) Standard Specifications for Highway Construction do not contain a mass concrete specification. The Auburn University Highway Research Center has been tasked with developing a specification for ALDOT mass concrete construction.

1.2 PROJECT OBJECTIVES

The primary objective of this project is to develop an ALDOT mass concrete construction specification. This specification should include recommendations for concrete temperature prediction, mass concrete size designation, materials requirements, temperature limits, temperature monitoring, and requirements for a thermal control plan. In order to accomplish these objectives, the following secondary objectives were pursued:

- Evaluate the state of the art in mass concrete specifications implemented by agencies across the United States,
- Measure in-place temperatures of typical ALDOT mass concrete members during construction and survey those members for signs of thermal cracking and DEF,
- Determine the accuracy of ConcreteWorks' temperature and cracking risk predictions,
- Quantify the effects of supplementary cementing materials (SCMs) on thermal cracking and DEF in mass concrete members,
- Quantify the effect of different coarse aggregates on the probability of thermal cracking in mass concrete placements,
- Develop a representative approach to determine the temperature difference to minimize the risk of early-age thermal cracking,
- Develop a method for ALDOT to designate members as mass concrete members,
- Use ConcreteWorks and field temperature measurements to develop temperature control requirements for ALDOT mass concrete construction, and
- Provide recommendations for the assembly and placement of temperature sensors in mass concrete members.

1.3 RESEARCH APPROACH

The following tasks were performed to accomplish the research objectives of this project:

- Task 1 consisted of a literature review of current mass concrete specifications in each state.
- Task 2 consisted of measuring the in-place concrete temperatures of seven mass concrete elements from ALDOT bridge projects.
- Task 3 used ConcreteWorks to model each ALDOT field element to assess the accuracy of ConcreteWorks' concrete temperature predictions.
- Task 4 used an established method to determine the impact of long-term temperature differences on the performance of mass concrete.
- Task 5 involved the development of an ALDOT mass concrete specification.

1.4 REPORT ORGANIZATION

Chapter 2 contains a literature review with information pertinent to mass concrete, including analysis, construction, and associated distresses. Chapter 3 summarizes the mass concrete specifications currently in use by ACI, the Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), and state DOTs around the country. Chapter 4 details the experimental plan implemented in this project. Chapter 5 summarizes the results from the field instrumentation data. Chapter 6 discusses the results of the statistical ConcreteWorks analysis and analysis of ALDOT mass concrete field elements. Chapter 7 discusses the project results that contributed to the development of an ALDOT mass concrete specification. Chapter 8 contains the project summary, conclusions, and recommendations for further research.

Appendix A contains a complete draft of an ALDOT mass concrete specification. Appendix B through Appendix H contain additional information and data from the seven ALDOT mass concrete elements instrumented during Task 3. Appendix I contains output data from the ConcreteWorks analysis in Chapter 6. Appendix J contains example calculations of a thermal and shrinkage cracking analysis performed on each of the seven ALDOT elements

Chapter 2

Literature Review

2.1 INTRODUCTION

The two unique distresses that originate in mass concrete elements are thermal cracking and DEF. The causes, mechanisms, mitigation practices, and examples of thermal cracking and DEF are covered in Sections 2.2 and 2.3, respectively. Section 2.4 contains the current requirements for reinforcing steel to control cracking in select specifications. Section 2.5 contains some common temperature control strategies, and Section 2.6 contains an introduction to ConcreteWorks, the software used to predict temperatures and early-age cracking risks for various mass concrete elements.

2.2 THERMAL CRACKING

2.2.1 Thermal Stress Development

When cement hydrates, heat is released, and high temperatures can develop at the core of a mass concrete member because heat becomes trapped due to the thickness of the element. In contrast, concrete hydrating close to the edge of the element can dissipate heat quickly because of its proximity to the outside environment. This non-uniform dissipation of heat creates differential temperature changes across the cross section, causing differential expansion and contraction within an element. This leads to the build-up of tensile stresses within an element due to “internal restraint” of movement (Bamforth 2007). When contraction of the element upon cooling is restricted by external boundary conditions, this is referred to as “external restraint” (Bamforth 2007). These two types of restraint lead to the development of stresses within the concrete due to temperature changes. Depending on the dimensions of the element, differential thermal effects will usually dissipate within 10 to 20 days, at which time most of the concrete has equilibrated with the ambient temperature (Base and Murray 1978). Therefore, the effects of temperature differences will be most prevalent at early ages, but this does not eliminate the need to design mass concrete elements for long-term temperature effects as well.

The rate of cement hydration is initially greater than the rate of heat loss to the environment, causing a rise in temperature and expansion of the concrete, which introduces compressive stresses, primarily in the core where the rate of heat loss is the least (Bamforth 2007). As the rate of cement hydration slows down, heat generation progressively gives way to gradual heat loss to the surrounding environment. This cooling gradually produces greater tensile stresses than the initially introduced compressive stresses, first at the edges and more gradually at the core. The lower rate of heat loss at the core and the greater rate of heat loss at the edge results in internal restraint. The resulting stress development causes tensile cracks to form at the concrete surface induced by the temperature difference between the edge and the core. These early-age cracks are often just surface cracks and tend to close as the cooling phase ends (Bernander 1998).

Figure 2-1 demonstrates the concept of surface cracking along with the concept of internal cracking due to heat loss in the core. In Figure 2-1, as the now hardened concrete cools, the large change in temperature at the core can lead to the contraction of core concrete and the formation of tensile stresses. Also at early-ages an elevated temperatures phenomena like creep occur. This relieves some of the tensile stresses at the edge, lessening surface crack widths. Simultaneously, the core concrete can finish cooling at a lower temperature than the edge, resulting in a stress reversal. This contributes to the reduction of surface crack widths and the potential for internal cracking to occur (Bamforth 2007).

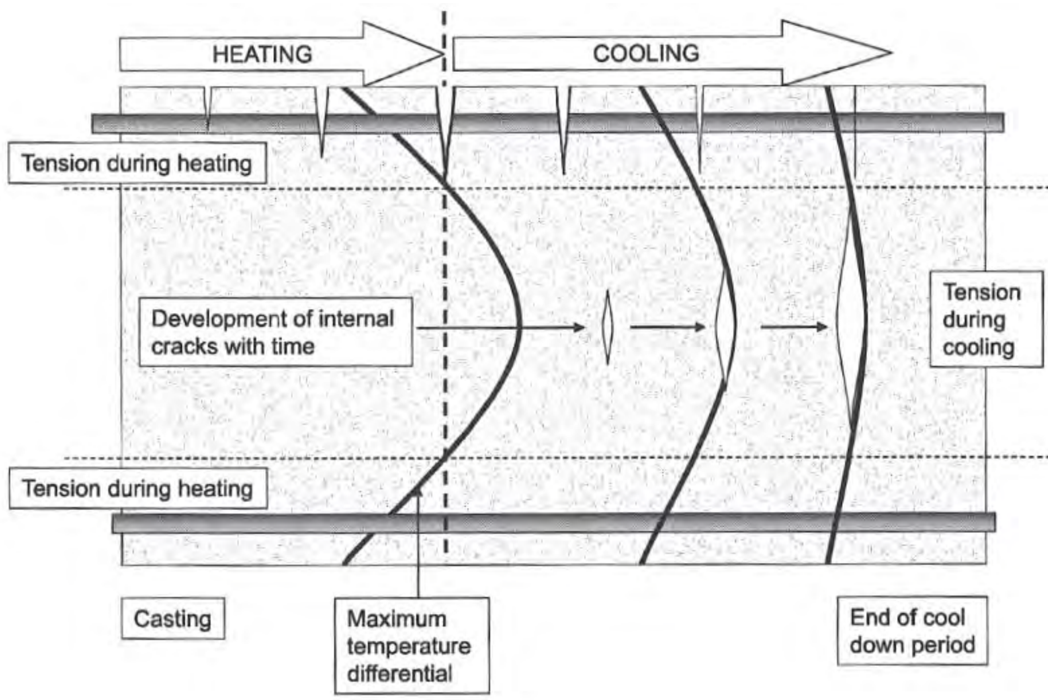


Figure 2-1: Development of cracks in massive concrete members subject to only internal restraint caused by temperature differences (Bamforth 2007)

The larger the mass concrete element, the greater the temperature difference will be, and thus the greater the cracking risk. The largest temperature difference in a cross-section is typically between the corner and core of a rectangular element's cross section, but the corner is typically an area of low internal restraint (Tankasala 2017). Cracking due to internal restraint will typically occur along the face of the member where restraint is highest, as illustrated by the red zones in Figure 2-2.

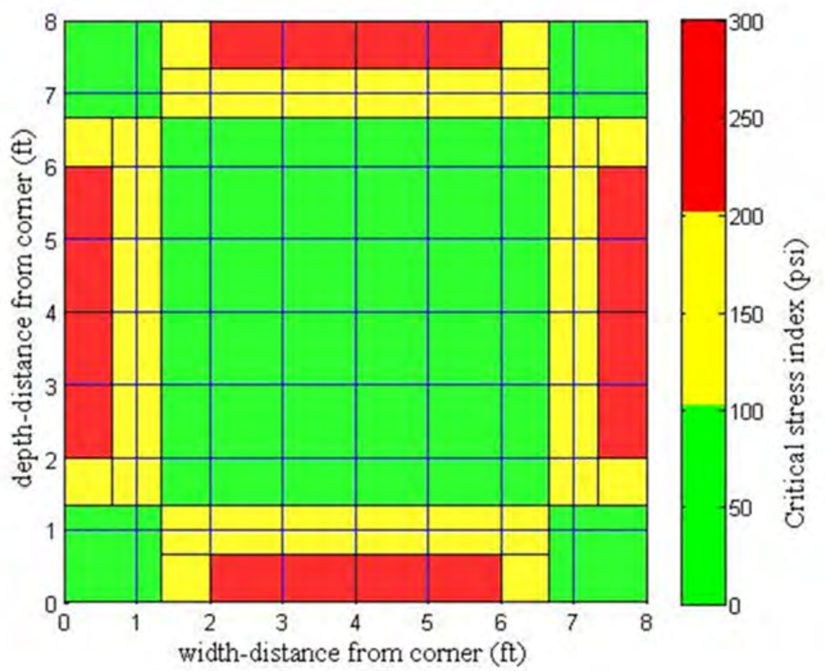


Figure 2-2: Most likely locations (shown in red) to develop thermal cracks due to the effect of internal restraint (Tankasala et al. 2017)

External restraint occurs when the concrete member's volume change is resisted externally at the element's boundary (Rostasy et al. 1998). Common types of external restraint are end restraint and continuous restraint. End restraint refers to deformation in an element being impeded by an adjacent member, such as at the end or inner supports of a beam. Continuous restraint results from one element being cast on top of subsoil, rock, or another element. The most common type of this restraint is a wall being cast on top of a foundation, resulting in cracks along the wall (Rostasy et al. 1998). Figure 2-3 below depicts the situation in which the contraction of a concrete wall is continuously restrained by its previously cast footing.

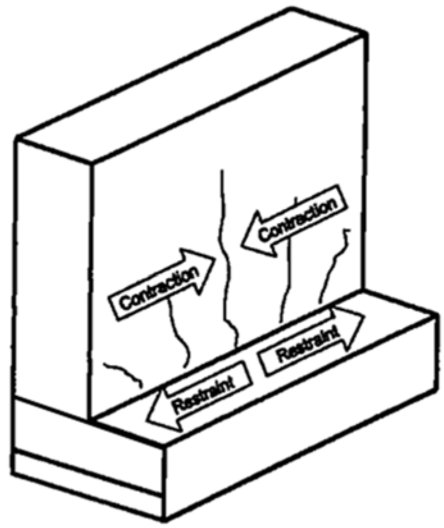


Figure 2-3: Restraint of a concrete member by an adjacent element (adapted from Bamforth 2007)

Thermal stresses do not begin to develop until the concrete reaches final set. Figure 2-4 demonstrates the concept of final set and a zero-stress temperature as illustrated by Schindler and McCullough (2002). When concrete is first placed, no stresses develop in the concrete due to the concrete being in a fluid state. The final-set temperature is the temperature at which the concrete is restrained against drying shrinkage and temperature changes for the first time (Schindler and McCullough 2002). This is also when the concrete can begin to develop strength. After this point, the core temperature of the concrete continues to rise due to continued cement hydration, causing thermal expansion. This expansion induces a compressive stress on the newly restrained concrete. As the concrete begins to cool and contract it reaches a point of zero stress (point B), after which the concrete experiences tensile stresses for the first time. Further cooling and temperature differences within the element can lead to increased tensile stresses. When tensile stress exceeds tensile strength at any location in the element, thermal cracks begin to develop, as seen in the lower half of Figure 2-4.

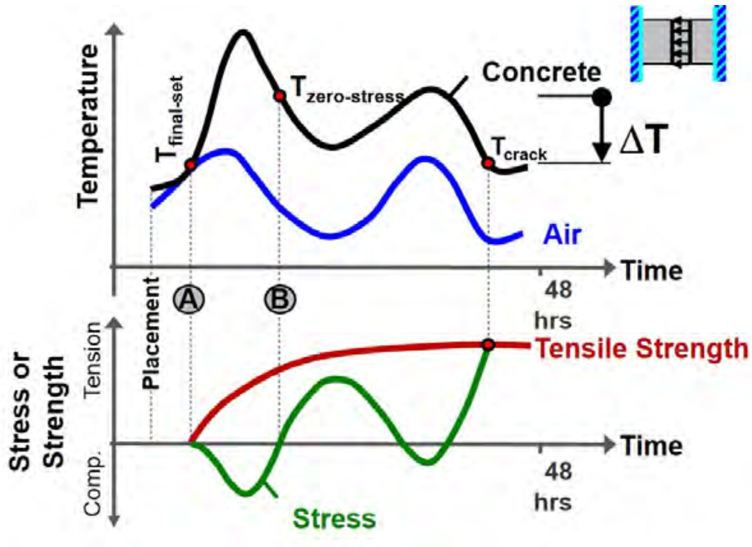


Figure 2-4: Thermal cracking mechanism at placement (Schindler and McCullough 2002)

2.2.2 Equation for Thermal Strain

Bamforth and Price (1995) used an equation for thermal strain to quantify the thermal stress in a concrete member. This equation was developed by Hughes (1971) to simplify the design process by comparing the restrained tensile strain induced during the cooling period from peak to ambient temperature with the tensile strain capacity of the concrete (Bamforth 2007). Equation 2-1 shows this relationship between tensile strain capacity and induced tensile strain due to temperature change.

$$\varepsilon_{tsc} = K \times CTE \times \Delta T_{max} \times R \qquad \textbf{Equation 2-1}$$

Where,

ε_{tsc}	=	tensile strain capacity ($\mu\varepsilon$),
K	=	creep modification factor (unitless),
CTE	=	concrete coefficient of thermal expansion (in./in./°F),
ΔT_{max}	=	maximum allowable temperature difference (°F), and
R	=	restraint factor [0 = unrestrained; 1 = full restraint] (unitless).

The creep modification factor, K, adjusts the concrete stiffness to account for the effects of creep (ACI 209 1982). Bamforth (1995) recommended a constant value of 0.8 to account for creep and relaxation in concrete at early ages. In 2007, Bamforth changed the recommended value to 0.65 to account for the effects of creep and sustained loading due to the sustained temperature gradient across a mass concrete element's cross section at early ages.

Next the coefficient of thermal expansion (CTE) of the concrete is based primarily on the type of coarse aggregate used (Browne 1972). A larger concrete CTE translates into a larger volume change whenever a temperature change is induced. Figure 2-5 shows common CTE values for concretes containing typical aggregates. The two coarse aggregates most common to Alabama are limestone (6 $\mu\varepsilon$/°C) and siliceous river gravel (11 $\mu\varepsilon$ /°C).

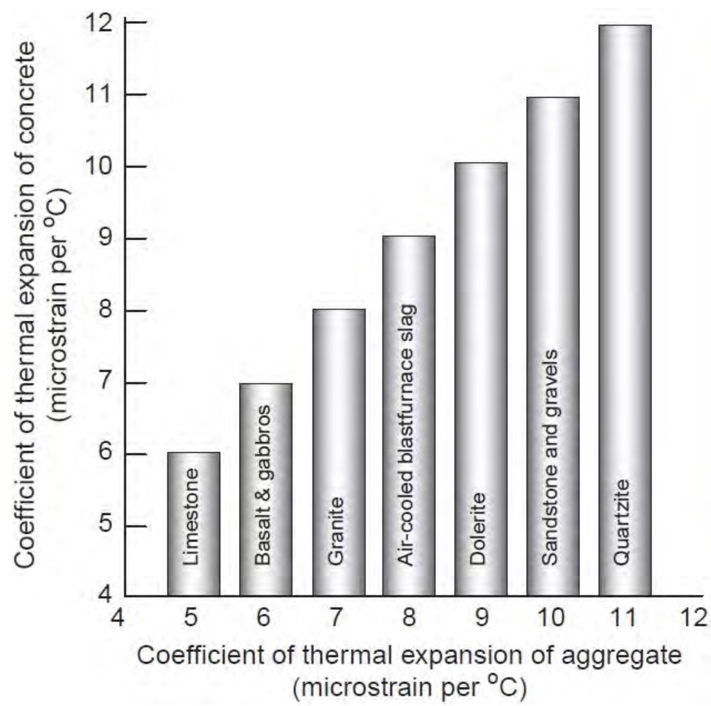


Figure 2-5: Influence of aggregate type on concrete CTE ($\Delta°\mathrm{F} = \mathbf{1.8} \times \Delta°\mathrm{C}$)
(Mehta and Monteiro 2006)

The restraint factor in Equation 2-1, R, is an estimated representation of internal restraint in a concrete member. Bamforth (2007) derives a recommended restraint factor of 0.42 based on limiting temperature values and property data available. This assumption corresponds to the restraint conditions found in some typical mass concrete elements (ACI 207 2007), but internal restraint can be a very difficult factor to quantify. This is because the restraint factor is dependent upon member geometry, boundary conditions, and the age of the concrete. Bamforth (2007) provides guidelines for determining an approximate restraint factor based on these conditions.

The temperature difference, ΔT_{max}, is the maximum allowable temperature difference between the core and any location in the element (see Section 2.2.5). ACI 207 (2007) defines temperature difference as "the cooling of the surface concrete relative to the more stable internal temperature." As previously stated, mass concrete members can experience large temperature differences. A greater allowable temperature difference depends on the variables listed above in Equation 2-1. Table 2-1 from Bamforth and Price (2007) provides some temperature difference limits at early ages when using various coarse aggregate types and different restraint factors.

Table 2-1: Temperature difference limits (°C) based on assumed typical values of CTE and tensile strain capacity (Bamforth and Price 2007) ($\Delta°\mathrm{F} = \mathbf{1.8} \times \Delta℃$)

Aggregate Type	**Gravel**	**Granite**	**Limestone**	**Lightweight**
Coefficient of Thermal Expansion ($\mu\varepsilon$/°C)	13.0	10.0	9.0	9.0
Tensile Strain Capacity ($\mu\varepsilon$)	65	75	85	115
Limiting temperature change for different restraint factors, R (°C)				
R = 1.00	6	9	12	20
R = 0.80	8	12	16	25
R = 0.60	11	17	22	34
R = 0.42	**20**	28	35	53
R = 0.30	24	36	46	71

The temperature limit of 20°C (36°F) is significant because this value is widely used and became the allowable temperature difference recommended by ACI Committee 301 (2016) for all mass concrete construction. Note that this limit, as proposed by Bamforth and Price (2007), is the limiting temperature difference for concrete using gravel aggregate (higher CTE) at the internal restraint factor of 0.42. This recommended limit of 36°F (35°F in some cases) has been adopted by many states as their maximum allowable temperature difference for all mass concrete construction, regardless of aggregate type (see Figure 3-2). However, based on the work done by Bamforth, higher temperature difference limits can be allowed when using aggregates such as granite [28°C (50°F)] and limestone [35°C (63°F)] because of their lower CTEs and higher tensile strain capacity.

2.2.3 Modeling Early-Age Thermal Stresses in Concrete Elements

Bamforth's above work related to thermal cracking assumed a constant creep modification factor of 0.65. However, because thermal cracking effects are highly dependent upon time at early ages (Bazant and Baweja 2000), an investigation into the benefits of a time-dependent creep modification factor is warranted. Bazant and Baweja (2000) developed a model for predicting creep, shrinkage, and temperature effects in concrete structures known as the B3 Model, and it will be reviewed herein.

2.2.3.1 Predicting Early-Age Strength & Stiffness of Concrete Using the Bazant-Baweja B3 Model

To quantify creep in a concrete element, the strength and stiffness of the concrete must first be determined. Equation 2-2 from the B3 model is used to calculate compressive strength at any time.

$$f_{cmt} = \left(\frac{t}{a + bt}\right) f_{cm28} \qquad \textbf{Equation 2-2}$$

Where,

f_{cmt}	=	concrete mean compressive strength at any time t (psi),
t	=	age of the concrete (days),
a	=	4.0 for moist-cured concrete with Type I cement (days),
b	=	0.85 for moist-cured concrete with Type I cement (unitless), and
f_{cm28}	=	concrete mean compressive strength at 28 days (psi).

Equation 2-3 is an alternative method for calculating concrete tensile strength from Raphael (1984) that was verified through splitting tensile applications and use of the concrete compressive strength. This equation was selected because it was developed based on mass concrete, and the splitting tensile testing represents the primary mechanism of thermal cracking in massive concrete elements (Raphael 1984).

$$f_t = 1.7 f_c^{2/3} \qquad \textbf{Equation 2-3}$$

Where,

f_t	=	concrete tensile strength (psi), and
f_c	=	concrete compressive strength (psi).

The stiffness of concrete, E_c, is also calculated as a function of compressive strength as seen in Equation 2-4 (ACI 318 2014):

$$E_C = 33(w_c)^{1.5}\sqrt{f_c} \qquad \textbf{Equation 2-4}$$

Where,

E_c	=	concrete modulus of elasticity (psi),
w_c	=	concrete unit weight (pcf), and
f_c	=	concrete compressive strength (psi).

Using normal weight concrete, Equation 2-4 becomes

$$E_C = 57000\sqrt{f_c} \qquad \textbf{Equation 2-5}$$

Where,

E_C	=	concrete modulus of elasticity (psi), and
f_c	=	concrete compressive strength (psi).

The parameters for the B3 Model are summarized in ACI 209.2R (2008). Default values for parameters may be used if specific data are unavailable at the time of design (ACI 209 2008). Equation 2-6 is the mean 28-day compressive strength used when only the design strength, fc, is known.

$$f_{cm28} = f_c + 1200$$ **Equation 2-6**

Where,

f_{cm28} = concrete mean compressive strength at 28 days (psi), and

f_c = concrete design compressive strength (psi).

A creep coefficient, $\varphi(t, t_o)$, is used in the B3 Model to represent the effects of creep as seen in Equation 2-7:

$$\varphi(t, t_o) = E(t_o)J(t, t_o) - 1$$ **Equation 2-7**

Where,

$\varphi(t, t_o)$ = creep coefficient,

$J(t, t_o)$ = average compliance function,

$E(t_o)$ = static modulus of elasticity at the age of concrete loading,

t = age of concrete, and

t_o = age of concrete loading.

The compliance function, $J(t, t_o)$, considers instantaneous strain due to unit stress, basic creep, and drying creep in Equation 2-8:

$$J(t, t_o) = q_1 + C_o(t, t_o) + C_d(t, t_o, t_c)$$ **Equation 2-8**

Where,

q_1 = instantaneous strain due to unit stress,

$C_o(t, t_o)$ = compliance function for basic creep,

$C_d(t, t_o, t_c)$ = compliance function for drying creep, and

t_c = age drying began (end of moist curing).

In Equation 2-9 the creep coefficient can then be converted into a creep modification factor, allowing Bamforth's thermal cracking equation (Equation 2-1) to be modeled as a time dependent quantity:

$$K(t) = \frac{1}{1 + \varphi(t, t_o)}$$ **Equation 2-9**

Where,

$\varphi(t, t_o)$ = creep coefficient, and

$K(t)$ = creep modification factor.

2.2.3.2 The Modified B3 Model

The compliance function from the B3 Model underestimates the effects of creep and relaxation in concrete at early ages (Byard and Schindler 2015). It also does not account for the development of the modulus of elasticity with age. Byard and Schindler (2015) developed a modified B3 Model which incorporates two modifications into the existing B3 Model's compliance function to improve its accuracy at early ages.

2.2.4 Effects of Thermal Cracking

Thermal cracking impacts performance as well as durability. The width of thermal cracks varies depending on the severity of the thermal gradient. Regardless of crack width, surface cracks may lead to deleterious effects on durability especially when in combination with shrinkage, ambient temperature cycles, and aggressive environments (Bernander 1998). Figure 2-6 shows surface cracking due to thermal stresses. Cracking of this kind exposes the embedded reinforcement, which leads primarily to corrosion, which will worsen the cracks (Emmons 1993).

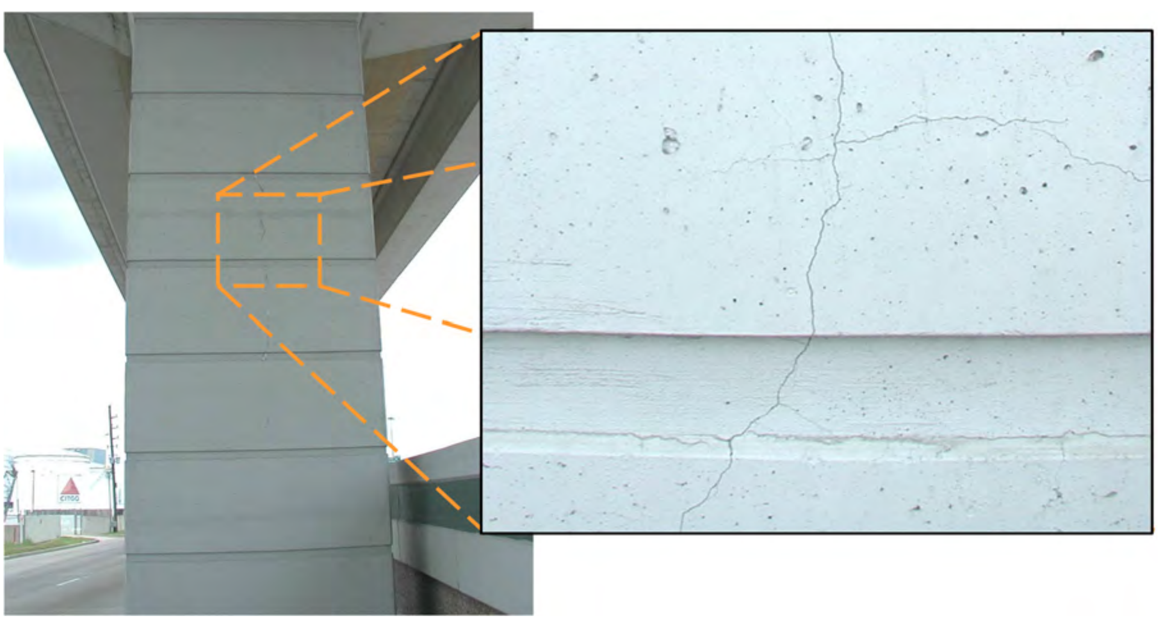


Figure 2-6: Thermal cracking in a Texas bridge element (Photo courtesy of Dr. J.C. Liu)

2.2.5 Limiting Temperature Differences

ACI 207 (2007) describes temperature difference as “the cooling of the surface concrete relative to the more stable internal temperature.” If the surface cools too rapidly, tensile stresses will form, which leads to thermal cracking. To avoid these tensile stresses, a maximum allowable temperature difference can be established during construction. This temperature difference limit will help ensure the concrete element cools at a more uniform rate by ensuring the temperature difference at any two points within the element does not induce a tensile stress greater than the strength of the concrete. Several state DOTs have already implemented maximum temperature difference limits and thermal control plans to control the cooling of mass concrete (see Chapter 3). Table 2-1 from Bamforth contains some values for a maximum allowable temperature difference based on certain assumed parameters. The most commonly adopted value is 20°C (35°F), as will be discussed in Chapter 3. See Section 2.5.1 and Table 2-8 for some recommendations on the control of the temperature difference.

2.3 DELAYED ETTRINGITE FORMATION

2.3.1 Background

DEF is an internal sulfate attack that causes expansion in the hydrated cement paste due to elevated concrete temperatures at early-age and the exposure of concrete to moisture (Pavoine et al. 2006). Late last century, some concrete pavements and precast members had begun showing signs of premature degradation, but no cause had been determined. This sparked an investigation into the cause of these crack formations (PCA 2001). The chemical composition of the mixture components was examined. The effects of high temperatures and the presence of moisture were also examined. However, little was learned about DEF, and the phenomenon continued to be a topic of discussion for many years. In 2001, Taylor, Famy, and Scrivener (2001) proposed a now widely accepted theory for the formation of DEF in ordinary portland cement (OPC) concrete. This theory suggests that chemistry of concrete materials, concrete paste microstructure, and curing temperature are the three main factors upon which DEF-induced expansion is dependent (Taylor et al. 2001).

2.3.2 Causes of DEF

Delayed ettringite formation occurs when high concrete temperatures affect the chemical reactions that take place during the hydration period (Folliard et al. 2006). In addition to these high concrete temperatures, the concrete material must then be wet or moist, intermittently or permanently, for the damaging effects to occur (Taylor et al. 2001). Ettringite ($C_3A \cdot 3CaSO_4 \cdot 32H_2O$) is a crystalline substance created during the hydration process by the reaction of calcium aluminates (C_3A and C_4AF) with gypsum (CSH_2) (Folliard et al. 2006). At temperatures over 158ºF in plain portland cement concrete systems, the chemistry allowing ettringite to form is affected, and the chemical compounds are trapped within the concrete paste (Taylor et al. 2001). After the curing cycle is complete, the concrete's exposure to moisture allows ettringite to reform in the hardened concrete.

The widely accepted maximum concrete temperature in plain portland cement concrete is 158°F (Taylor et al. 2001). If concrete exceeds this temperature during early age, the presence of DEF is more likely. However, this maximum temperature limit is not absolute. This threshold can vary based on certain conditions that affect the maximum core temperature and the formation of DEF. For example, the amount of sulfate present in the concrete affects the growth and expansion of ettringite (Pavoine et al. 2006). Table 2-3 summarizes the work done by Folliard et al. (2008). This work was adopted as limits by ACI 201.2R (2016) to increase the maximum concrete temperature limit based on the use of SCMs.

The constant presence of excess moisture is essential in the formation of DEF. For DEF to occur, the material must be wet or moist (Taylor et al. 2001). Scherer (1999) concluded that the "driving force (of ettringite) is provided by the supersaturation, and the hydrostatic pressure that is needed to stop growth increases with the degree of supersaturation." According to current understanding, the presence of moisture provides a pore solution for ettringite components, in which ettringite can form using sulfates within the cement composition (Stark and Bollmann 2000). This leads to the cracking and degradation seen from DEF.

In addition to these two criteria necessary for the presence of DEF in mass concrete, there are other less influential factors that contribute to the development of DEF as well. Pavoine, Divet, and Fenouillet (2006) concluded that the following parameters also contribute to the occurrence of DEF:

- Alkali levels in the concrete,
- Initial cracking of the concrete,
- Sulfates present in the clinker,
- Sulfates present in the cement, and
- Chemical admixtures.

2.3.3 Effects of DEF

The expansion resulting from DEF formation, much like with alkali-silica reaction (ASR), results in cracking (Folliard et al. 2006). The proposed cracking mechanism due to ettringite formation is illustrated below in Figure 2-7.

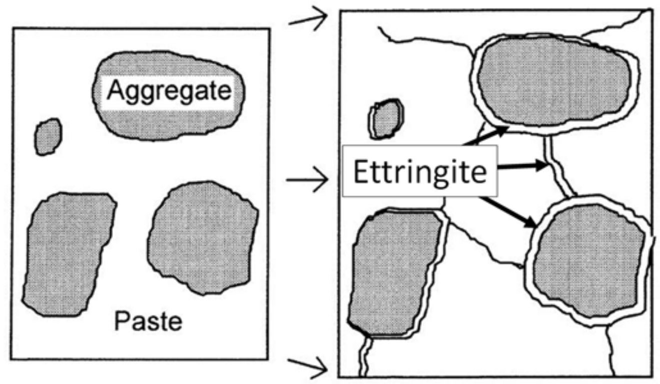


Figure 2-7: Proposed DEF expansion mechanism (Taylor et al. 2001)

This figure suggests that uneven paste expansion causes cracking at the paste-aggregate interface. Ettringite then forms and fills these cracks. If concrete subject to DEF is then reheated, ettringite expands further and induces internal stresses, further cracking the concrete (Taylor et al. 2001).

Structural damage in mass concrete due to DEF has been reported in the United States on limited occasions. In 2008 a seal foundation in Georgia was found to have cracks up to 6 in. wide filled with ettringite. Microscopic analysis of core samples taken concluded that DEF was the primary cause of structural damage (McCall 2013). Conclusions in a Georgia Institute of Technology report estimated a maximum concrete temperature of 200°F and a maximum temperature difference of 111°F (Kurtis et al. 2012). Figure 2-8 depicts the measured cracking pattern in this seal footing (McCall 2013).

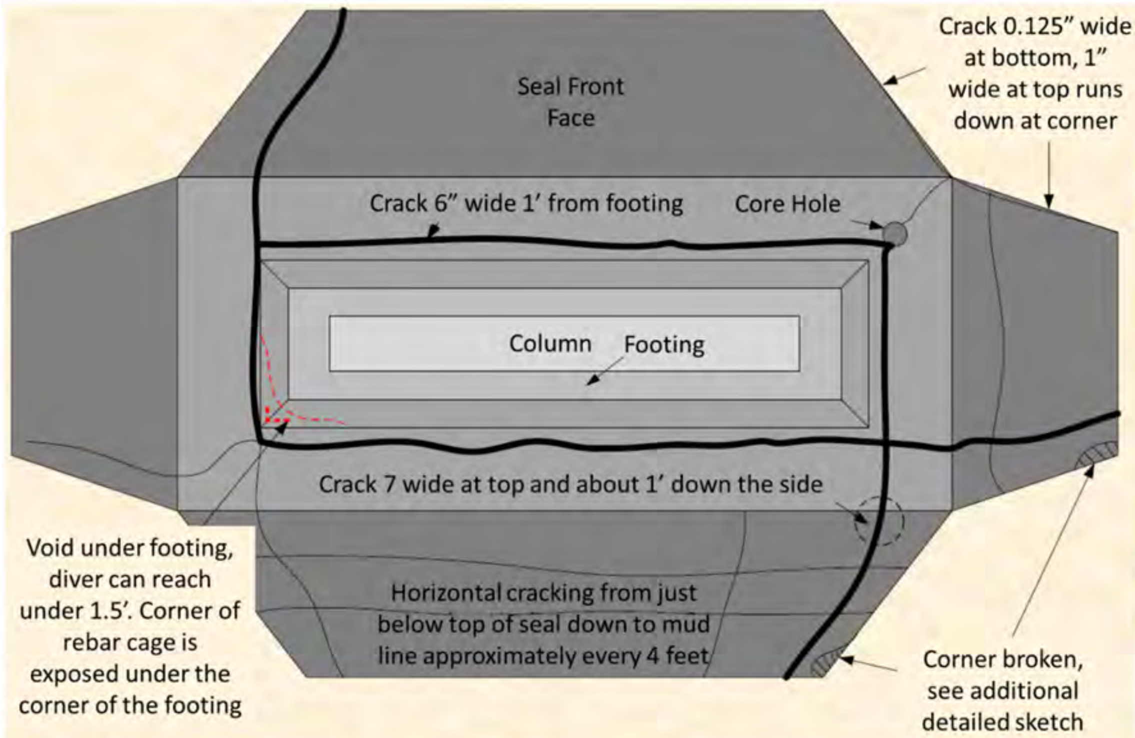


Figure 2-8: Plan view of seal footing with cracks due to DEF (McCall 2013)

DEF was also determined to be the primary cause of cracking in at least one column on the San Antonio Y Overpass in San Antonio, TX (Thomas et al. 2008). Researchers discovered that Type III cement was used in the concrete mixture for these columns (Thomas et al. 2008). Type III cement produces more heat during hydration, contributing to the potential presence of DEF. Figure 2-9 below shows the most severe cracking found at that site.

A group of researchers later examined the same bridge to develop a method for diagnosing DEF in concrete elements. Concrete cores were taken in order to examine samples with a scanning electron microscope. Back scattered electron images of these samples showed significant cracking at the aggregate-paste interface, allowing ettringite to form in these cracks, as seen in Figure 2-10.

Figure 2-9: DEF induced cracking in San Antonio Y overpass (Thomas et al. 2008)

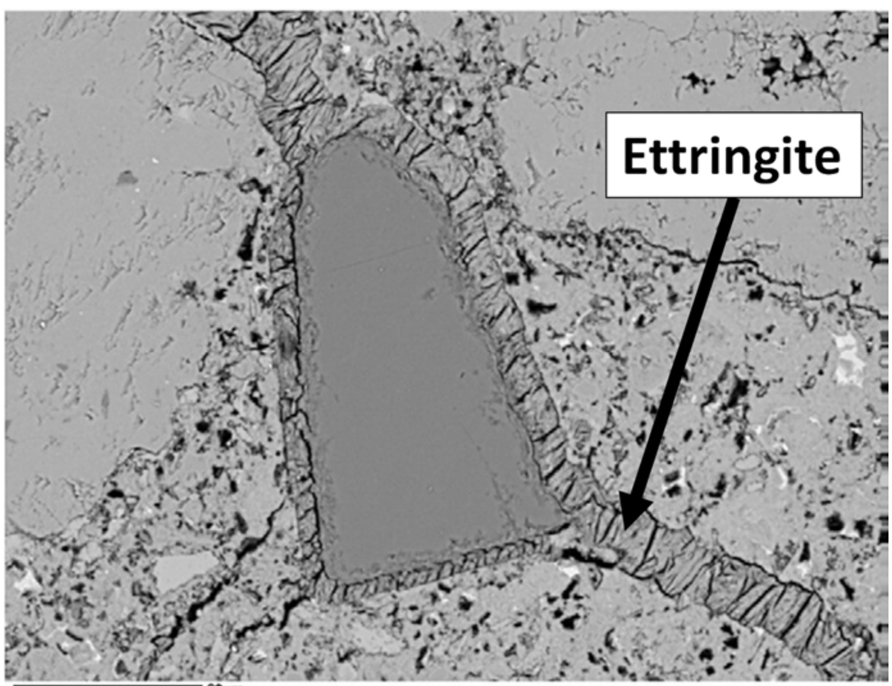


Figure 2-10: Ettringite filled cracks around coarse aggregate removed from San Antonio Y overpass (adapted from Thomas et al. 2008)

2.3.4 Limiting the Maximum Concrete Temperature

As stated in Section 2.3.2, the most widely accepted maximum concrete temperature limit is 158°F (70°C) (Folliard et al. 2006). This limit became widely accepted after the work done by Taylor, Famy, and Scrivener (2001). However, the temperature limit at which DEF forms may be altered by the use of SCMs (ACI 201.2R 2016). Maximum temperature control strategies implemented during construction to meet the maximum concrete temperature limit are discussed in Section 2.5.2.

2.3.4.1 Use of SCMs

The use of supplementary cementing materials (SCMs) in concrete can potentially increase the maximum concrete temperature limit (Folliard et al. 2006). The use of fly ash (FA) or slag cement (SC) reduces the amount of sulfate in the concrete (Thomas et al. 2008). Since using SCMs provides a different chemical composition than solely using portland cement, using SCMs can also provide a concrete mixture less susceptible to DEF (Myuran et al. 2015).

Research to assess the validity of the 158°F limit and potentially develop a higher temperature limit by using SCMs was performed by TxDOT. This study determined that the following five SCMs are effective at mitigating DEF-induced expansion at temperatures above 158°F when used in sufficient quantities: Class F fly ash, Class C fly ash, slag cement, metakaolin, and ultra-fine fly ash. These claims are supported by expansion test results shown in Figure 2-11.

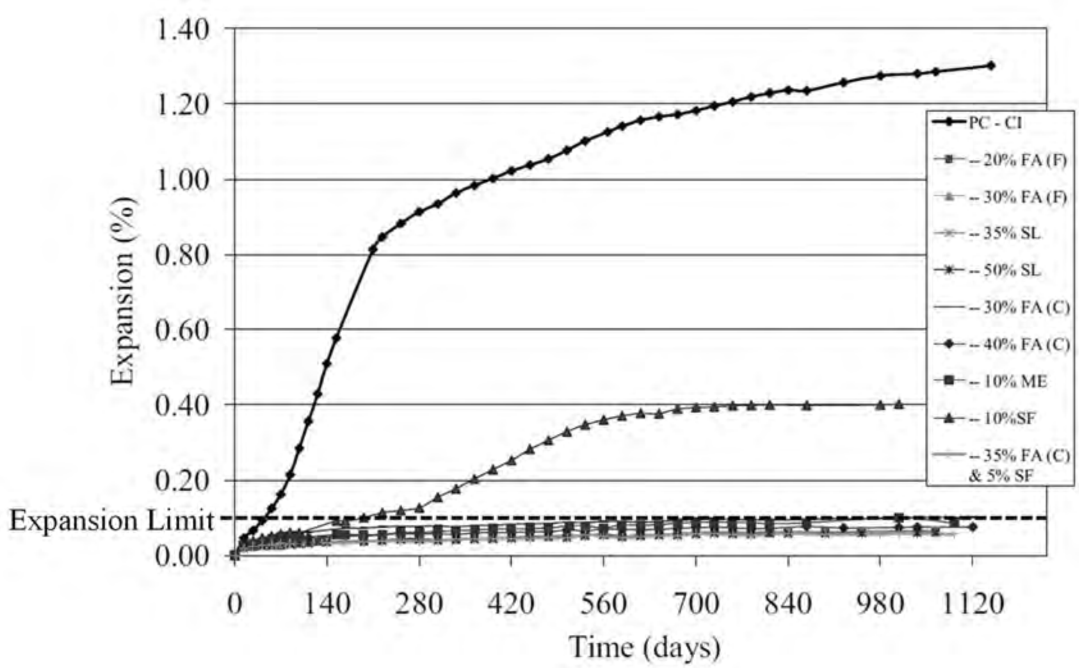


Figure 2-11: Expansion test results for mortar bars with plain portland cement in combination with SCMs cured at 203°F (95°C) (adapted from Folliard et al. 2006)

The two mortar bars to show signs of expansion above the expansion limit of 0.10% contained either only plain portland cement or a 10% silica fume replacement. Silica fume not used in combination with other SCMs is not effective at mitigating the effects of DEF (Folliard et al. 2008). Both Class C and Class F fly ash can reduce DEF induced expansion, but Class F tends to be the less expansive of the two (Folliard et al. 2006). Figure 2-12 compares the 14-day expansion tests of a 20% Class F fly ash replacement with 35% and 40% Class C fly ash replacements.

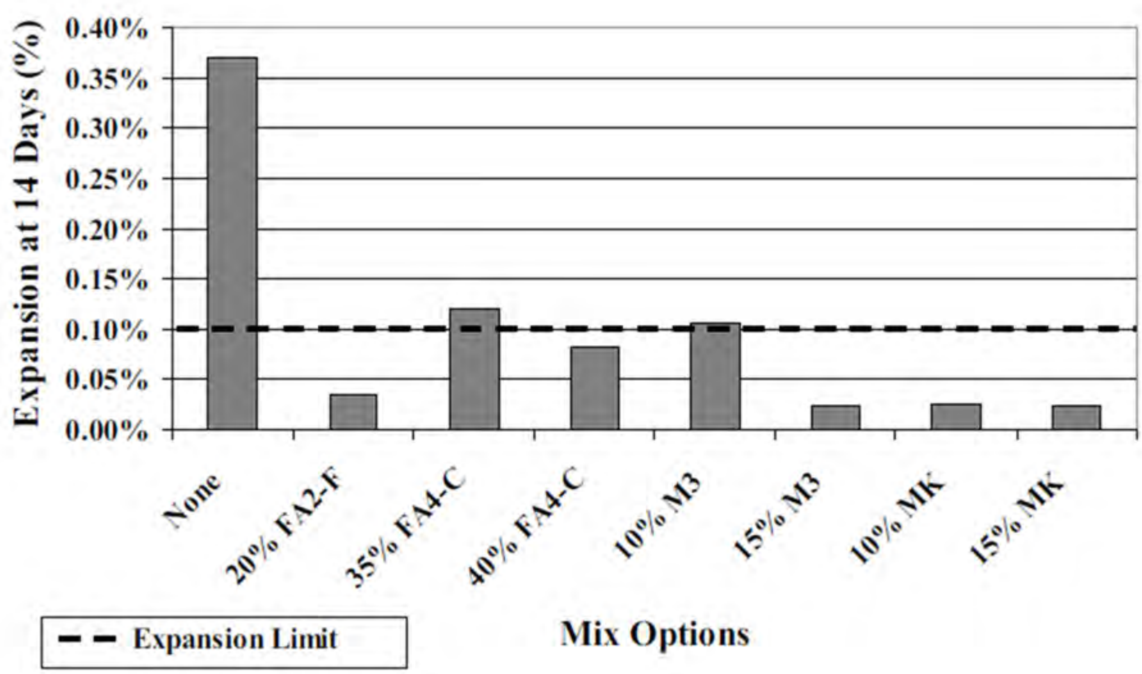


Figure 2-12: Comparative expansion of Class F fly ash at higher replacement levels (Folliard et al. 2006)

The TxDOT study concluded with Folliard et al. (2006) recommending a new set of temperature limits in 2006. Table 2-2 summarizes these new temperature limits; however, the types of and replacement quantities of SCMs were not specified.

Table 2-2: Temperature limit recommendations for TxDOT (Folliard et al. 2006)

Concrete Element Type	Maximum Temperature Limit
Precast girders with plain concrete (No SCMs)	150°F
Precast girders with concrete containing SCMs	170°F
Mass concrete	160°F

It is unknown why the 170°F limit shown in Table 2-2 does not apply to the mass concrete placements that contain SCMs. Many mass concrete specifications require low heat cements and SCMs which help reduce the risk of DEF (see Table 3-7). Based on this original work by Folliard et al. (2006), ACI 201.2R (2016) adopted the temperature limits in Table 2-3.

Table 2-3: Recommended measures for reducing the potential for DEF in concrete exposed to elevated temperatures at early ages (adopted from ACI 201.2R, 2016)

Maximum Concrete Temperature, T_c	Prevention Required
$T_c \leq 158°F$	No prevention required
$158°F < T_c \leq 185°F$	Use one of the following approaches to minimize the risk of expansion: 1. Portland cement meeting requirements of ASTM C150 moderate or high sulfate-resisting and low-alkali cement with a fineness value less than or equal to 430 m^2/kg 2. Portland cement with a 1-day mortar strength (ASTM C109) less than or equal to 2850 psi 3. Any ASTM C150 portland cement in combination with the following proportions of pozzolan or slag cement: a) Greater than or equal to 25 percent fly ash meeting the requirements of ASTM C618 for Class F fly ash b) Greater than or equal to 35 percent fly ash meeting the requirements of ASTM C618 for Class C fly ash c) Greater than or equal to 35 percent slag cement meeting the requirements of ASTM C989 d) Greater than or equal to 5 percent silica fume (meeting ASTM C1240) in combination with at least 25 percent slag cement e) Greater than or equal to 5 percent silica fume (meeting ASTM C1240) in combination with at least 20 percent Class F fly ash f) Greater than or equal to 10 percent metakaolin meeting ASTM C618 4. An ASTM C595/C595M or ASTM C1157/C1157M blended hydraulic cement with the same pozzolan or slag cement content in Item 3
$T_c > 185°F$	The internal concrete temperature should not exceed 185°F (85°C) under any circumstances.

In addition to raising the maximum concrete temperature limit, the use of SCMs may also be part of an effective temperature control strategy. The maximum in-place concrete temperature may be lower due to SCM replacement at certain levels. The adiabatic temperature rise of fly ash and slag cement replacements compared to a purely Type I cement mixture are demonstrated in Figure 2-13.

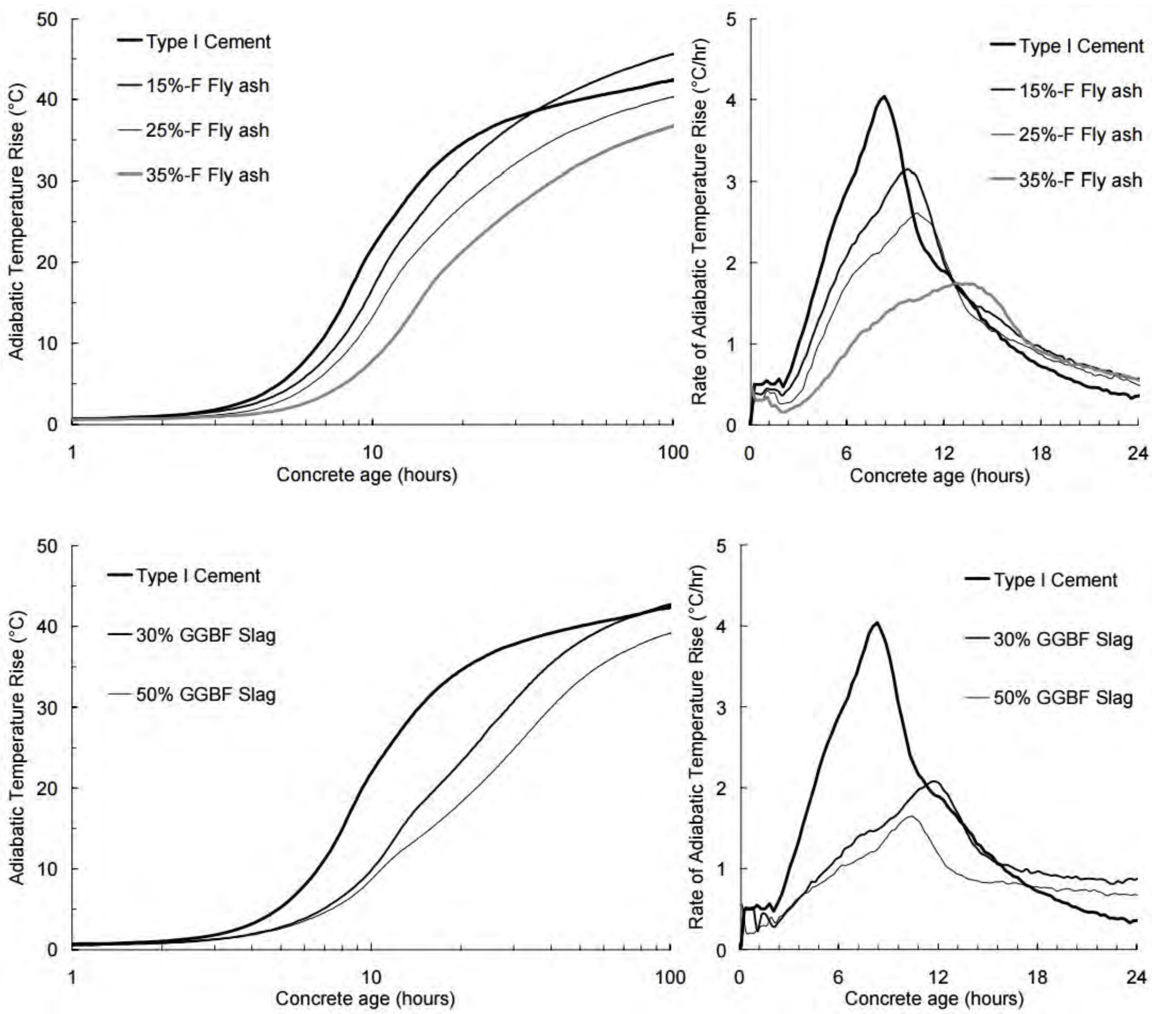


Figure 2-13: Effect of fly ash and slag cement replacement on heat of hydration (Schindler and Folliard 2005) (°F = 1.8 x °C + 32)

2.4 MINIMUM STEEL REINFORCEMENT REQUIREMENTS TO CONTROL CRACKING

The focus of this requirement review was directed at establishing the minimum amount of reinforcement required to control cracking due to thermal and shrinkage effects in mass concrete elements. Steel reinforcement is provided to control crack widths and withstand tensile stresses in concrete members when cracking occurs (Gilbert 1992). Service loads are an expected cause of cracking, but cracking may occur due to numerous other causes including shrinkage and temperature effects. Thermal cracking is addressed in Section 2.2. Shrinkage effects lead to direct tension cracking in restrained elements by causing axial tension throughout an element (Base and

Murray 1979). To avoid excessively wide cracks, adequate amounts of well-anchored reinforcement must be provided where significant tension is expected (Gilbert 1992).

Direct tension forces as a result of shrinkage and thermal effects act on the entire cross section to contribute to the formation of through cracks. In direct tension situations, such as a restrained slab, shrinkage will typically cause a few widely spaced cracks that can penetrate the entire member. If insufficient reinforcement is provided in these situations, yielding of the reinforcement will occur resulting in wide, unserviceable cracks (Gilbert 1992). The number of cracks and the final crack widths depend on the length of the member, the quantity of reinforcement provided, the quality of bond between concrete and reinforcement, the amount of shrinkage, and the concrete strength.

Because of the size of mass concrete members, differential shrinkage will cause a maximum buildup of tension forces at the surface and compression at the interior, similar to the effects of temperature differences (Base and Murray 1979). Base and Murray (1978) investigated several examples of restrained reinforced members between 0.5 m and 1.0 m thick (20” to 40”) with similar configurations. They noted that in some cases the concrete contained wide cracks that propagated through the entire member, and in other cases no cracking occurred. The difficulty lies in determining the amount of reinforcement to provide.

ACI 224R (2001) provides a guide to reasonable crack widths based on exposure conditions, as shown in Table 2-4. The required amount of shrinkage and temperature reinforcement provided should be based upon this table. Gilbert (1992) and Bamforth (2007) also suggest a maximum crack width of 0.3 mm (0.012 in.) for members with exposed surfaces, durability, and aesthetic significance.

Table 2-4: ACI 224R guide to tolerable crack widths in RC under service loads

Exposure Condition	Tolerable Crack Width	
Dry air, protective membrane	0.016 in.	0.41 mm
Humidity, moist air, soil	0.012 in.	0.30 mm
De-icing chemicals	0.007 in.	0.18 mm
Seawater and seawater spray; wetting and drying	0.006 in.	0.15 mm
Water-retaining structures	0.004 in.	0.10 mm

2.4.1 Current Shrinkage and Temperature Reinforcement Requirements

The amount of reinforcement required to control cracking depends on the level of crack control required, the restraint, and the total amount of shrinkage (Bamforth 2007). The maximum acceptable crack width depends on the type of structure, the environment, and the consequences of excessive cracking (Gilbert 1992). The current minimum reinforcing steel requirements of ACI

318 (2014), AASHTO (2016), and AS 3600 (2009) (Australian Standard for Concrete Structures 2009) are reviewed and compared in this section. The Australian standard was reviewed because of the higher minimum amount of reinforcement required.

ACI 318 (2014) provides minimum shrinkage and temperature reinforcement ratios for beams, slabs, and walls as shown in Table 2-5. The minimum ratio of 0.18% is empirical but has been used satisfactorily for many years (ACI 318 2014). However, in the case of slabs and elements subject to significant restraint (such as massive elements), it may be necessary to increase the amount of reinforcement required to control cracks due to shrinkage and thermal effects in both principal directions (ACI 318 2014; PCI MNL 120; Gilbert 1992).

Table 2-5: Minimum reinforcement requirements from ACI 318 (2014)

<table>
<tr><td rowspan="5">Beams and slabs (Table 24.4.3.2)</td><th colspan="2">Reinforcement Type</th><th>f_y, psi</th><th colspan="2">Minimum reinforcement ratio, ρ</th></tr>
<tr><td colspan="2">Deformed bars</td><td>< 60000</td><td colspan="2">0.0020</td></tr>
<tr><td colspan="2" rowspan="3">Deformed bars or welded wire reinforcement</td><td>60,000</td><td colspan="2">0.0018</td></tr>
<tr><td rowspan="2">> 60000</td><td rowspan="2">Greater of:</td><td>$(0.0018*60000)/f_y$</td></tr>
<tr><td>0.0014</td></tr>
<tr><td rowspan="4">Cast-in-place Walls (Table 11.6.1)</td><th>Reinforcement Type</th><th>Bar/wire size</th><th>f_y, psi</th><th colspan="2">Minimum transverse reinforcement ratio, ρ_t</th></tr>
<tr><td rowspan="3">Deformed bars</td><td rowspan="2">≤ No. 5</td><td>≥ 60,000</td><td colspan="2">0.0020</td></tr>
<tr><td>< 60,000</td><td colspan="2">0.0025</td></tr>
<tr><td>> No. 5</td><td>Any</td><td colspan="2">0.0025</td></tr>
</table>

Note: f_y = yield stress of steel reinforcement

The AASHTO LRFD Bridge Design Specifications (2016) requirements for shrinkage and temperature reinforcement are shown in Equation 2-10 and Table 2-6. Shrinkage and temperature reinforcement requirements are similar to ACI 318 (2014), but the total required reinforcement area to gross cross-sectional area ratio of 0.18% has been redefined in Equation 2-10 to show that the total required reinforcement, $A_s = 0.0018bh$, is distributed evenly around the perimeter of the element (AASHTO 2016). This method provides a more uniform approach for finding the amount of shrinkage and temperature steel in elements of any size. The coefficient in Equation 2-10 is the product of 0.0018, f_y = 60 ksi, and A_s = 12.0 in./ft, and it has the units of kips/in.-ft. The result of this equation and the reinforcement ratio limit presented in Table 2-6 have units of in.^{2}/ft and define the area of reinforcement in each direction in each face. AASHTO also defines that shrinkage and temperature steel shall be placed near any concrete surface subject to daily temperature changes. Table 2-6 also contains the maximum spacing limits AASHTO required for shrinkage and temperature reinforcement.

$$A_s \geq \frac{1.30bh}{2(b+h)f_y}$$ **Equation 2-10**

Where,

A_s = area of reinforcement per foot, in each direction, and on each face (in^2/ft),

b = least width of element cross section (in.),

h = least thickness of element cross section (in.), and

f_y = specified yield strength of reinforcing bars ≤ 75 ksi.

Table 2-6: Minimum reinforcement requirements from AASHTO LRFD (2016)

<table>
<tr><th>Section</th><th>Requirement</th></tr>
<tr><td rowspan="3">Shrinkage and Temperature Reinforcement (5.10.8)</td><td>0.11 ≤ $\boldsymbol{A_s}$ (in^2/ft) ≤ 0.60</td></tr>
<tr><td>Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete.</td></tr>
<tr><td>Spacing of S&T reinforcement bars shall not exceed:
• 3.0 times the component thickness, or 18.0 in.
• 12.0 in. for walls and footings greater than 18.0 in. thick
• 12.0 in. for other components greater than 36.0 in. thick</td></tr>
</table>

The Australian Standard code for concrete structures (2009) requires reinforcement ratios for controlling crack widths much greater than those required by ACI 318 (2014) or AASHTO (2016). For walls, the AS 3600 minimum reinforcement requirements are summarized in Table 2-7. The minimum amount of reinforcement required by AS 3600 for walls is 0.25% for any sized bar as opposed to the 0.20% required by ACI 318 for any sized bar up to a No. 5. The AS 3600 minimum reinforcement requirements are based on the level of exposure to the environment. Exposure Classes A2, B1 and B2 are most similar to the non-coastal exposure environments found in Alabama. Class A2 is for temperate climates, and Classes B1 and B2 are for near coastal and coastal climates, respectively, such as Mobile, Alabama.

Table 2-7: Minimum transverse reinforcement for walls (Australian Standard 3600, 2009)

	Exposure Class	**Level of Crack Control**	ρ_{min}	**Special Note**
Section 11.7.2	A1 & A2	Minor	0.0025	For walls longer than 8 m, additional horizontal crack control reinforcement may be needed at the base of the wall to control thermal cracking during hydration.
		Moderate	0.0035	
		Strong	0.0060	
	B1, B2, C1, & C2	All	**0.0060**	

2.4.2 Predicting Crack Widths

Bamforth's method of predicting crack widths and reinforcing steel stress in mass concrete elements are discussed in this section. The prediction of crack widths is used in this study to determine the amount of reinforcement needed to control crack widths in mass concrete elements. This method was used to estimate the long-term crack behavior of instrumented ALDOT elements to assess the efficiency of the reinforcement provided.

2.4.2.1 Bamforth's Method

Bamforth (2007) presents a method for calculating the required amount of shrinkage and temperature reinforcement based on early-age and long-term cracking due to temperature and shrinkage effects. For early-age predictions, Bamforth assumes that the most likely time of cracking will occur between three and seven days, and he recommends using concrete properties at three days in most cases. For long-term predictions, Bamforth recommends using 28-day concrete properties. The long-term temperature differences used in this method are the difference in the peak concrete core temperature and the lowest recorded long-term temperature. The full method involves the following four steps:

1. Define the allowable crack width associated with early-age thermal cracking,
2. Estimate the magnitude of restrained strain and the risk of cracking, CR,
3. Estimate the crack-inducing strain, ε_{cr}, and
4. Check the predicted crack width, w.

Refer to Appendix J for example calculations from this method. This method follows the requirements of Eurocode 2: 1992-1-1 and Eurocode 2: 1992-3 (Bamforth 2007). European design code requirements are not discussed here, but all coefficients used for this method are based on recommended values from the Eurocode 2: Design of Concrete Structures, unless other values were recommended for use by Bamforth.

Defining the allowable crack width has to do with the relationship between crack width and functionality (Bamforth 2007). Crack width limits in Table 2-4 are an excellent aid for designers when considering crack control reinforcement.

The level of restraint must be considered as well. Accurately estimating the level of external and internal restraint requires knowledge of many concrete factors, all of which contribute to the overall restraint factor, R, which ranges from 0.0 to 1.0. Bamforth provides guidance on how to estimate external and internal restraint, which is largely based on ACI 207.2R (2007). As noted in Section 2.2.2, Bamforth (2007) recommends using an internal restraint factor of 0.42 for predicting early-age cracking.

The crack-inducing strain, $\varepsilon_{cr}(t)$, in Equation 2-11 is used in Bamforth's method to derive crack widths. It is equal to the restrained component of the free strain, $\varepsilon_r(t)$, minus half of the tensile strain capacity at time, t. In this section the time, t, is referred to as either early-age (EA) or long-term (LT). The early-age values are used to estimate a typical average crack width, and the long-term values are used to estimate the worst-case scenario for a crack widening during the coldest part of the year.

$$\varepsilon_{cr}(t) = \varepsilon_r(t) - 0.5\varepsilon_{ctu}(t)$$ **Equation 2-11**

Where,

$\varepsilon_{cr}(t)$	=	crack-inducing strain (mm/mm),
$\varepsilon_r(t)$	=	restrained strain (mm/mm), and
$\varepsilon_{ctu}(t)$	=	tensile strain capacity of concrete under sustained loading (mm/mm).

The free strain for early-age, $\varepsilon_{free}(EA)$, in Equation 2-12 is the total amount of autogenous and thermal shrinkage that would occur with no external restraint. The temperature change, T_1, is the difference between the maximum core concrete temperature and the concrete temperature when it matches ambient weather conditions.

$$\varepsilon_{free}(EA) = T_1\alpha_c + \varepsilon_{ca}(EA)$$ **Equation 2-12**

Where,

$\varepsilon_{free}(EA)$	=	total unrestrained strain when core reaches ambient temperature (mm/mm),
T_1	=	temperature difference in peak core temperature and core at ambient temp. (°C),
α_c	=	concrete coefficient of thermal expansion (mm/mm /°C), and
$\varepsilon_{ca}(EA)$	=	autogenous shrinkage when core reaches ambient temperature (mm/mm).

Bamforth (2007) defines the residual free strain for long-term crack widths $\varepsilon_{free}(LT)$, in Equation 2-13 as the difference in autogenous shrinkage, the total drying shrinkage, and the thermal shrinkage for T_2 that would occur. The long-term temperature difference, T_2 , is the difference between the ambient temperature from T_1 and the lowest predicted (or recorded) concrete temperature for the year. This low temperature will occur during the coldest time of the year.

$$\varepsilon_{free}(LT) = T_2\alpha_c + \delta\varepsilon_{ca} + \varepsilon_{cd} \qquad \textbf{Equation 2-13}$$

Where,

$\varepsilon_{free}(LT)$	=	residual free strain from end of T_1 until 28-day value (mm/mm),
T_2	=	temperature difference in core ambient temperature and lowest core temp. (°C),
α_c	=	concrete coefficient of thermal expansion (mm/mm /°C),
$\delta\varepsilon_{ca}$	=	difference in autogenous shrinkage at early-age and 28 days (mm/mm), and
ε_{cd}	=	ultimate drying shrinkage (mm/mm).

The restrained strain, $\varepsilon_r(t)$, is equal to the free strain multiplied by the appropriate restraint factors. Bamforth (2007) calculates the early-age restrained strain, $\varepsilon_r(EA)$, in Equation 2-14 by multiplying the early-age free strain by the external restraint factor and the creep factor.

$$\varepsilon_r(EA) = R_1K_1\left[\varepsilon_{free}(EA)\right] \qquad \textbf{Equation 2-14}$$

Where,

$\varepsilon_r(EA)$	=	restrained strain at early-age (mm/mm),
R_1	=	restraint factor at early-age (unitless),
K_1	=	creep factor (unitless), and
$\varepsilon_{free}(EA))$	=	free strain at early-age (mm/mm).

Bamforth (2007) calculates the long-term restrained strain, $\varepsilon_r(LT)$, in Equation 2-15 by multiplying the difference in free strain from early-age to 28 days by the creep factor and the appropriate restraint factors.

$$\varepsilon_r(LT) = K_1[R_2T_2\alpha_c + R_3(\delta\varepsilon_{ca} + \varepsilon_{cd})] \qquad \textbf{Equation 2-15}$$

Where,

$\varepsilon_r(LT)$	=	long-term restrained strain (mm/mm),
K_1	=	creep factor (unitless),

R_2 = long-term restraint factor for thermal strains (unitless),

T_2 = temperature difference in core ambient temperature and lowest core temperature,

α_c = concrete coefficient of thermal expansion (mm/mm /°C),

R_3 = long-term restraint factor for drying shrinkage (unitless),

$\delta\varepsilon_{ca}$ = difference in autogenous shrinkage at early-age and 28 days (mm/mm), and

$\varepsilon_{cd}(t)$ = ultimate drying shrinkage (mm/mm).

Bamforth (2007) uses the maximum predicted crack spacing, S_{max}, to predict the long-term crack width in Equation 2-16. This calculation uses the properties of the concrete cross section and reinforcing steel to determine the maximum crack spacing.

$$S_{max} = 3.4c + 0.425\frac{k_1 d}{\rho_{eff}}$$ **Equation 2-16**

S_{max} = maximum predicted crack spacing (mm),

c = concrete cover (mm),

k_1 = coefficient for bond properties of reinforcement. Bamforth recommends a value of 0.8 (unitless),

d = reinforcing bar diameter (mm), and

ρ_{eff} = ratio of reinforcement to effective area of concrete in tension around reinforcement (unitless).

Equation 2-17 defines the effective reinforcement ratio, ρ_{eff}, used in finding the crack spacing. This ratio uses the area of reinforcement and the effective surface depth that will affect the predicted width of cracks.

$$\rho_{eff} = \frac{A_s}{1000h_{e_{eff}}}$$ **Equation 2-17**

ρ_{eff} = ratio of reinforcement to effective area of concrete in tension around reinforcement (unitless),

A_s = amount of reinforcement provided (mm^2/m), and

$h_{e_{eff}}$ = effective surface depth of concrete in tension (mm).

Equation 2-18 defines the effective surface depth, $h_{e_{eff}}$, used to find the effective area of concrete in tension. This value considers the properties of the concrete cross section and reinforcing bar size when defining the effective depth.

$$h_{e_{eff}} = Lesser\ of: \begin{cases} 2.5(c + 0.5d) \\ h/2 \end{cases} \qquad \textbf{Equation 2-18}$$

$h_{e_{eff}}$	=	effective surface depth of concrete in tension (mm),
c	=	concrete cover (mm),
d	=	reinforcing bar diameter (mm), and
h	=	depth of concrete member (mm).

To predict the width of long-term cracks, Bamforth provides Equation 2-19, using the total crack-inducing strain (early-age plus long-term) and the crack spacing to estimate the crack width.

$$w(LT) = \varepsilon_{cr}(EA + LT) * S_{\max} \qquad \textbf{Equation 2-19}$$

$w(LT)$	= predicted long-term crack width (mm),
$\varepsilon_{cr}(EA + LT)$	= total crack-inducing strain, early-age plus long-term (mm/mm), and
$S_{\max}$	= maximum predicted crack spacing (mm).

2.5 Temperature Control Strategies

To control the maximum concrete temperature difference and maximum concrete core temperature, temperature control strategies may be implemented. Methods of monitoring maximum core temperatures may assist in this process. Contained in this section are methods implemented successfully in the past to complete these tasks. There are strategies for before, during, and after construction, and each will affect concrete temperatures differently.

2.5.1 Maximum Temperature Difference Control

Some of the methods used to control temperature differences are more passive, such as proper formwork removal time, proper material selection, proper concrete placement planning, and coordinating the construction schedule to align with cooler ambient temperatures. Other, more active ways of cooling concrete include precooling coarse aggregate, running cold water through embedded pipes, and covering fresh concrete with insulating blankets (ACI 207.2R 2007). These techniques can also be used to control the maximum concrete temperature difference in the element.

2.5.1.1 Formwork Removal

Proper formwork selection and removal times contribute to controlling the temperature difference in mass concrete elements, especially in cold climates. During a cold weather placement, poorly insulated concrete elements are susceptible to rapid heat loss at the surface, leading to large temperature differences (ACI 207.2R 2007). Steel forms are an example of poor insulation, as they can act as heat sinks when used on mass concrete placements. This rapid heat loss at contact with the formwork also leads to large temperature differences. Wooden forms and blanket insulated forms have much better insulating properties than steel. This prevents rapid heat loss at the surface and allows the entire element to cool at a more uniform rate (ACI 207.2R 2007).

If forms are removed prematurely, or the ambient temperature is much colder than the element, “thermal shock” can occur. This phenomenon occurs as a result of the significant tensile stresses induced when the concrete surface is exposed to lower temperatures. This exposure of the surface to the cold air causes immediate heat loss at the edge relative to the well-insulated concrete core, leading to the development of thermal cracks (ACI 207.2R 2007). By leaving formwork in place for the proper amount of time, effects of this issue can be much less severe, as illustrated by Gajda and Alsamsam (2006) in Figure 2-14.

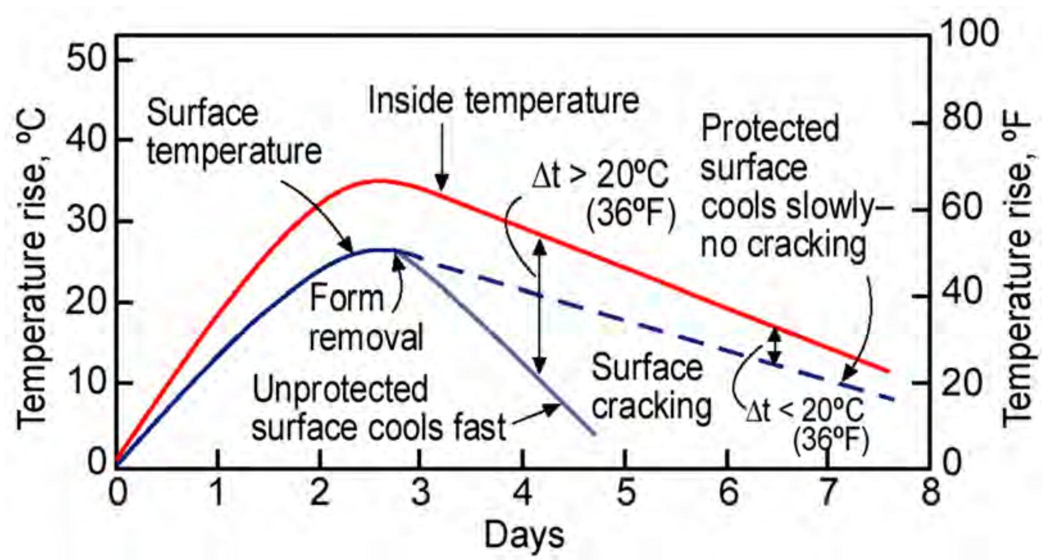


Figure 2-14: The effects of early formwork removal time (Gajda and Alsamsam 2006)

$$(°\mathrm{F} = \mathbf{1.8} \times °\mathrm{C} + \mathbf{32})$$

In Figure 2-14, when formwork is removed after three days the difference between the core and edge temperature increases drastically due to the still elevated core temperature. By allowing formwork to provide insulation for a longer time period, the edge temperature is not allowed to decrease at a slower rate, preventing a temperature difference that would induce thermal cracking. An example of cracking because of early formwork removal can be seen below in Figure 2-15. This core sample was taken from a footing where an insulation blanket blew off during construction in cold weather. The cold weather caused thermal shock and induced significant stresses within this member that led to thermal cracking (Gajda and Alsamsam 2006).

Figure 2-15: Core from footing exposed to thermal shock (Gajda and Alsamsam 2006)

2.5.1.2 Coarse Aggregate Selection

The coarse aggregate selected significantly impacts the potential for thermal cracking, because of the aggregate's CTE and the tensile strain capacity that the aggregate provides (see Equation 2-1 and Table 2-1). A coarse aggregate with a lower CTE will improve resistance to thermal cracking because the CTE of coarse aggregate has the greatest effect on concrete's overall CTE (Browne 1972). The two primary types of coarse aggregate used in Alabama are siliceous river gravel and limestone. Typical CTE values for concrete containing siliceous river gravel and limestone are 6.95 × 10-6 in./in./°F and 5.52 × 10-6 in./in./°F, respectively (Schindler et al. 2010). Concrete containing a limestone coarse aggregate has a higher resistance to thermal cracking because it can tolerate a higher temperature difference than river gravel before the same tensile stress is induced.

The coarse aggregate selected affects a concrete's tensile strain capacity (ε_{tsc}). The tensile strain capacity represents the amount of strain a concrete member can withstand without developing a crack. Values of 65 $\mu\varepsilon$ and 85 $\mu\varepsilon$ were assigned to river gravel and limestone, respectively, by Bamforth (2007). Bamforth and Price (1995) assigned slightly different values of 70 $\mu\varepsilon$ and 90 $\mu\varepsilon$. Based on the ratio of these values, concrete with limestone aggregate can withstand 30% more tensile strain than river gravel concrete before cracking.

2.5.1.3 Use of Multiple Placement Lifts Instead of One Continuous Placement

As described in Section 1.1, a concrete element's least dimension most greatly affects the temperature problems associated with mass concrete. By pouring a mass concrete element in separate lifts many of these problems can be avoided. Separate lifts allow the least dimension of the mass concrete element to be reduced, thereby reducing the maximum concrete temperature and maximum temperature difference. For example, a mat foundation with dimensions of 100 ft x 100 ft x 30 ft has a least dimension of 30 ft. In an element this size, extremely high core temperatures will occur, and DEF and thermal cracking may become issues. However, if the same element were to be poured in three equal lifts, the least dimension of each lift would be only 10 ft. While still considered a mass concrete pour, this element size could much more readily disperse trapped heat to the environment, which may lower the risk of damaging effects from thermal cracking and DEF (ACI 207.4R 2005).

2.5.2 Maximum Concrete Temperature Control

Controlling the maximum concrete temperature is the most effective way of addressing all the concerns raised thus far, especially DEF. Lowering the maximum temperature will help lessen the risk of thermal cracking if the maximum concrete temperature difference is lowered, and the risk of DEF will also be lessened by lowering the maximum concrete temperature. There are precooling and post-cooling methods for accomplishing this task.

2.5.2.1 Pre-Cooling Methods

Pre-cooling concrete or concrete materials can be a simple preventative technique when dealing with mass concrete construction (ACI 207.4R 2005). The most effective method of pre-cooling is to use a well-engineered low heat concrete (Gajda et al. 2005). Placing concrete at lower temperatures typically lowers the maximum in-place concrete temperature at a 1 to 1 ratio (Gajda and Vangreen 2002). For a 10°F decrease in the concrete placement temperature, there will be a 10°F reduction in the maximum concrete temperature. A simple method like placing concrete during cooler ambient conditions affects the maximum concrete temperature as well. Below are four techniques available for when further pre-cooling is needed (Gajda et al. 2005).

1. Evaporative Cooling: The process of evaporative cooling involves wetting an aggregate stockpile by sprinkling and using evaporation to remove heat from the material. This is the most economical method of cooling aggregate (Gajda et al., 2005). The amount of cooling that takes place depends on several factors including wind, relative humidity, and ambient temperature (ACI 207.4R 2005).
2. Ice: Replacing a certain amount of mixing water with ice is the most common way of pre-cooling. The ice lowers the temperature of the mixing water and also removes heat as the ice melts. Ice can replace up to 80 percent of the batch water (Gajda et al. 2005) and can lower the concrete temperature by approximately 21°F (ACI 207.4R 2005).
3. Chilled water: This method simply chills the mixing water before batching takes place. This can lower the maximum concrete temperature by approximately 8°F (ACI 207.4R 2005).
4. Liquid Nitrogen: While local availability should be considered when considering this technique, liquid nitrogen is an effective pre-cooling method (ACI 207.4R 2005). Adding liquid nitrogen to a concrete mixture is the most effective method of lowering maximum concrete temperatures by more than 20°F (Gajda et al. 2005). Injection of the nitrogen directly into the ready-mix concrete truck drum is recommended (ACI 207.4R 2005). This allows the contractor the freedom to make on-site adjustments to the placement temperature.

2.5.2.2 Post-Cooling Techniques

Post-cooling techniques are used to remove heat generated during the hydration process, whereas pre-cooling is done prior to placement. This is done primarily through the use of internal cooling pipes placed inside the element prior to casting concrete. Once the element has been cast, cool water is pumped through these pipes removing heat from the inside of the concrete. The variables that determine how much heat is removed include the pipe material, pipe diameter, temperature of the water, and length of the pipe (Kim et al. 2000). The main disadvantages of these pipes are their installation and maintenance during construction. For this reason, only durable materials like aluminum, steel, and heavy-duty PVC should be used for this application (ACI 207.4R 2005). Figure 2-16 from Kim et al. (2000) demonstrates the usefulness of installing cooling pipes in mass concrete members. There is a difference of approximately 10°C (18°F) between the predicted maximum concrete temperature without cooling pipes and the measured temperature with cooling pipes.

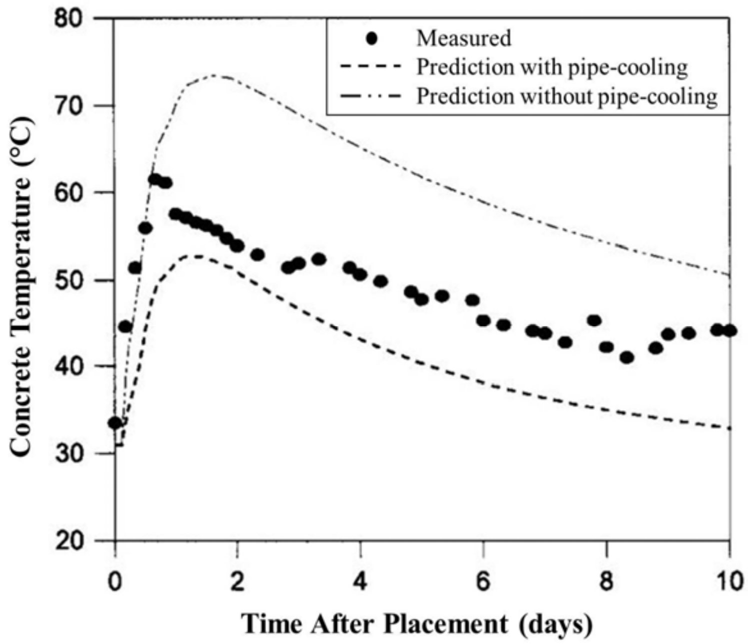


Figure 2-16: Effect of cooling pipes (adapted from Kim et al. 2000) ($°\mathrm{F} = 1.8 \times °\mathrm{C} + 32$)

2.5.3 Thermal Control Plan

To regulate internal concrete temperatures, a concrete thermal control plan (TCP) is typically used for a specified number of days. The TCP requires that the contractor meet and maintain specific temperature requirements throughout the construction phase of the element. These temperature requirements depend on the project parameters and element size. The following information from the Georgia Department of Transportation (2013) details the typical requirements found in a TCP:

- Concrete mixture proportions,
- Strategies for placement time/date, pre-cooling, etc.,
- Curing method and duration of cooling,
- An approximate maximum concrete temperature and temperature difference,
- Direction on the type of temperature monitoring and recording system to be used,
- The required locations for temperature sensors within the element,
- Instruction on how often measurements will take place and any additional measurements needed if maximum limits are approached, and
- A recommendation for a 28-day thermal stress model.

A full list of state DOTs with mass concrete TCP requirements may be found in Table 3-11 and Figure 3-4.

2.5.3.1 Internal Temperature Monitoring Methods

The TCP contains a description of the type of temperature monitoring system to be used, the location of temperature sensors, and the duration of the temperature monitoring. Descriptions of sensor locations typically require that sensors be placed at the core and multiple edges of the element. This is done to monitor the maximum in place temperatures at those locations and also to get the real-time temperature difference between the edge and core of the element. These sensors (core and edge) are often tied to the surrounding reinforcement for convenience. Often more monitoring systems than needed are prescribed in case one system should fail (ACI 207.4R 2005). This also allows for comparison of data from different systems for accuracy. Table 3-9 contains a full list of state DOTs with a requirement for temperature monitoring systems in mass concrete.

There are several types of sensors available for use. These types include thermocouples, intelliRock, iButton and more. More information on the capabilities and construction of the iButton sensors used can be found in Section 4.2.1.

2.5.4 Bamforth's Summary of Options to Help Mitigate Early-Age Thermal Cracking

Table 2-8 from Bamforth (2007) contains a summary of recommendations for the prevention, mitigation, and control of early-age thermal cracking. Many of these options are listed and discussed in other parts of this text. Table 2-8 is presented because it provides a simple summary from Bamforth (2007) on the best and worst choices that relate to thermal cracking. Note that Bamforth recommends that the best choice of aggregate is an angular aggregate with a low CTE. This choice corresponds to the limestone aggregate commonly used by ALDOT in concrete construction.

Table 2-8: Summary of available options to mitigate thermal cracking (Bamforth 2007)

Factor	Worst Choice	Best Choice	Comments
Concrete mixture parameters			
Aggregate shape	Rounded	Angular	Better strain capacity may be partially offset by higher cement content
Aggregate type	High CTE	Low CTE	
Cementitious	CEM1 (100% cement)	Addition of FA, slag cement	
Admixtures (excluding latex and polymers)	None	Water reducers, superplasticizers	To reduce cement content, check durability requirements
Placing temperature	High	Low	Cooling of constituents using chilled water, ice, or liquid nitrogen
Construction practice			
Ambient temperature	High	Low	Night time concreting is beneficial
In situ cooling	Cooling pipes are effective, but expensive. Surface cooling by water spray applicable for sections < 20 in. thick		
Formwork for sections under 20" thick	Insulated, plywood with long striking time	GRP, steel, striking time not significant	To permit rapid heat loss and reduce T_1
Formwork material in mass concrete	Steel, GRP	Plywood, insulated with long striking time	To minimize thermal gradients, keep upper surface insulated
Insulation for mass concrete	None	Thermal blanket	Effective for isolated element with low external restraint
Construction sequence	Alternate bay between lifts	Sequential construction or short infill bays	Not significant if using full movement joints
Period between successive lifts	Long	Short	First pour may be insulated to reduce difference effects. Slip-forming is beneficial as casting is continuous
Movement joints	None, partial movement joints	Full movement joints	Full movement joints require dowels and sealing
Prestressing at base			Not normally economic
Reinforcement distribution	Large diam. bars at wide spacing	Small diam. bars at close spacing	To increase the surface area of reinforcement

2.6 CONCRETEWORKS

ConcreteWorks is a mixture proportioning, temperature prediction, and chloride diffusion service-life software package developed to aid engineers in predicting concrete behavior in certain types of elements, including many which are mass concrete (Concrete Durability Center 2005). It is available for free on the TxDOT website. This software permits the user to input specific details about the concrete materials, cement chemistry, placement conditions, and other features unique to a project to predict mass concrete temperature and cracking risk behavior.

2.6.1 Inputs

There are nine categories of inputs available for ConcreteWorks as listed in Table 2-9. For any inputs that are not available at the time of modeling, default values are available (Concrete Durability Center 2005). This gives the engineer/designer the freedom to customize a project model to the degree which they are able. Default values generally provide reliable temperature predictions, but accuracy will obviously increase as more project specific inputs are manually applied to the model.

Table 2-9: ConcreteWorks input categories (Concrete Durability Center 2005)

Input Category	Specific Inputs
General	• Time, date, and location of placement • Duration of analysis (1-14 days)
Shape	• Type and shape of element
Dimensions	• Member dimensions specific to element shape
Mixture Proportions	• Batch weights and properties of all materials
Material Properties	• Chemical composition, hydration properties of cement • Type of aggregates used and corresponding CTEs
Mechanical Properties	• Maturity function, equivalent age, and early age creep inputs
Construction	• Placement temperature, form type, method and duration of curing
Environment	• Ambient weather data
Corrosion	• Details about reinforcing steel used

2.6.2 Outputs

Using the given information, the software will output predictions including short-term temperature development profiles across the entire element. The concrete stress and strength development can also be calculated based on the use of maturity functions. See Table 2-10 for a complete list of outputs available for mass concrete through ConcreteWorks.

Table 2-10: ConcreteWorks mass concrete outputs (adapted from Concrete Durability Center 2005)

Mass Concrete Member Type	Chloride Service Life	Thermal Cracking Risk	Temperature Prediction	Stress/Strength Development
Rectangular Column	X	X	X	X
Rectangular Footing	X	X	X	X
Partially Submerged Rectangular Footing	X	X	X	X
Rectangular Bent Cap	X	X	X	X
T-Shaped Bent Cap	X		X	X
Circular Column	X		X	X
Drilled Shaft	X		X	X

A feature of ConcreteWorks that was very useful for this project is the ability to download predicted temperature profiles at any location in the element. This allows for viewing of temperature differences between the core and edge of a modeled element in tandem with the associated cracking risk. Figure 2-17 is an example of ConcreteWorks' outputs summary and temperature difference/cracking risk plot for the first 168 hours. The member type used was a rectangular footing modeled after an ALDOT bridge pedestal placed in Scottsboro, Alabama.

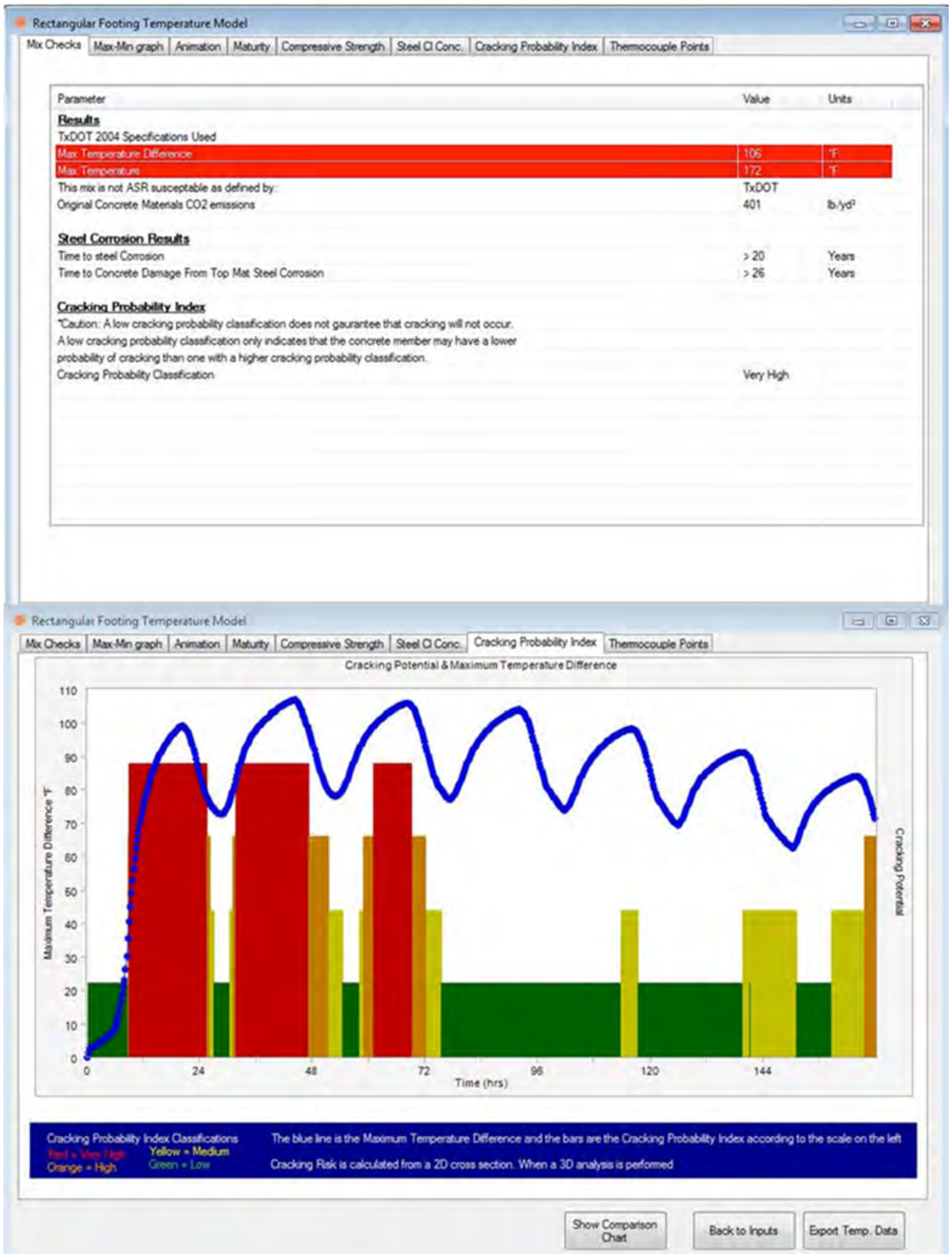


Figure 2-17: Example ConcreteWorks output summary (top) and ΔT/cracking risk profile (bottom)

2.6.2.1 Maximum Limits

In the "Results" summary, the "Max Temperature Difference" is highlighted red because the value is greater than 35°F. The "Max Temperature" is highlighted red because the value exceeds 158°F. ConcreteWorks will automatically do this if the TxDOT 2004 specification is selected (Concrete Durability Center 2005). These values match the maximum limits for TxDOT (see Figure 3-2 and

Figure 3-3), except that the maximum temperature limit has been raised from 158°F to 160°F in the Texas Standard Specifications for Highway and Bridge Construction (2014).

2.6.2.2 Cracking Risk

The cracking risk profile shown in Figure 2-17 is color-coded according to cracking probability and tensile stress-strength ratio (Concrete Durability Center 2005). See Table 2-11 for the definitions of the four colors used. This quantification allows for a quick assessment of potential durability issues associated with the current mixture proportions. Preventative measures can then be taken to lower the cracking risk. Notice the “Cracking Probability Index” for the given example is “very high”.

Table 2-11: ConcreteWorks cracking risks (adapted from Concrete Durability Center 2005)

Color	Cracking probability	Tensile stress-strength ratio (x)
Green	Low	$x < 0.60$
Yellow	Medium	$0.60 \leq x < 0.67$
Orange	High	$0.67 \leq x < 0.72$
Red	Very High	$x \geq 0.72$

Chapter 3

Review of U.S. Mass Concrete Specifications

3.1 Review of Current Mass Concrete Specifications

The following section details the various ways that agencies throughout the United States have defined the use of mass concrete through standard specifications. These agencies include ACI, AASHTO, FHWA, and various state departments of transportation.

3.1.1 ACI Committee 301

The Specifications for Structural Concrete (2016) as reported by ACI Committee 301 contains a useful summary of mass concrete information. This information includes the least dimension designation recommended by ACI Committee 207 (2005) as 4 ft, as well as the requirement listed in Table 3-1 to manage the temperatures reached.

Table 3-1: Mass concrete requirements of ACI 301 (2016)

Material Requirements	Temperature Monitoring	Temperature Requirements	Thermal Control Plan
• Type III Cement shall not be used unless specified by the engineer • Use hydraulic cement with a low heat-of-hydration or use a portland cement in combination with Class F fly ash and/or slag cement	• Place two sensors at the center of the largest portion of the placement and two sensors no more than 2 in. from center of the nearest exterior surface. The second sensor at each location is for redundancy • Temperatures should be recorded no less often than every 12 hours	• Maximum concrete temperature placement of 95°F • Maximum concrete temperature of 160°F • Maximum temperature difference of 35°F	A thermal control plan (TCP) must be submitted and approved by the project engineer

ACI 301 applies maximum limits to the amounts of SCMs used in normal concrete that also apply to mass concrete. The maximum amount of fly ash permitted is 25% by mass, and the maximum amount of slag cement permitted is 50% by mass. The total amount of these two SCMs together may not exceed 50% by mass.

3.1.2 ACI Committee 201

In addition to the requirements set forth by ACI 301, ACI Committee 201 (2016) provides recommendations for adaptable temperature requirements to minimize the deleterious effects of DEF in the Guide to Durable Concrete. These recommendations state that if a concrete temperature greater than 158°F is unavoidable then the measures in Table 3-2 should be adopted.

Table 3-2: Recommended measures for reducing the potential for DEF in concrete exposed to elevated temperatures at early ages (ACI 201.2R 2016)

Maximum Concrete Temperature, T	Prevention Required
T ≤ 158°F	No prevention required
158°F < T ≤ 185°F	Use one of the following approaches to minimize the risk of expansion: 1. Portland cement meeting requirements of ASTM C150 moderate or high sulfate-resisting and low-alkali cement with a fineness value less than or equal to 430 m^2/kg 2. Portland cement with a 1-day mortar strength (ASTM C109) less than or equal to 2850 psi 3. Any ASTM C150 portland cement in combination with the following proportions of pozzolan or slag cement: g) Greater than or equal to 25 percent fly ash meeting the requirements of ASTM C618 for Class F fly ash h) Greater than or equal to 35 percent fly ash meeting the requirements of ASTM C618 for Class C fly ash i) Greater than or equal to 35 percent slag cement meeting the requirements of ASTM C989 j) Greater than or equal to 5 percent silica fume (meeting ASTM C1240) in combination with at least 25 percent slag cement k) Greater than or equal to 5 percent silica fume (meeting ASTM C1240) in combination with at least 20 percent Class F fly ash l) Greater than or equal to 10 percent metakaolin meeting ASTM C618 4. An ASTM C595/C595M or ASTM C1157/C1157M blended hydraulic cement with the same pozzolan or slag cement content in Item 3
T > 185°F	The internal concrete temperature should not exceed 185°F (85°C) under any circumstances.

The above recommendations are based on the work of Ghorab et al. (1980), Ramlochan et al. (2003), and Thomas (2001, 2008) which shows that the risk associated with concrete cured at high temperatures can be effectively eliminated by the incorporation of SCMs (ACI 201.2R 2016). This table is based on Table 2-3 which proposes a similar maximum temperature specification based on the amount of SCMs present.

3.1.3 FHWA and AASHTO

The FHWA Standard Specification for the Construction of Roads and Bridges (2014) does not contain any specification specific to mass concrete. It does contain requirements for concrete construction that apply to mass concrete. This information is shown in Table 3-3.

Table 3-3: Concrete specifications for FHWA FP-14

Material Requirements	Temperature Monitoring System	Temperature Requirements
• Maximum percent of total cementing material by mass: • Fly ash: 25% • Slag cement: 50% • FA+SC: 50%	Provide a maturity meter that conforms to AASHTO T 325 and can collect and store maturity data for at least 14 days	• Concrete temperature at placement between 50°F and 80°F • Maximum temperature differential of 35°F • Maximum concrete core temperature of 140°F for high-strength concrete and 160°F for prestressed concrete • Minimum edge temperature of 45°F

AASHTO Construction Specifications (2016) currently contains no mass concrete specifications. However, a maximum temperature requirement of 160°F for precast construction is in place.

3.1.4 State DOTs

A review of each state's standard construction specifications was performed to gather information pertinent to mass concrete construction. Table 3-4 separates all U.S. states DOTs into those with a mass concrete specification and those without one. Table 3-5 through Table 3-11 summarize the mass concrete requirements found for each state DOT. Figure 3-1 through Figure 3-4 are included to illustrate these requirements across the United States.

Table 3-4: State DOT mass concrete specifications

DOTs with a mass concrete specification		DOTs without a mass concrete specification	
Arkansas	Mississippi*	Alabama	Nebraska
California	New Hampshire*	Alaska	Nevada
Connecticut	New Jersey	Arizona	New Mexico
Delaware	New York*	Colorado	North Dakota
Florida	North Carolina*	District of Columbia	Oklahoma
Georgia	Ohio	Hawaii	Oregon
Idaho	Rhode Island	Illinois	Pennsylvania
Iowa	South Carolina	Indiana	South Dakota
Kentucky	Texas	Kansas	Tennessee
Louisiana	Virginia	Maryland	Utah
Maine*	West Virginia	Michigan	Vermont
Massachusetts*		Minnesota	Washington
		Missouri	Wisconsin
		Montana	Wyoming

*Specification is not specific to mass concrete but includes requirements applicable to mass concrete construction

Table 3-5: State DOT mass concrete specification reference document

DOT	Reference Document
Arkansas	2014 Standard Specification
California	2015 Standard Specification
Connecticut	2016 Standard Specification
Delaware	2016 Standard Specification
Florida	2015 Standard Specification
Georgia	2013 Special Provision to Standard Specification
Idaho	2012 Standard Specification
Iowa	2014 Standard Specification
Kentucky	2012 Special Note for Standard Specification
Louisiana	2016 Standard Specification
Maine	2014 Standard Specification
Massachusetts	2012 Supplemental Specification
Mississippi	2004 Standard Specification
New Hampshire	2016 Standard Specification
New Jersey	2016 Standard Specification
New York	2016 Standard Specification
North Carolina	2012 Standard Specification
Ohio	2016 Construction and Material Specifications
Rhode Island	2015 Supplement to Standard Specification
South Carolina	2007 Standard Specification
Texas	2014 Standard Specification
Virginia	2014 Standard Specification
West Virginia	2010 Special Provision to Standard Specification

Table 3-6: State DOT designation as mass concrete

DOT	Mass Concrete Definition
California	• CIP pile with diameter > 8 ft (temperature monitoring required for diameter > 14 ft) • All other elements with least dimension > 7 ft
Connecticut	• Any concrete placement (excluding underwater) with least dimension > 5 ft in each of three different directions • Any circular concrete placement (excluding underwater) with diameter > 6 ft and height > 5 ft
Delaware	As designated in the contract documents
Florida	As designated in the contract documents
Georgia	• Any element with least dimension > 5 ft • Any drilled shaft with least dimension > 6 ft
Idaho	Any element with least dimension > 4 ft
Iowa	• Any footing with least dimension > 5 ft • Any structural element with least dimension > 4 ft
Kentucky	Any structural element (excluding drilled shafts) with least dimension > 6 ft
Louisiana	Any concrete placement with least dimension > 4 ft
Massachusetts	As specified on the construction plans
New Jersey	As shown on the construction plans
Ohio	• Any concrete placement with least dimension > 5 ft • Any drilled shaft with diameter > 7 ft
Rhode Island	Any element with a ratio of total volume to surface area > 0.6 and minimum dimension of 3 ft in any 3 planes
South Carolina	• Any element with least dimension > 5 ft • Any circular element with diameter > 6 ft and height > 5 ft
Texas	Any concrete placement (excluding drilled shafts) with least dimension > 5 ft
West Virginia	Any concrete placement (excluding drilled caissons) with least dimension > 4 ft

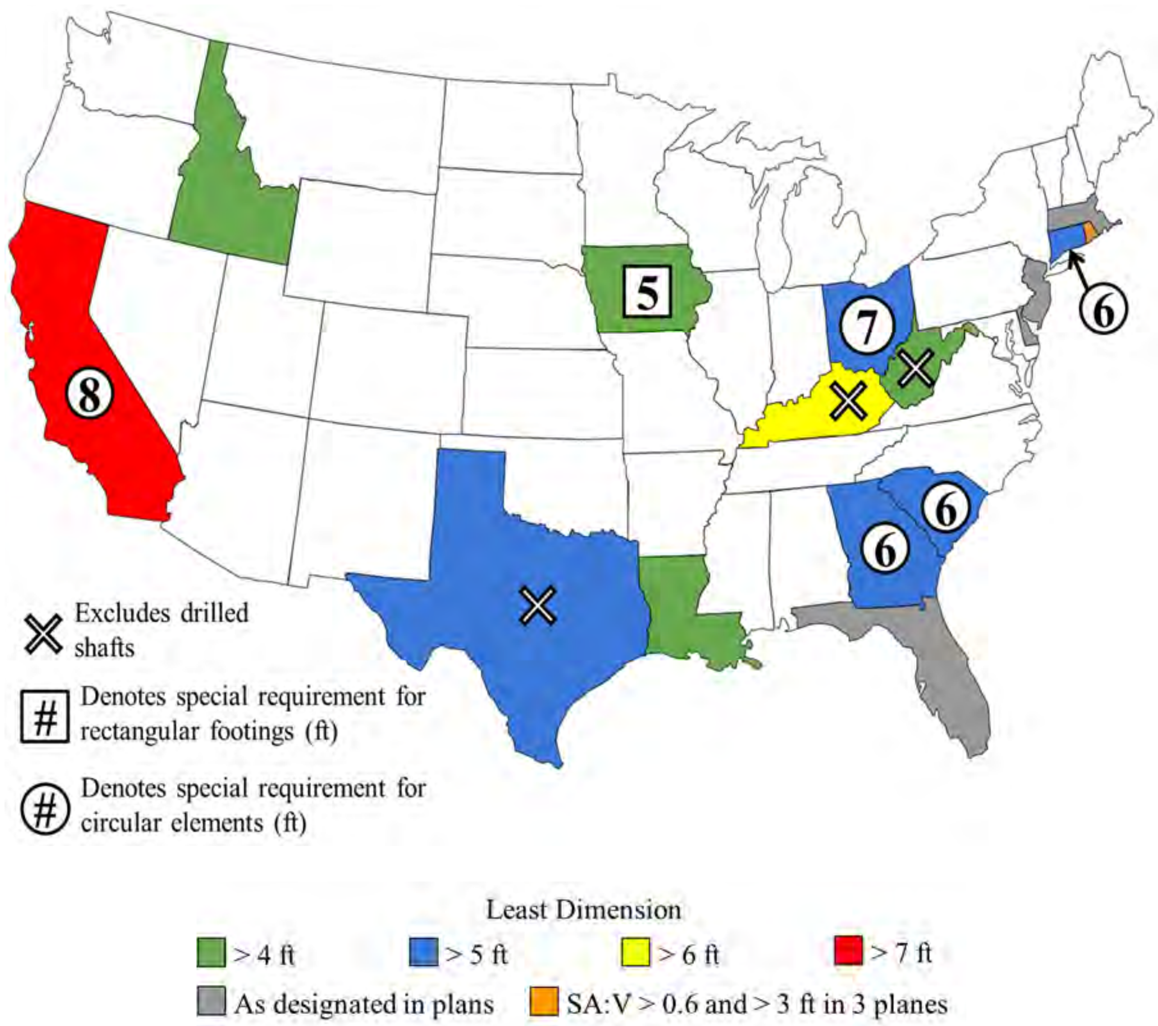


Figure 3-1: State DOT mass concrete designation

Table 3-7: State DOT material requirements for mass concrete

DOT	Material Requirements (all percentages are of total cementing material by weight)
Arkansas	• Use Type II (MH) cement • Fly ash content ≤ 120 pcy

California	• Total cementing content ≥ 505 pcy
	• Total cementing content of CIP piles based on diameter:
	  • (8 ft ≤ d ≤ 10 ft) ≤ 750 pcy
	  • (10 ft < d ≤ 14 ft) ≤ 720 pcy
	• Fly ash content of CIP piles (with diameter > 8 ft) ≥ 25%
	• Fly ash content range: 25-35%
	• Slag cement content range: 50-75%

Florida	• Use Type II (MH) cement
	• Heat of hydration at 7 days ≤ 80 cal/g
	• Fly ash content range: 18-50%
	  • If core temperature is expected to exceed 165°F: 35-50%
	• Slag cement content range: 50-70%
	  • If combined with silica fume, ultrafine fly ash, metakaolin: 50-55%
	• Minimum 20% fly ash and 40% portland cement when using ternary mixture

Georgia	• Type III cement, Class C fly ash, silica fume, and metakaolin are NOT permitted • Class F fly ash content range: 25-40% • Slag cement content ≤ 75% • Aggregate testing for ASR susceptibility recommended
Iowa	• Use only Type I/II cement • Total cementing content ≥ 560pcy • Class C fly ash content ≤ 20% • Air entrainment required
Kentucky	• Use Type I(SM), Type IS, or blended cement that conforms to ASTM C595 • Slag constituent in Type IS is limited to 50% of the mass of the portland blast furnace slag • Class F fly ash content range: 25-30% • Slag cement content ≤ 50%
Louisiana	• Use Type II, Type IP, or Type IS cement • Heat of hydration at 7 days ≤ 70 cal/g • Fly ash content range: 20-50% • Grade 100 & 120 slag cement content range: 50-70%
Massachusetts	Recommends following mixing guide in special provision (not found)
Mississippi	Defines Class C concrete for use in massive reinforced sections
Ohio	• Type III cement and accelerating admixtures not permitted • Total cementing content ≥ 470 pcy Fly ash & slag cement content ≤ 50%
Texas	Type III cement not permitted
Virginia	• Total cementing content for massive unreinforced ≥ 423 pcy Total cementing content for massive lightly reinforced ≥ 494 pcy
West Virginia	• Class F fly ash content ≤ 25% • Slag cement content ≤ 50% Total SCM content ≤ 50%

Table 3-8: State DOT temperature requirements

DOT	Concrete Temperature Requirements
Arkansas	• Placement temperature range: 50-75°F • Maximum temperature difference: 36°F
California	• Maximum temperature: 160°F • Maximum temperature difference determined by TCP
Connecticut	Maximum placement temperature: 85°F

Delaware	• Maximum temperature: 160°F
	• Maximum temperature difference:
	  • First 24 hours: 30°F
	  • 24-48 hours: 40°F
	  • 2-7 days: 50°F
	  • 7-14 days: 60°F

Florida	• Maximum temperature: 180°F • Maximum temperature difference: 35°F
Georgia	• Maximum placement temperature: 85°F unless approved by TCP • Maximum temperature: 158°F and within 70°F of mean annual ambient temperature • Maximum temperature difference: 35°F
Idaho	Maximum temperature difference: 35°F

Iowa	• Placement temperature range: 40-70°F
	• Maximum temperature during heat dissipation: 160°F
	• Maximum temperature difference for an element with least dimension of 6.5 ft or less:
	  • First 24 hours: 20°F
	  • 24-48 hours: 30°F
	  • 48-72 hours: 40°F
	  • After 72 hours: 50°F

Kentucky	• Maximum placement temperature: 70°F • Maximum temperature: 160°F • Maximum temperature difference: 35°F

Louisiana	• Maximum placement temperature: 85°F
	  • If the internal temperature of plastic concrete exceeds 85°F, care should be taken that succeeding batches do not exceed 90°F
	  • Absolute maximum placement temperature: 95°F
	• Maximum temperature: 160°F
	• Maximum temperature difference: 35°F

New Hampshire	Maximum temperature: 160°F

Table 3-8: State DOT Temperature Requirements (continued)

New Jersey	• Maximum temperature: 160°F • Maximum temperature difference: 35°F
New York	Maximum temperature difference: 30°F
North Carolina	Maximum temperature: 160°F
Ohio	• Maximum temperature: 160°F • Maximum temperature difference: 36°F
Rhode Island	• Maximum temperature: 155°F • Maximum temperature difference: 35°F • Performance based temperature difference limit can be determined as a function of the maturity curve
South Carolina	• Maximum placement temperature: 80°F • Maximum temperature difference: 35°F
Texas	• Placement temperature range: 50-75°F • Maximum temperature: 160°F • UT determined that 185°F can be used for concrete containing SCMs through TxDOT sponsored research • Maximum temperature difference: 35°F
West Virginia	• Maximum temperature: 160°F • Maximum temperature difference: 40°F • Higher max. temperature difference can be used if it can be proven that deleterious effects will be avoided

The temperature difference limits imposed by Delaware, Iowa, and Rhode Island in Table 3-8 are particularly interesting because they incorporate a time-dependent concrete temperature difference limit to account for the increasing tensile strength of maturing concrete. As concrete cures, it develops more strength and can withstand a higher temperature-induced stress. These types of limits that allow higher temperature restrictions are ideal for an ALDOT mass concrete specification and are further investigated in this report.

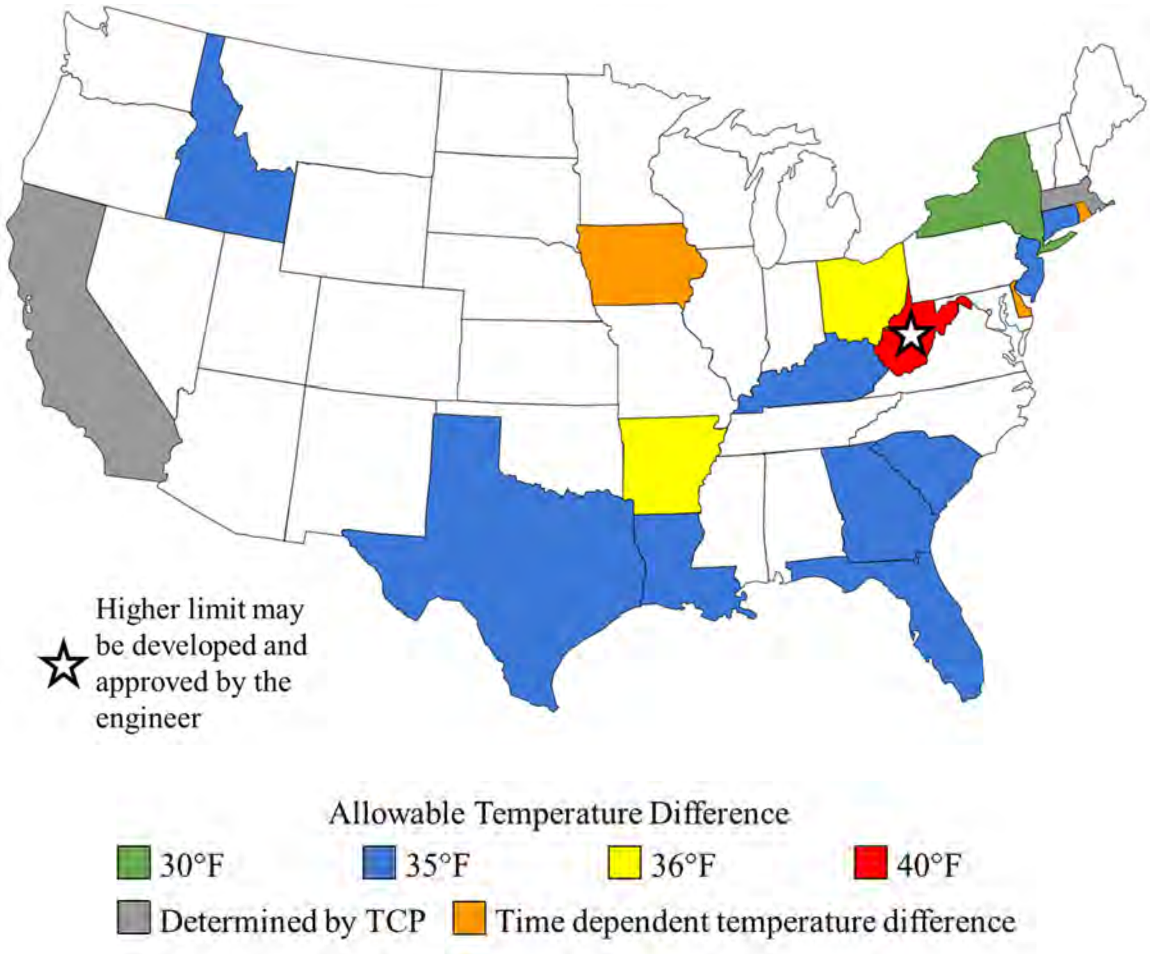


Figure 3-2: State DOTs allowable temperature difference

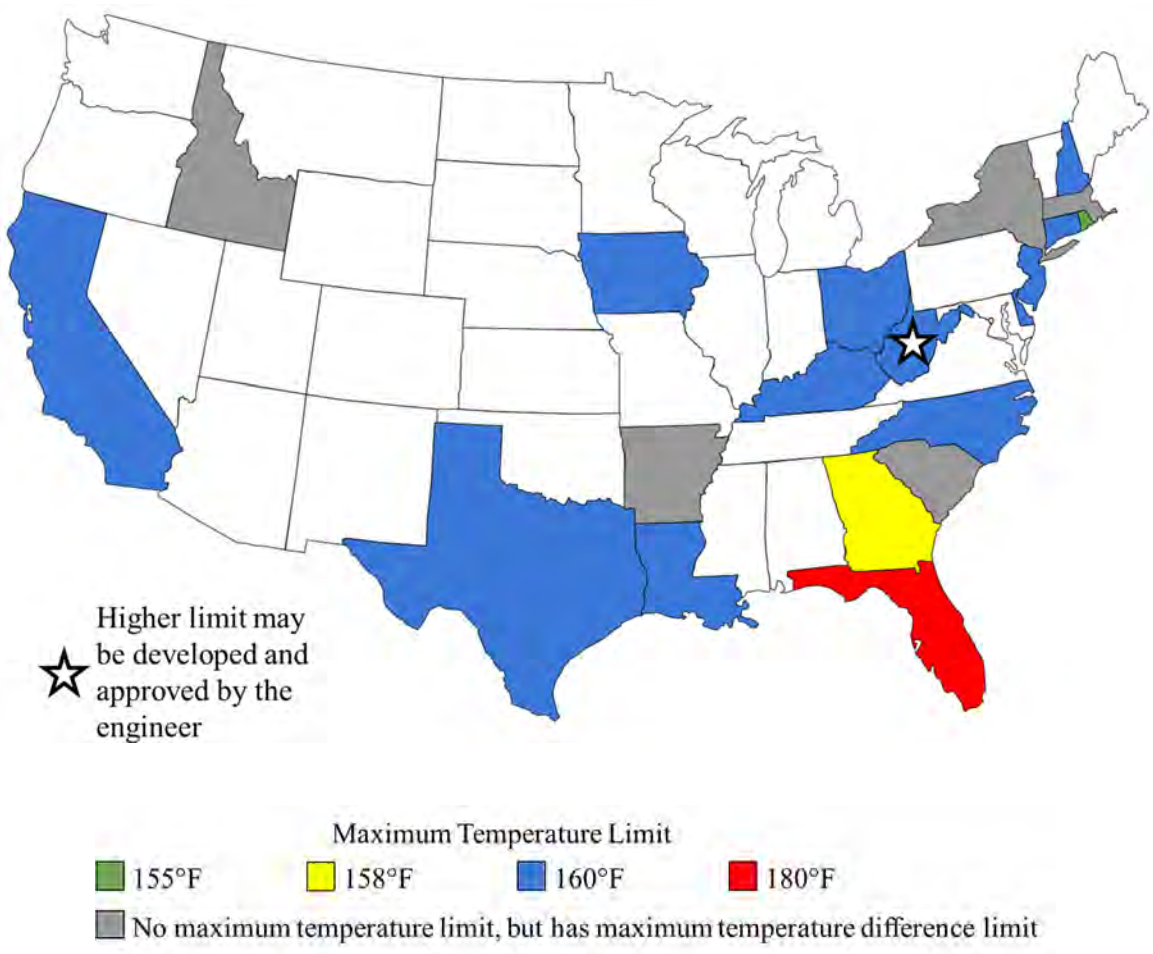


Figure 3-3: State DOTs maximum temperature limit

Table 3-9: State DOT temperature monitoring requirements

DOT	Temperature Monitoring Equipment and Requirements
Arkansas	Monitor temperature for 7 days
California	• Record temperature hourly and monitor until internal temperature is falling • Sensors placed in hottest location, outer edge, corner, and top
Delaware	• Monitor temperature until the maximum temperature difference is reached and the decreasing temperature difference is confirmed as defined in the TCP • Record temperature at a maximum interval of 15 minutes • Provide 2 independent sets of sensors to monitor interior and exterior temperatures • Locate monitoring points at geometric center and 2 in. from surface along the shortest line from center to surface
Florida	• Monitor temperature until the maximum temperature difference is reached and the decreasing temperature difference is confirmed as defined in the TCP • Record temperature at interval no greater than 6 hours • Measured concrete core and exterior surface temperatures approved by engineer
Georgia	• Record temperature hourly (Notify engineer if core and edge reach 140°F and 30°F, respectively) • Provide 2 independent sets of sensors to monitor the center, midpoint of side closest to center, midpoint of top surface, midpoint of bottom surface, and corner furthest from center (edge sensor placed 2-6 in. from surface) to be approved by engineer
Idaho	Monitor temperature for 7 days
Iowa	• Monitor temperature for duration of curing cycle • Record temperature hourly • Provide 2 independent sets of sensors • Locate at least 10 sensors at specified points
Kentucky	• Record temperature every 4 hours • Temperature difference is not recorded until 12 hours after placement • Locate 2 sensors at core and 2 sensors at edge
Louisiana	• Record temperature at maximum interval of 6 hours • Provide 2 independent monitoring systems • Locate sensors at center and edge of placement
New Jersey	Monitor temperature for 15 days OR until core temperature is within 35°F of lowest ambient temperature after placing
Ohio	• Monitor temperature for 28 days after placement • Provide 2 independent sets of sensors

Table3-9: State DOT temperature monitoring requirements (continued)

Rhode Island	• Monitor temperature for minimum of 90 days • Record temperature at maximum interval of 1 hour • Specific data logger shall be used that can determine maturity of concrete • Provide a minimum of 9 sensors for placements less than 500 cubic yards and 17 sensors for placements over 500 cubic yards • Locate a pair of sensors at: peak temperature location, lowest temperature location, 1 in. from top edge directly above core, center of side edge. Use 1 sensor to record ambient temperature • Sensors in each pair shall be between 6-18 in. apart
South Carolina	Locate sensors at center and 2 in. from surface
Texas	• Monitor temperature for 4 days unless otherwise approved • Provide 2 independent sets of sensors • Locate edge sensors no more than 3 in. from surface
West Virginia	• Monitor temperature for 28 days after placement • Record temperature every hour • Provide 2 independent sets of sensors • Locate sensors at core, top face, and center of side furthest away from core

Table 3-10: State DOT in-place concrete curing requirements

DOT	In-Place Concrete Curing Requirements
Arkansas	• Forms shall remain in place for at least 4 days anytime ambient temperature falls below 40°F • All mass concrete shall be cured by free moisture; water curing shall be provided for all exposed surfaces for 14 days
California	Cooling pipes must be removable to a depth of at least 4 in.
Delaware	Do not remove temperature control mechanisms until core temperature is within 25°F of ambient temperature
Florida	Do not remove TCP materials until core temperature is within 50°F of ambient temperature
Iowa	Continue temperature control until core temperature is within 50°F of average ambient temperature
Kentucky	• Maintain thermal control until core temperature is within 35°F of average ambient temperature • Insulate concrete until thermal control is finished
Maine	Massive elements are not to exceed a temperature change of more than 30°F in a 24-hour period during curing
Massachusetts	• Concrete placed at 70°F shall be protected by an enclosure with tight wooden forms at least 5/8 in. thick except at corners and where 2 edge surfaces meet • Artificial heat shall be provided to maintain the minimum required concrete surface temperature
New Hampshire	Maintain curing until temperature is within 50°F of ambient temperature
New York	Surface temperature of sections > 2 ft in thickness shall not drop faster than 18°F in a 24-hour period
North Carolina	• Provide measures to retain moisture when curing at elevated temperatures • Maintain a relatively uniform rate of increase in temperature within curing enclosure of approximately 40°F/hr
Rhode Island	• Do not remove forms until estimated strength of surface exceeds 2500 psi based on lowest indicated maturity AND the mean temperature difference core and ambient is < 30°F • Side forms may be removed after 12 hours except when ambient temperature is below 50°F
Texas	• Only water curing is permitted in mass concrete construction • Forms or insulating membrane must remain in place for 4 days after placement
West Virginia	• Maintain curing for 7 days • Water curing is not permitted; white polyethylene sheeting shall be used to prevent moisture loss

Table 3-11: State DOT thermal control plan requirements

DOT	Thermal Control Plan
Arkansas	TCP developed by contractor, approved by engineer
California	TCP developed by contractor, approved/monitored by engineer
Delaware	• TCP and thermal behavior analysis shall be prepared by specialty engineer for the contractor and approved by project engineer 45 days before placement • Specialty engineer must inspect monitoring system
Florida	TCP and mixture designs developed by contractor, approved by engineer
Georgia	TCP developed by contractor, approved by engineer 30 days before placement
Iowa	• TCP developed by contractor, approved/monitored by engineer 30 days before placement • TCP developed by licensed Temperature Control Engineer if element minimum dimension > 6.5 ft
Kentucky	TCP developed by contractor, approved/monitored by engineer 30 days before placement
Louisiana	TCP developed by contractor, approved/monitored by engineer
New Jersey	TCP developed by contractor, approved by engineer 30 days before placement
Ohio	TCP developed by contractor, approved by engineer 10 days before placement
Rhode Island	• "General TCP" developed by contractor, approved by engineer before first mass concrete placement. This includes plans for a mock up 60 days prior • "Specific TCP" then submitted by contractor for each unique placement other than what is covered in the "General TCP"
South Carolina	Mass Concrete Placement Plan developed by contractor, approved by engineer (contains TCP, anticipated thermal developments, and monitoring system details)
Texas	Use ConcreteWorks or other approved method to develop TCP
West Virginia	Temperature control requirements to be detailed by engineer prior to construction

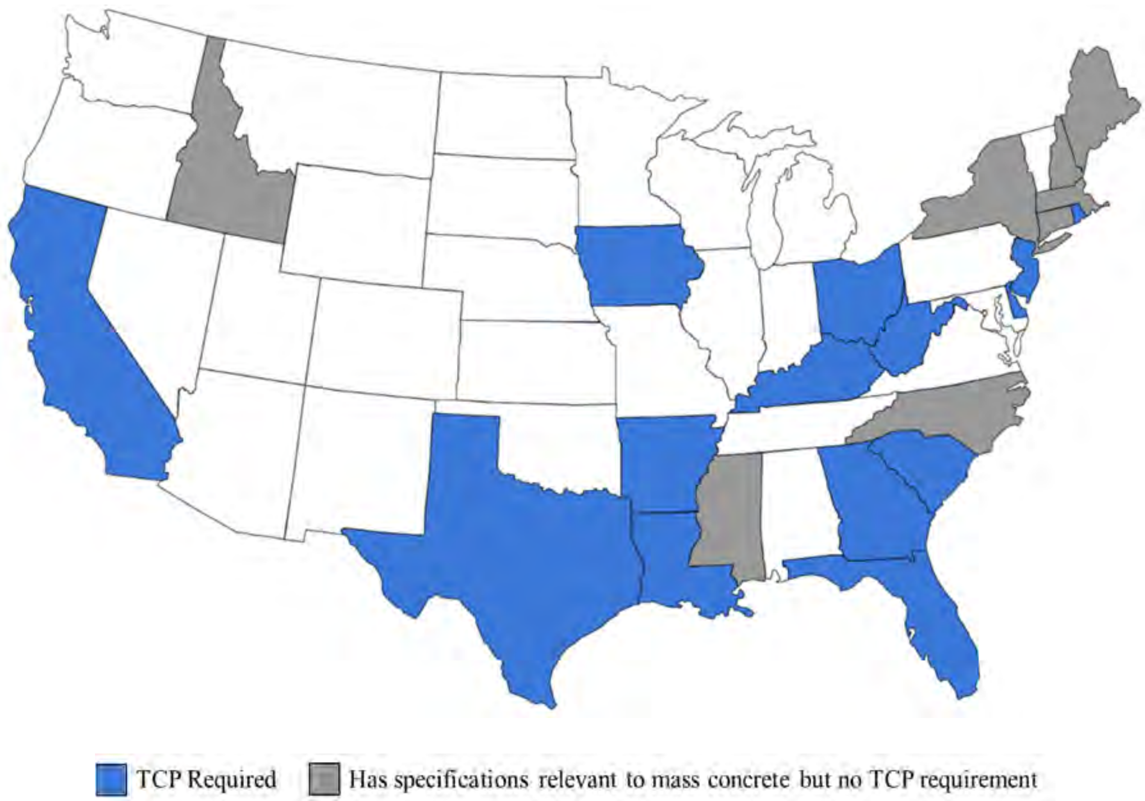


Figure 3-4: State DOTs with a TCP requirement

Chapter 4

Experimental Plan

4.1 INTRODUCTION

The experimental plan for this project was developed to help achieve the project objectives outlined in Section 1.2.: to provide guidance on mass concrete designation, material requirements, temperature monitoring, and temperature prediction for future ALDOT mass concrete projects. This chapter provides an overview of the ConcreteWorks analysis, numerical models, field instrumentation techniques, and lab testing that are all part of the experimental plan.

Seven bridge elements were instrumented from July 2015 to July 2016. Figure 4-1 shows the locations of these field elements around the state. These elements varied in shape, size, coarse aggregate type, and placement conditions, and their main attributes are summarized in Table 4-1. These differing attributes were the primary motivation for selecting these members, allowing temperature data to be collected in a variety of scenarios to help researchers decide what qualifications would designate an ALDOT concrete element as mass concrete. The least dimensions of most elements were between 4 and 6 feet. This range covers common state DOT mass concrete size thresholds, and an investigation with ALDOT elements was desired to determine a threshold using Alabama materials before assigning a least dimension threshold to the ALDOT mass concrete specification.

Figure 4-1: Location of instrumented elements located in the state of Alabama (adapted from Geology.com 2017)

Table 4-1: Seven ALDOT elements instrumented between July 2015 and July 2016

<table>
<tr><th>Element Type</th><th>Location</th><th>Placement Season</th><th>Least Dimension (ft)</th></tr>
<tr><td>Wall</td><td>Harpersville</td><td rowspan="5">Summer</td><td>4.0</td></tr>
<tr><td>Pedestal</td><td rowspan="2">Scottsboro</td><td>10.0</td></tr>
<tr><td rowspan="4">Bent Cap</td><td>6.5</td></tr>
<tr><td>Albertville</td><td>6.5</td></tr>
<tr><td>Brewton</td><td>6.5</td></tr>
<tr><td>Elba</td><td rowspan="2">Winter</td><td>6.5</td></tr>
<tr><td>Column</td><td>Birmingham</td><td>5.0</td></tr>
</table>

4.2 FIELD INSTRUMENTATION

The instrumentation of ALDOT bridge elements under construction provided temperature data crucial in completing many objectives of this project. These data allowed researchers to study the concrete temperature behavior of these potential mass concrete elements. It was compared to the ConcreteWorks temperature predictions in the same elements. It was also compared to the temperature difference limits produced by numerical modeling. All of this contributed to the recommendation of temperature limits for the mass concrete specification detailed in Appendix A. Instrumentation of these elements was accomplished by use of iButton sensors that is described in the following section.

4.2.1 Temperature Sensors

The sensor selected for this project was the DS1921G Thermocron iButton Device produced by Maxim Integrated. This model of sensor was selected because it can record and store 2048 data points and operates within a temperature range of -40°F to 185°F. The iButton is encased in stainless steel, which ensures that the sensor will be protected from construction activities and high pressures expected in mass concrete applications. The OneWireViewer software associated with this sensor allows for user programming of the start time and frequency of data recordings. Because the sampling intervals can be changed and the sensor attaches a time stamp to all data points, it is possible to collect months of temperature data in real time without having to frequently visit the site.

4.2.1.1 Temperature Sensor Assembly

Temperature sensors were assembled in the Auburn University Structural Research Laboratory. The iButton sensor was placed in a Keystone battery holder in order to solder on 22-gauge, unshielded, 2-conductor wire. This assembly was then layered with Static Control Epoxy Coating 3525 produced by General Polymers. This was done to waterproof and electrically insulate the assembly in preparation for embedding the sensor in fresh concrete. Oven testing was done with coated and uncoated sensors to confirm that the epoxy would not affect the iButton’s temperature measurement accuracy.

The other end of the wire was connected to an RJ-11 telephone jack, which could be plugged into a USB reader. This USB reader was plugged into a laptop computer, allowing data to be collected via the OneWireViewer software. The components and completed assembly may be seen below in Figure 4-2.

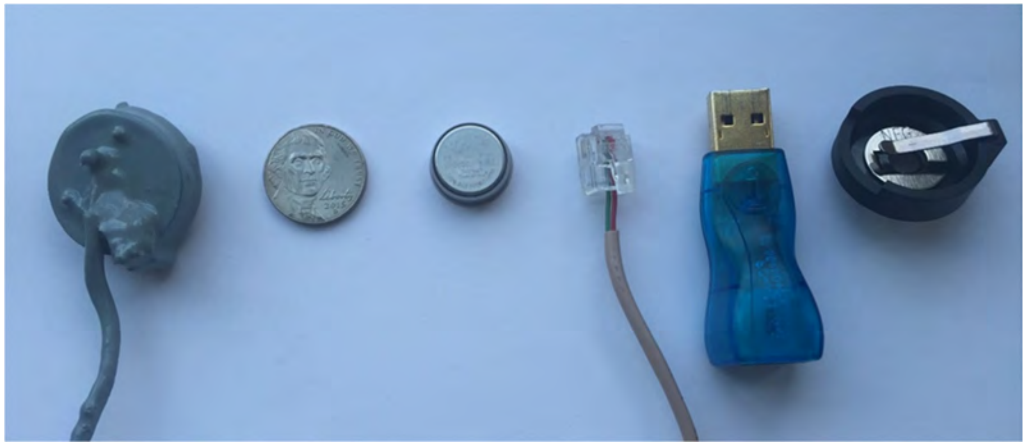


Figure 4-2: iButton components and complete assembly (with nickel for scale) (from L to R: completed sensor, nickel, iButton, RJ-11 attached to 22-gauge wire, USB reader, and Keystone battery clip)

4.2.1.2 Programming and Installation of Sensors

Programming of the sensors began with assigning sensors to specific locations within the intended element by using a unique serial number of each iButton. Once the placement date and time was known, the sensors were set to begin recording approximately four hours before placement. At the start of each placement, the temperature recording frequency was set for every 15 minutes in order to very accurately map the temperature profile for the first 14 days. After this time period, the recording interval was changed to every 3 hours, making sure to record a measurement at approximately 3:00 PM daily, which is close to the expected peak temperature time. At this interval there is enough sensor memory to store 256 days of data.

Installation of assembled sensors took place one or two days prior to placement, after the reinforcement cage was set in place. At a minimum, there were typically two sets of six sensors installed across two cross sections within the element for a total of twelve sensors. At each cross section, two sensors were installed at the geometric center. Two were installed at the middle of the side edge between 2 to 6 in. from the surface. Two sensors were installed directly above the core sensors at the middle of the top surface between 2 to 6 in. from the surface. Two sensors were used at every location for redundancy. Figure 4-3 and Figure 4-4 illustrate the typical location of temperature sensors.

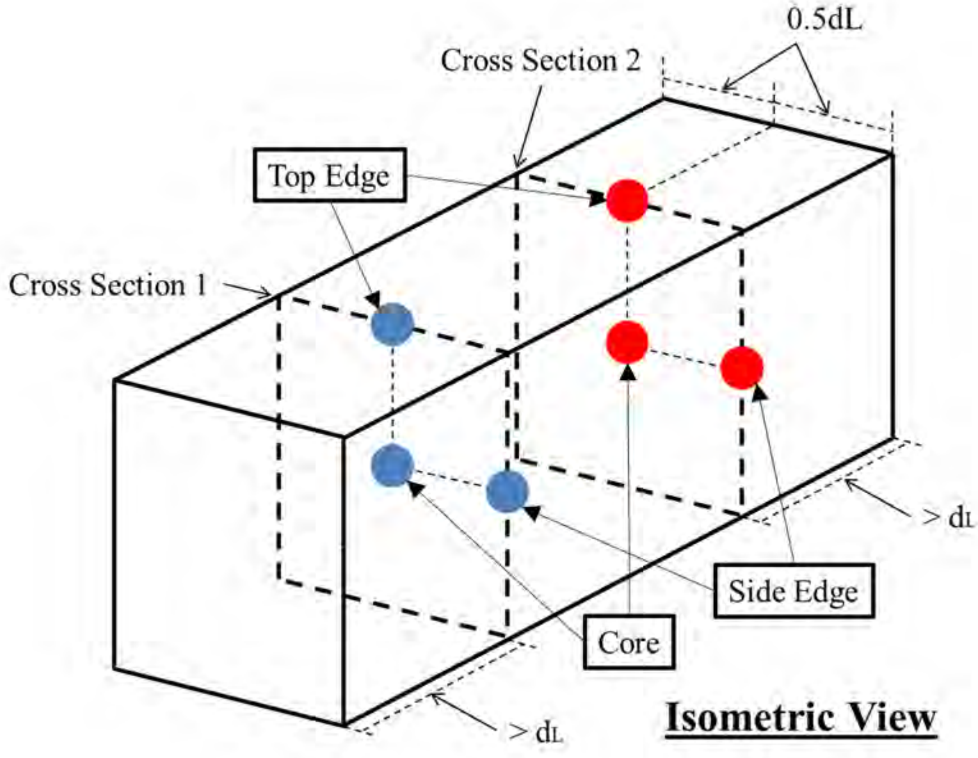


Figure 4-3: Example layout of temperature sensors in a bent cap
(d_L = least dimension) (n.t.s.)

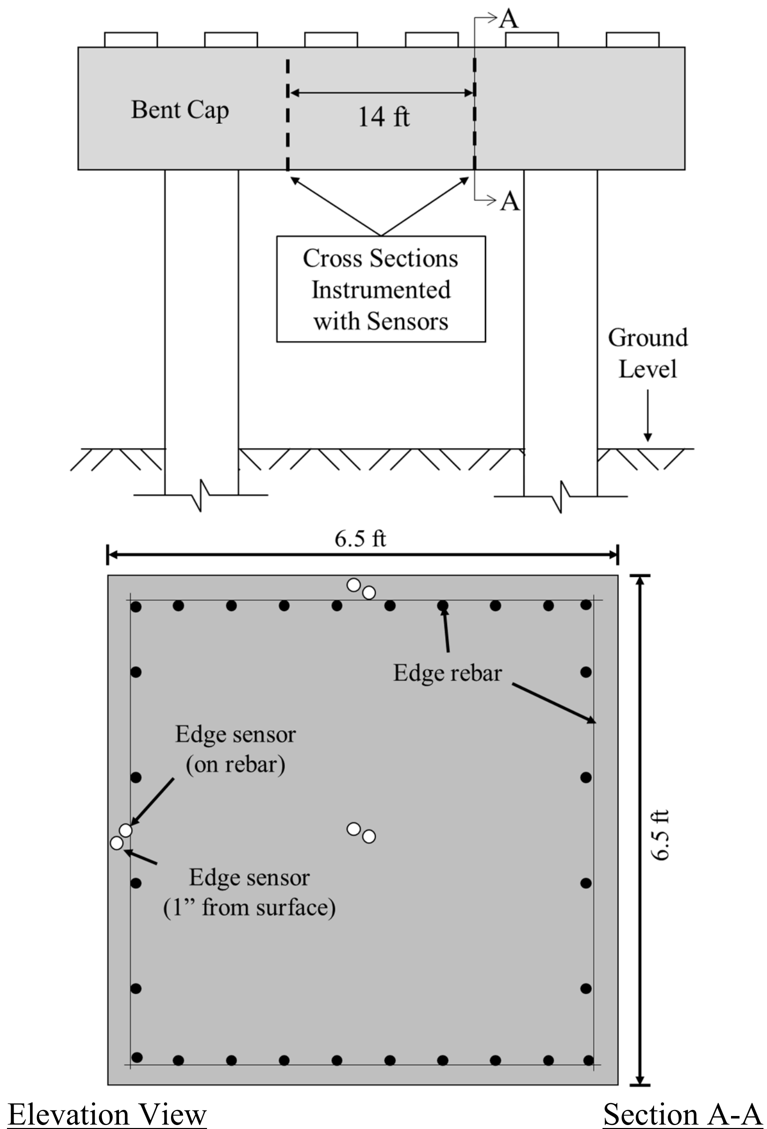


Figure 4-4: Example layout of temperature sensor cross sections in a bent cap (left) and temperature sensors within each cross section (right) (n.t.s.)

To provide guidance on what could be defined as the “edge”, one sensor at each edge location was tied directly to the rebar (the “rebar edge”) while the other one was extended to within the cover distance (the “cover edge”). This sensor layout allowed comparisons to be made between the two edge locations to determine if attaching the sensors directly to the rebar would be sufficient, as this would be the most practical placement location during future ALDOT projects. These two sensor locations are shown in Figure 4-5. Plastic cable ties were used to attach sensors to the rebar cage at the closest location possible to the core, side edge, and top edge.

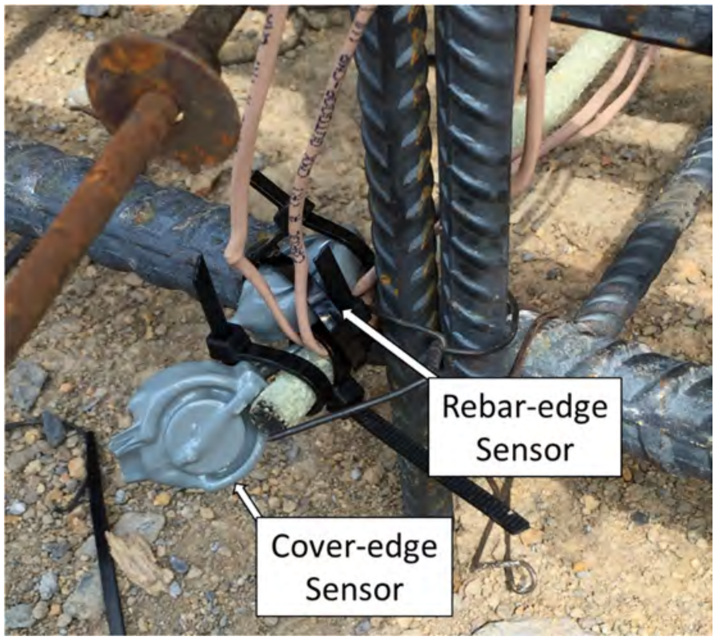


Figure 4-5: Example of cover-edge and rebar-edge sensors in Harpersville crashwall

4.3 CONCRETEWORKS ANALYSIS

4.3.1 Objectives

ConcreteWorks was utilized to help achieve two objectives of this project. The first objective of this analysis was to assess the accuracy of ConcreteWorks by modeling each of the instrumented ALDOT bridge elements to compare predictions with measured field data. Thanks to the assistance provided by ALDOT project managers, all input information for each element instrumented was made available for modeling purposes.

The second objective was to complete a ConcreteWorks analysis for 480 different placement scenarios, all modeled using parameters most relevant to ALDOT construction. This was done to determine which inputs had the greatest impact on maximum concrete temperature, maximum concrete temperature difference, and thermal cracking risk. The element type chosen for these analyses was a rectangular column, because of ConcreteWorks' capability of producing a cross-sectional stress profile for this element using 2-D structural analysis. A specialized version of ConcreteWorks was used to assess the 7-day stress profile at various locations on each cross section.

4.3.2 Significant Variables

To complete the first objective, it was important to focus on the variables that have a more significant impact on temperatures in mass concrete. Five ConcreteWorks inputs were isolated and varied to determine each of their effects. Because of this approach, a total of 480 test runs were performed. Table 4-2 lists the five inputs that were varied in this analysis. To quantify the effect of each variable on the corresponding outputs, the ANOVA Single Factor Test was used with a 95% confidence interval.

Table 4-2: ConcreteWorks analysis isolated variables

Input	Number of Input Variations	Input Descriptions
Placement Location	2	Huntsville Mobile
Placement Date	6	Jan. 15 @ 12:00 PM Mar. 15 @ 10:00 AM May 15 @ 8:00 AM July 15 @ 8:00 AM Sep. 15 @ 10:00 AM Nov. 15 @ 12:00 PM
Coarse Aggregate Type	2	Limestone River gravel
SCM Type	4	100% PC (0% SCM) 30% Class F FA sub. 30% Class C FA sub. 50% slag cement sub.
Least Dimension (ft)	5	4 5 6 7 8

Huntsville and Mobile were chosen as the two placement locations because they represent the typically warmest and coolest ambient conditions in Alabama. Default weather data from each of these locations were used at the specified times. ConcreteWorks also contains default data for Montgomery and Birmingham, but were not selected because Mobile and Huntsville capture the climactic extremes of Alabama.

Different placement times were used to take advantage of ConcreteWorks' default weather data for different seasons and times of day. Times were selected based on Alabama's seasonal ambient temperatures and common construction placement schedules. Contractors typically place concrete earlier in the day during warmer months to take advantage of cooler temperatures. During cooler winter months contractors wait for warmer ambient conditions in the middle of the day. For each scenario, it was assumed that the concrete placement temperature was equal to the ambient temperature at that time.

Limestone and river gravel are the two most commonly available coarse aggregates in Alabama. The primary change in input when switching coarse aggregate type is the concrete CTE for each aggregate. Based on previous research performed at Auburn University, the standard input values for limestone and river gravel were 5.52 × 10-6 in./in./°F and 6.95 × 10-6 in./in./°F, respectively (Schindler et al. 2010). CTE testing of each specific concrete used in the elements instrumented was performed as part of this experimental plan.

The SCM types selected are those most commonly used by ALDOT. The replacement levels were selected to represent the current upper limit allowed by ALDOT Section 501 (2012). ConcreteWorks contains default values for SCM properties that affect mixture proportions and hydration properties of the concrete. The properties of the SCMs used on each instrumented project were also collected with samples of each SCM.

Least dimensions of 4-7 ft are common mass concrete designations by other state DOTs (see Table 3-6). Because there is no strict rule that defines a mass concrete least dimension, several sizes were analyzed to determine at what point Alabama's input parameters caused the model to experience excessive maximum temperatures, temperature differences, and cracking risks.

4.4 NUMERICAL MODELING

4.4.1 Maximum Concrete Temperature Difference

An objective of this project was to develop an improved method for determining the maximum concrete temperature difference to minimize the risk of thermal cracking. Based on the information in Section 2.2, the time dependent relationship between the maximum temperature difference and concrete tensile strength is the link through which to accomplish this.

Equation 2-1 uses concrete properties devoid of the consideration that tensile strength, modulus of elasticity, and creep effects change with time. By modifying this equation, a time-dependent maximum temperature difference may be established, as shown in Equation 4-1. By using Equation 2-2 through Equation 2-9, the known CTE values for Alabama coarse aggregates, and a restraint factor, a time-dependent temperature difference limit for use in an ALDOT mass concrete specification will be determined and compared to the measured data from field projects.

$$\Delta T_{max}(t) = \frac{f_t(t)}{E_c(t) \times K(t) \times CTE \times R} \quad \textbf{Equation 4-1}$$

$\Delta T_{max}(t)$	=	allowable concrete temperature difference as a function of time (°F),
$f_t(t)$	=	concrete tensile strength as a function of time (psi),
$E_c(t)$	=	modulus of elasticity as a function of time (psi),
$K(t)$	=	creep and sustained loading modification factor as a function of time
CTE	=	concrete coefficient of thermal expansion (in./in./°F), and
R	=	Restraint factor (0 = unrestrained; 1 = full restraint).

4.4.2 Prediction of Long-Term Cracking in Instrumented ALDOT Elements Using Bamforth's Method

A secondary objective of this plan is to use Bamforth's crack prediction method (Section 2.4.2.1) to assess the long-term effects of concrete mass temperature differences (or bulk temperature differences) on the instrumented field elements. This bulk temperature difference refers to the "long-term maximum internal temperature change of a large concrete mass as it cools from as internal peak temperature to a stable temperature approximately equal to the ambient temperature" (ACI 207.2R 2007). Bamforth (2007) uses this temperature difference to predict long-term crack widths in mass concrete elements restrained by adjacent members. Figure 4-6 illustrates the bulk temperature difference by showing how a temperature drop in the core can lead to a large long-term bulk temperature difference. This is in contrast to the temperature difference between the core and edge, which affects the concrete most at early ages, as shown on the left part of Figure 4-6. The crack widths predicted with Bamforth (2007) are compared to crack widths measured on the field elements during follow up inspections.

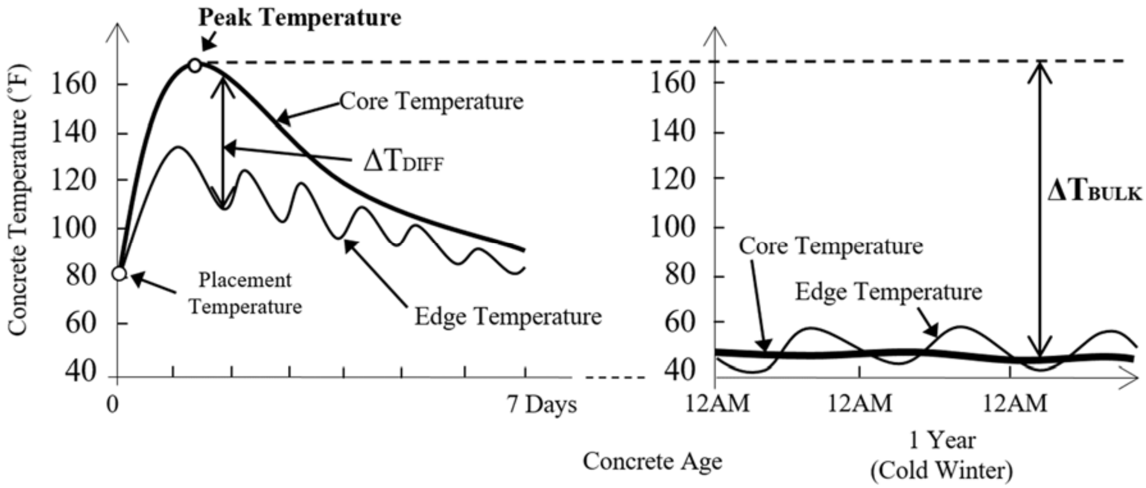


Figure 4-6: Contrasting early-age temperature differences to long-term bulk temperature differences in mass concrete

4.5 LABORATORY TESTING

Laboratory testing of concrete materials sampled from each ALDOT field element was conducted in the Auburn University Structural Research Laboratory to obtain user inputs for ConcreteWorks. By modifying default values with project-specific values, the accuracy of the ConcreteWorks predictions was greatly improved for each element. To test each concrete mixture, enough materials were gathered to make a 1.5 ft^3 batch of concrete for each field element.

4.5.1 Collecting Raw Materials

Aggregates, cement, and SCMs were gathered from the batch plants that supplied the concrete for each instrumented element. The materials were sealed and stored until use. The approved mixture proportions were obtained from the ALDOT project manager for each site. The chemical compositions of all cements and SCMs were obtained from the supplying manufacturers.

4.5.2 Mixing Procedure and Test Specimens

Each mixture was prepared in accordance with ASTM C 192 (2010) using the following procedure:

- A specified amount of mortar is used to coat the inside of the mixer as to prevent loss of paste during mixing. The coating must not affect the w/cm of the specified mixture.
- All aggregates and 80% of the mixing water are added to the mixer.
- Mix for two minutes.
- Cementing materials and the remaining batch water are added to the mixer
- Mix for three minutes.
- Rest for three minutes.

- Mix for two minutes.

A slump test was performed in accordance with AASHTO 119 (2007), and an air content test was performed in accordance with AASHTO T 152 (2005). It was the goal of these tests to be within an acceptable range of the values recorded during the field elements' placements. If fresh properties were acceptable then four 6 x 12 in. cylinders and two 4 x 8 in. cylinders were prepared for testing in accordance with ASTM C 192 (2015).

4.5.3 Hydration Parameters Testing

Upon placement in the mold, one 6 x 12 in. cylinder was weighed and the placement temperature recorded. It was then placed in a semi-adiabatic calorimetry testing apparatus called a Q-Drum for no less than seven days. Adiabatic temperature data taken from each test were used to calculate ConcreteWorks hydration parameters based on heat of hydration models developed by Schindler and Folliard (2005). All Q-Drum testing was done in accordance with guidelines provided by the manufacturer, iQuadrel Services. Figure 4-7 shows the Q-Drum testing apparatus without the lid and a cylinder inside the drum.

Figure 4-7: Q-Drum testing apparatus

4.5.4 Compressive Strength and Modulus of Elasticity Testing

Three 6 x 12 in. cylinders were prepared to test the 28-day modulus of elasticity and compressive strength of each batch. These tests were done in accordance with ASTM C 39 (2010) and ASTM C 469 (2010).

4.5.5 Coefficient of Thermal Expansion Testing

Two 4 x 8 in. cylinders were prepared to test each concrete's CTE in accordance with AASHTO T 336 (2011). Figure 4-8 shows the CTE water bath, water pump, and LVDTs used for testing. The concrete samples were submerged in a water bath that cycled between 50°F (10°C) and 122°F (50°C) daily, while measuring the change in length of each specimen. Each test was run until consistent results were recorded, and each concrete CTE was calculated from this information. Two CTE tests were performed for each mixture, and the results were averaged.

Figure 4-8: CTE testing apparatus

Chapter 5

Measured Behavior of ALDOT Mass Concrete Elements

5.1 INTRODUCTION

Seven ALDOT bridge elements were instrumented with temperature sensors from July 2015 to July 2016. Temperatures were measured at the core and at least two sides of every element. The field data collected are presented here along with the following information for each site:

- Project information and initial site observations when available,
- Sensor installation locations,
- 14-day concrete temperature profiles from the core and edges,
- 14-day concrete temperature difference profiles for the core and edges,
- Long-term temperature and temperature difference profiles when available,
- Results of visual inspections upon return to site,
- Results of crack width predictions using Bamforth's method, and
- Results of laboratory testing.

For additional information on each project (mixture proportions, cement composition, weather data) refer to Appendices B through H.

As many as twelve temperature sensor profiles were measured for each element. However, for simplicity all figures in this chapter containing temperature data display only the core and side-rebar temperature profiles for each element that resulted in the largest temperature difference. A summary of the recorded temperatures of interest for all elements is presented in Section 5.9. A summary of crack width predictions for all elements is presented in Section 5.10. For all measured temperature profiles for each element refer to Appendix B through Appendix H.

5.2 ALBERTVILLE BENT CAP

The following section summarizes the work done on the Albertville bent cap. The mixture proportions, cementing material compositions, weather data, and all temperature profiles for this element may be found in Appendix B.

5.2.1 Project Information

Table 5-1 contains the basic project information for this element. Figure 5-1 shows this element under construction.

Table 5-1: Albertville bent cap project information

Item	Entry
Placement Date	7/31/2015
Placement Time	6:00 a.m.
Placement Location	Albertville, Alabama
Member Type	Rectangular bent cap
Member Dimensions (ft)	6.5 × 6.5 × 40
Cement Type	I/II
Total Cementing Materials Content (pcy)	567
SCM Type (% Replacement)	Class F Fly Ash (25%)
Coarse Aggregate Type	Limestone
Form Type	Wood
Placement Temperature (°F)	85

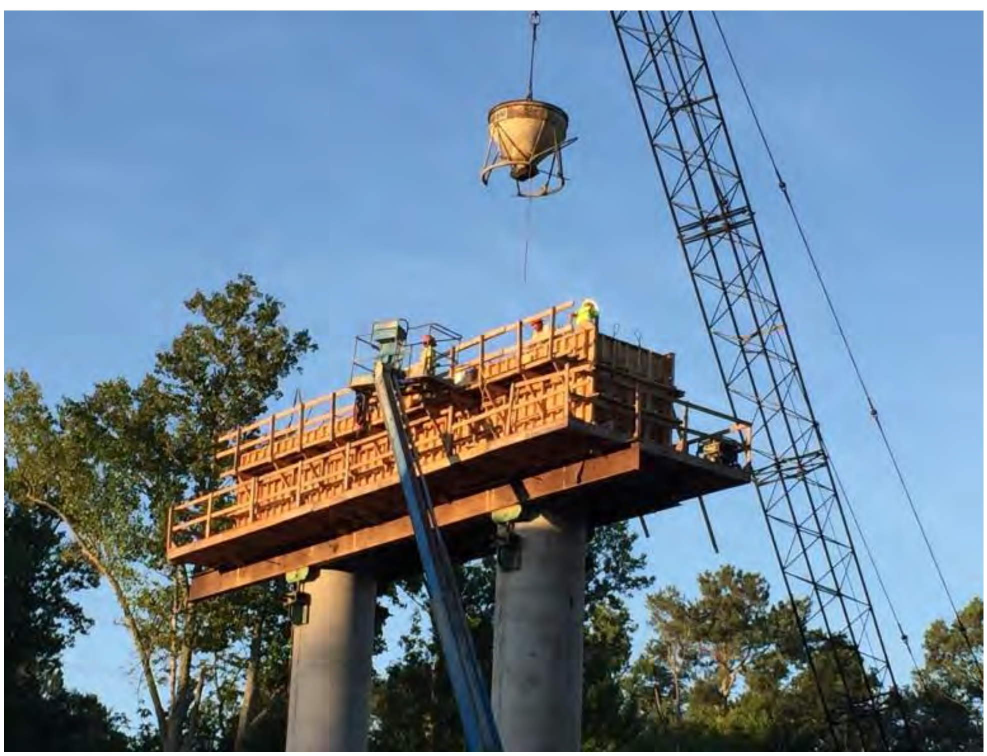


Figure 5-1: Albertville bent cap

5.2.2 Sensor Locations

Figure 5-2 shows the approximate temperature sensor cross section locations and temperature sensor locations within each of these cross sections.

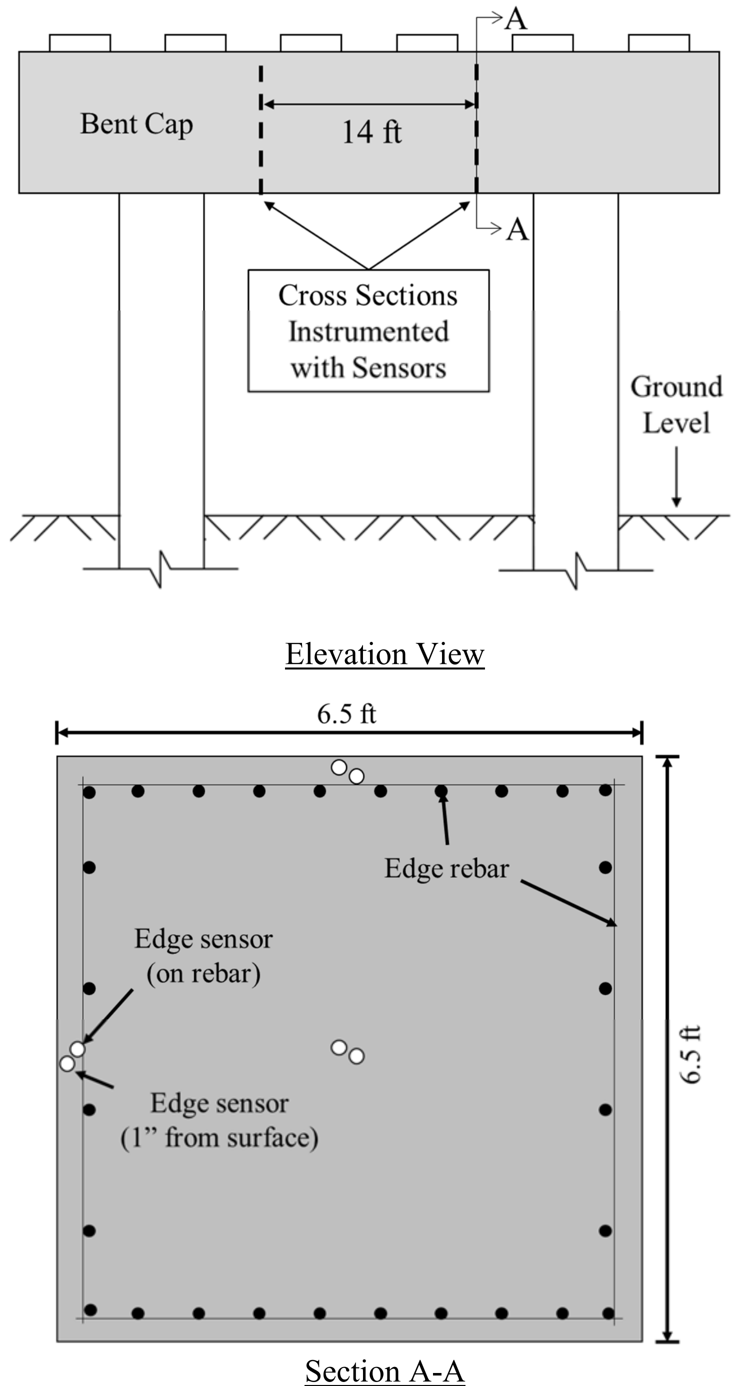


Figure 5-2: Albertville bent cap instrumented cross section locations (top) and temperature sensor locations on these cross sections (bottom)

5.2.3 14-Day Temperature Data

Figure 5-3 and Figure 5-4 show the 14-day temperature data and temperature difference data, respectively.

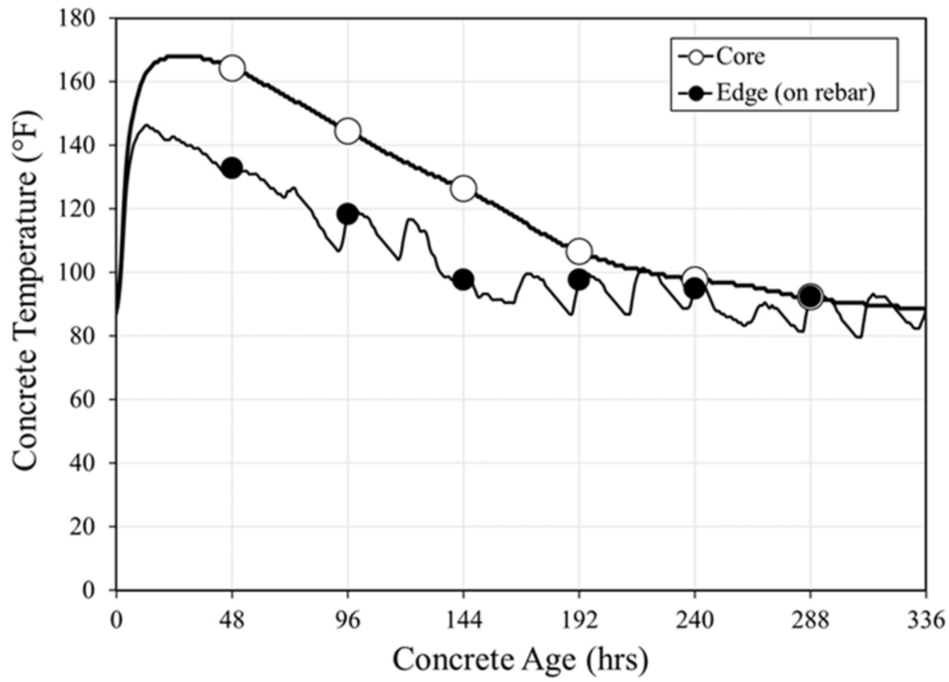


Figure 5-3: Albertville bent cap 14-day temperature data

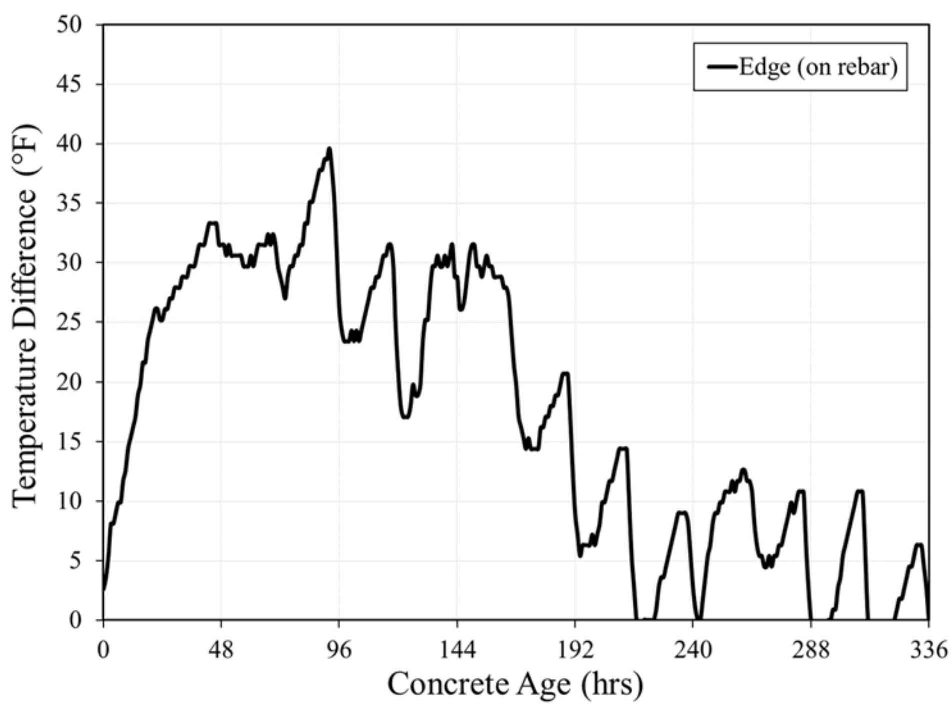


Figure 5-4: Albertville bent cap 14-day temperature difference data

5.2.4 Long-Term Temperature Data

The long-term concrete temperatures recorded for this bent cap are shown in Figure 5-5.

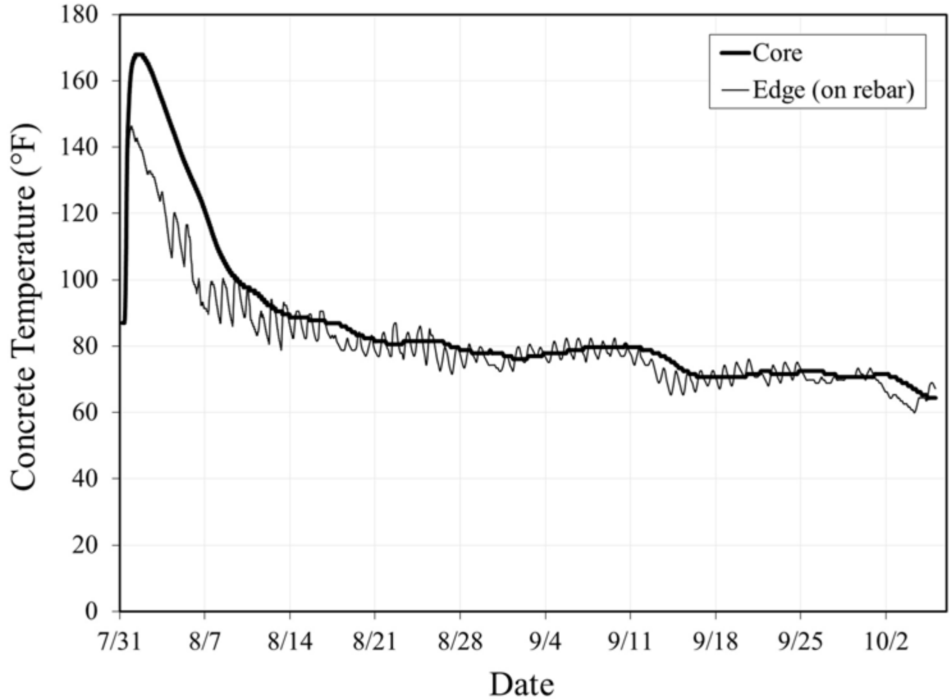


Figure 5-5: Albertville bent cap long-term temperature data

5.2.5 Visual Inspections

Upon returning to the project site on August 14, 2015, researchers found no sign of thermal cracking or DEF. On October 6, 2015, another site visit was made to collect more temperature data, and no signs of distress were found at that time. Because the element would become inaccessible as construction continued, the sensor lead wires were cut at that time.

5.2.6 Bamforth Crack Prediction Results

Table 5-2 contains the results from Bamforth's long-term crack prediction method for the Albertville bent cap. The type of restraint for this member is ends only, because the bent cap is restrained by the two supporting piers. The restraint factor was determined by modeling the field element in SAP 2000 for as-built conditions and then again for full restraint against contraction. For the as-built condition, the supporting circular piers were considered fixed at ground level, which is conservative as they will provide a high restraint condition due to their size. The provided reinforcement ratio refers to all of the longitudinal face reinforcing steel that would cross a vertical crack through the bent cap. The long-term bulk temperature difference refers to the difference between the maximum recorded concrete temperature and the long-term recorded low temperature; however, for this element no data were recorded during the winter months, so a long-term bulk temperature difference was not recorded. The value for long-term crack width is based on Bamforth's prediction method outlined in Section 2.4.2.1.

Table 5-2: Albertville bent cap predicted crack widths based on Bamforth's method

Item	Entry
Type of Restraint	Ends only
Provided reinforcement ratio, ρ (%)	0.55
Restraint factor	0.30
Concrete age at low temperature (days)	DNR
Long-term bulk temperature difference (°F)	DNR
Predicted long-term crack widths (in.)	DNR

Note: DNR = did not record

5.2.7 Results of Laboratory Testing

The results of the Q-drum, CTE, concrete compressive strength, and modulus of elasticity tests performed by Auburn researchers are shown below in Table 5-3.

Table 5-3: Albertville bent cap laboratory testing results

Property	Value
Total Heat of Hydration, H_u (J/kg)	375,000
Activation Energy, E (J/mol)	31,800
Hydration Slope Parameter, β	1.574
Hydration Time Parameter, τ (hr)	15.74
Ultimate Degree of Hydration, α_u	0.846
Concrete CTE (in./in./°F × 10^6)	4.82
28-day Concrete Mean Compressive Strength, f_{cm28} (psi)	5,300
28-day Concrete Modulus of Elasticity, E_{cm28} (psi)	4,750,000

5.3 HARPERSVILLE CRASHWALL

The following section summarizes the work done on the Harpersville crashwall. The mixture proportions, cementing material compositions, weather data, and all temperature profiles for this element may be found in Appendix C.

5.3.1 Project Information

Table 5-4 contains the basic project information for this element. Figure 5-6 shows this element under construction.

Table 5-4: Harpersville crashwall project information

Item	Entry
Placement Date	8/24/2015
Placement Time	10:20 a.m.
Placement Location	Harpersville, Alabama
Member Type	Crashwall
Member Dimensions (ft)	48 × 4 × 10
Cement Type	I/II
Total Cementing Materials Content (pcy)	535
SCM Type (% Replacement)	Class C Fly Ash (20%)
Coarse Aggregate Type	Limestone
Form Type	Wood
Placement Temperature (°F)	85

Figure 5-6: Harpersville crashwall under construction

5.3.2 Sensor Locations

Figure 5-7 shows the approximate temperature sensor cross section locations and the temperature sensor locations within each of these cross sections.

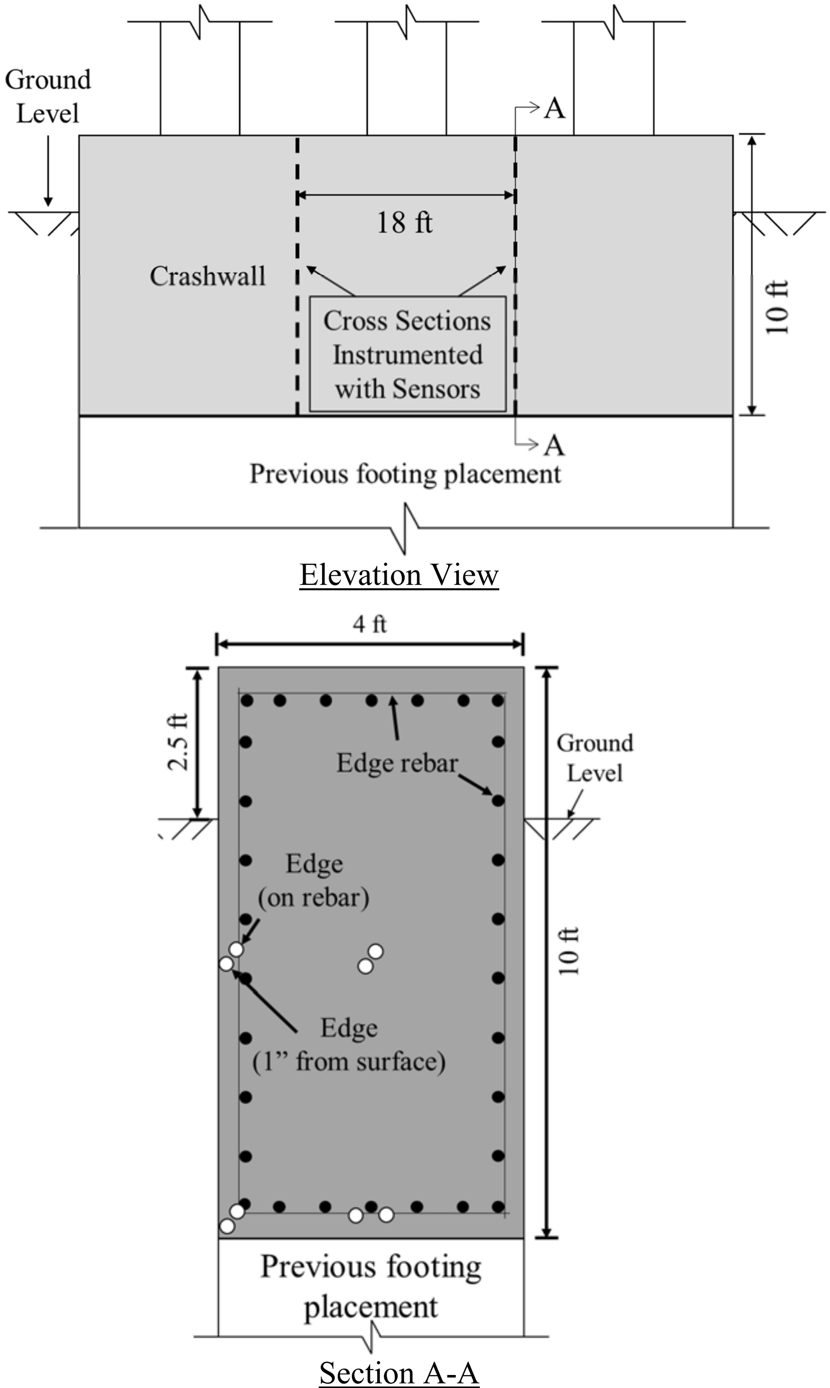


Figure 5-7: Harpersville crashwall instrumented cross section locations (top) and temperature sensor locations on these cross sections (bottom)

5.3.3 14-day Temperature Data

Figure 5-8 and Figure 5-9 show the 14-day temperature data and temperature difference data, respectively.

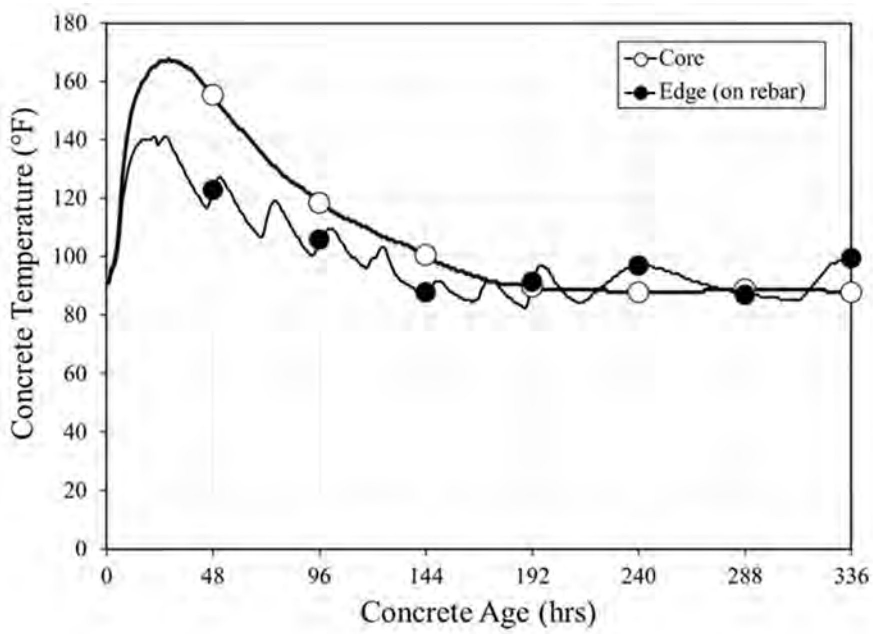


Figure 5-8: Harpersville crashwall 14-day temperature data

Figure 5-9: Harpersville crashwall 14-day temperature difference data

5.3.4 Long-term Temperature Data

The long-term concrete temperatures recorded for this crashwall are shown in Figure 5-10.

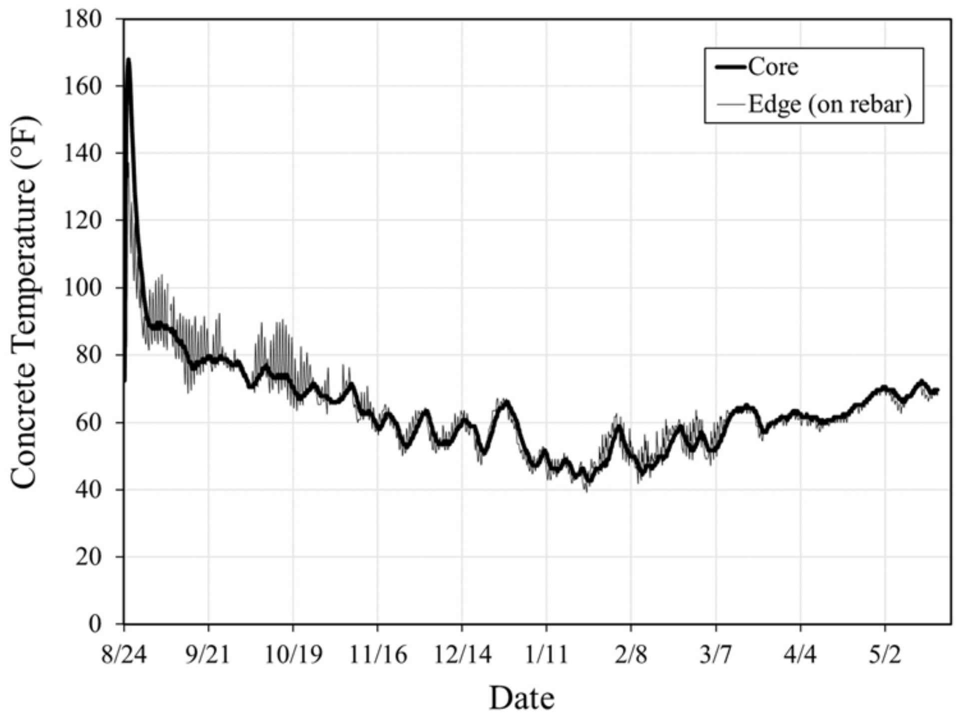


Figure 5-10: Harpersville crashwall long-term temperature data

5.3.5 Visual Inspections

Upon returning to the project site on September 7, 2015, researchers found no sign of thermal cracking or DEF. On January 8, 2016 and May 19, 2016, site visits were made during which researchers discovered one cross section of sensor wires had become inaccessible due to the presence of backfill and riprap at the base of the wall. No signs of distress were found at that time. On January 31, 2017, 526 days after placement, a site visit was made and minor signs of cracking (approx. 0.005 in.) were discovered in the crashwall close to mid-length.

5.3.6 Bamforth Crack Prediction Results

Table 5-5 contains the results from Bamforth's long-term crack prediction method for the Harpersville crashwall. The type of restraint for this member is continuous along its bottom edge, because this wall was cast on the footing that was previously cast. The restraint factor was based on Bamforth's recommendation of 0.5 for walls similar to this case. The provided reinforcement ratio refers to all of the reinforcing steel around the face of the wall that would intersect a potential vertical crack in the wall. The long-term bulk temperature difference refers to the difference between the maximum recorded concrete temperature and the long-term recorded low temperature for this element. The value for long-term crack width is based on Bamforth's prediction method outlined in Section 2.4.2.1. The only observed cracking in this member is near the top of the crashwall and further cracking may have occurred beneath ground surface that could not be observed.

Table 5-5: Harpersville crashwall predicted crack widths based on Bamforth's method

Item	Entry
Type of Restraint	Continuous edge restraint
Provided reinforcement ratio, ρ (%)	0.17
Restraint factor	0.5
Concrete age at low temperature (days)	153
Long-term bulk temperature difference (°F)	125
Predicted long-term crack widths (in.)	0.009

5.3.7 Results of Laboratory Testing

The results of the Q-drum, CTE, concrete compressive strength, and modulus of elasticity tests performed by Auburn researchers are shown below in Table 5-6.

Table 5-6: Harpersville crashwall results of laboratory testing

Property	Value
Total Heat of Hydration, H_u (J/kg)	437,500
Activation Energy, E (J/mol)	34,500
Hydration Slope Parameter, β	1.138
Hydration Time Parameter, τ (hr)	13.69
Ultimate Degree of Hydration, α_u	0.758
Concrete CTE (in./in./°F × 106)	4.47
28-day Concrete Mean Compressive Strength, f_{cm28} (psi)	6,200
28-day Concrete Modulus of Elasticity, E_{cm28} (psi)	6,160,000

5.4 SCOTTSBORO PEDESTAL

The following section summarizes the work done on the Scottsboro pedestal. The mixture proportions, cementing material compositions, weather data, and all temperature profiles for this element may be found in Appendix D.

5.4.1 Project Information

Table 5-7 contains the basic project information for this element. Figure 5-11 shows this element before and after construction. The concrete cover between the side rebar and the formwork is approximately six inches as compared to the two to three inches that was used for the other instrumented projects. This pedestal was placed in three lifts, and the dark grey line in the right picture of Figure 5-11 is the construction joint between lifts two and three.

Table 5-7: Scottsboro pedestal project information

Item	Entry
Placement Date	9/3/2015
Placement Time	10:20 a.m. – 3:55 p.m.
Placement Location	Scottsboro, Alabama
Member Type	Pedestal
Member Dimensions (ft)	10 × 12.5 × 34
Cement Type	I/II
Total Cementing Materials Content (pcy)	620
SCM Type (% Replacement)	Class F Fly Ash (20%)
Coarse Aggregate Type	Limestone
Form Type	Steel
Placement Temperature (°F)	95

Figure 5-11: Scottsboro pedestal prior to placement (top) and post construction (bottom)

5.4.2 Sensor Locations

Figure 5-12 shows the approximate temperature sensor cross section locations and the temperature sensor locations within each of these cross sections.

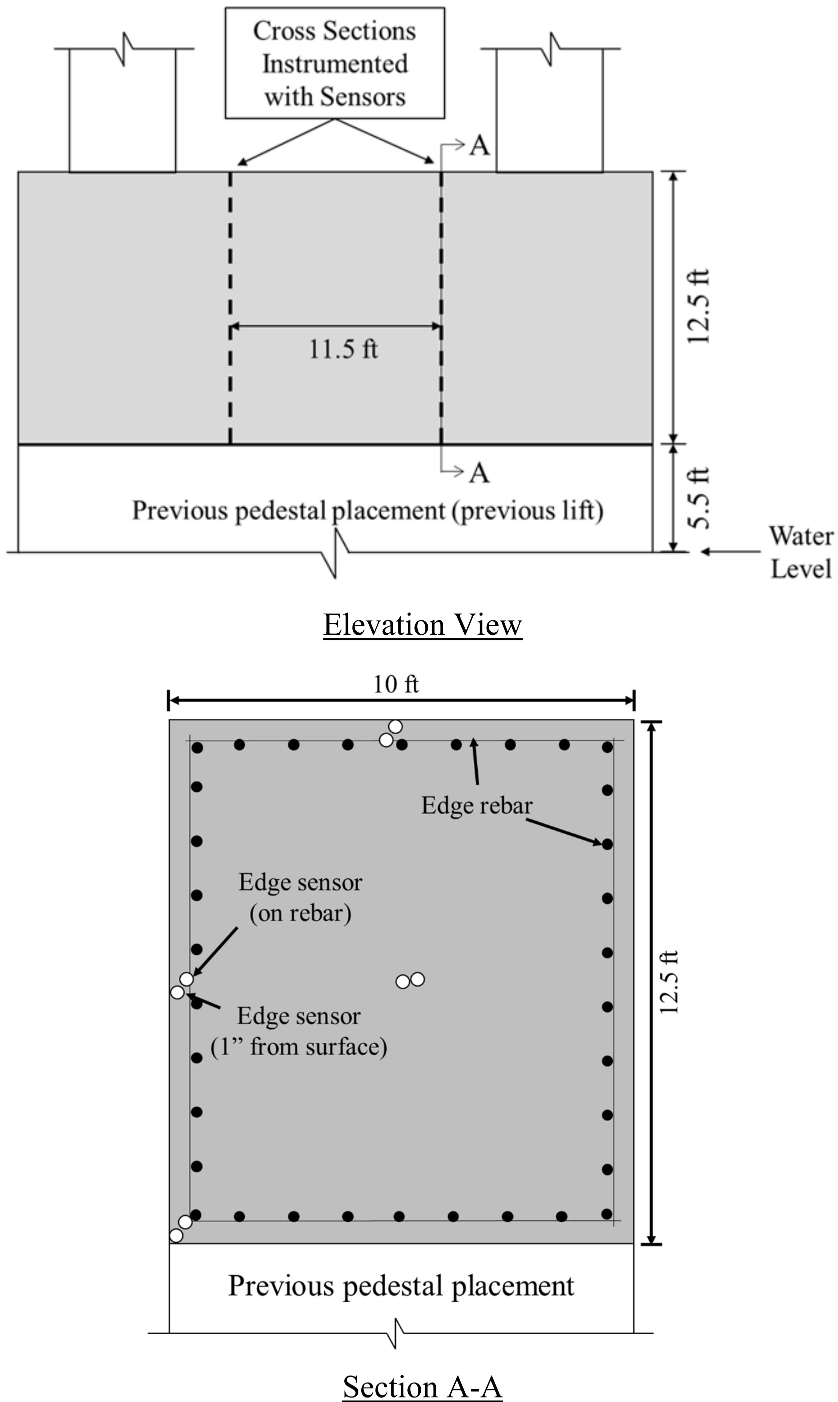


Figure 5-12: Scottsboro pedestal instrumented cross section locations (top) and temperature sensor locations on these cross sections (bottom)

5.4.3 14-day Temperature Data

Figure 5-13 and Figure 5-14 show the 14-day temperature data and temperature difference data, respectively.

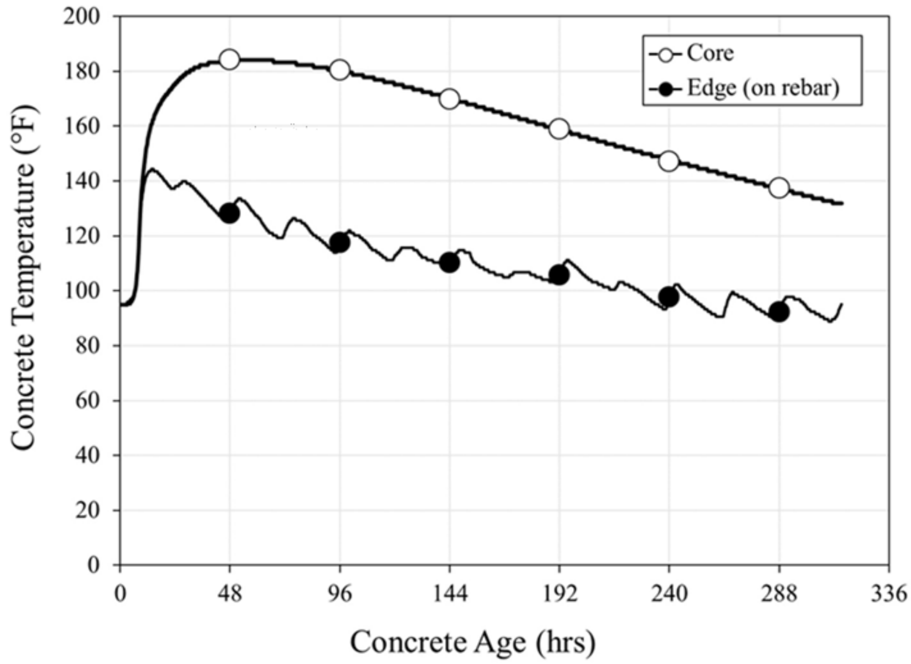


Figure 5-13: Scottsboro pedestal 14-day temperature data

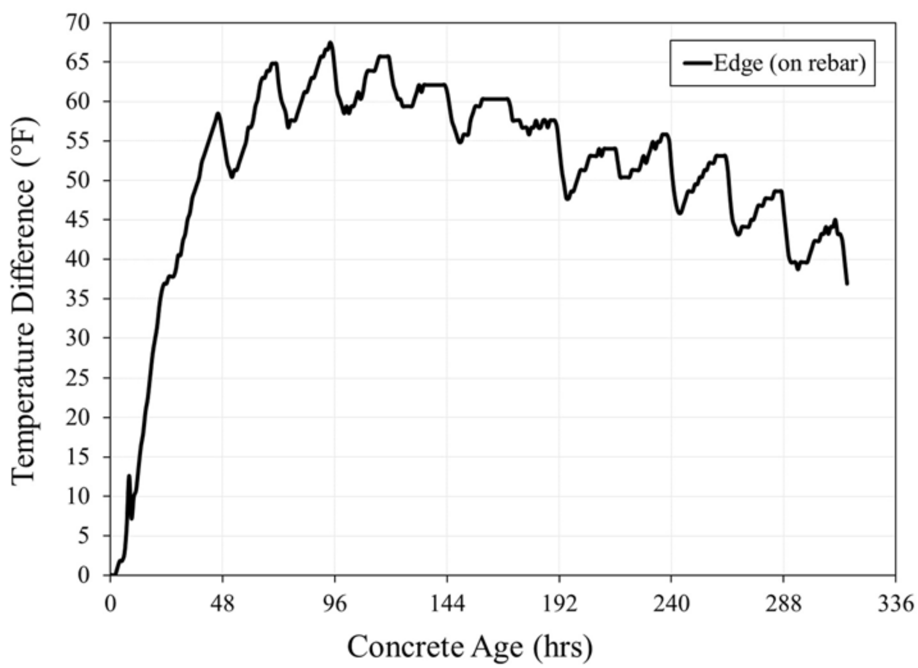


Figure 5-14: Scottsboro pedestal 14-day temperature difference data

5.4.4 Long-term Temperature Data

No long-term data were obtained for this element. Upon returning to the project site after construction it was discovered the sensor wires had been cut. After construction ended the lead wire location was inaccessible.

5.4.5 Visual Inspections

Upon returning to the project site on September 16, 2015, researchers found no sign of thermal cracking or DEF. Upon returning to the project site on May 19, 2016, researchers discovered the lead wires had been cut sometime during construction. No signs of thermal cracking or DEF were found. On February 9, 2017, 525 days after placement, another site visit was made by canoe to perform a visual inspection, and signs of thermal cracking were discovered at this time.

The instrumented element is approximately five feet above the water level. Because the inspection was made from a canoe, this made inspection of the instrumented element impossible. Instead, researchers were able to examine the top of the previously placed pedestal section. It is on this section where the documented thermal cracks were located. Refer to Figure 5-17 for approximate locations of the documented cracks. Cracks 1 and 2 are located on the north face of the pedestal at the indicated locations. Cracks 3 and 4 are located on the south face of the pedestal at the same indicated locations

Figure 5-15 through Figure 5-19 document these cracks that were found on the Scottsboro pedestal. The cracks found on the second lift stopped before reaching the surface of the water below, and the cracks did not continue up into the third lift that was instrumented for research. The largest found width and average width of each crack may be found in Table 5-8. The absolute largest crack width measured was approximately 0.022 in., which exceeds 0.012 in. that is the goal for elements with this type of exposure condition.

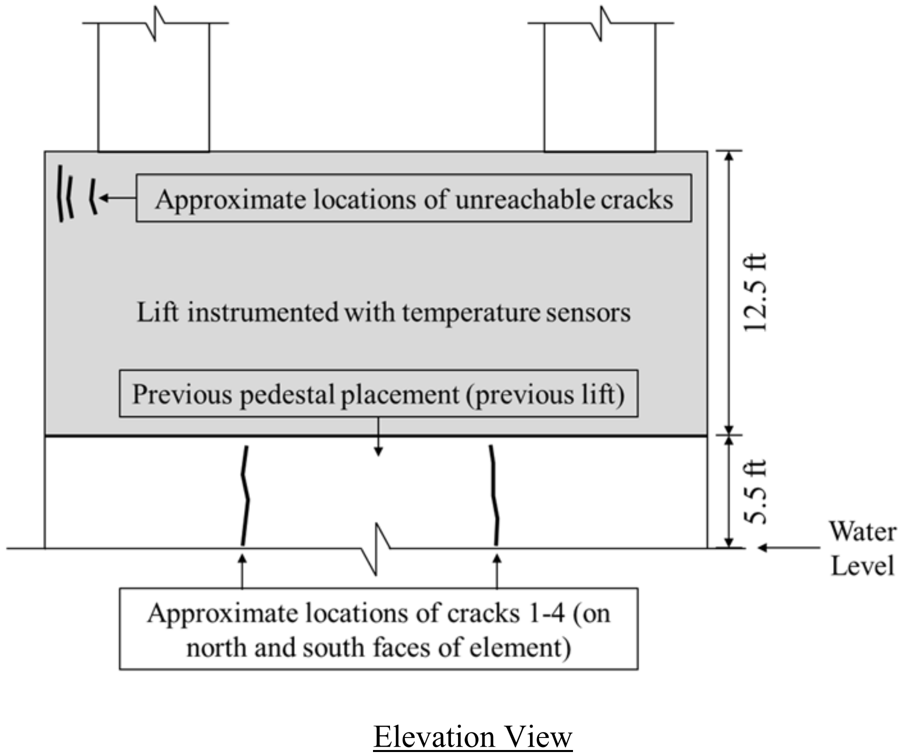


Figure 5-15: Approximate locations of all cracks on Scottsboro pedestal

Figure 5-16: Scottsboro pedestal and bent cap on day of inspection on 2/9/2017

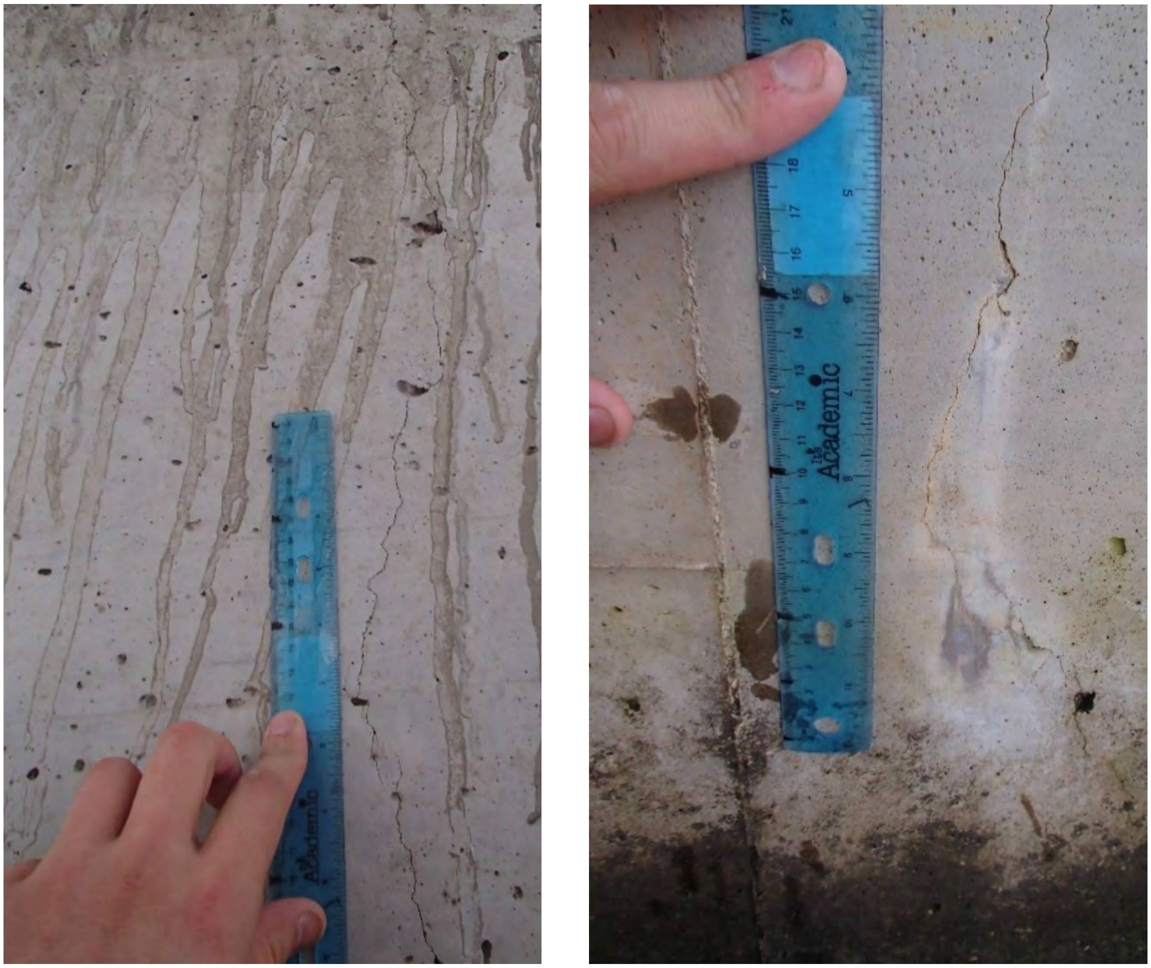


Figure 5-17: Cracks in Scottsboro pedestal on day of inspection on 2/9/2017

Figure 5-18: Scottsboro pedestal cracks observed at top of pedestal (unreachable)

Figure 5-19: Scottsboro pedestal close-up of cracks observed at top of pedestal (unreachable)

Table 5-8: Scottsboro pedestal crack width measurements

Crack No.	Measurement Type	Value
Crack 1	Largest width	0.022 in.
	Average width	0.016 in.
Crack 2	Largest width	0.021 in.
	Average width	0.016 in.
Crack 3	Largest width	0.018 in.
	Average width	0.013 in.
Crack 4	Largest width	0.013 in.
	Average width	0.010 in.

5.4.6 Bamforth Crack Prediction Results

Table 5-9 contains the results from Bamforth's long-term crack prediction method for the Scottsboro pedestal. The type of restraint for this member is continuous edge restraint along the bottom edge. The restraint factor was calculated using the method developed by Bamforth (2007), which is based on ACI 207.2R (2007). The provided reinforcement ratio refers to all of the circular ties around the face of the pedestal that would intersect a potential vertical crack in the pedestal. The long-term bulk temperature difference refers to the difference between the maximum recorded concrete temperature and the long-term recorded low temperature; however, for this element no data were recorded during the winter months so a long-term bulk temperature difference was not recorded. The value for long-term crack width is based on Bamforth's prediction method outlined in Section 2.4.2.1. Because the instrumented lift of the Scottsboro pedestal was inaccessible during the visual inspection, the crack widths gathered from the second lift may not reflect the crack predictions that were based on the temperature profile of the third lift.

Table 5-9: Scottsboro pedestal predicted crack widths based on Bamforth's method

Item	Entry
Type of Restraint	Continuous edge restraint
Provided reinforcement ratio, ρ *(%)*	0.04
Restraint factor	0.8
Concrete age at low temperature (days)	DNR
Long-term bulk temperature difference (°F)	DNR
Predicted long-term crack widths (in.)	DNR

Note: DNR = did not record

5.4.7 Results of Laboratory Testing

The results of the Q-drum, CTE, concrete compressive strength, and modulus of elasticity tests performed by Auburn researchers are shown below in Table 5-10.

Table 5-10: Scottsboro pedestal results of laboratory testing

Property	Value
Total Heat of Hydration, H_u (J/kg)	391,000
Activation Energy, E (J/mol)	34,600
Hydration Slope Parameter, β	1.598
Hydration Time Parameter, τ	15.29
Ultimate Degree of Hydration, α_u	0.786
Concrete CTE (in./in./°F × 10^6)	4.04
28-day Concrete Mean Compressive Strength, f_{cm28} (psi)	6,000
28-day Concrete Modulus of Elasticity, E_{cm28} (psi)	5,400,000

5.5 SCOTTSBORO BENT CAP

The following section summarizes the work done on the Scottsboro bent cap. The mixture proportions, cementing material compositions, weather data, and all temperature profiles for this element may be found in Appendix E.

5.5.1 Project Information

Table 5-11 contains the basic project information for this element. Figure 5-20 shows this element under construction.

Table 5-11: Scottsboro bent cap project information

Item	Entry
Placement Date	9/18/2015
Placement Time	11:00 a.m.
Placement Location	Scottsboro, Alabama
Member Type	Rectangular bent cap
Member Dimensions (ft)	6.5 × 7.5 × 41
Cement Type	I/II
Total Cementing Materials Content (pcy)	620
SCM Type (% Replacement)	Class F Fly Ash (20%)
Coarse Aggregate Type	Limestone
Form Type	Steel
Placement Temperature (°F)	87

Figure 5-20: Scottsboro bent cap under construction

5.5.2 Sensor Locations

Figure 5-21 shows the approximate temperature sensor cross section locations and the temperature sensor locations within each of these cross sections.

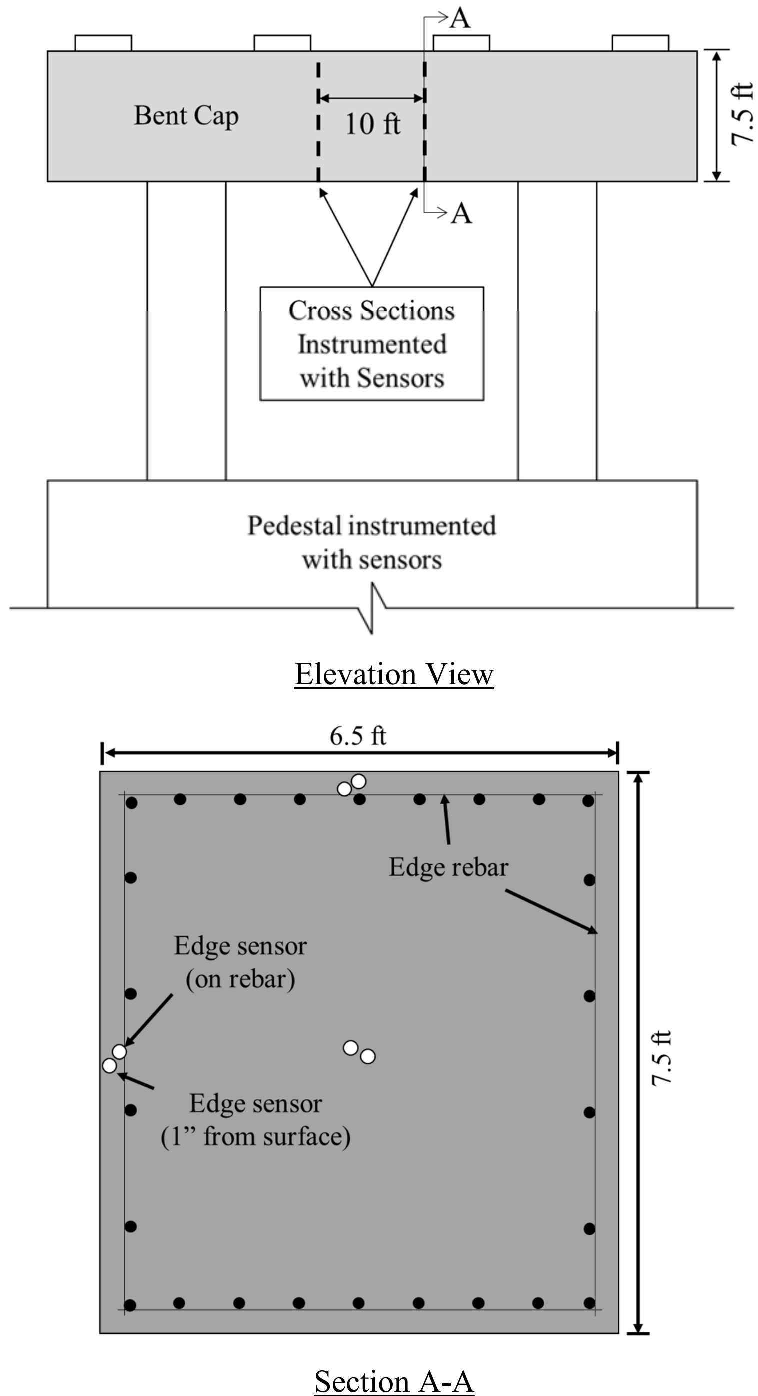


Figure 5-21: Scottsboro bent cap instrumented cross section locations (top) and temperature sensor locations on these cross sections (bottom)

5.5.3 14-day Temperature Data

Figure 5-22 and Figure 5-23 show the 14-day temperature data and temperature difference data, respectively.

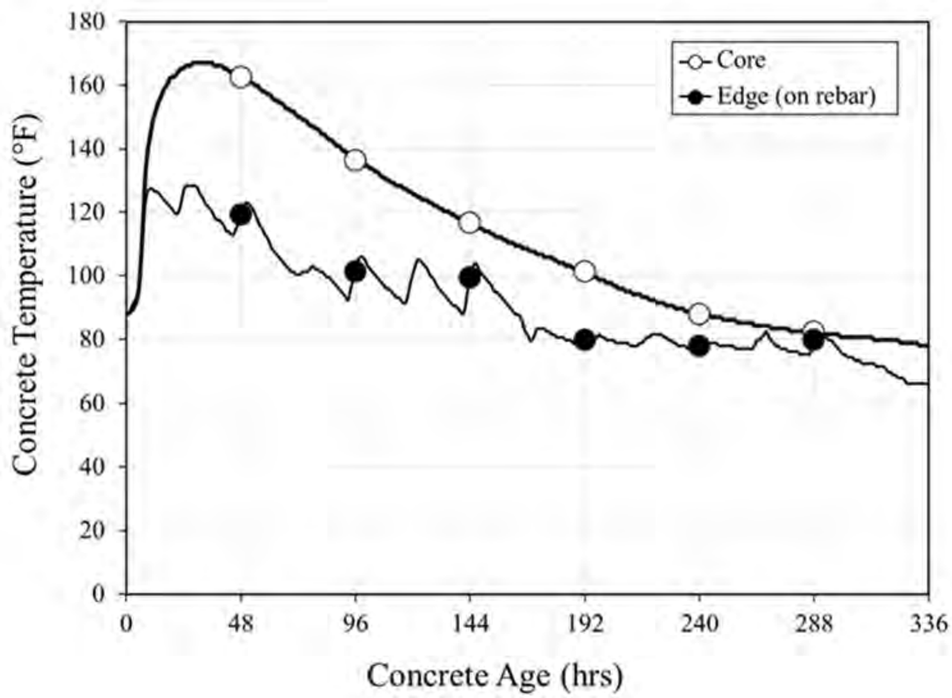


Figure 5-22: Scottsboro bent cap 14-day temperature data

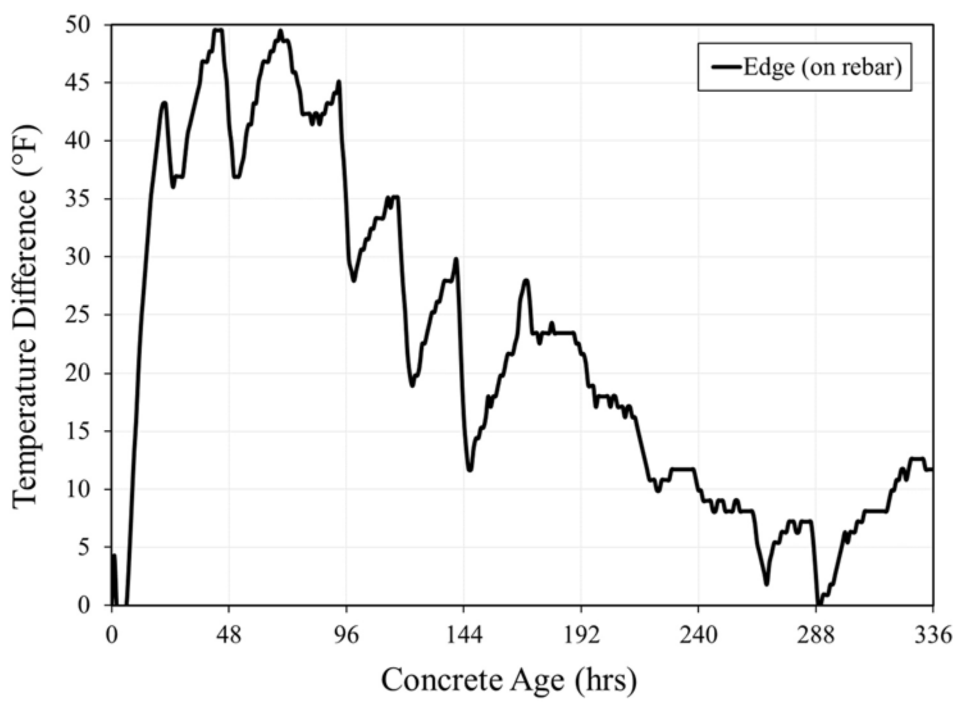


Figure 5-23: Scottsboro bent cap 14-day temperature difference data

5.5.4 Long-term Temperature Data

No long-term temperature data were collected due to the temperature sensor lead wires being cut sometime during construction.

5.5.5 Visual Inspections

A site visit took place on October 6, 2015, fourteen days after construction. No signs of thermal cracking were observed at that time. An additional site visit took place on May 19, 2016, and again no distresses were discovered. At this time it was discovered that the temperature sensor lead wires had been cut by construction workers. Because of this and the difficulty in accessing the bent cap, no further site visits were conducted to this bent cap.

5.5.6 Bamforth Crack Prediction Results

Table 5-12 contains the results from Bamforth's long-term crack prediction method for the Scottsboro bent cap. The type of restraint for this member is ends only, because the bent cap is restrained by the two supporting piers. The restraint factor was determined by modeling the field element in SAP 2000 for as-built conditions and then again for full restraint against contraction. For the as-built condition, the supporting circular piers were considered fixed at ground level, which is conservative as they will provide a high restraint condition due to their size. The provided reinforcement ratio refers to all of the longitudinal reinforcement that crosses a potential vertical crack. The long-term bulk temperature difference refers to the difference between the maximum recorded concrete temperature and the long-term recorded low temperature for this element. For this element no long-term data were recorded during the winter months, so a long-term bulk temperature difference was not recorded. The value for long-term crack width is based on Bamforth's prediction method outlined in Section 2.4.2.1.

Table 5-12: Scottsboro bent cap predicted crack widths based on Bamforth's method

Item	Entry
Type of Restraint	Ends only
Provided reinforcement ratio, ρ (%)	0.56
Restraint factor	0.3
Concrete age at low temperature(days)	DNR
Long-term bulk temperature difference (°F)	DNR
Predicted long-term crack widths (in.)	DNR

Note: DNR = did not record

5.5.7 Results of Laboratory Testing

The results of the Q-drum, CTE, concrete compressive strength, and modulus of elasticity tests performed by Auburn researchers are shown below in Table 5-13.

Table 5-13: Scottsboro bent cap laboratory testing results

Property	Value
Total Heat of Hydration, H_u (J/kg)	391,000
Activation Energy, E (J/mol)	34,600
Hydration Slope Parameter, β	1.598
Hydration Time Parameter, τ (hr)	15.29
Ultimate Degree of Hydration, α_u	0.786
Concrete CTE (in./in./°F × 10^6)	4.04
28-day Concrete Mean Compressive Strength, f_{cm28} (psi)	6,000
28-day Concrete Modulus of Elasticity, E_{cm28} (psi)	5,400,000

5.6 ELBA BENT CAP

The following section summarizes the work done on the Elba bent cap. The mixture proportions, cementing material compositions, weather data, and all temperature profiles for this element may be found in Appendix F.

5.6.1 Project Information

Table 5-14 contains the basic project information for this element. Figure 5-24 shows this element under construction.

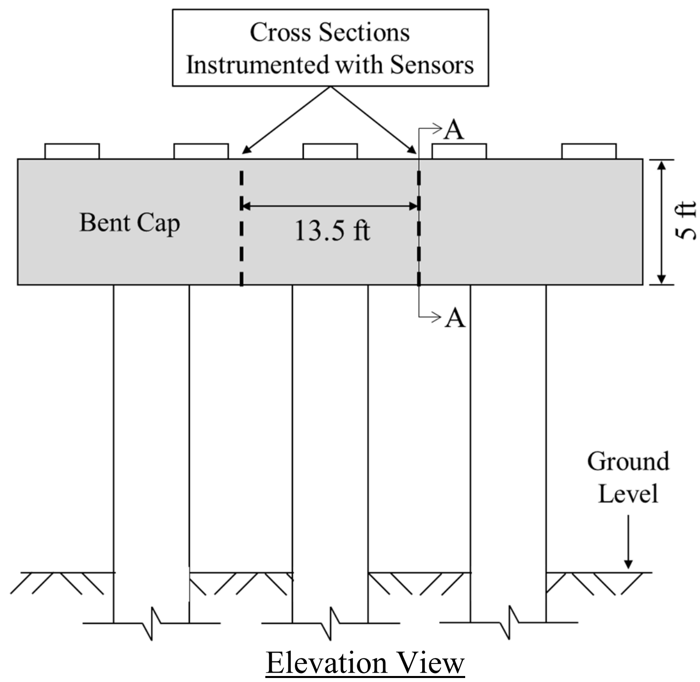
Table 5-14: Elba bent cap project information

Item	Entry
Placement Date	12/18/2015
Placement Time	11:00 a.m.
Placement Location	Elba, Alabama
Member Type	Rectangular bent cap
Member Dimensions (ft)	5 × 5.5 × 42
Cement Type	I/II
Total Cementing Materials Content (pcy)	550
SCM Type (% Replacement)	Class F Fly Ash (20%)
Coarse Aggregate Type	#57/#67 River Gravel
Form Type	Wood
Placement Temperature (°F)	72

Figure 5-24: Elba bent cap during construction (top) and post construction (bottom)

5.6.2 Sensor Locations

Figure 5-25 shows the approximate temperature sensor cross section locations and the temperature sensor locations within each of these cross sections.



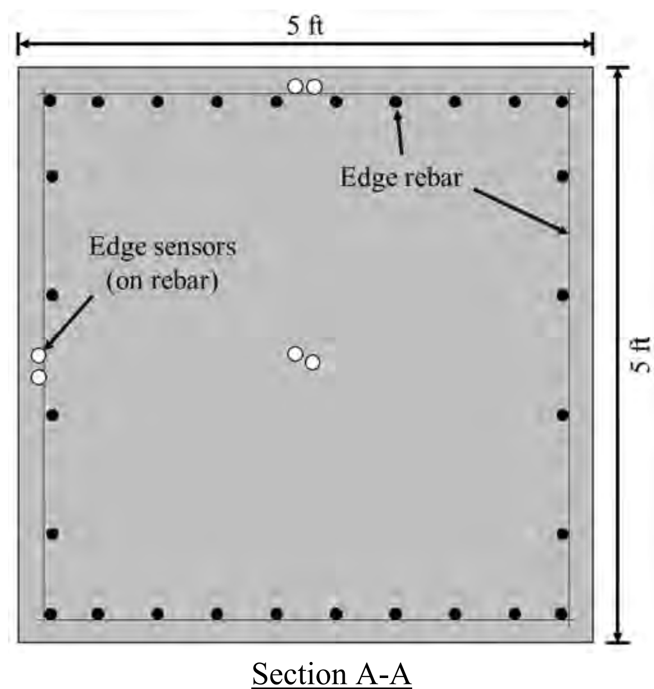


Figure 5-25: Elba bent cap instrumented cross section locations (top) and temperature sensor locations on these cross sections (bottom)

5.6.3 14-day Temperature Data

Figure 5-26 and Figure 5-27 show the 14-day temperature data and temperature difference data, respectively.

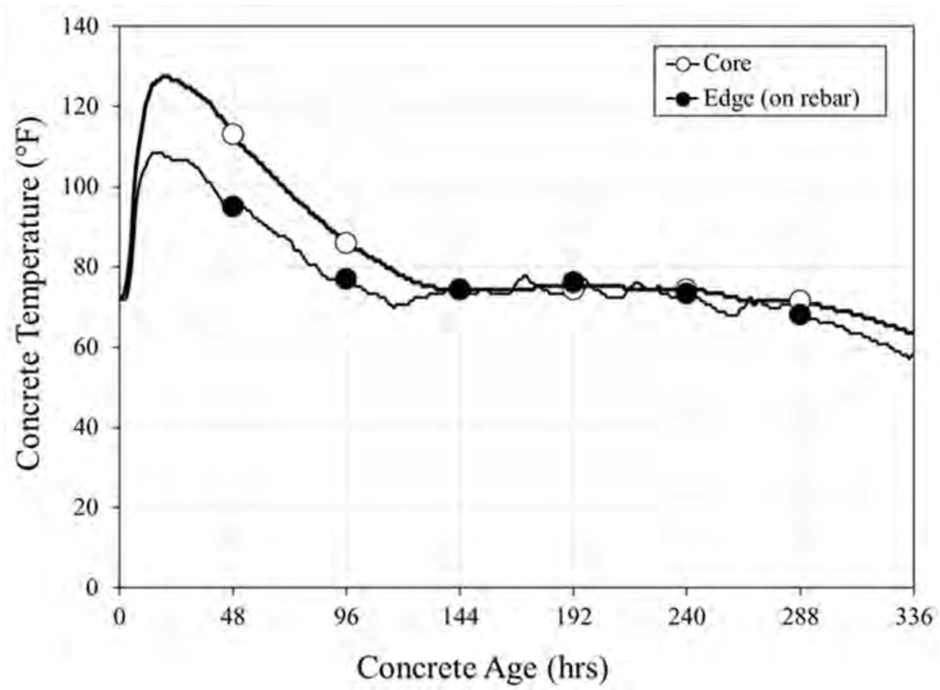


Figure 5-26: Elba bent cap 14-day temperature data

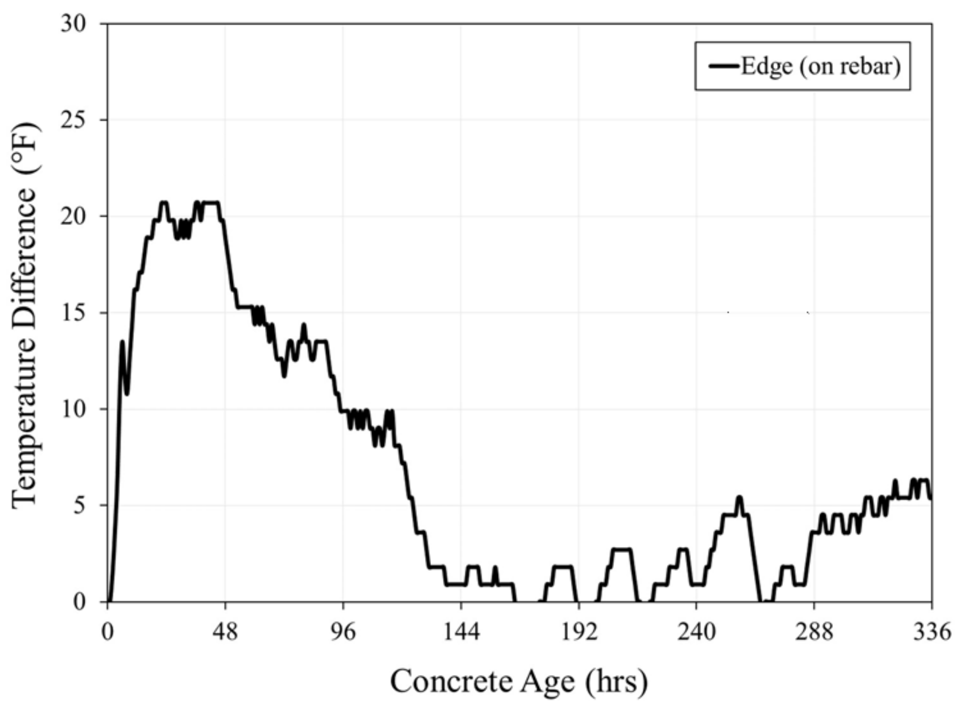


Figure 5-27: Elba bent cap 14-day temperature difference data

5.6.4 Long-term Temperature Data

The long-term concrete temperatures recorded for this bent cap are shown in Figure 5-28.

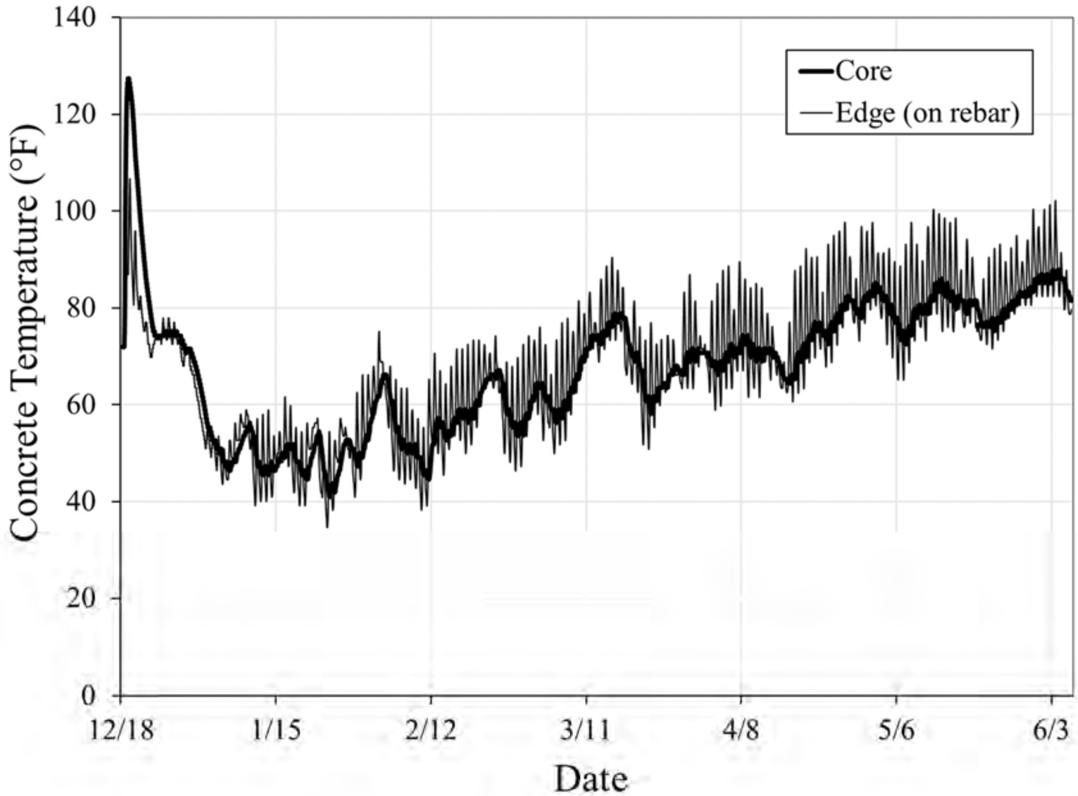


Figure 5-28: Elba bent cap long-term temperature data

5.6.5 Visual Inspections

A site visit took place on January 4, 2015, fourteen days after construction. No signs of thermal cracking were observed at that time. A second site visit took place on June 6, 2016, and again no distresses were discovered. A third site visit took place on March 16, 2016, and again no distresses were discovered. On this third visit the temperature sensors were unresponsive and no data were recovered. It is unknown why the sensors malfunctioned, but because access would become difficult in the future, the sensor lead wires were cut at this time.

5.6.6 Bamforth Crack Prediction Results

Table 5-15 contains the results from Bamforth's long-term crack prediction method for the Elba bent cap. The type of restraint for this member is ends only, because the bent cap is restrained by the two supporting piers. The restraint factor was determined by modeling the field element in SAP

2000 for as-built conditions and then again for full restraint against contraction. For the as-built condition, the supporting circular piers were considered fixed at ground level, which is conservative as they will provide a high restraint condition due to their size. The long-term bulk temperature difference refers to the difference between the maximum recorded concrete temperature and the long-term recorded low temperature for this element. Since the Elba bent cap was poured in the winter, the concrete age at which the recorded low temperature occurred is only 37 days. This temperature was assumed to be the long-term low temperature, and calculations were performed assuming this temperature occurred later in time. The provided reinforcement ratio refers to all of the longitudinal reinforcement that crosses the cross section of a potential vertical crack. The value long-term crack width is based on Bamforth's prediction method outlined in Section 2.4.2.1.

Table 5-15: Elba bent cap predicted crack widths based on Bamforth's method

Item	Entry
Type of Restraint	Ends only
Provided reinforcement ratio, ρ (%)	0.95
Restraint factor	0.3
Concrete age at low temperature (days)	37
Long-term bulk temperature difference (°F)	86
Predicted long-term crack widths (in.)	0.002

5.6.7 Results of Laboratory Testing

The results of the Q-drum, CTE, concrete compressive strength, and modulus of elasticity tests performed by Auburn researchers are shown below in Table 5-16.

Table 5-16: Elba bent cap laboratory testing results

Property	Value
Total Heat of Hydration, H_u (J/kg)	411,000
Activation Energy, E (J/mol)	34,700
Hydration Slope Parameter, β	1.035
Hydration Time Parameter, τ (hr)	11.67
Ultimate Degree of Hydration, α_u	0.794
Concrete CTE (in./in./°F × 10^6)	6.39
28-day Concrete Mean Compressive Strength, f_{cm28} (psi)	4,700
28-day Concrete Modulus of Elasticity, E_{cm28} (psi)	3,950,000

5.7 BIRMINGHAM COLUMN

The following section summarizes the work done on the Birmingham column. The mixture proportions, cementing material compositions, weather data, and all temperature profiles for this element may be found in Appendix G.

5.7.1 Project Information

Table 5-17 contains the basic project information for this element. Figure 5-29 shows this element post construction.

Table 5-17: Birmingham column project information

Item	Entry
Placement Date	1/21/2016
Placement Time	9:55 a.m. – 11:20 a.m.
Placement Location	Birmingham, Alabama
Member Type	Rectangular Column
Member Dimensions (ft)	4.5 × 4.5 × 20
Cement Type	I/II
Total Cementing Materials Content (pcy)	600
SCM Type (% Replacement)	Class C Fly Ash (20%)
Coarse Aggregate Type	Limestone
Form Type	Wood
Placement Temperature (°F)	61

Figure 5-29: Birmingham column post construction with sensor lead wires visible

5.7.2 Sensor Locations

Figure 5-30 shows the approximate temperature sensor cross section locations and the temperature sensor locations within each of these cross sections.

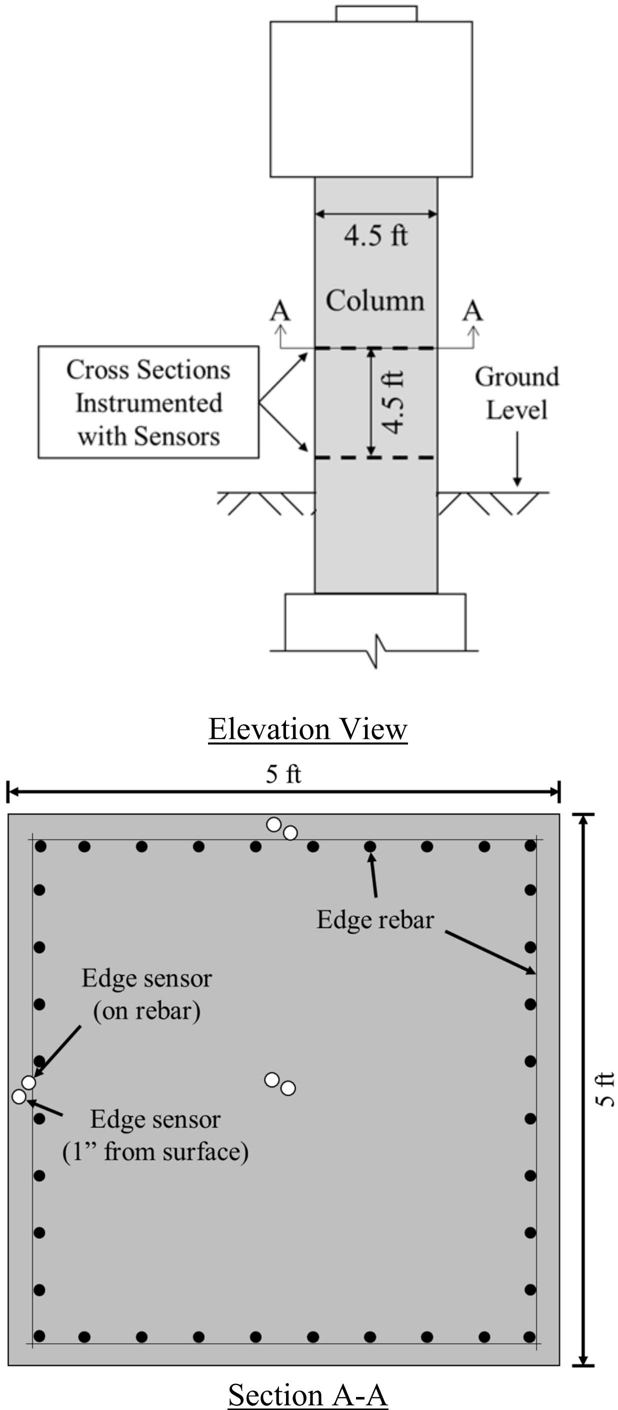


Figure 5-30: Birmingham column instrumented cross section locations (top) and temperature sensor locations on these cross sections (bottom)

5.7.3 14-day Temperature Data

Figure 5-31 and Figure 5-32 show the 14-day temperature data and temperature difference data, respectively.

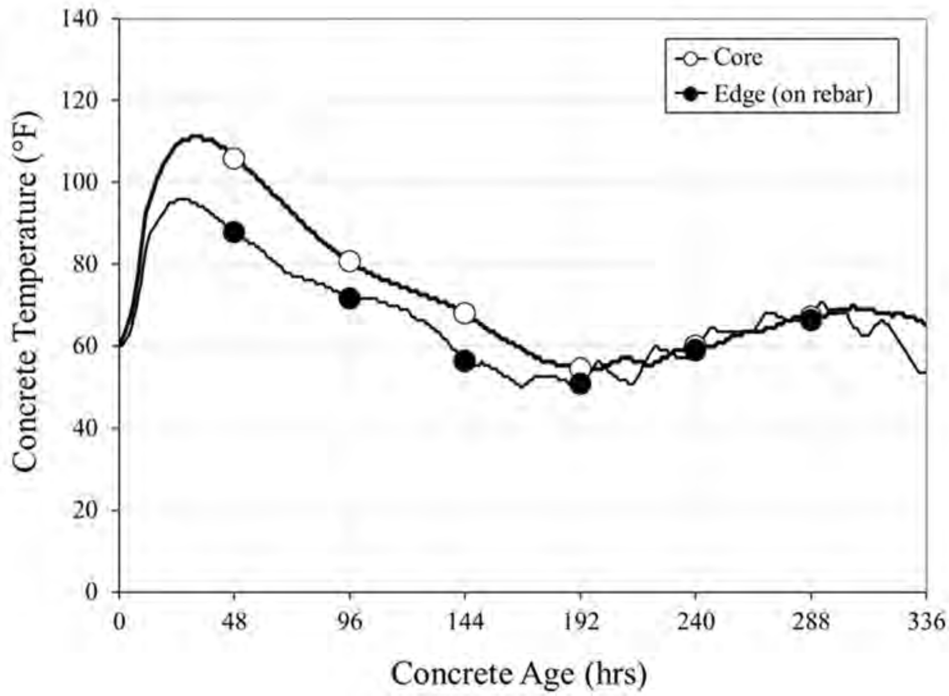


Figure 5-31: Birmingham column 14-day temperature data

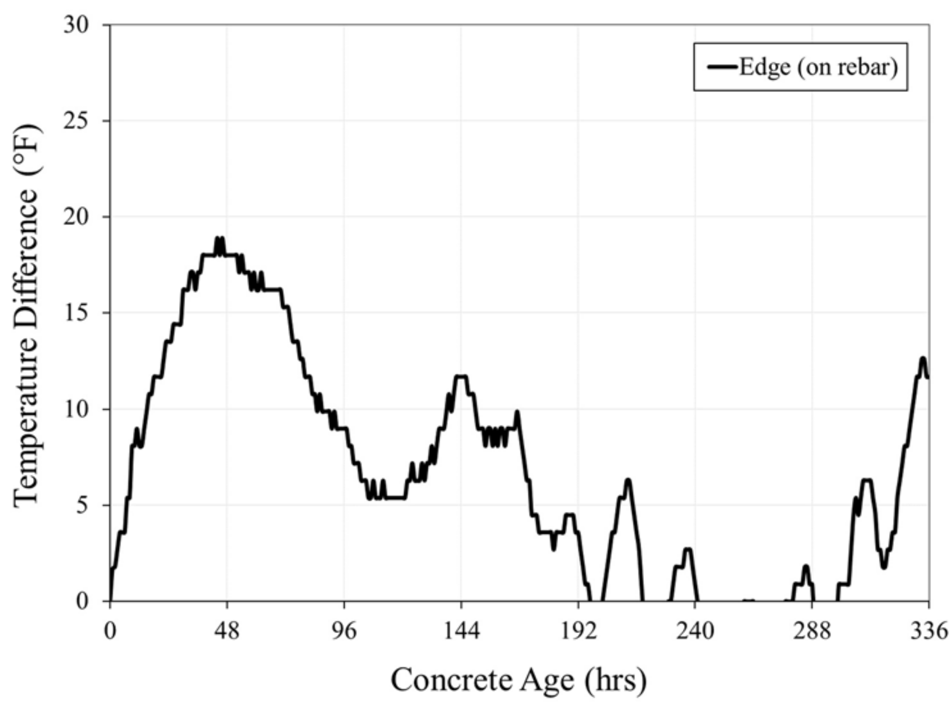


Figure 5-32: Birmingham column 14-day temperature difference data

5.7.4 Long-term Temperature Data

The long-term concrete temperatures recorded for this column are shown in Figure 5-33.

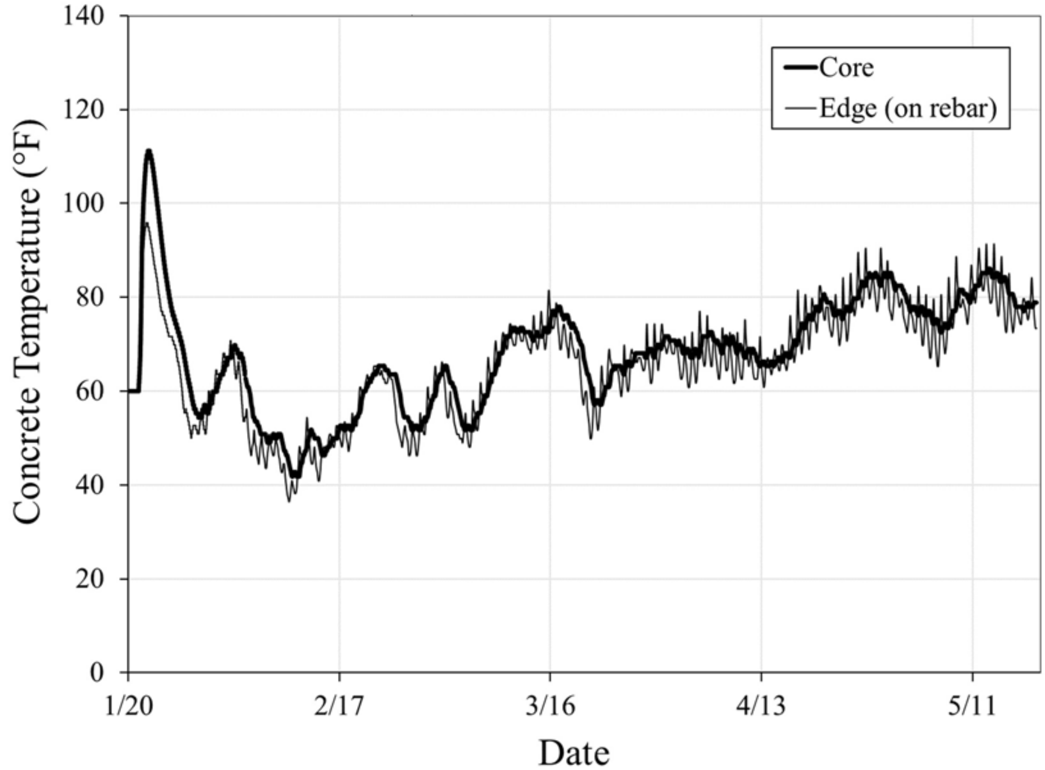


Figure 5-33: Birmingham column long-term temperature data

5.7.5 Visual Inspections

A site visit took place on February 4, 2016, 14 days after construction. No signs of thermal cracking were observed at that time. A second site visit was conducted on May 19, 2016, and again no distresses were discovered. A third site visit took place on January 31, 2017, and again no distresses were discovered. At this time it was discovered that the temperature sensor lead wires had been cut by construction workers. Because of this no further site visits are recommended.

5.7.6 Bamforth Crack Prediction Results

Table 5-18 contains the results from Bamforth's long-term crack prediction method for the Birmingham column. The worst-case restraint for this member is continuous along its bottom edge, where it is cast on a footing. This classification was used because shrinkage and temperature cracking was expected to appear vertically, perpendicular to the circular tie reinforcement.

Bamforth's restraint level of 0.7 was used for this condition because of the high restraint at the footing. The provided reinforcement ratio refers to all of the ties around the face of the column that would intersect a potential vertical crack in the column. The long-term bulk temperature difference refers to the difference between the maximum recorded concrete temperature and the long-term recorded low temperature for this element. Since the Birmingham column was poured in the winter, the concrete age at which the recorded low temperature occurred is only 20 days. This temperature was assumed to be the long-term low temperature, and calculations were performed assuming this temperature occurred later in time. The value for long-term crack width is based on Bamforth's prediction method outlined in Section 2.4.2.1. Because no signs of distress were discovered on the Birmingham column, there are no field data with which to compare these crack width predictions.

Table 5-18: Birmingham column predicted crack widths based on Bamforth's method

Item	Entry
Type of Restraint	Continuous edge
Provided reinforcement ratio, ρ (%)	0.19
Restraint factor	1.0
Concrete age at low temperature (days)	20
Long-term bulk temperature difference (°F)	69
Predicted long-term crack widths (in.)	0.006

5.7.7 Results of Laboratory Testing

The results of the Q-drum, CTE, concrete compressive strength, and modulus of elasticity tests performed by Auburn researchers are shown below in Table 5-19.

Table 5-19: Birmingham column laboratory testing results

Property	Value
Total Heat of Hydration, H_u (J/kg)	451,000
Activation Energy, E (J/mol)	34,700
Hydration Slope Parameter, β	0.948
Hydration Time Parameter, τ (hr)	13.01
Ultimate Degree of Hydration, α_u	0.786
Concrete CTE (in./in./°F × 10^6)	5.49
28-day Concrete Mean Compressive Strength, f_{cm28} (psi)	7,000
28-day Concrete Modulus of Elasticity, E_{cm28} (psi)	6,850,000

5.8 BREWTON BENT CAP

The following section summarizes the work done on the Brewton bent cap. The mixture proportions, cementing material compositions, weather data, and all temperature profiles for this element may be found in Appendix H.

5.8.1 Project Information

Table 5-20 contains the basic project information for this element. Figure 5-34 shows this element during construction.

Table 5-20: Brewton bent cap project information

Item	Entry
Placement Date	7/15//2016
Placement Time	6:00 a.m. – 10:20 a.m.
Placement Location	Brewton, Alabama
Member Type	Rectangular Bent Cap
Member Dimensions (ft)	6.0 × 6.5 × 45
Cement Type	I/II
Total Cementing Materials Content (pcy)	534
SCM Type (% Replacement)	Class F Fly Ash (20%)
Coarse Aggregate Type	Gravel
Form Type	Wood
Placement Temperature (°F)	74

Figure 5-34: Brewton bent cap during bridge construction

5.8.2 Sensor Locations

Figure 5-35 shows the approximate temperature sensor cross section locations and the temperature sensor locations within each of these cross sections.

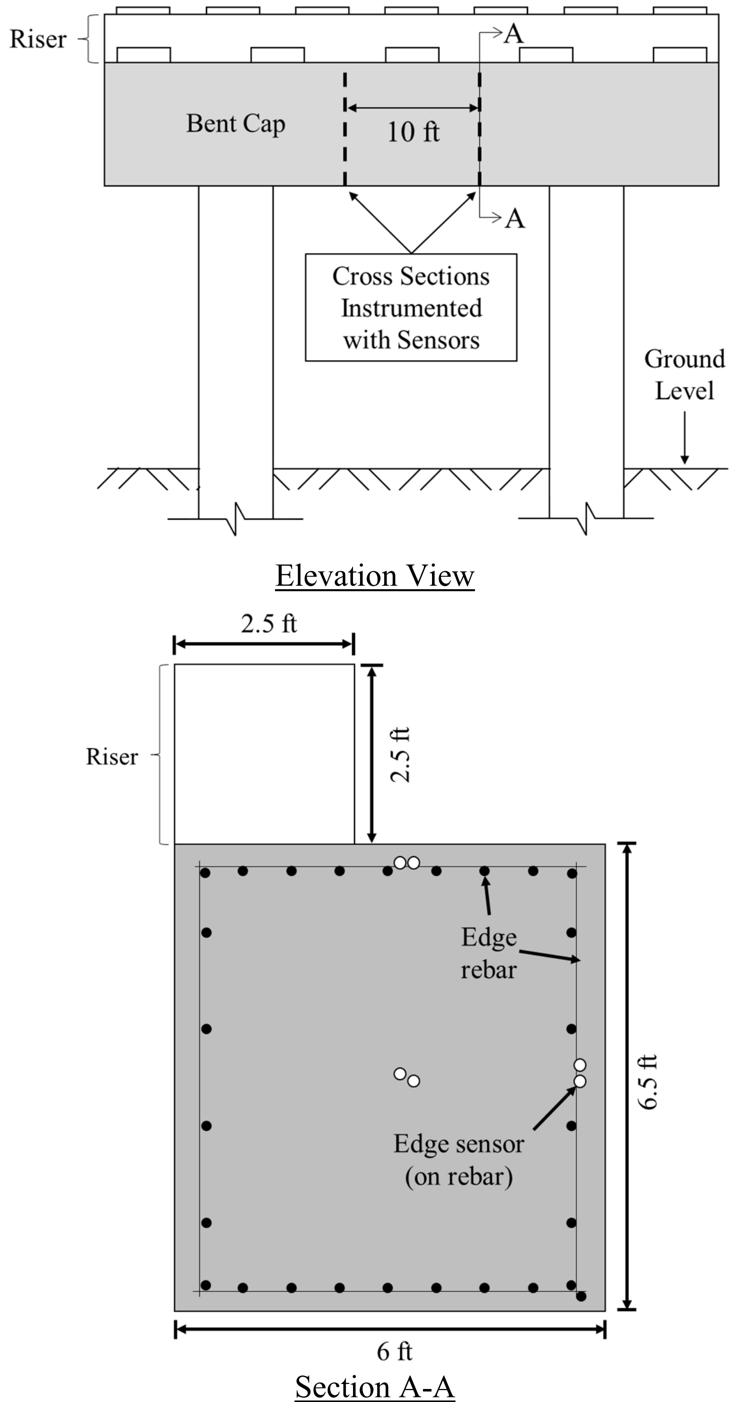


Figure 5-35: Brewton bent cap instrumented cross-section locations (top) and temperature sensor locations on these cross sections (bottom)

5.8.3 14-day Temperature Data

Figure 5-36 and Figure 5-37 show the 14-day temperature data and temperature difference data, respectively.

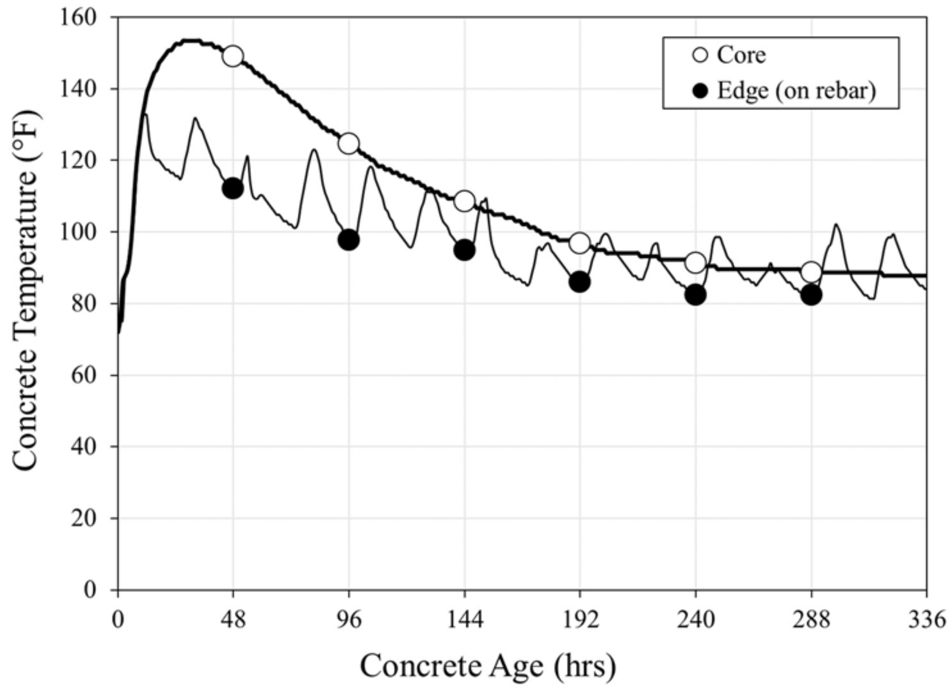


Figure 5-36: Brewton bent cap 14-day temperature data

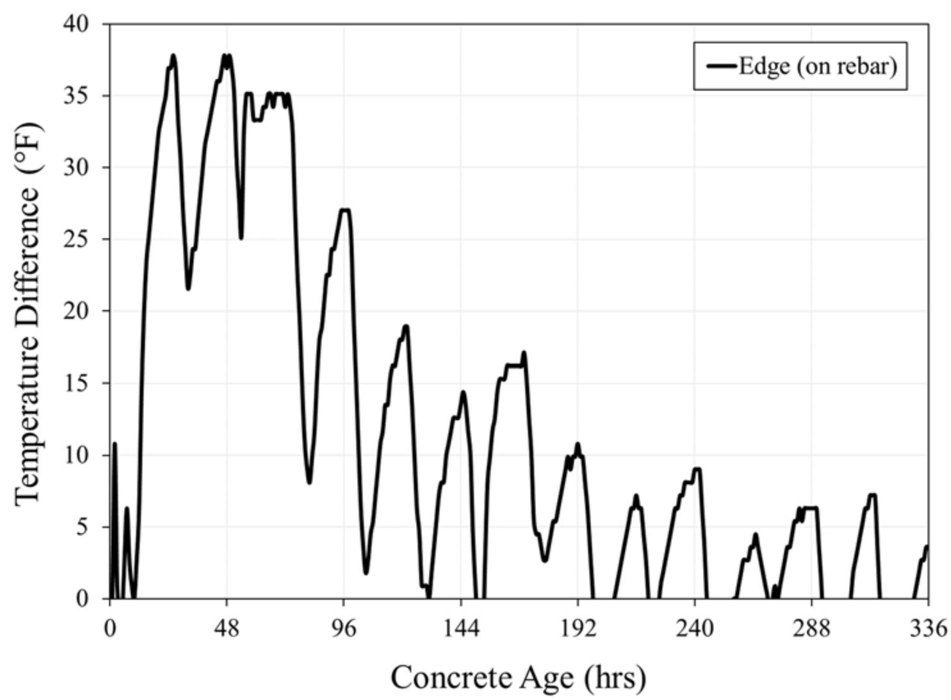


Figure 5-37: Brewton bent cap 14-day temperature difference data

5.8.4 Long-term Temperature Data

The long-term concrete temperatures recorded for this bent cap are shown in Figure 5-38. There is a gap in temperature data from August 1 to August 9, because researchers were unable to access temperature sensors until August 9, and the sensors reached capacity on August 1.

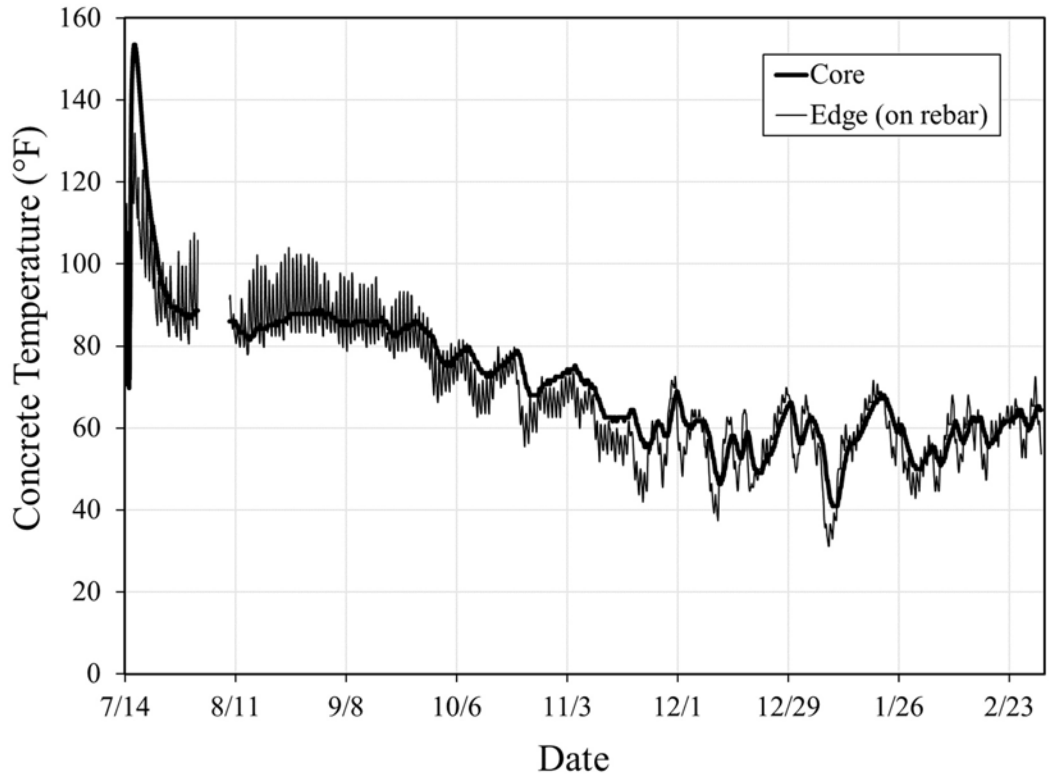


Figure 5-38: Brewton bent cap long-term temperature data

5.8.5 Visual Inspections

A site visit took place on August 9, 2016, 25 days after construction. No signs of thermal cracking were observed at that time. A second site visit took place on November 1, 2016, and again no distresses were discovered. A third site visit took place on March 3, 2017. During this visit, a series of narrow vertical cracks were discovered at the middle of the element on both the north and south faces of the element. Each of these cracks are approximately two inches in length, and the largest crack width found was less than 0.005 inches in width.

5.8.6 Bamforth Crack Prediction Results

Table 5-21 contains the results from Bamforth's long-term crack prediction method for the Brewton bent cap. The type of restraint for this member is ends only, because the bent cap is restrained by the two supporting piers. The restraint factor was determined by modeling the field element in SAP 2000 for as-built conditions and then again for full restraint against contraction. For the as-built condition, the supporting circular piers were considered fixed at ground level, which is conservative as they will provide a high restraint condition due to their size. The provided reinforcement ratio refers to all of the longitudinal reinforcement that crosses the cross section of a potential vertical crack. The long-term bulk temperature difference refers to the difference between the maximum recorded concrete temperature and the long-term recorded low temperature for this element. The value for long-term crack width is based on Bamforth's prediction method outlined in Section 2.4.2.1.

Table 5-21: Brewton bent cap predicted crack widths based on Bamforth's method

Item	Entry
Type of Restraint	Ends only
Provided reinforcement ratio, ρ (%)	0.72
Restraint factor	0.41
Concrete age at low temperature (days)	178
Long-term bulk temperature difference (°F)	113
Predicted long-term crack widths (in.)	0.003

5.8.7 Results of Laboratory Testing

The results of the Q-drum, CTE, concrete compressive strength, and modulus of elasticity tests performed by Auburn researchers are shown below in Table 5-22.

Table 5-22: Brewton bent cap results of laboratory testing

Property	Value
Total Heat of Hydration, H_u (J/kg)	396,000
Activation Energy, E (J/mol)	39,800
Hydration Slope Parameter, β	1.638
Hydration Time Parameter, τ (hr)	9.393
Ultimate Degree of Hydration, α_u	0.857
Concrete CTE (in./in./°F × 10^6) (assumed)	6.00
28-day Concrete Mean Compressive Strength, f_{cm28} (psi)	4800
28-day Concrete Modulus of Elasticity, E_{cm28} (psi)	3,950,000

5.9 Summary and Discussion of Temperature Data

Tables Table 5-23 and Table 5-24 list the maximum recorded concrete temperature, the maximum recorded concrete temperature difference (early-age), the recorded concrete core bulk temperature difference (long-term), and the minimum recorded concrete core temperature (long-term) for all field elements. The early-age concrete temperatures are taken from the 14-day temperature data for each element. These temperatures are important for early-age thermal cracking and DEF. The long-term temperature data were taken from the long-term data that were collected from each project (if any). The "long-term bulk temperature difference" refers to the temperature difference between the concrete core at its maximum temperature and at its minimum temperature. The minimum concrete core temperature contains a value only if temperature recordings were made during the coldest months of the year. The Albertville bent cap, for example, contains the letters DNR (did not record) in the minimum concrete temperature column because temperature data were not available for this element during the winter months.

Table 5-23: Summary of maximum early-age temperature values from instrumented elements

Element	Placement Date	Least Dimension (ft)	Maximum Concrete Temperature	Concrete Age (hours)	Maximum Temperature Difference	Concrete Age (hours)
Albertville Bent Cap	7/31/15	6.5	168°F	20	40°F	92
Harpersville Crashwall	8/24/15	4.0	168°F	27	42°F	44
Scottsboro Pedestal	9/3/15	10.0	185°F	45	68°F	93
Scottsboro Bent Cap	9/18/15	6.5	167°F	27	50°F	42
Elba Bent Cap	12/18/15	5.0	127°F	18	21°F	22
Birmingham Column	1/21/16	4.5	111°F	30	19°F	42
Brewton Bent Cap	7/15/16	6.5	154°F	27	38°F	26

Table 5-24: Summary of significant long-term temperature values from instrumented ALDOT elements

Element	Minimum Concrete Temperature (Core)	Bulk Temperature Difference (Core)	Concrete Age at Minimum Temperature(days)
Albertville Bent Cap	DNR	DNR	-
Harpersville Crashwall	43°F	125°F	153
Scottsboro Pedestal	DNR	DNR	-
Scottsboro Bent Cap	DNR	DNR	-
Elba Bent Cap	41°F	86°F	37
Birmingham Column	42°F	69°F	20
Brewton Bent Cap	41°F	113°F	178

Note: DNR = did not record

5.10 SUMMARY AND DISCUSSION OF CRACK PREDICTION DATA

Table 5-25 summarizes the predicted and measured crack widths for all elements. For elements where no long-term data were recorded a bulk temperature difference was determined based on the minimum ambient weather data to compare the resulting crack widths.

Table 5-25: Summary of predicted crack widths from Bamforth's crack calculator

Element	Bulk temperature difference (long-term)	Predicted Crack Width (in.)
Albertville Bent Cap	130°F*	0.003
Harpersville Crashwall	125°F	0.009
Scottsboro Pedestal	150°F*	0.022
Scottsboro Bent Cap	130°F*	0.002
Elba Bent Cap	86°F	0.002
Birmingham Column	69°F	0.005
Brewton Bent Cap	113°F	0.003

Note: * denotes when long-term bulk temperature difference was assumed based on ambient weather data

Chapter 6

Results and Discussion of ConcreteWorks Analysis

6.1 Accuracy of ConcreteWorks

Before use for ALDOT mass concrete, ConcreteWorks must first be proven as an accurate tool for the prediction of concrete temperatures in mass concrete elements constructed in Alabama. This was done by modeling each field element from Chapter 5 in ConcreteWorks using the appropriate inputs for each element. One of ConcreteWorks' limitations is that it is only capable of calculating stresses for a maximum of seven days. This limited the comparison with field elements to seven days after placement.

To accurately model each field element, detailed data were needed from each project. A BMT 174 concrete testing results form was obtained from ALDOT for each project. Raw materials were also collected from batch plants that produced the project concrete to perform laboratory testing to determine all necessary inputs for ConcreteWorks. Weather data for each site location were obtained from the National Oceanic and Atmospheric Administration (NOAA) for the seven days following placement. Finally, the type of formwork and time of formwork removal were recorded for each project. For each ConcreteWorks prediction, any sudden increase or decrease in the edge temperature is generally due to removal of the formwork.

6.1.1 Comparing ConcreteWorks Temperature Predictions to 7-day Field Data

To validate ConcreteWorks, temperature prediction histories were needed for each element at the locations where actual temperature sensors were installed inside the field elements. Figure 6-1 through Figure 6-14 show all of the ConcreteWorks comparisons to measured, in-place concrete temperatures of the ALDOT field elements. For a cross section modeled in ConcreteWorks, temperature histories may be downloaded for any location within that cross section. These temperature predictions from ConcreteWorks were then compared to the real-time temperature data recorded by the temperature sensors. Sensor locations were recorded during sensor installation prior to concrete placement; however, it is possible that some sensors may have moved from their original positions during concrete placement. The following figures compare the results of ConcreteWorks with the field elements for both core and edge locations. Each graph shows 168 hours (seven days) of temperature data. In the case of the Harpersville crashwall, ConcreteWorks overestimated the effect of formwork removal on the edge temperature change.

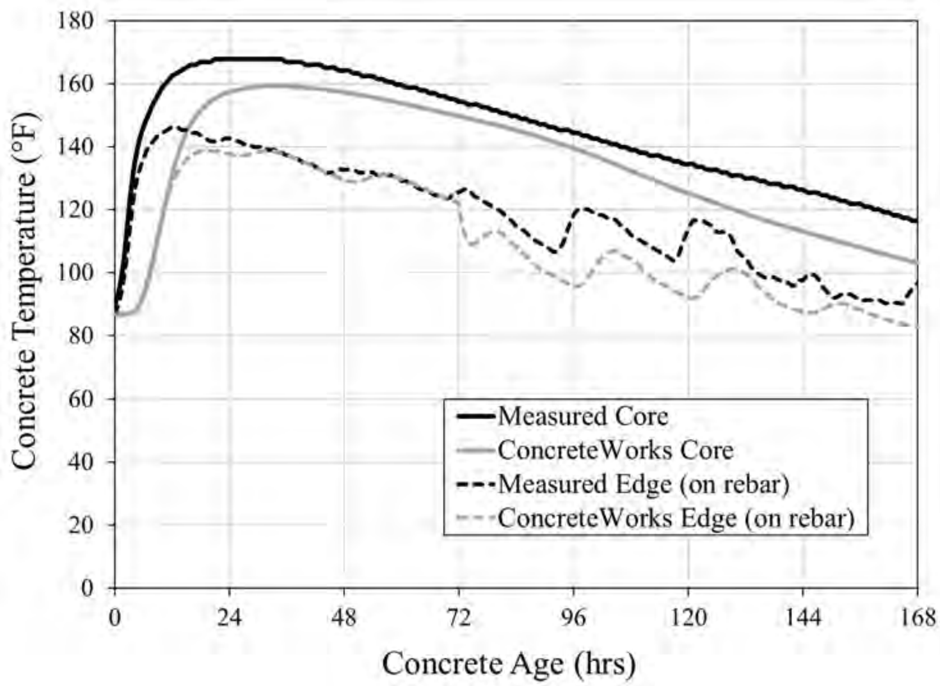


Figure 6-1: Albertville bent cap measured temperature data versus ConcreteWorks predictions

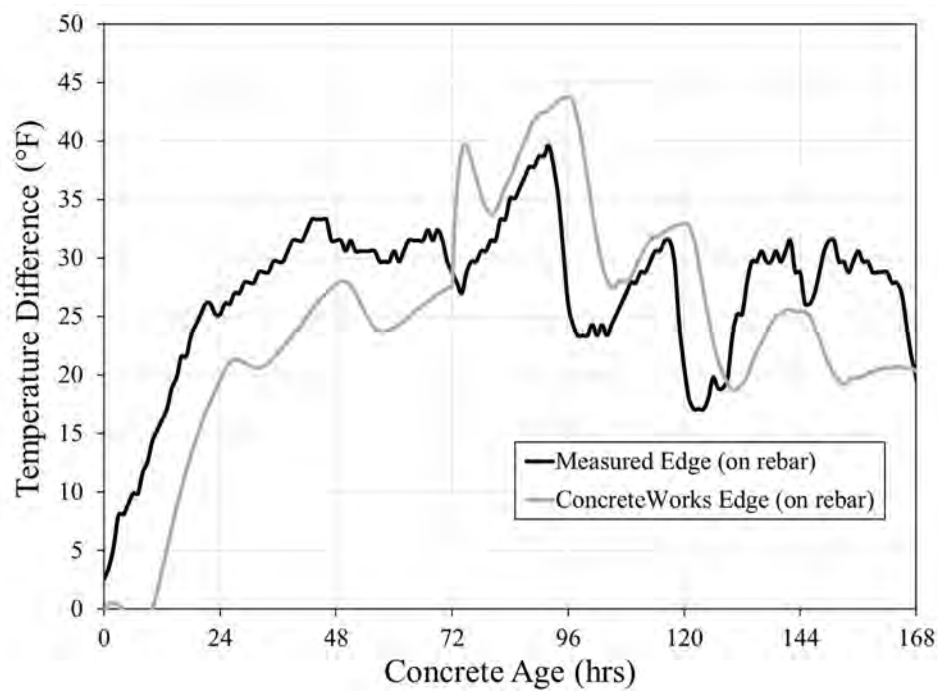


Figure 6-2: Albertville bent cap measured temperature difference data versus ConcreteWorks predictions

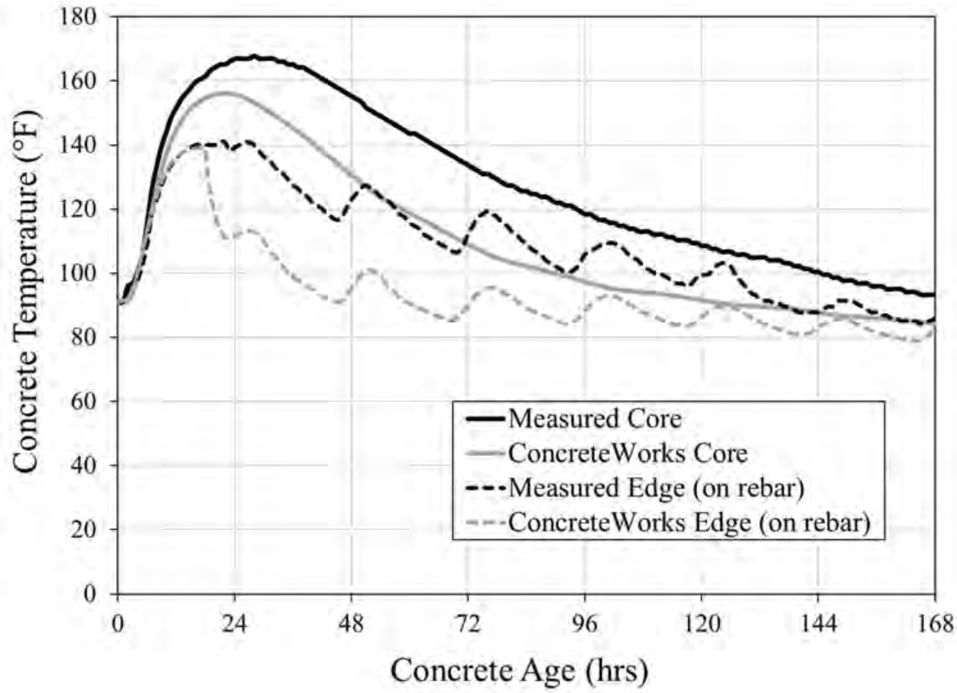


Figure 6-3: Harpersville crashwall measured temperature data versus ConcreteWorks predictions

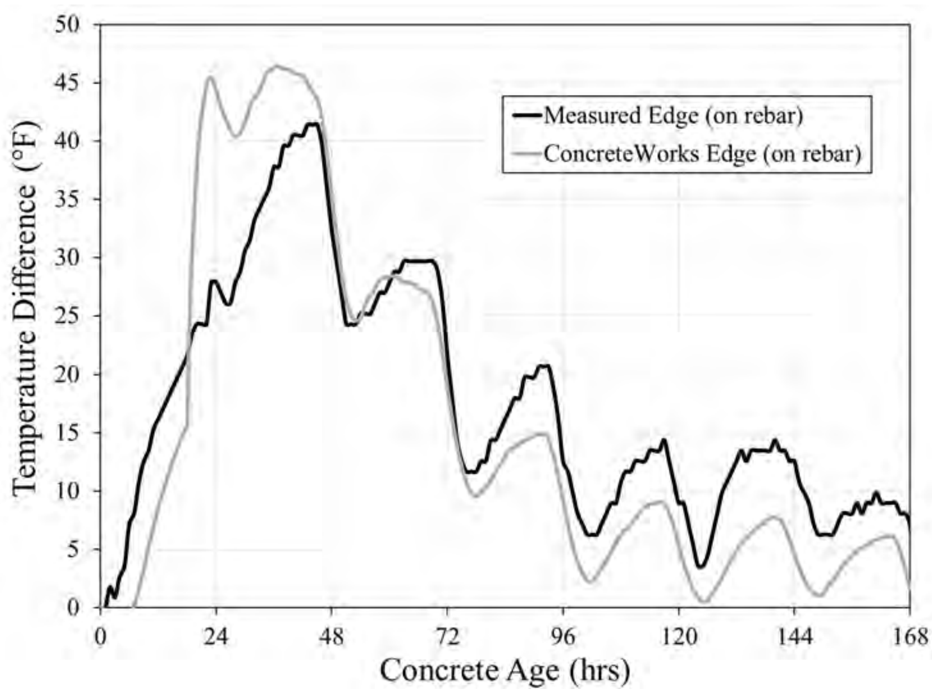


Figure 6-4: Harpersville crashwall measured temperature difference data versus ConcreteWorks predictions

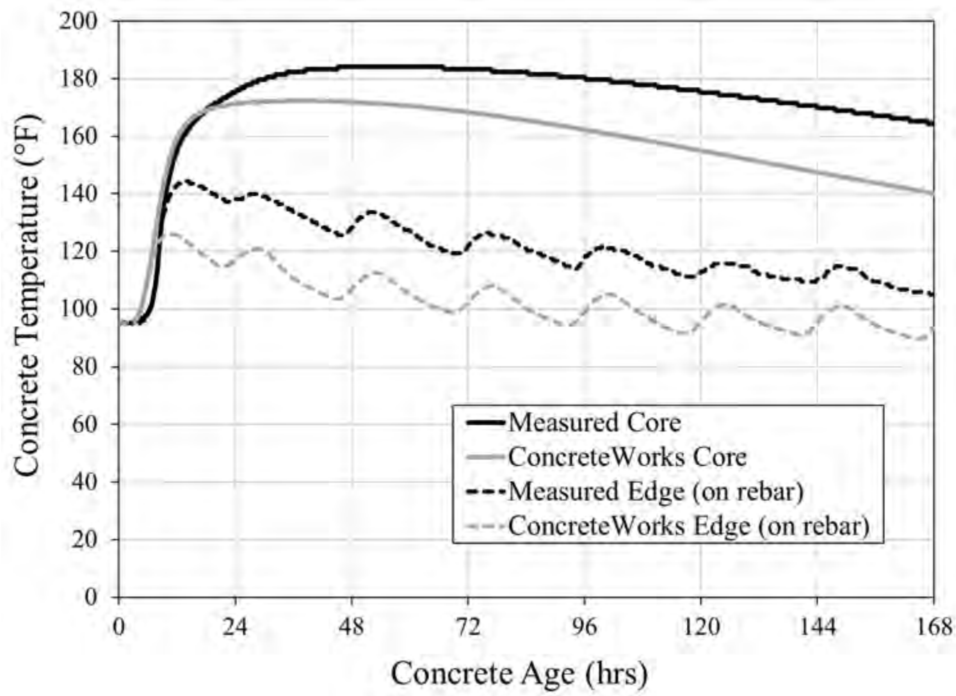


Figure 6-5: Scottsboro pedestal measured temperature data versus ConcreteWorks predictions

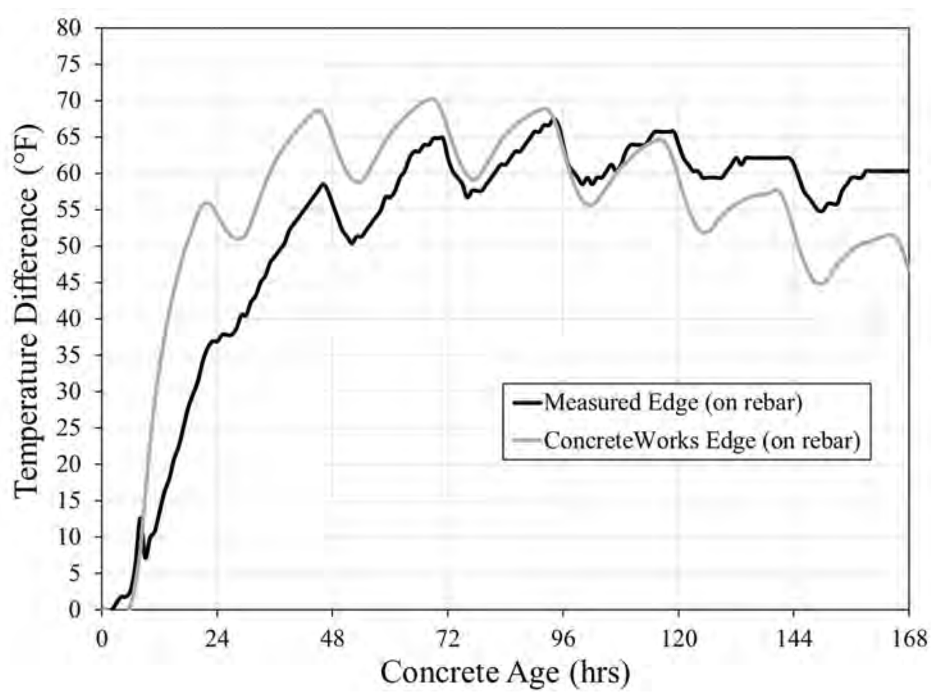


Figure 6-6: Scottsboro pedestal measured temperature difference data versus ConcreteWorks predictions

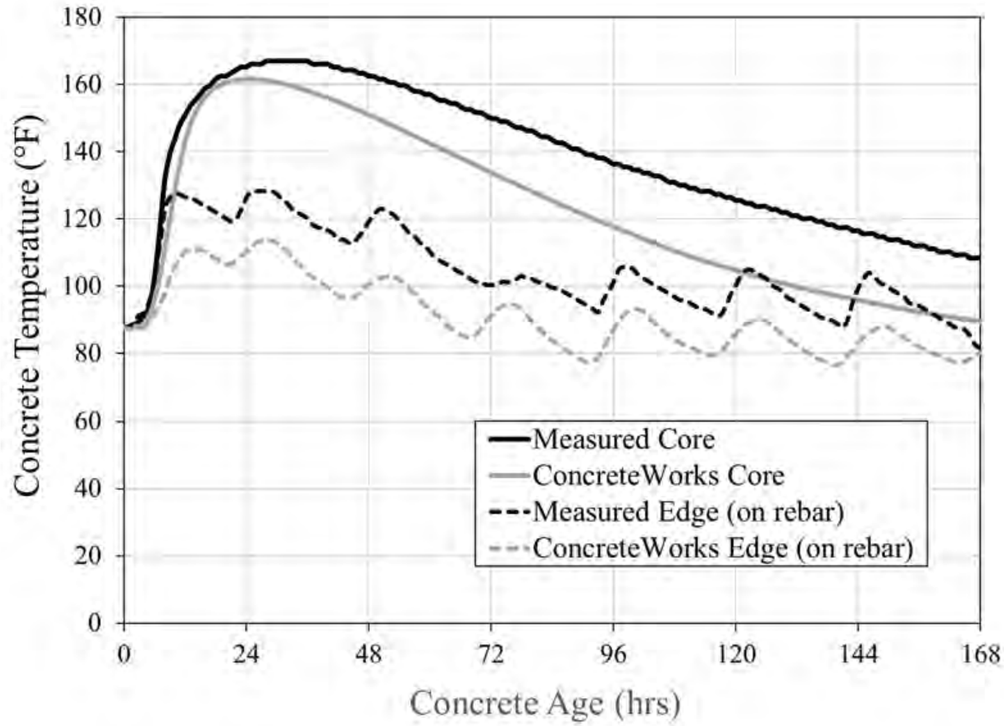


Figure 6-7: Scottsboro bent cap measured temperature data versus ConcreteWorks predictions

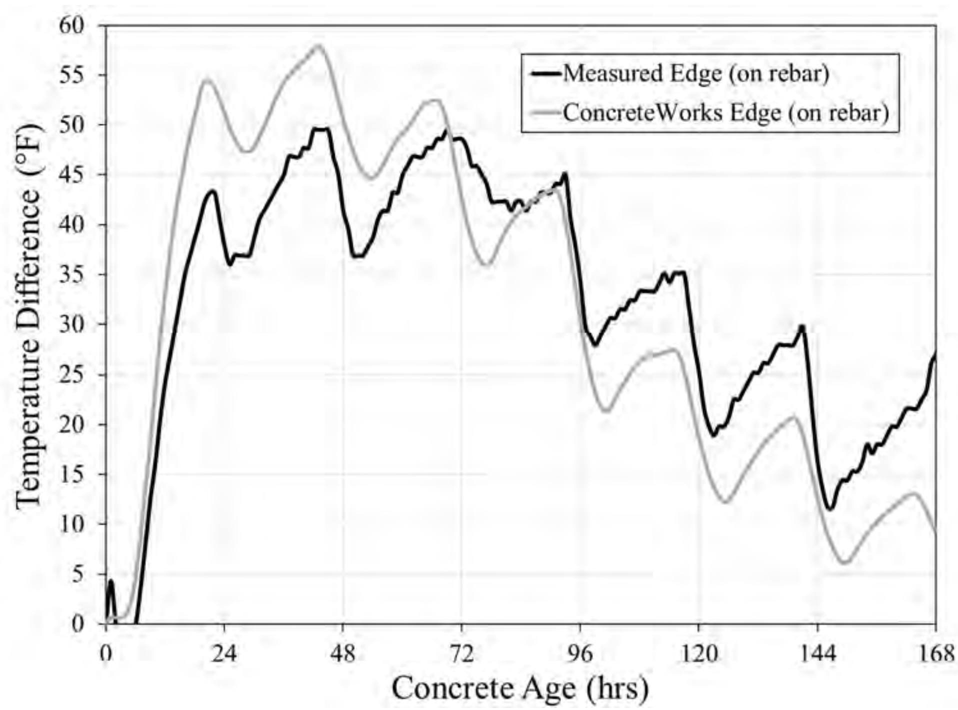


Figure 6-8: Scottsboro bent cap measured temperature difference data versus ConcreteWorks predictions

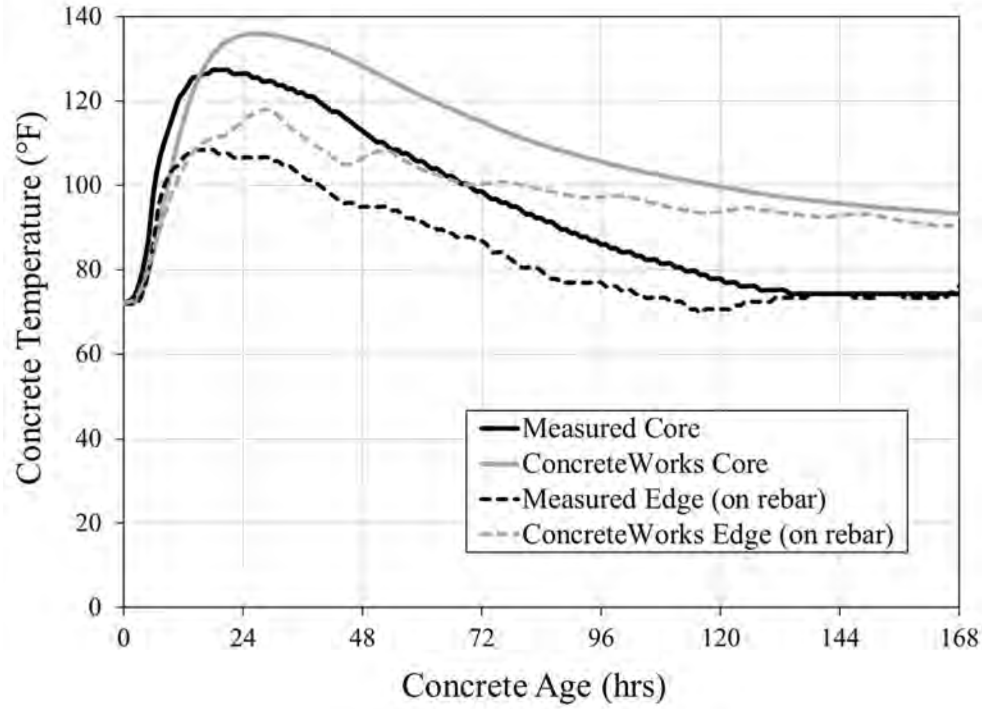


Figure 6-9: Elba bent cap measured temperature data versus ConcreteWorks predictions

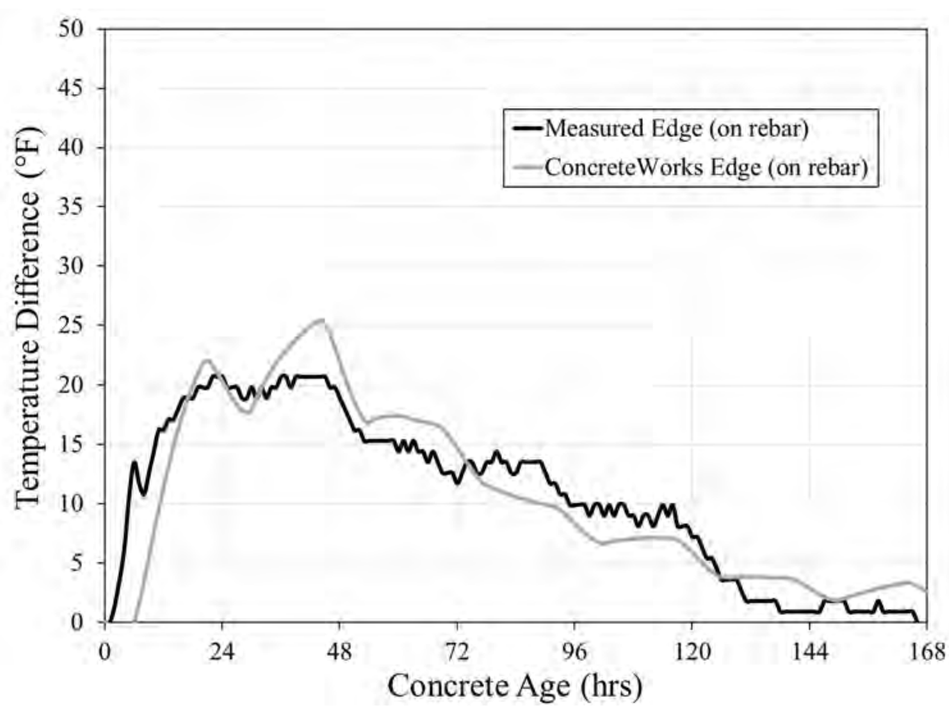


Figure 6-10: Elba bent cap measured temperature difference data versus ConcreteWorks predictions

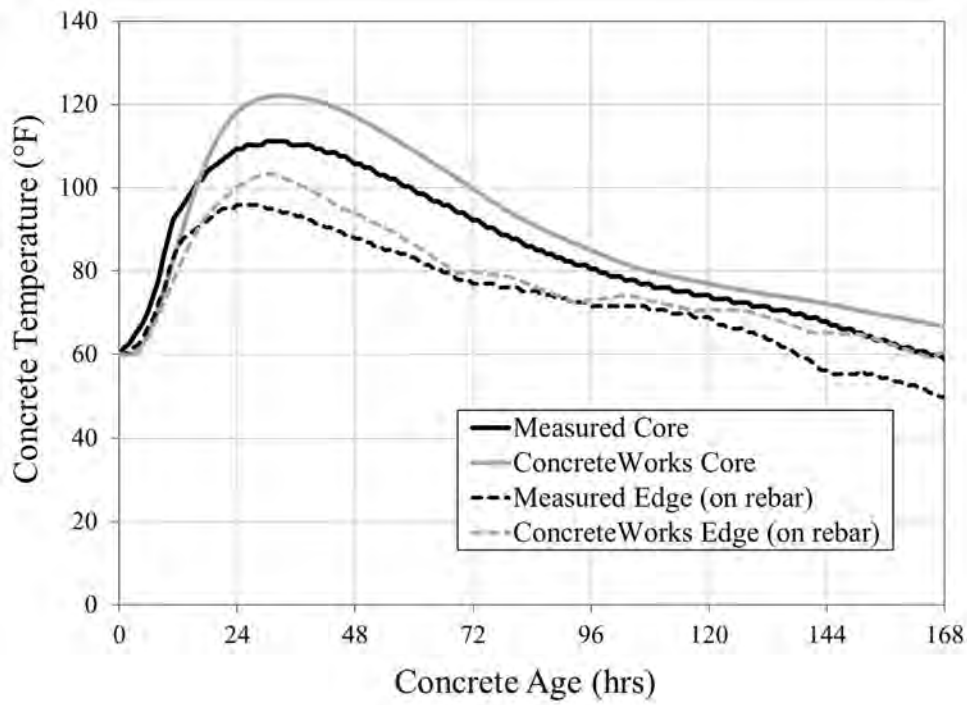


Figure 6-11: Birmingham column measured temperature data versus ConcreteWorks predictions

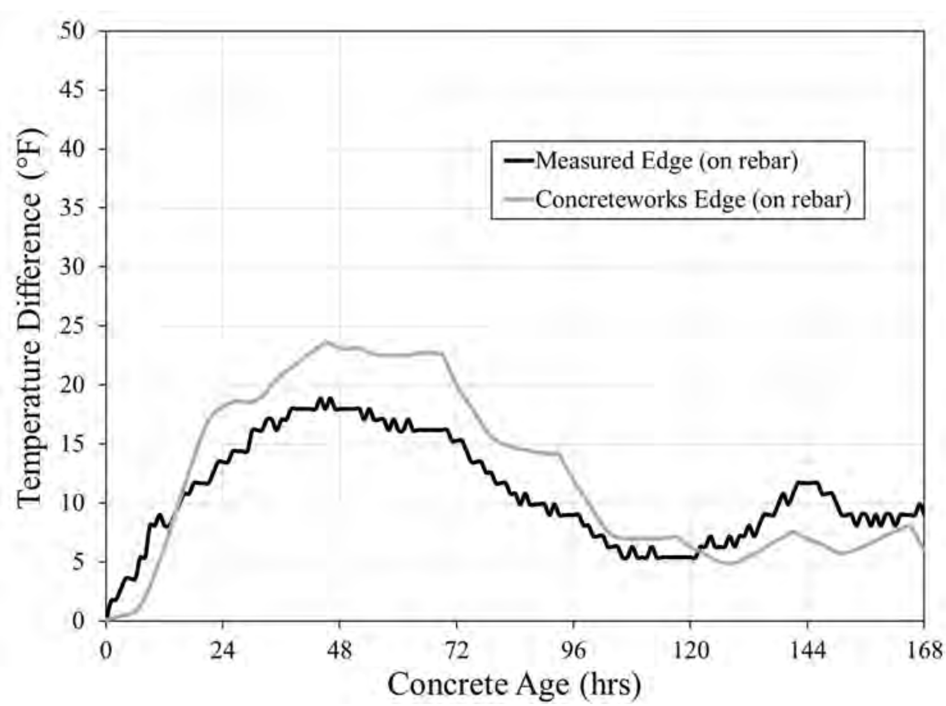


Figure 6-12: Birmingham column measured temperature difference data versus ConcreteWorks predictions

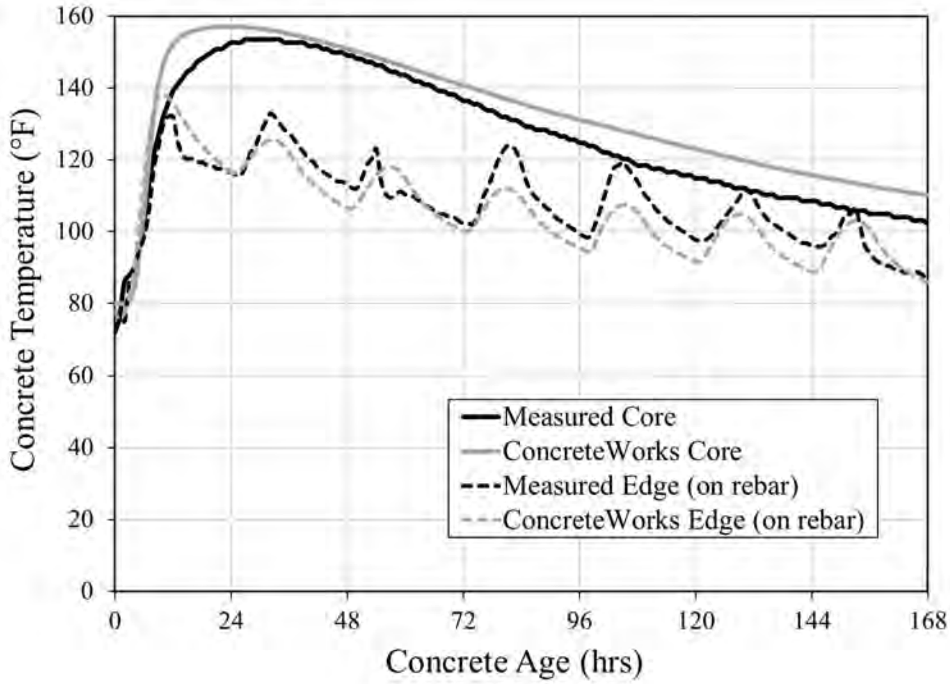


Figure 6-13: Brewton bent cap measured temperature data versus ConcreteWorks predictions

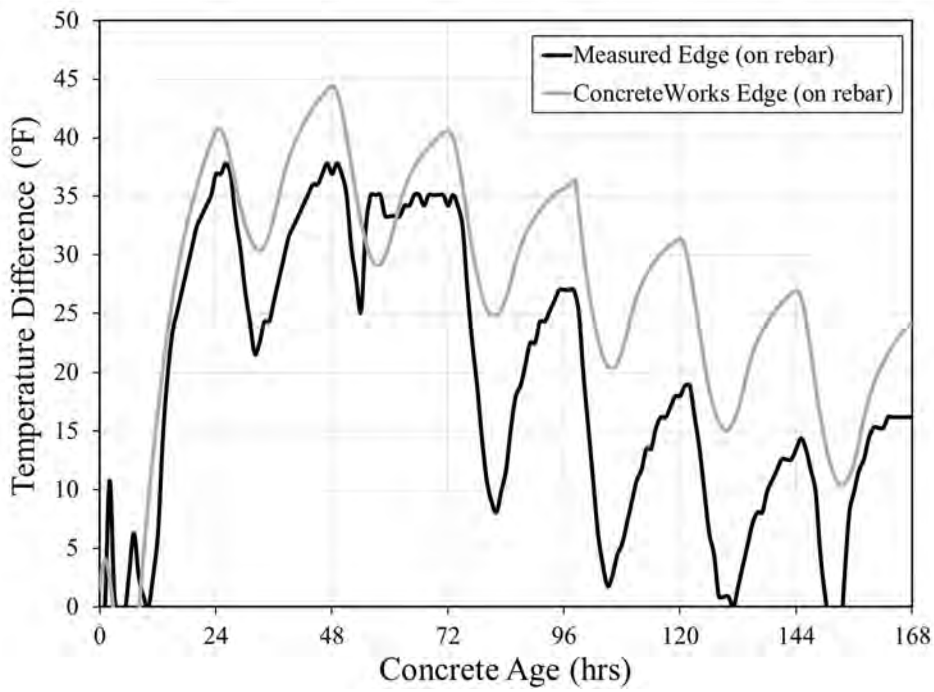


Figure 6-14: Brewton bent cap measured temperature difference data versus ConcreteWorks predictions

6.1.2 Discussion and Assessment of Accuracy of ConcreteWorks

ConcreteWorks' accuracy as a mass concrete temperature prediction tool was assessed by defining a range of acceptable values greater than or less than the measured in-place concrete temperatures and concrete temperature differences as defined in Table 6-1. The acceptable error ranges take into consideration variability of the measured concrete temperatures. The iButton sensors are precise to within 1.8°F up until 158°F (70°C) after which their precision changes to 2.3°F.

Table 6-1: Acceptable error ranges for ConcreteWorks temperature predictions

Accuracy Category	Maximum Concrete Temperature	Maximum Concrete Temperature Difference
Excellent	± 0 to 5 °F	± 0 to 3 °F
Good	± 5 to 10 °F	± 3 to 6 °F
Acceptable	± 10 to 15 °F	± 6 to 9 °F
Poor	± 15 °F and greater	± 9 °F and greater

Table 6-2 summarizes the comparisons between measured concrete temperatures and ConcreteWorks predicted temperatures. Negative errors indicate that ConcreteWorks under-predicted the corresponding maximum temperature values. All ConcreteWorks predictions were classified as "Acceptable" or better. Two maximum concrete temperature predictions were classified as "Good", and two were classified as "Excellent". Five maximum concrete temperature difference predictions were classified as "Good", and one was classified as "Excellent". Based on this assessment, ConcreteWorks was deemed accurate enough for use in the prediction of ALDOT mass concrete elements' temperature behaviors.

Table 6-2: Accuracy of ConcreteWorks maximum temperature predictions compared to measured maximum field temperatures

Element	Maximum Concrete Temperature			Maximum Concrete Temperature Difference		
	Measured	CW Predicted	Error	Measured	CW Predicted	Error
Albertville Bent Cap	168°F	159°F	-9°F	40°F	44°F	+4°F
Harpersville Crashwall	168°F	156°F	-12°F	41°F	46°F	+5°F
Scottsboro Pedestal	185°F	172°F	-13°F	68°F	70°F	+2°F
Scottsboro Bent Cap	167°F	162°F	-5°F	50°F	58°F	+8°F
Elba Bent Cap	127°F	136°F	+9°F	21°F	25°F	+4°F
Birmingham Column	111°F	122°F	+11°F	19°F	24°F	+5°F
Brewton Bent Cap	154°F	157°F	+3°F	38°F	44°F	+6°F

Note: Cell background color matches convention in Table 6-1.

6.2 Summary of ConcreteWorks Statistical Analysis

As previously described in Section 4.3, ConcreteWorks was used in this project to determine what factors most greatly affect the cracking risk of members that could be designated mass concrete in the state of Alabama. This was done through a statistical analysis of 480 ConcreteWorks analysis runs, each using a unique placement scenario. The input variables assessed were coarse aggregate type, SCM type, placement location, placement date, and element least dimension. The resulting information found in Section 6.2.2 is used as a guide for selecting appropriate temperature limits, least dimensions, and in the development of temperature control requirements for an ALDOT mass concrete construction specification.

6.2.1 Variable Inputs Used in Statistical Analysis

A summary of the variables used in the statistical analysis may be found in Table 6-3. Two types of coarse aggregate were selected for use in this project to provide two different concrete CTEs. Limestone and siliceous river gravel were both selected because they are available throughout Alabama and are commonly used in ALDOT concrete mixtures. Past Auburn University research determined CTE values of 5.52 × 10-6 in./in./°F and 6.95 × 10-6 in./in./°F for limestone and river gravel, respectively (Schindler et al. 2010). As stated in Section 2.6.1, these variables were systematically selected to model concrete behavior in the state of Alabama.

Table 6-3: ConcreteWorks variables

Variable	Option	Reason
Coarse Aggregate	Limestone	Different CTE
	River Gravel	
SCM Use	100% portland cement	Different cementing materials composition
	30% Class F fly ash replacement	
	30% Class C fly ash replacement	
	50% slag cement replacement	
Location	Huntsville	Different ambient weather and placement conditions
	Mobile	
Placement Date/Time	January 15 at 12:00 PM	
	March 15 at 10:00 AM	
	May 15 at 8:00 AM	
	July 15 at 8:00 AM	
	September 15 at 10:00 AM	
	November 15 at 12:00 PM	
Least Dimension	4 ft	Different definitions of mass concrete
	5 ft	
	6 ft	
	7 ft	
	8 ft	

6.2.2 Results of Statistical Analysis of ConcreteWorks

To quantify the effect of each variable on the output of each test, the ANOVA Single Factor Test was used with a 95% confidence interval. The resulting p-values for each variable are listed in Table 6-4. The closer a p-value is to zero, the greater that input variable's influence on the corresponding output variable is. Because of the confidence level, any p-value greater than 0.05 is statistically insignificant.

Table 6-4: P-Values of input variables based on ANOVA Test

Input Variable	ANOVA P-Values		
	Maximum In-Place Concrete Temperature	**Maximum Concrete Temperature Difference**	**Maximum Concrete Cracking Risk**
Coarse Aggregate	0.0081	0.22	4.3×10^{-23}
SCM Use	1.2×10^{-9}	1.6×10^{-15}	4.7×10^{-10}
Placement Location	6.5×10^{-8}	0.029	0.58
Placement Date	5.1×10^{-127}	7.1×10^{-8}	1.5×10^{-3}
Element Least Dimension	1.5×10^{-9}	1.4×10^{-133}	2.4×10^{-44}

Based on these results, Table 6-5 shows the ranking of each input variable's impact on each output from greatest to least. For example, the placement date has the greatest impact on the maximum in-place temperature while the coarse aggregate type has the least. Any text that is struck through represents statistical insignificance for that variable.

Table 6-5: Ranking the statistical impact of input variables based on the ANOVA Single Factor Test

Maximum In-Place Concrete Temperature	Maximum Concrete Temperature Difference	Maximum Concrete Cracking Risk
1. Placement Date	1. Element Least Dimension	1. Element Least Dimension
2. SCM Use	2. SCM Use	2. Coarse Aggregate Type
3. Element Least Dimension	3. Placement Date	3. SCM Use
4. Placement Location	4. Placement Location	4. Placement Date
5. Coarse Aggregate Type	~~5. Coarse Aggregate Type~~	~~5. Placement Location~~

Note: Struck through text indicates statistical insignificance for that variable.

6.2.2.1 New Maximum Temperature Table from ACI 201

In 2016, ACI 201 released a set of recommendations that increase the maximum allowable in-place concrete temperature (see Table 3-2). These recommendations state that when using an SCM at the appropriate replacement levels, the maximum allowable in-place concrete temperature may be increased from 158°F to 185°F. Table 6-6 summarizes Table 3-2 as it pertains to the SCM replacements used in ConcreteWorks for this project. For this research project only fly ash and slag cement were considered when analyzing SCM replacement.

Table 6-6: ACI 201.2R Maximum in-place concrete temperature recommendations as they pertain to the statistical ConcreteWorks analysis (adapted from ACI 201.2R 2016)

Maximum Concrete Temperature, T	Prevention Required
T ≤ 158°F	No prevention required
158°F < T ≤ 185°F	Use one of the following approaches to minimize the risk of expansion: 1. Portland cement meeting requirements of ASTM C150 moderate or high sulfate-resisting and low-alkali cement with a fineness value less than or equal to 430 m^2/kg 2. Portland cement with a 1-day mortar strength (ASTM C109) less than or equal to 2850 psi 3. Any ASTM C150 portland cement in combination with the following proportions of pozzolan or slag cement: a) Greater than or equal to 25 percent fly ash meeting the requirements of ASTM C618 for Class F fly ash b) Greater than or equal to 35 percent fly ash meeting the requirements of ASTM C618 for Class C fly ash c) Greater than or equal to 35 percent slag cement meeting the requirements of ASTM C989 d) Greater than or equal to 5 percent silica fume (meeting ASTM C1240) in combination with at least 25 percent slag cement e) Greater than or equal to 5 percent silica fume (meeting ASTM C1240) in combination with at least 20 percent Class F fly ash f) Greater than or equal to 10 percent metakaolin meeting ASTM C618 4. An ASTM C595/C595M or ASTM C1157/C1157M blended hydraulic cement with the same pozzolan or slag cement content in Item 3
T > 185°F	The internal concrete temperature should not exceed 185°F (85°C) under any circumstances.

By using at least 25% Class F fly ash, 35% Class C fly ash, or 35% slag cement, the maximum allowable in-place temperature may exceed 158°F without the risk of excessive expansion. For this project, 30% replacement levels were chosen for both Class C and F fly ash because the ConcreteWorks analysis began in 2014 before these recommendations were published. The current Class F fly ash and slag cement replacement levels used in this analysis are greater than the recommended minimums, but the Class C fly ash replacement level is not. However, because this project's replacement level of 30% falls only 5% short of the recommendation, it is assumed that this change would only cause a slight difference in maximum temperature and cracking risk behavior and thus used the ConcreteWorks analyses results for the 30% Class C fly ash replacement. All ConcreteWorks runs containing SCMs satisfied the requirements from Table 6-6 by containing more than the minimum amount of fly ash or slag cement required to raise the maximum in-place concrete temperature. This could improve a mass concrete specification by removing some elements from the extra requirements for mass concrete construction, saving the State time and money.

6.2.2.2 Size Designation

The new maximum temperature rule of ACI 201.2R (shown in Table 6-6) was used to aid in the appropriate size designation for mass concrete members in a specification. The ConcreteWorks statistical analysis data were examined to determine what size member would exceed the maximum allowable in-place concrete temperature. Any element whose maximum in-place concrete temperature rose above:

- 158°F with no SCMs,
- 185°F with SCMs,
- Produced a cracking risk of “high”, or
- Produced a cracking risk of ”very high”

is designated as mass concrete and will require a thermal control plan (TCP).

Table 6-7 indicates that any concrete member using 100% portland cement with a least dimension of 4 ft or greater will require a TCP due to the high maximum in-place temperature. However, the use of SCMs will greatly reduce the need for a TCP by lowering the maximum temperature and the cracking risk. Use of limestone coarse aggregate will help in lowering the cracking risk, but it will not contribute greatly to reducing the maximum temperature (see also Table 6-5).

Table 6-7: Designation of members in need of a TCP based on ConcreteWorks analysis

SCM Use	Coarse Agg. Type	Element Least Dim.	T_{max} greater than 158 F?	T_{max} greater than 185 F?	Cracking Risk High or Very High?	TCP Needed ?
100% PC	Limestone	4 ft		N/A		
		5 ft	✓	N/A		✓
		6 ft	✓	N/A		✓
		7 ft	✓	N/A		✓
		8 ft	✓	N/A	✓	✓
	River Gravel	4 ft	✓	N/A		✓
		5 ft	✓	N/A	✓	✓
		6 ft	✓	N/A	✓	✓
		7 ft	✓	N/A	✓	✓
		8 ft	✓	N/A	✓	✓
30% FFA	Limestone	4 ft	N/A			
		5 ft	N/A			
		6 ft	N/A			
		7 ft	N/A			
		8 ft	N/A			
	River Gravel	4 ft	N/A			
		5 ft	N/A			
		6 ft	N/A			
		7 ft	N/A			
		8 ft	N/A		✓	✓
30% CFA	Limestone	4 ft	N/A			
		5 ft	N/A			
		6 ft	N/A			
		7 ft	N/A			
		8 ft	N/A			
	River Gravel	4 ft	N/A			
		5 ft	N/A			
		6 ft	N/A			
		7 ft	N/A		✓	✓
		8 ft	N/A		✓	✓
50% SL	Limestone	4 ft	N/A			
		5 ft	N/A			
		6 ft	N/A			
		7 ft	N/A			
		8 ft	N/A		✓	✓
	River Gravel	4 ft	N/A			
		5 ft	N/A			
		6 ft	N/A		✓	✓
		7 ft	N/A		✓	✓
		8 ft	N/A		✓	✓

Note: Shaded boxes with N/A = corresponding temperature limit not applicable due to SCMs

6.2.2.3 Maximum Concrete Temperature

As seen in Table 6-5, the maximum in-place concrete temperature is most affected by the placement date and use of SCM. Figure 6-15 shows the maximum temperature results of the ConcreteWorks analyses for a 6 ft square column cast in Mobile using limestone coarse aggregate. The placement dates directly affect the maximum temperature because of the ambient temperature's effect on the placement temperature of the concrete. Figure 6-15 demonstrates that the concrete placement temperature has the greatest effect on the ConcreteWorks maximum in-place temperature predictions. Concrete made with 100% portland cement (no SCM) generated enough heat to increase the maximum in-place temperature approximately 80°F relative to the placement temperature.

The same may be said about any of the concretes containing SCMs. The maximum in-place concrete temperatures for each type of SCM replacement are higher during warmer months when the placement temperature would be higher. Schindler and Folliard (2005) have demonstrated this concept by providing the theoretical adiabatic temperature rise of some SCMs in Figure 2-13. The data from Figure 6-15 also show the use of SCMs, especially Class F fly ash, has the potential to reduce the maximum in-place concrete temperature by 10 to 15°F. Concrete containing these types of SCMs typically produces less heat during hydration than concrete exclusively using portland cement (Schindler and Folliard 2005).

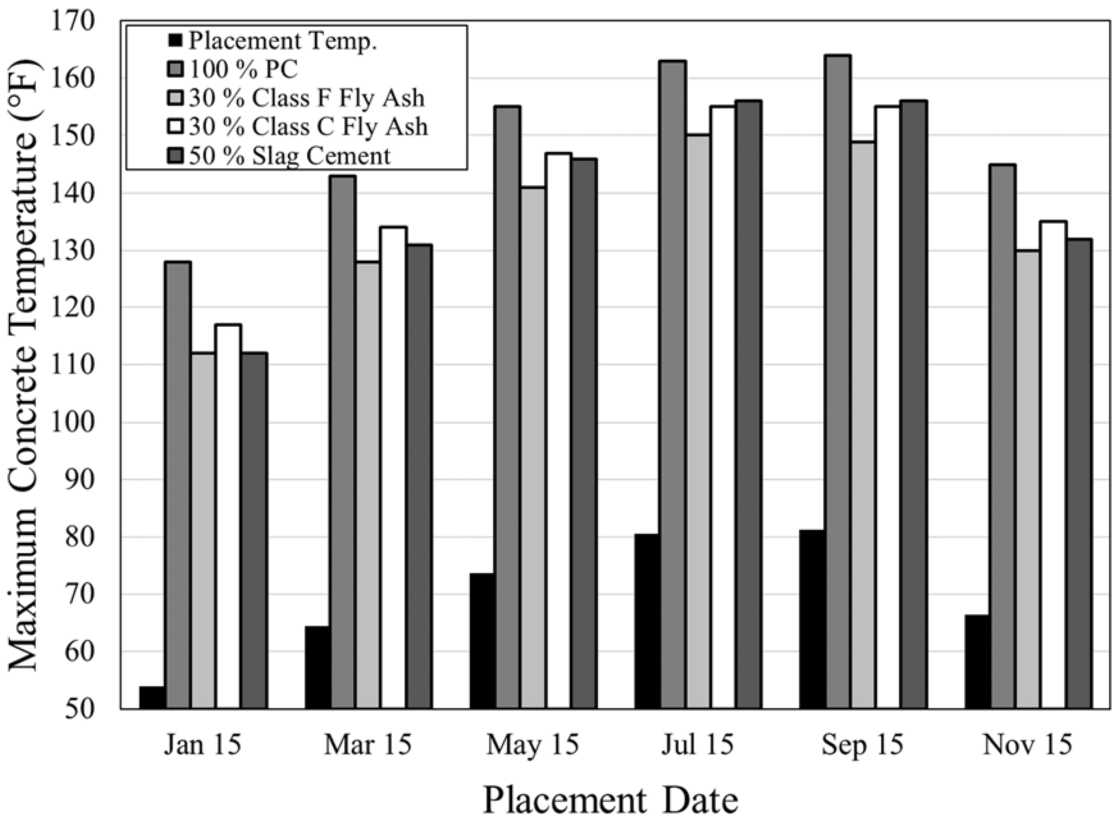


Figure 6-15: Effect of placement date and SCM type on maximum concrete temperature (Mobile, AL; Limestone; 6 ft x 6 ft)

The least dimension of an element will also affect the maximum in-place temperature, as seen in Figure 6-16. The maximum in-place concrete temperature for each element with the five least dimensions considered is seen in Figure 6-16. All elements are cast in Mobile during July (placement temp. = 80°F) using 100% portland cement. Concurrent with Table 6-5, the least dimension of a concrete element has a significant effect on the maximum in-place temperature, but its impact is not as great as the placement date or use of SCMs.

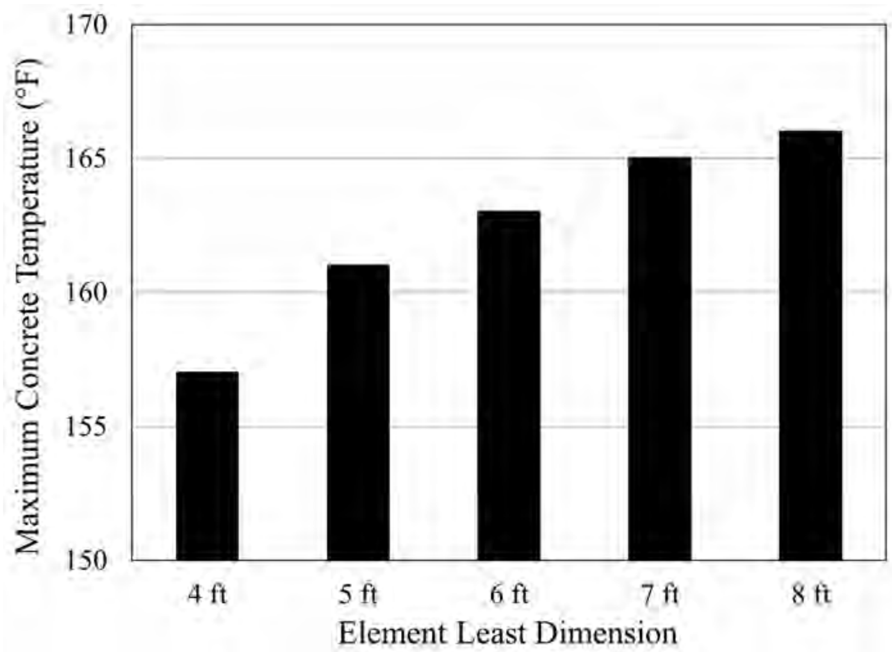


Figure 6-16: Effect of element least dimension on maximum concrete temperature (Mobile, AL; July 15; Limestone; 100% PC)

6.2.2.4 Maximum Concrete Temperature Difference

As shown in Table 6-5, the least dimension of an element and the use of SCMs have the greatest impact on the maximum concrete temperature difference. Figure 6-17 demonstrates this using the same square column as before, cast in Mobile on July 15th using limestone coarse aggregate with varying column sizes. Here the size of the element has the greatest impact on the maximum concrete temperature difference. Following the trend of this chart, the maximum temperature difference increases by approximately 5°F for every 1 ft of least dimension. Using SCMs will reduce the temperature difference by approximately 4°F to 8°F in each case.

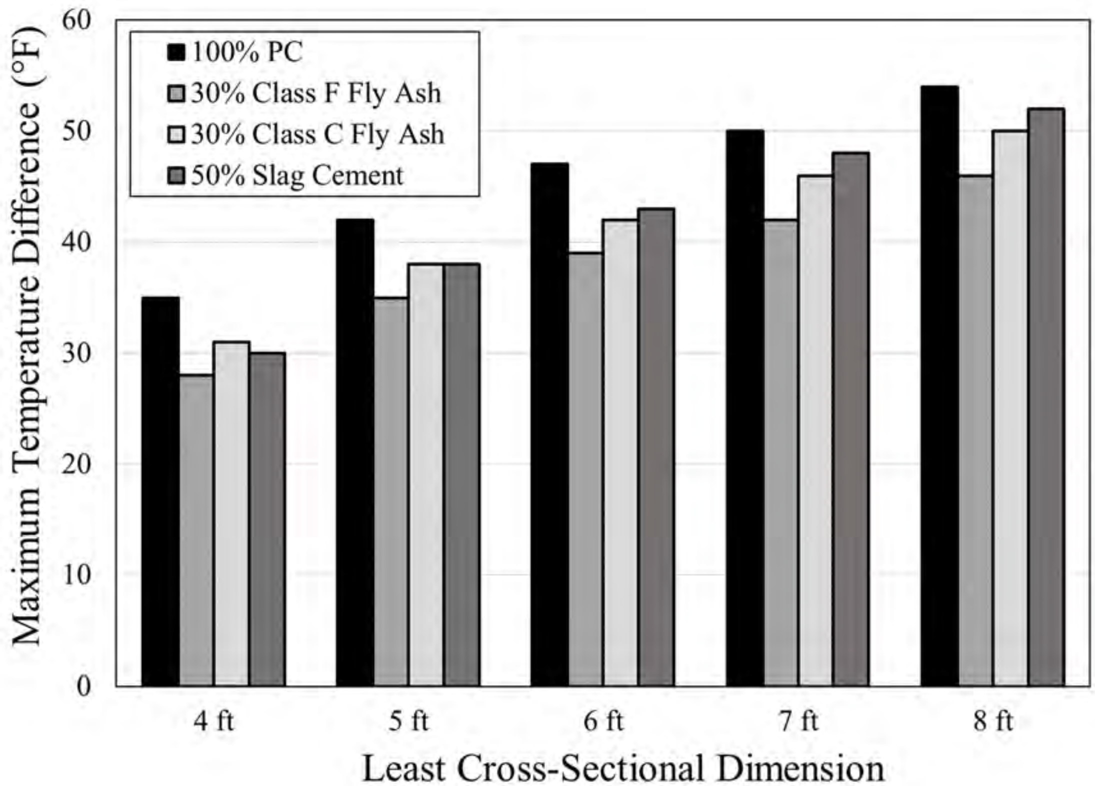
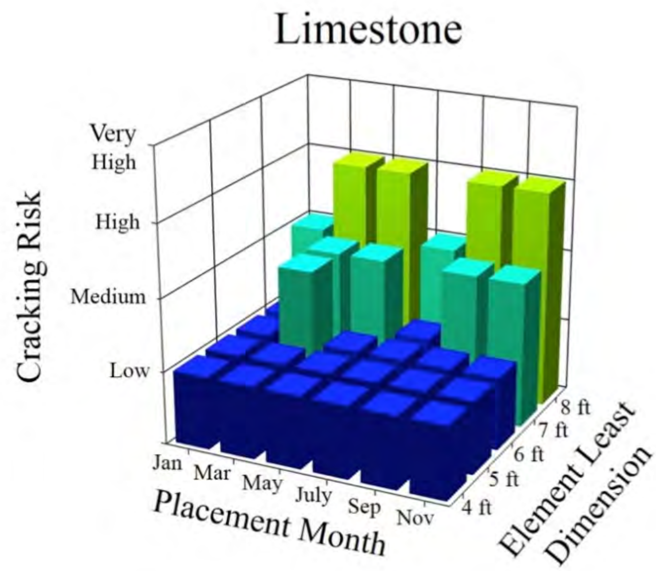


Figure 6-17: Effect of least dimension and SCM use on maximum concrete temperature difference (Mobile, AL; July 15; Limestone)

6.2.2.5 Cracking Risk

In ConcreteWorks, the "cracking risk" is obtained by comparing the development of the tensile stress to the development of the tensile strength at the same concrete maturity. Table 6-5 indicates that element least dimension and coarse aggregate type are the two factors that most greatly affect maximum cracking risk. Figure 6-18 illustrates the effect of the placement date, least dimension, and coarse aggregate type on the cracking risk. Once again, this analysis uses a square column cast in Mobile using 100% portland cement. This comparison emphasizes how much the least dimension and coarse aggregate type affect the cracking risk. By using limestone aggregate as opposed to river gravel, the cracking risk is greatly decreased due to the difference in CTE.



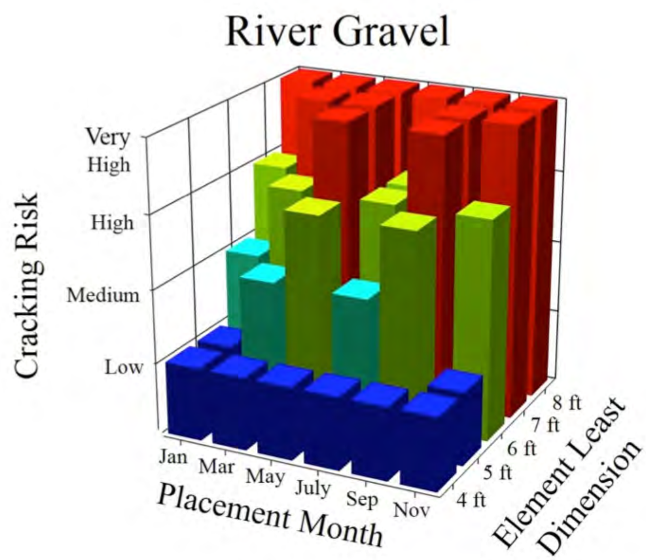


Figure 6-18: Effect of least dimension, coarse aggregate type, and placement date on maximum concrete cracking risk (Mobile, AL; 100% PCC)

The type of SCM used also influences the cracking risk. Figure 6-19 illustrates this by comparing cracking risk analyses of 100% portland cement concrete (PCC) and 30% Class F fly ash using the same square column cast in Mobile with river gravel coarse aggregate. The results demonstrate the effectiveness of SCMs on the cracking risk. Because of the lower maximum in-place concrete temperature associated with Class C fly ash, the thermal stresses are lower, and the cracking risk is lower. This figure also illustrates the significant impact of the least dimension on the cracking risk, with the smaller elements having the lowest cracking risks. Both Figure 6-18 and Figure 6-19 illustrate the relatively low impact that placement date has on the cracking risk. Regardless of the time of year, those elements of the same size and mixture proportions have comparable cracking risks.

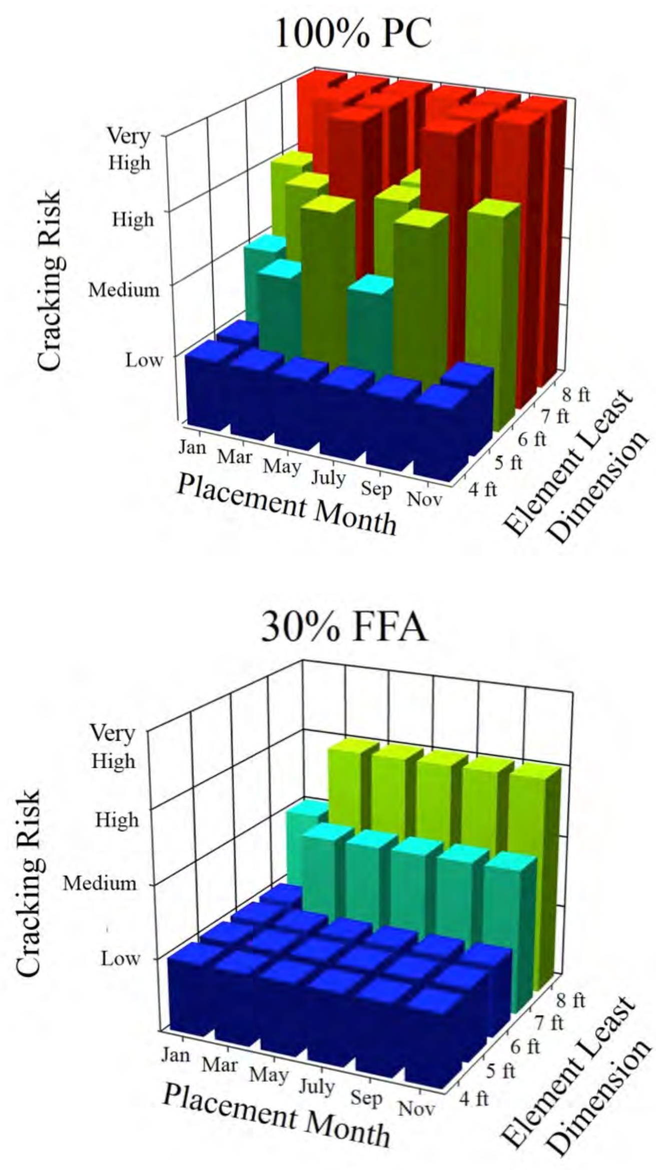


Figure 6-19: Effect of least dimension, SCM, and placement date on maximum concrete cracking risk (Mobile, AL; River Gravel)

6.2.2.6 Maximum Temperature Difference versus Cracking Risk

While a large maximum temperature difference may lead to thermal cracking, the concrete CTE and thus the type of coarse aggregate, plays a large part in determining what the cracking risk will be. A concrete with a low CTE will not shrink as much as a concrete with a high CTE. As stated in Chapter 2 and shown in Equation 2-1, the concrete with a low CTE can withstand a much larger temperature difference before developing thermal cracks. This leads to the conclusion that a concrete's CTE must be considered when developing a mass concrete specification.

Many mass concrete specifications implement thermal cracking mitigation procedures based solely on the expectation of a maximum temperature difference more than 35°F (20°C) without giving any consideration to the type of coarse aggregate used. From Table 2-1 it is clear that the 35°F limit developed by Bamforth (1995) specifically pertains to concrete containing river gravel, due to its greater CTE. Bamforth's limiting temperature difference for concrete with limestone using the same restraint factor is 63°F (35°C). Bamforth's conclusions correspond to the cracking risk results from the 480 ConcreteWorks analyses as summarized in Table 6-5. When comparing the cracking risk results, the elements containing limestone coarse aggregate have a lower cracking risk when all other variables are the same. Figure 6-20 illustrates this trend using the 7 ft square column with differing placement locations, placement dates, SCM type, and coarse aggregate type. In Figure 6-20 the x-axis is organized by different combinations of coarse aggregate type and SCM type. For each combination, the 7 ft column is analyzed at each placement location and time of year for a total of 96 ConcreteWorks runs. For example, the x-axis section labeled "100% PC – LS" contains the results from all runs using 100% portland cement (PC) and limestone aggregate (LS) at both locations and at each time. The section to its immediate right contains all runs of the same mixture, placement times, and locations using river gravel (RG). The gray shaded area is the maximum concrete temperature difference corresponding to each of the 96 analyses, to be read with the primary y-axis. The black line is the cracking risk that corresponds to each of these analyses, from "low" to "very high" to be read with the secondary y-axis.

Figure 6-20 illustrates the trend that in each case limestone aggregate is used, the cracking risks are much lower than the same concrete placements that differ only by using river gravel. The temperature profiles for each set of mixtures using limestone and river gravel are similar, but due to the limestone's much lower CTE, the concrete containing limestone experiences a much lower risk of cracking. Also to be noted is the drop in temperature difference and cracking risk between mixtures that use SCMs and those that use 100% portland cement. SCMs also help reduce the risk of cracking, but the risk of cracking is best reduced by use of a low CTE concrete. These results indicate the best method for determining the maximum allowable temperature difference may not be based solely on a fixed temperature difference limit (most commonly 35°F, see Table 3-8). The most effective method of determining a limiting temperature difference may be by use of a cracking risk or a similar approach that considers the type of coarse aggregate used and corresponding CTE.

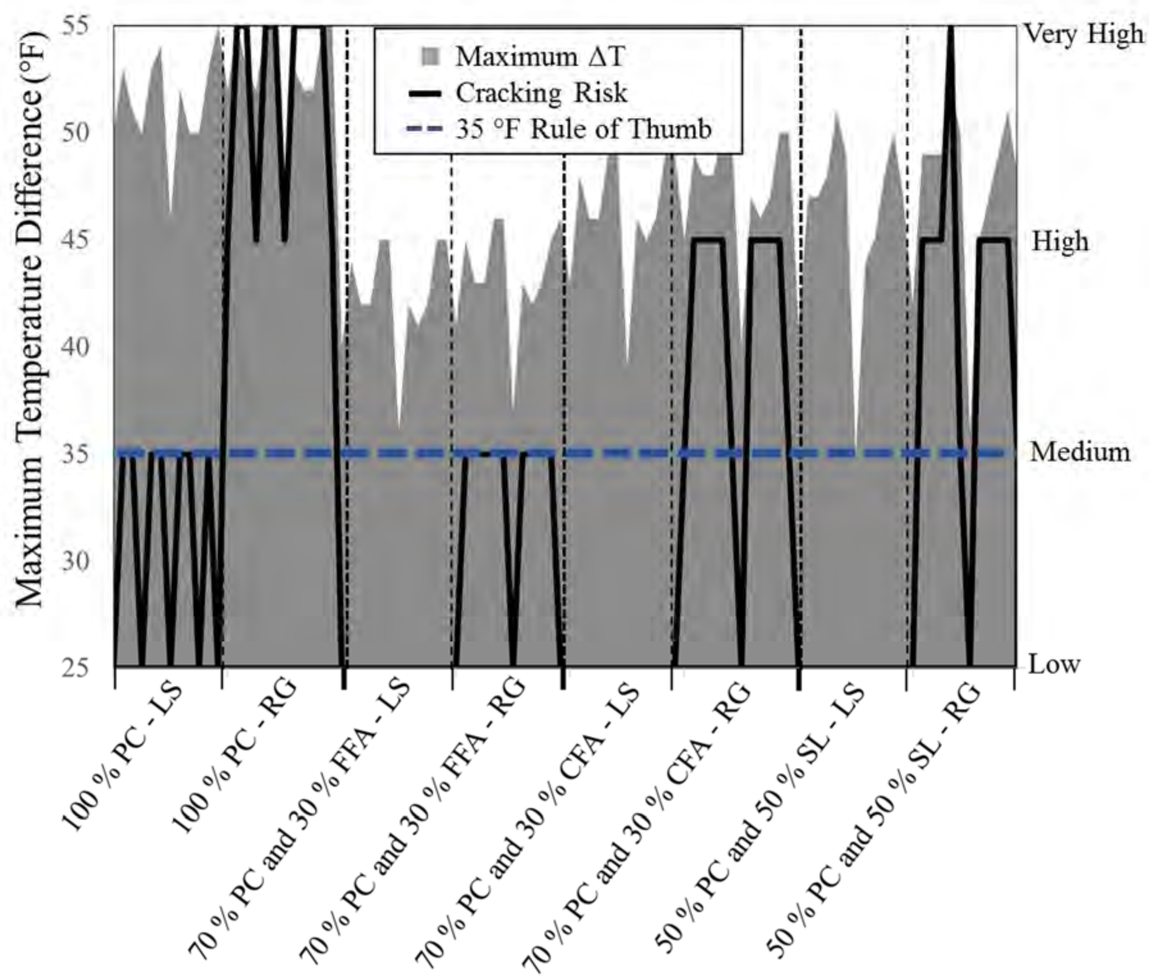


Figure 6-20: Comparing maximum temperature differences and their corresponding cracking risks on a 7 ft cross section

Chapter 7

Reinforcement Design and Specification Requirements for ALDOT Mass Concrete Elements

The purpose this project was to recommend a mass concrete specification for use in ALDOT construction. The following sections contain recommendations for shrinkage and temperature reinforcement design for mass concrete members as well as recommendations for a mass concrete specification. The mass concrete specification draft is presented in Appendix A.

7.1 SHRINKAGE AND TEMPERATURE REINFORCEMENT REQUIREMENTS FOR MASS CONCRETE

The summary of crack widths from visual inspections of field elements are presented in Table 7-1. A summary of the provided amount of shrinkage and temperature reinforcement for each field element and the predicted crack widths using Bamforth's prediction method are presented in Table 7-2. Table 7-3 contains the same summary, except for each element the amount of shrinkage and temperature reinforcement provided has been changed to the minimum amount required by AASHTO LRFD (2016), which is calculated using Equation 2-10. Table 7-4 contains the same summary with the minimum amount of shrinkage and temperature reinforcement set at 0.60% as required by AS 3600 (2009). The Australian requirement was included to compare the crack widths expected to result when a much greater minimum amount of reinforcement is specified. The bulk temperature differences used to find the maximum crack widths are the same as the bulk temperature differences used in Table 5-25.

Table 7-1: Crack width summary from visual inspections

Element	Crack Number	Largest width (in.)	Average width (in.)
Albertville Bent Cap	Could not inspect		
Harpersville Crashwall	1	0.005	
Scottsboro Pedestal	1	0.022	0.016
	2	0.021	0.016
	3	0.018	0.013
	4	0.013	0.010
Scottsboro Bent Cap	Could not inspect		
Elba Bent Cap	No cracks found		
Birmingham Column	No cracks found		
Brewton Bent Cap	Many minor cracks	< 0.005	

Table 7-2: Crack width prediction summary using amount of S&T reinforcement provided on plans

Element	$\rho_{provided}$ (%)	Predicted Crack Width (in.)
Albertville Bent Cap	0.55	0.003
Harpersville Crashwall	0.17	0.009
Scottsboro Pedestal	0.04	**0.022**
Scottsboro Bent Cap	0.56	0.002
Elba Bent Cap	0.95	0.002
Birmingham Column	0.19	0.006
Brewton Bent Cap	0.72	0.003

Note: Bolded values indicate crack widths in excess of 0.012 inches

Table 7-3: Crack width prediction summary using minimum amount of S&T reinforcement required by AASHTO (2016)

Element	A_s (in^2/ft)	Predicted Crack Width (in.)
Albertville Bent Cap	0.43	0.005
Harpersville Crashwall	0.37	0.007
Scottsboro Pedestal	0.60	0.009
Scottsboro Bent Cap	0.46	0.004
Elba Bent Cap	0.34	0.005
Birmingham Column	0.48	0.009
Brewton Bent Cap	0.41	0.007

Table 7-4: Crack width prediction summary using minimum amount of S&T reinforcement required by AS 3600 (2009)

Element	$\rho_{provided}$ (%)	Predicted Crack Width (in.)
Albertville Bent Cap	0.60	0.002
Harpersville Crashwall		0.003
Scottsboro Pedestal		0.003
Scottsboro Bent Cap		0.002
Elba Bent Cap		0.003
Birmingham Column		0.003
Brewton Bent Cap		0.004

Based on these predicted crack widths, the amount of longitudinal reinforcement currently provided in ALDOT bent caps is sufficient to limit crack widths and prevent through-cracking from occurring due to long-term bulk temperature changes. For the bent caps, much more reinforcement is supplied than the minimum amount of shrinkage and temperature reinforcement required by AASHTO LRFD (2016). The amount of reinforcement provided in these elements limits the predicted crack widths in Table 7-2 to less than the predicted crack widths in Table 7-3 using the minimum required amount of reinforcement. The amount of reinforcement provided in the bent caps is comparable to the amount required by the Australian standard, as are the crack widths. While it is not likely that engineers designed these members with through-cracking in mind, they incidentally provided more than enough reinforcement to limit crack widths to less than the service level requirement of 0.012 inches. This claim is supported by the visual inspections of the available bent caps as well as the results of crack predictions. Brewton was the only inspected bent cap that showed signs of long-term thermal cracking, and based on the amount of reinforcement provided, these cracks are not expected to grow beyond a serviceable limit, as defined in Table 2-4.

The column and crashwall also contain an adequate amount of shrinkage and temperature reinforcement. The amount of reinforcement provided is comparable to the minimum amount required by AASHTO (2016). The longitudinal reinforcement in the crashwall and the transverse reinforcement in the column provide the necessary reinforcement to keep cracks from becoming excessively large. Based on predicted crack widths these elements will experience some cracking, but none that will cause serviceability issues. The visual inspections revealed minor cracking in the crashwall whereas no cracking was observed in the column. For these reasons, it is concluded that current ALDOT design of crashwalls and columns is adequate for long-term shrinkage and temperature demands.

In contrast, the Scottsboro pedestal does not contain enough shrinkage and temperature reinforcement to prevent thermal cracking associated with mass concrete. The visual inspection alone (Table 7-1) showed signs of crack widths exceeding serviceability limits (0.012 in.). Crack predictions show that if the amount of reinforcement provided was 0.60 in^2/ft—the minimum amount required by AASHTO (2016) (Table 7-3)—then crack widths would not be expected to exceed 0.012 inches. The provided transverse reinforcement for this element consists of No. 4 bars at 12 inch spacing on center, which satisfies the maximum spacing requirement for shrinkage and temperature bars in ACI 318 (2014) and AASHTO LRFD (2016) (see Table 2-6). However, this does not satisfy the minimum amount of required shrinkage and temperature steel as seen in Table 7-3. It is recommended that consideration be given to the amount of transverse reinforcement provided for these pedestals to mitigate the effects of long-term thermal and shrinkage cracking by supplying a minimum of 0.60 in2/ft as required by AASHTO (2016). AASHTO also states that shrinkage and temperature reinforcement must be placed near surfaces exposed to daily temperature changes and in structural mass concrete (Table 2-6). Consideration should be given to this design specification when placing shrinkage and temperature reinforcement.

7.2 Maximum Concrete Temperature Limit

The maximum concrete temperature limit must be established to minimize the risk of DEF. As discussed in Section 6.2.2.1, the use of SCMs aid in the mitigation of DEF (Folliard et al. 2006). As such, based on ACI 201.2R (2016) a temperature limit of 185°F is recommended for mass concrete elements that meet the SCM type and use requirements of ACI 201.2R (2016). Florida has adopted a limit of 180°F, and West Virginia allows the maximum concrete temperature to exceed 160°F if approved by the project engineer (see Table 3-8). The limit of 160°F is still recommended for any element using only portland cement or a limited content of Class C fly ash, Class F fly ash, or slag cement. Note that the 158°F limit as recommended by Taylor et al. (2001) is rounded up to 160°F.

Several states have adopted 160°F as the maximum concrete temperature limit (see Figure 3-3). Given the recent publication of Table 6-6 by ACI 201.2R (2016), more states may change their mass concrete specifications to allow higher maximum concrete temperatures. Additionally, high-early strength (Type III) cement shall not be used in any element with a least dimension equal to or greater than 4.0 ft., unless approved in writing by the Engineer. It is recommended that this statement be added to the Master Proportions Table in ALDOT Article 501.02.

7.3 MASS CONCRETE SIZE DESIGNATION

Based on the ConcreteWorks analysis (see Table 6-7) and the least dimensions designations from other state specifications (see Table 3-6), the most conservative size designation for a mass concrete member shall be taken as 4 ft. This designation is used by ACI 301 (2016) and some state DOTs as a conservative value, and it should be used when no properties about the concrete mixture proportions are known.

The results of the ConcreteWorks statistical analysis shown in Table 6-5 reveal that the least dimension of a mass concrete member has a significant effect on the maximum concrete temperature, maximum concrete temperature difference, and cracking risk. Based on Table 6-7 the least dimension that should be designated as mass concrete will change based on the cementing materials and the type of coarse aggregate used, especially when incorporating the most current maximum concrete temperature limits from ACI 201 (2016) (Table 6-6). Use of SCMs is thus very beneficial in mass concrete because they reduce the maximum temperature and the potential for DEF. Use of a low-CTE coarse aggregate will reduce the cracking risk and thereby thermal cracking, but it will not significantly aid in lowering the maximum in-place concrete temperature or temperature difference (see Table 6-5, Figure 6-20). Table 7-5 contains a summary of the element sizes from Table 6-7 that should be designated as mass concrete.

Table 7-5: Least dimension designation based on ConcreteWorks analysis

SCM Use	Coarse Aggregate Type	Element Least Dimension	Reason for Designation
100% PC without SCMs	Any	4.0 ft	High risk of DEF without SCMs
≥ 25% FFA	Limestone	> 8.0 ft	High cracking risk
	River Gravel	6.0 ft	
≥ 35% CFA	Limestone	> 8.0 ft	High cracking risk
	River Gravel	6.0 ft	
≥ 50% SL	Limestone	7.0 ft	High cracking risk
	River Gravel	5.0 ft	

It is recommended that for any element containing 100% portland cement, 4.0 ft should be the mass concrete least dimension. As seen in Table 6-7, concrete containing limestone aggregate has a lower cracking risk for larger elements than concrete containing river gravel. Therefore, when combined with sufficient amounts of fly ash or slag cement, elements containing limestone aggregate may have a greater least dimension before being classified as mass concrete. When the SCM requirements of Table 6-6 are met, any concrete containing limestone aggregate should be designated mass concrete at a least dimension of 7.0 ft. A concrete that does not contain limestone aggregate and a sufficient amount of fly ash or slag cement to satisfy Table 6-7 should be classified as mass concrete at a least dimension of 5.0 ft.

Based on this summary, a mass concrete size designation was developed based on the cementing materials and coarse aggregate types used. Figure 7-1 is a flowchart detailing the size designation. Because limestone is commonly used in Alabama, the 7 foot least dimension designation is based on the use of limestone coarse aggregate. The use of "other" coarse aggregate types is in reference to river gravel in this study, but also refers to granite and any other coarse aggregate with a higher concrete CTE than limestone.

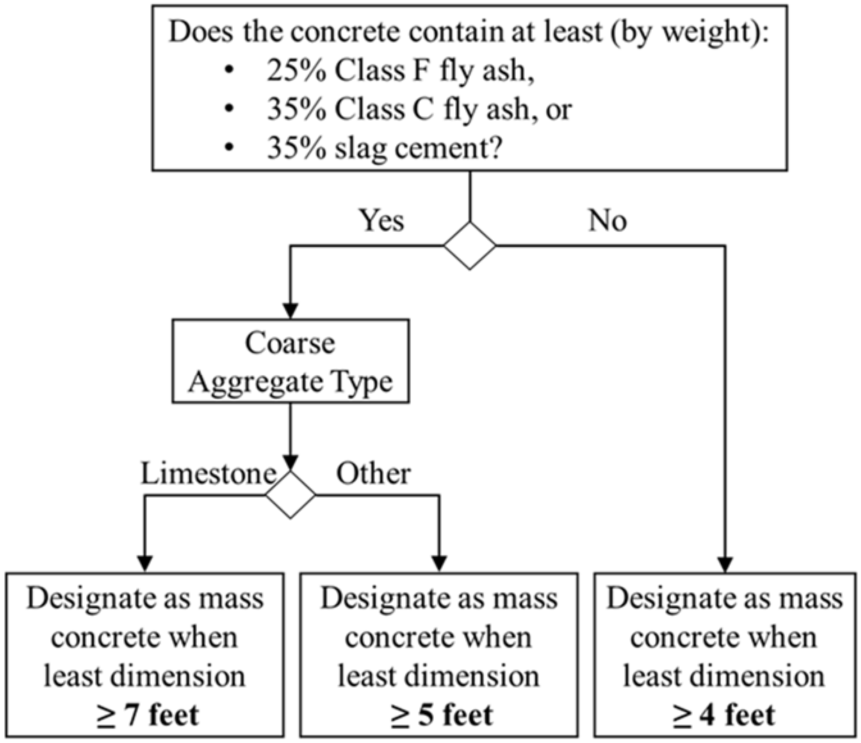


Figure 7-1: Mass concrete least dimension selection flow chart

7.4 MAXIMUM CONCRETE TEMPERATURE DIFFERENCE LIMIT

7.4.1 Limit When No Information about Aggregate Is Known

The maximum concrete temperature difference limit must be established to minimize the risk of thermal cracking. As discussed in Section 2.2.2, the default historical temperature difference limit of 35°F (20°C) comes from research done by Bamforth and Price (2007) that includes siliceous river gravel as the coarse aggregate. Table 7-6 (from Section 2.2.2) details temperature difference limits recommended by Bamforth and Price (2007) for other coarse aggregates, such as 63°F (35°C) for limestone coarse aggregate.

Table 7-6: Temperature difference limits (°C) based on assumed typical values of CTE and tensile strain capacity (Bamforth and Price 2007) ($\Delta°\mathrm{F} = \mathbf{1.8} \times \Delta℃$)

Aggregate Type	Gravel	Granite	Limestone	Lightweight
Coefficient of Thermal Expansion ($\mu\varepsilon$/°C)	13.0	10.0	9.0	9.0
Tensile Strain Capacity ($\mu\varepsilon$)	65	75	85	115
Limiting temperature change for different restraint factors, R (°C):				
R = 1.00	6	9	12	20
R = 0.80	8	12	16	25
R = 0.60	11	17	22	34
R = 0.42	**20**	**28**	**35**	**53**
R = 0.30	24	36	46	71

While the recommended temperature difference value should remain 35°F when no information about the coarse aggregate is known, Bamforth's research shows promise for developing an age-dependent temperature difference limit based on the predicted development of concrete tensile strength and creep effects with time, which is discussed in the following section.

7.4.2 Development of an Age-Dependent Maximum Concrete Temperature Difference Limit for an ALDOT Specification

To develop this age-dependent concrete temperature difference limit, Equation 2-1 was modified to produce Equation 7-1. The supporting equations to calculate a time-dependent concrete tensile strength and elastic modulus may be found in Equation 2-2 through Equation 2-5 (Bazant and Baweja 2000). The method for calculating a time-dependent creep modification factor may be found in Equation 2-7 through Equation 2-9.

$$\Delta T_{max}(t) = \frac{f_t(t)}{E_c(t) \times K(t) \times CTE \times R}$$ **Equation 7-1**

Where,

$\Delta T_{max}(t)$ = allowable concrete temperature difference as a function of time (°F),

$f_t(t)$ = concrete tensile strength as a function of time (psi),

$E_c(t)$ = concrete modulus of elasticity as a function of time (psi),

$K(t)$ = creep modification factor as a function of time (unitless),

CTE = concrete coefficient of thermal expansion (in./in./°F), and

R = Restraint factor (0 = unrestrained; 1 = full restraint) (unitless).

Because the modeling of creep effects is so significant in the prediction of thermal cracks in mass concrete, the Bamforth (2007) creep coefficient of 0.65 (Section 2.2.2) was tested against the time-dependent creep modification factors calculated using the B3 Model (Section 2.2.3.1) and the Modified B3 Model (Byard and Schindler 2015) (Section 2.2.3.2). The variables used in calculating the B3 and Modified B3 Model compliance values are found in Table 7-7.

Table 7-7: Variable inputs for B3 and Modified B3 compliance calculations

Input	Variable	Value
Specified 28-day compressive strength	f'_{c28}	4,000 psi
Cement Content	c	620 lb/yd^3
Empirical Constants	m	0.50
	n	1.00
Water-cement ratio	w/c	0.44
Aggregate-cement ratio	a/c	4.79

7.4.3 Bamforth, B3 Model, Modified B3 Model – Compliance and Creep Factors

The calculated compliance values for the B3 and Modified B3 compliance functions are plotted using time-steps of 0-24 hours, 24-48 hours, 48-72 hours, 72-120 hours, and 120-168 hours (7 days total) in Figure 7-2. As stated in Section 2.2.3.2, the original B3 Model underestimates creep effects at early ages (Byard and Schindler 2015). This is seen in the significant difference in the two models at 24 hours, but the two compliance functions converge as the concrete ages. Further proof of this is seen in the difference in creep factors in Figure 7-3. The sudden jumps at 24 hours, 72 hours, and 120 hours reflect the ages at which loads were applied (to). Based on Figure 7-3, the Modified B3 Model is more appropriate than the original B3 Model for an early-age thermal cracking analysis (Byard and Schindler 2015).

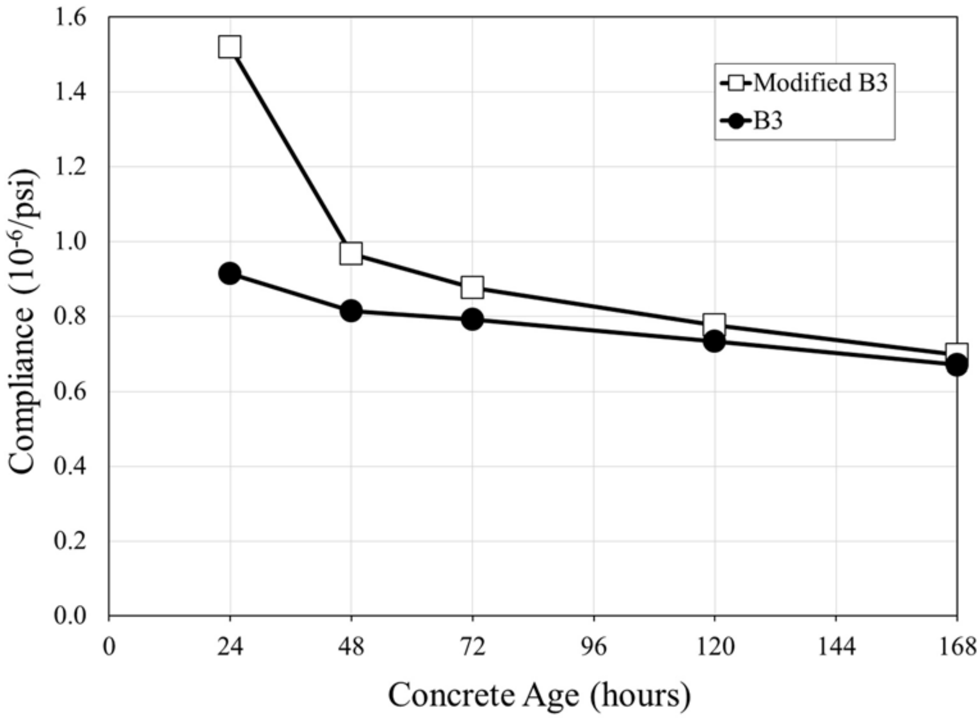


Figure 7-2: Compliance values: B3 versus Modified B3

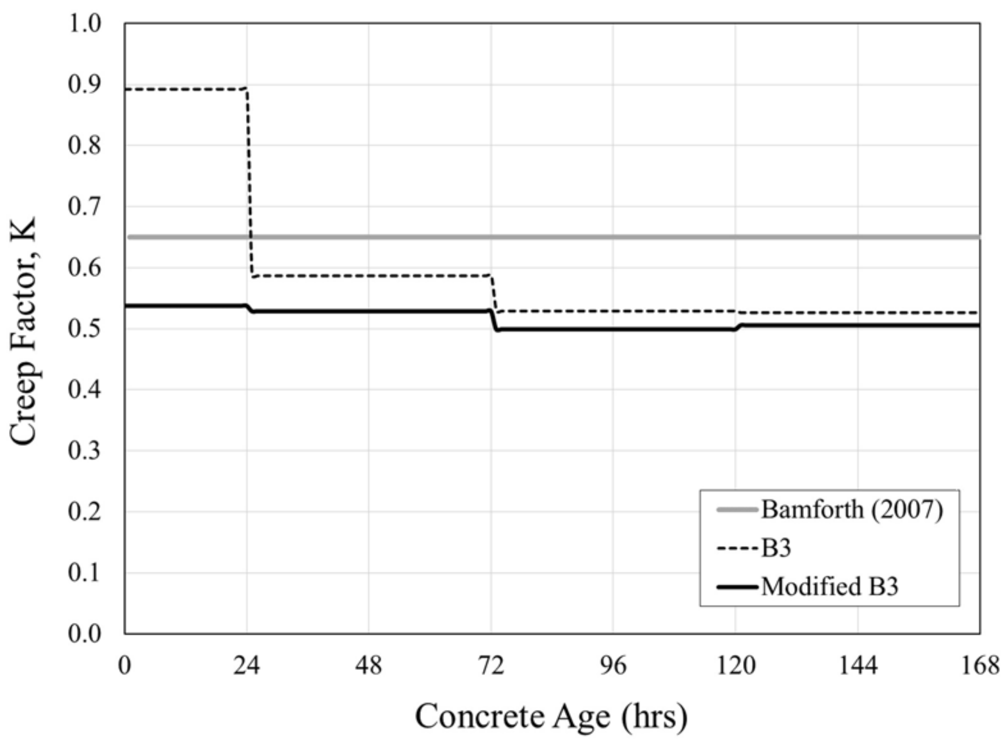


Figure 7-3: Creep factors: Bamforth versus B3 versus Modified B3

7.4.4 Comparing the Bamforth and Modified B3 Model Maximum Temperature Difference Limits

Age-dependent concrete temperature difference limits were calculated with Equation 7-1 using the inputs in Table 7-8. Both the Bamforth and Modified B3 Model creep factors were used for comparison. Equation 7-2 is an example calculation of Equation 7-1 at 24 hours using the Modified B3 Model creep factor. The resulting age-dependent temperature difference limits for limestone and river gravel concretes are plotted in Figure 7-4 and Figure 7-5, respectively. The sudden jumps in the Modified B3 Model plots indicate a change in loading age, t_o.

Table 7-8: Input values for example time-dependent temperature difference limit

Parameter	Value
Time, t (hours)	0 to 168
Restraint Factor, R	0.50
Specified 28-day compressive strength, f'_{c28} (psi)	4,000
Concrete CTE (in./in./°F) (limestone, river gravel)	5.52×10^{-6}, 6.95×10^{-6}
Creep factor, K (Bamforth, Modified B3)	0.65, time-dependent

Equation 7-2

$$\Delta T_{max}(t) = \frac{f_t(24)}{E_c(24) \times K(24) \times CTE_{LS} \times R}$$

$$= \frac{178\, psi}{1655000\, psi \times 0.538 \times 5.52 \times 10^{-6}\ in./in./°\mathrm{F} \times 0.50}$$

$$= \mathbf{61}\ °\mathrm{F}$$

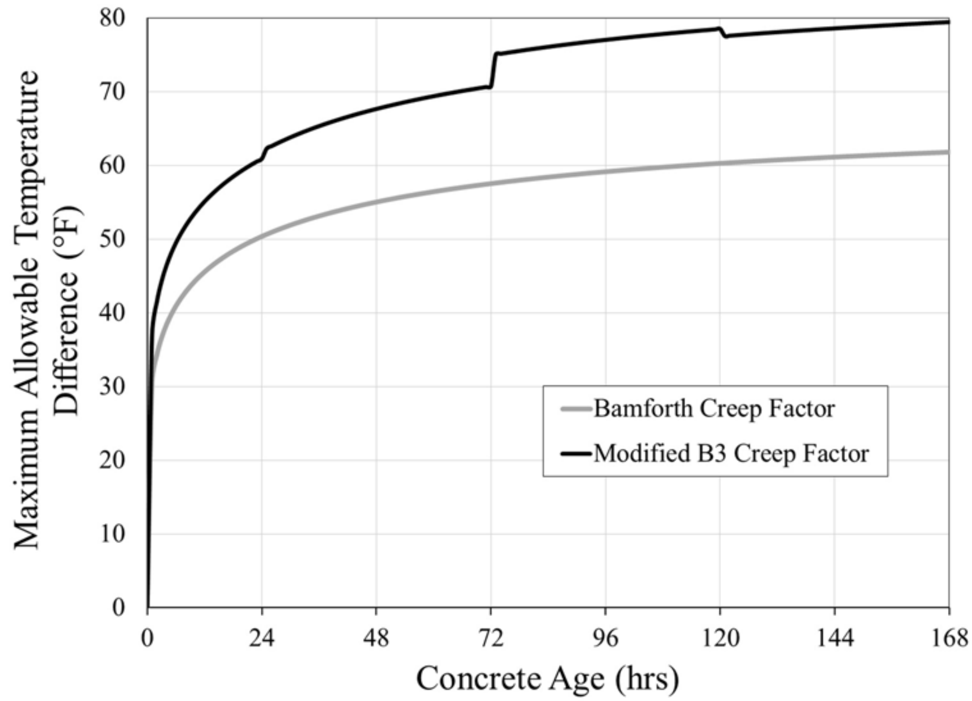


Figure 7-4: 7-day maximum allowable temperature difference limits – Limestone

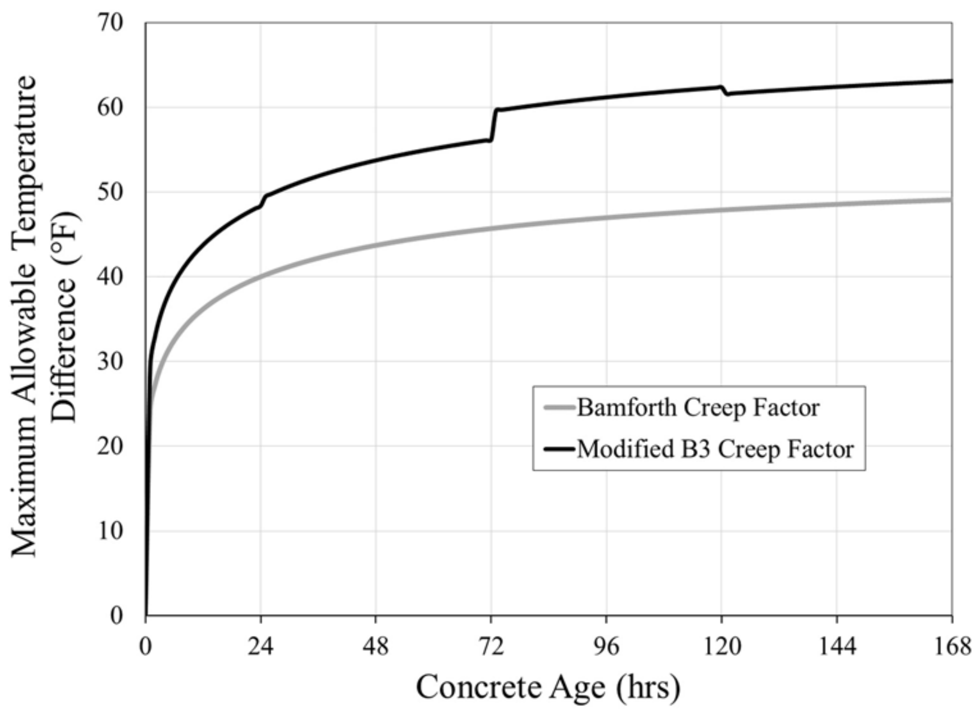


Figure 7-5: 7-day maximum allowable temperature difference limits – River gravel

Bamforth's simple, constant creep factor of 0.65 produced a more conservative maximum allowable temperature difference than the Modified B3 factor in both Figure 7-4 and Figure 7-5, and it was also within 10°F of the values of the Modified B3 factor in both cases. Therefore, for simplicity of application, Bamforth's constant creep factor is recommended for ALDOT use.

7.4.5 Age-Dependent Allowable Temperature Difference Limit

Figure 7-6 is a comparison of the maximum allowable temperature differences with Equation 7-1 for limestone and river gravel using Bamforth's constant creep factor of 0.65 and a restraint factor of 0.5. The inputs are the same from Table 7-8, except that for additional safety the mean compressive strength development (where f_{c28m} = f'_{c28} + 1,200 psi) is used to determine the concrete modulus of elasticity.

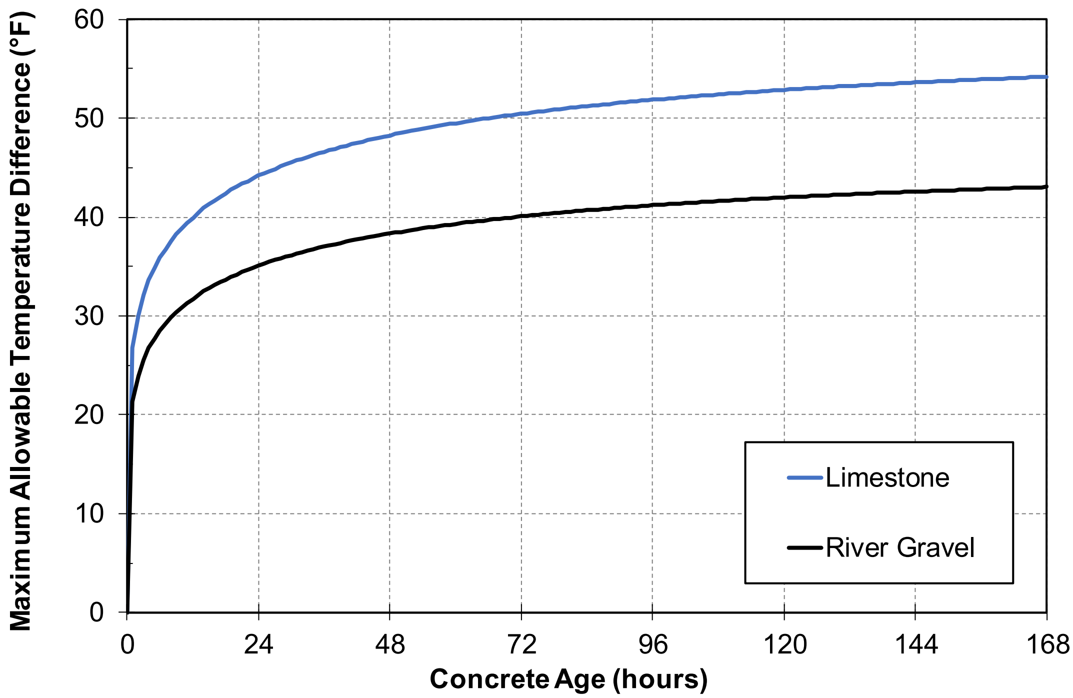


Figure 7-6: Maximum allowable temperature difference limits – Bamforth

Table 7-9 is a list of temperature difference limits from Figure 7-6 for concrete containing river gravel. Ideally, this is the type of table to include in an ALDOT mass concrete specification to give contractors and engineers a quick and easy reference by which to check the maximum allowable temperature difference. Notice that the early-age temperature difference at 24 hours is 35°F, which is similar to Bamforth's recommendation for river gravel in Table 7-6. The Iowa DOT—as seen in Table 3-8—uses an age-dependent mass concrete temperature difference limit like the one seen here.

Table 7-9: Maximum allowable temperature differences – River gravel concrete

Concrete Age, *t* (hours)	Maximum Allowable Temperature Difference, $\Delta T_{max}(t)$ (°F)
12	32
24	35
36	37
48	38
60	39
72	40
84	41
96	41
108	42
120	42
132	42
144	43
156	43
168	43

7.4.5.1 Modifying the Coefficient of Thermal Expansion

Table 7-9 is for river gravel concrete with a CTE of 6.95 × 10-6 in./in., but the temperature difference values may be adjusted for other CTE values by modifying the temperature differences by a ratio of two concrete CTEs. Equation 7-3 introduces a CTE modification factor by which this is accomplished. Once the CTE modification factor is known, the temperature difference may be modified by using Equation 7-4. By using these equations in conjunction with Table 7-9, an age-dependent temperature difference limit may be found for any mass concrete member whose CTE is known.

$$F_{CTE} = \frac{6.95 \times 10^{-6}\ in./in./°F}{CTE_{measured}}$$ **Equation 7-3**

Where,

F_{CTE} = concrete CTE modification factor (unitless),

$CTE_{measured}$ = measured concrete CTE (in./in./°F).

$$\Delta T_{max,modified}(t) = F_{CTE} \times \Delta T_{max}(t) \quad \textbf{Equation 7-4}$$

Where,

$\Delta T_{max,modified}(t)$ = modified allowable maximum temperature limit (°F),

F_{CTE} = concrete CTE modification factor (unitless), and

$\Delta T_{max}(t)$ = allowable maximum temperature limit as a function of concrete age (°F) (listed in Table 7-9).

It is best to test the concrete CTE using AASHTO T 336 (2011), but default values for different types of aggregate in Alabama may be found in Table 7-10. If the coarse aggregate type is unknown, then an age-dependent allowable temperature difference limit should not be used.

Table 7-10: Default CTE values for concrete made with typical Alabama coarse aggregate types (Schindler et al. 2010)

Coarse Aggregate Type	Concrete CTE (in./in./°F)
River Gravel	6.95×10^{-6}
Limestone	5.52×10^{-6}
Granite	5.60×10^{-6}

7.4.6 Tiered Maximum Concrete Temperature Difference Limit

Three different approaches have been discussed in Section 7.4 for predicting an allowable temperature difference limit:

1. When the concrete CTE for a specific project has been measured,
2. When the coarse aggregate type is known and a corresponding default CTE value is used, and
3. When the coarse aggregate type and CTE value are unknown.

These three categories (or tiers) are listed in Table 7-11 along with the method for calculating the specified temperature difference limit. Ideally, this table would be included in an ALDOT mass concrete specification.

Table 7-11: Tiered allowable temperature difference limit for use in a mass concrete specification.

Tier	Requirement	Specification
I	CTE of project concrete is tested according to AASHTO T336	Use ***known*** CTE value to calculate ***age-dependent ΔT limit*** in accordance with Table 7-9 and modified with Equation 7-4
II	Coarse aggregate type of project concrete is known, but concrete CTE has not been tested	Use ***default*** CTE value from Table 7-10 to calculate ***age-dependent ΔT limit*** in accordance with Table 7-9 and modified with Equation 7-4
III	Coarse aggregate type of project concrete is unknown	Use maximum ΔT limit of ***35 °F***

7.4.6.1 Comparison of Tiered Maximum Concrete Temperature Difference Limit to Field Data

To demonstrate the effectiveness of the tiered specification described in Table 7-11, temperature difference limits for every tier were calculated for each of the seven field elements and plotted against the respective measured temperature difference in Figure 7-7 through Figure 7-13. To satisfy Tier I, Table 7-12 contains the tested CTE values for each of the field elements. Five of the seven field elements exceeded the Tier III limit of 35°F. Two of the seven elements exceeded the Tier II limit. None of the recorded data exceeded the Tier I temperature limit.

Table 7-12: Tested CTE values for instrumented ALDOT field elements

Element	Coarse Aggregate Type	CTE (in./in./°F)
Albertville Bent Cap	Limestone	4.82×10^{66}
Harpersville Crashwall	Limestone	4.47×10^{-6}
Scottsboro Pedestal	Limestone	4.04×10^{-6}
Scottsboro Bent Cap	Limestone	4.04×10^{-6}
Elba Bent Cap	River Gravel	6.49×10^{-6}
Birmingham Column	Limestone	5.49×10^{-6}
Brewton Bent Cap	River Gravel	6.00×10^{-6} (assumed)

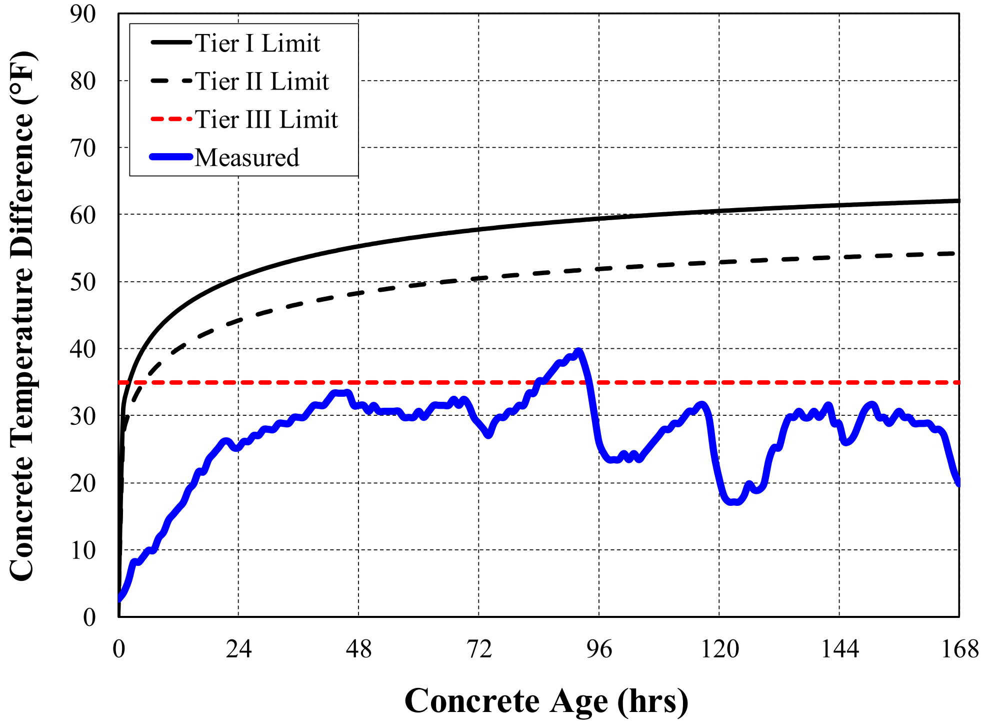


Figure 7-7: Albertville bent cap: Temperature difference data versus potential limits

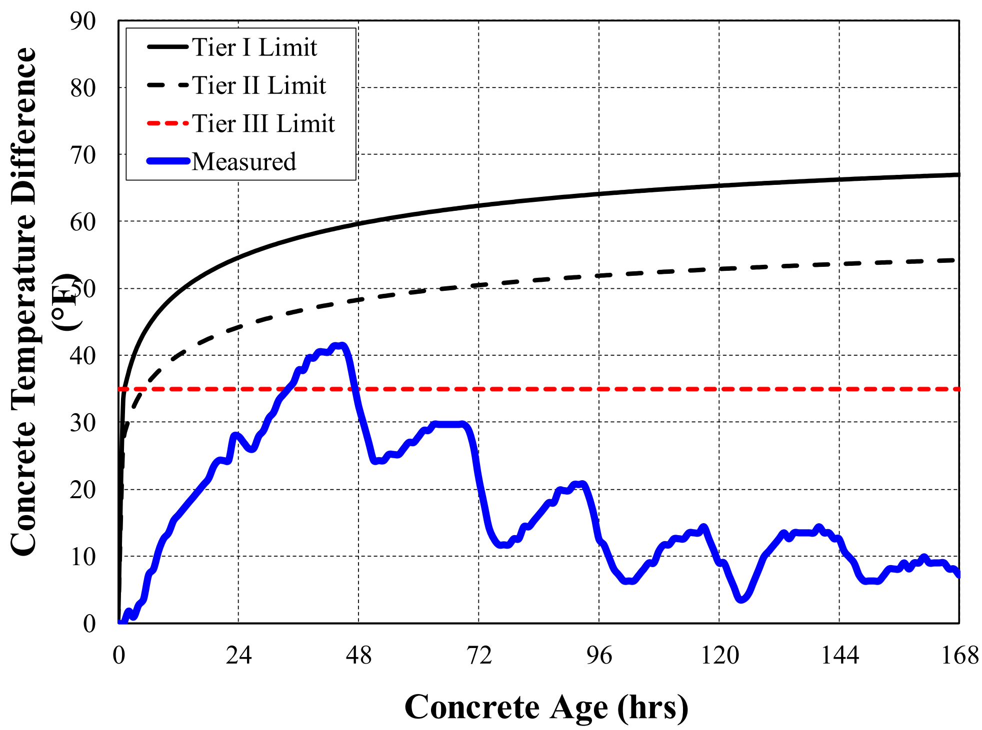


Figure 7-8: Harpersville crashwall: Temperature difference data versus potential limits

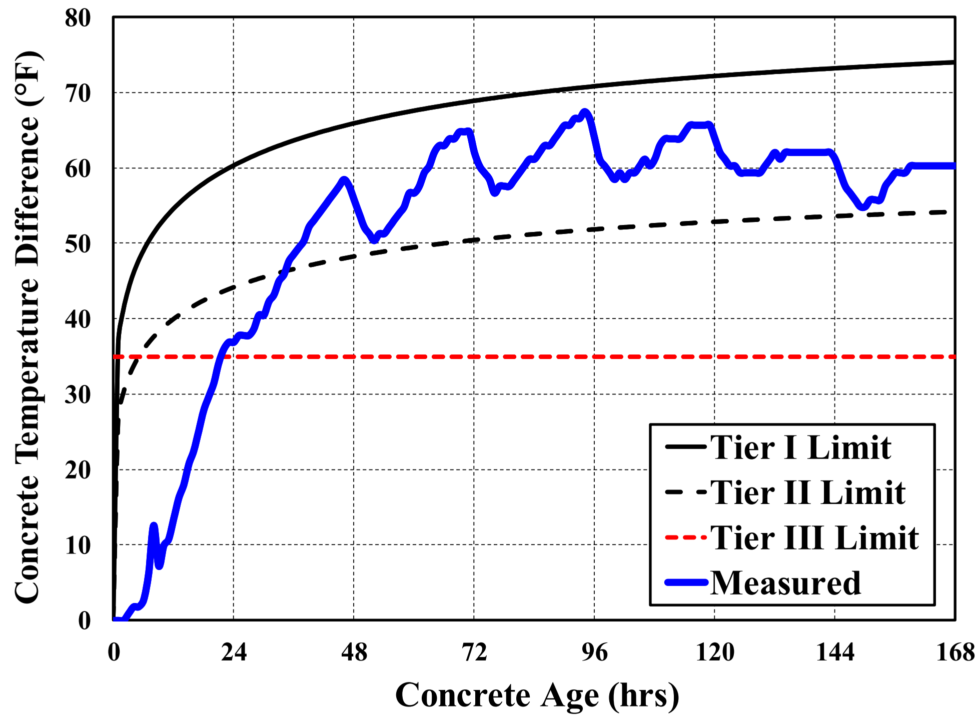


Figure 7-9: Scottsboro pedestal: Temperature difference data versus potential limits

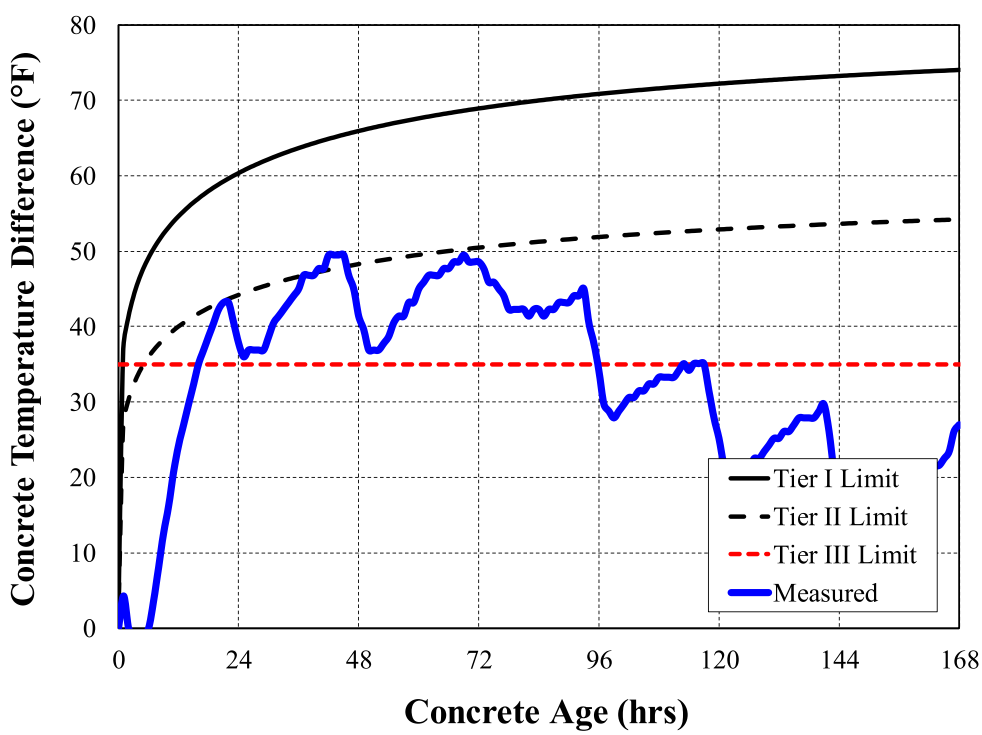


Figure 7-10: Scottsboro bent cap: Temperature difference data versus potential limits

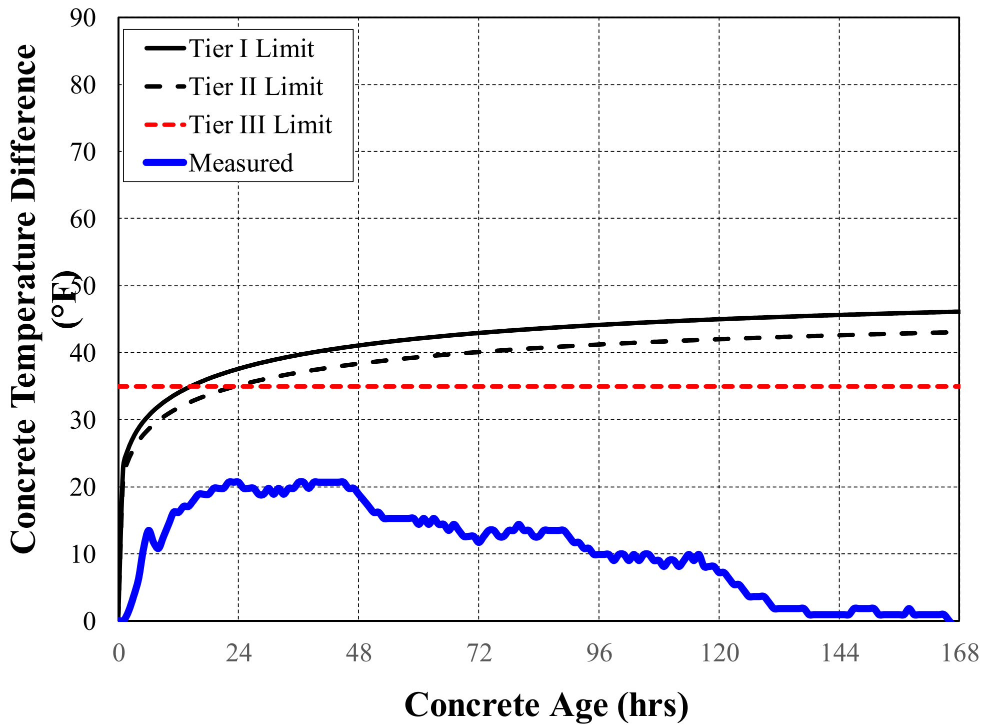


Figure 7-11: Elba bent cap: Temperature difference data versus potential limits

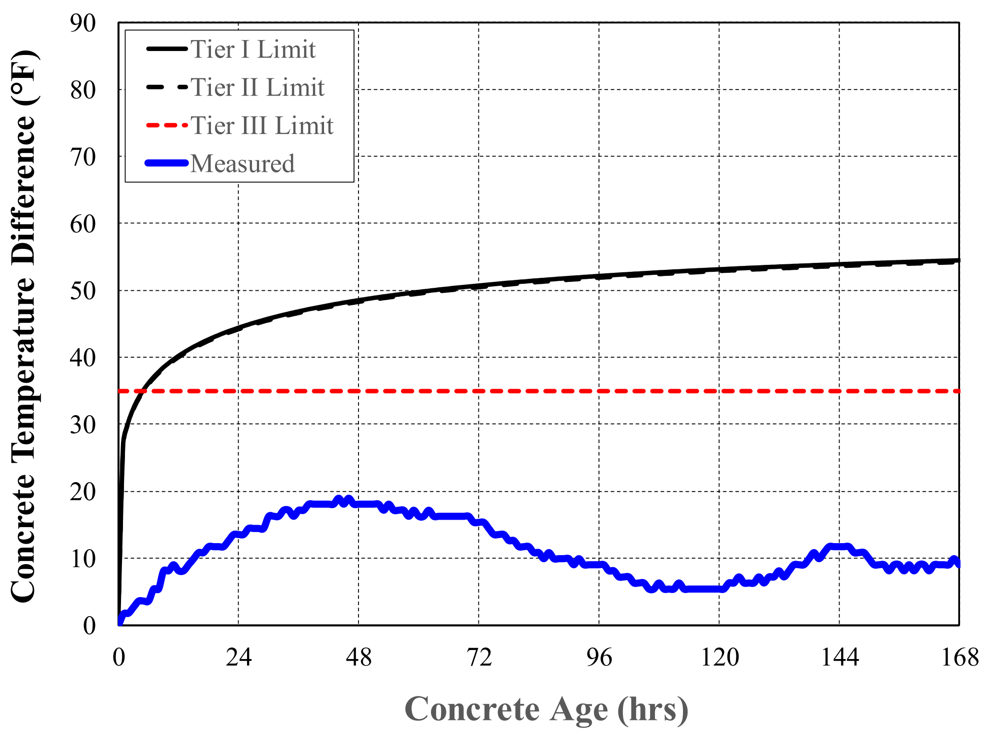


Figure 7-12: Birmingham column: Temperature difference data versus potential limits

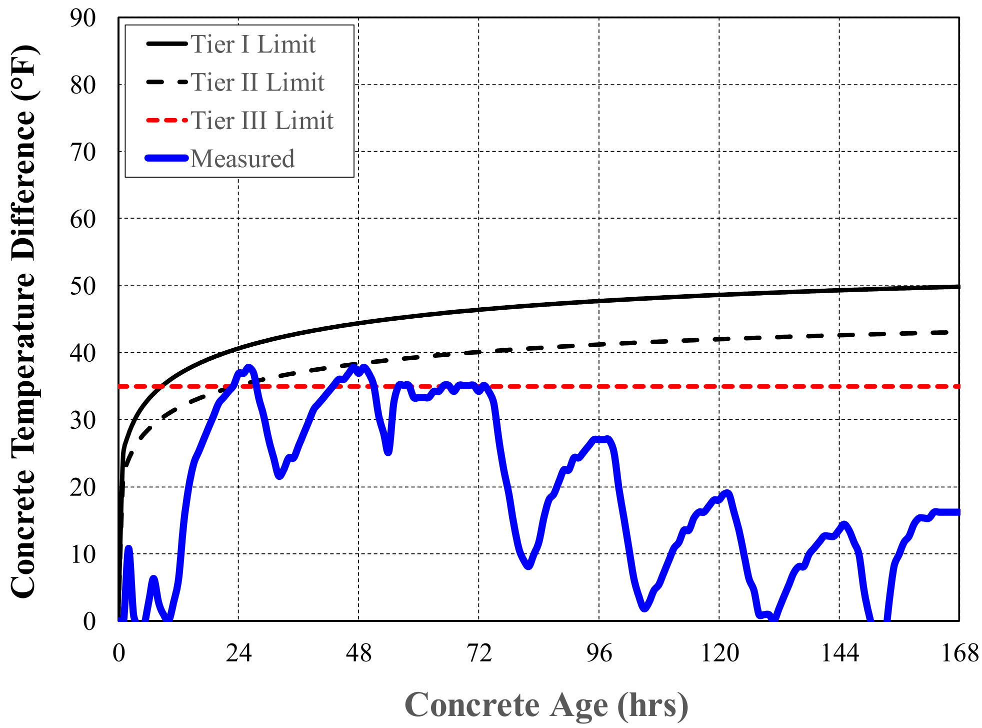


Figure 7-13: Brewton bent cap: Temperature difference data versus potential limits

The Scottsboro pedestal (Figure 7-9) and the Brewton bent cap exceeded the Tier II age-dependent temperature difference limit at approximately 34 hours and 23 hours, respectively. The Scottsboro pedestal reached a maximum temperature difference at 94 hours, and the temperature difference remained above the Tier II limit for more than 7 days. Signs of thermal cracking were observed in the Scottsboro pedestal. These cracks may become a serviceability problem. More shrinkage and temperature reinforcement may have resulted in decreased crack widths, making a large temperature difference less detrimental to the mass concrete element.

The Brewton bent cap exceeded the Tier II temperature limit at approximately 24 hours. Signs of thermal cracking were observed, but these cracks are minimal in width and are not expected to affect the serviceability of the bent cap. The amount of shrinkage and temperature reinforcement provided is suspected to have kept these crack widths small, and therefore the maximum temperature difference of 37°F is acceptable. The Harpersville crashwall also showed signs of minimal thermal cracking, but the measured crack widths were very small. No other instrumented elements showed signs of thermal cracking.

In conclusion, based on the fact that large thermal cracks were not found in any of the elements except the Scottsboro pedestal, the Tier III limit of 35°F is extremely conservative and would make it difficult for contractors trying to meet this temperature restriction, given that appropriate amounts of shrinkage and temperature reinforcement are provided. The Tier I and II temperature difference limits are more representative of observed in-place concrete behavior, and if implemented, these temperature limits would be more equitable for contractors while still providing some guidance on what temperature difference limits in mass concrete members should be.

7.5 TEMPERATURE MONITORING RECOMMENDATIONS

Temperature monitoring of mass concrete elements is needed to ensure that temperature limits are not exceeded. The temperature monitoring sensors must be able to, at minimum, perform the functions of the temperature sensors described in Section 4.2.1. It is recommended that one cross section be instrumented, but having multiple sensors in each location for redundancy is important because it is common for temperature sensors to be damaged or lost during concrete placement. Therefore, it is recommended that two independent sensors be installed at each prescribed location.

7.5.1 Duration of Temperature Monitoring

For each of the ALDOT elements instrumented, the maximum in-place concrete temperature and maximum temperature difference were reached within 7 days for recorded data and ConcreteWorks predictions. It is therefore recommended that ALDOT mass concrete temperature monitoring take place for the first 7 days after placement. Arkansas and Idaho also require that mass concrete be monitored for 7 days after concrete placement.

7.5.2 LOCATION OF TEMPERATURE MONITORING SENSORS

Installing the temperature sensors in the proper locations is essential for capturing the maximum in-place concrete temperature and maximum temperature difference. Therefore, the proper installation cross section as well as the sensor locations within this cross section must be carefully selected. The selected cross section should be as near to the middle of the element as possible without being near or in contact with any free ends or other elements connected to the mass concrete element. Figure 7-14 and Figure 7-16 display proper cross section locations on a bent cap and column, respectively. Figure 7-15 displays inappropriate cross section locations for a bent cap. These basic cross-section location concepts apply to columns, crashwalls, pedestals, footings, and any other elements that may be considered mass concrete. Sensors at each cross section should be positioned (a) as near to the center of the cross section as possible, (b) in the middle of the side edge, and (c) in the middle of the top edge. Two sensors should be provided at each location for a total of six sensors per cross section. The proper sensor locations on a rectangular and circular cross section are shown in Figure 7-17 and Figure 7-18.

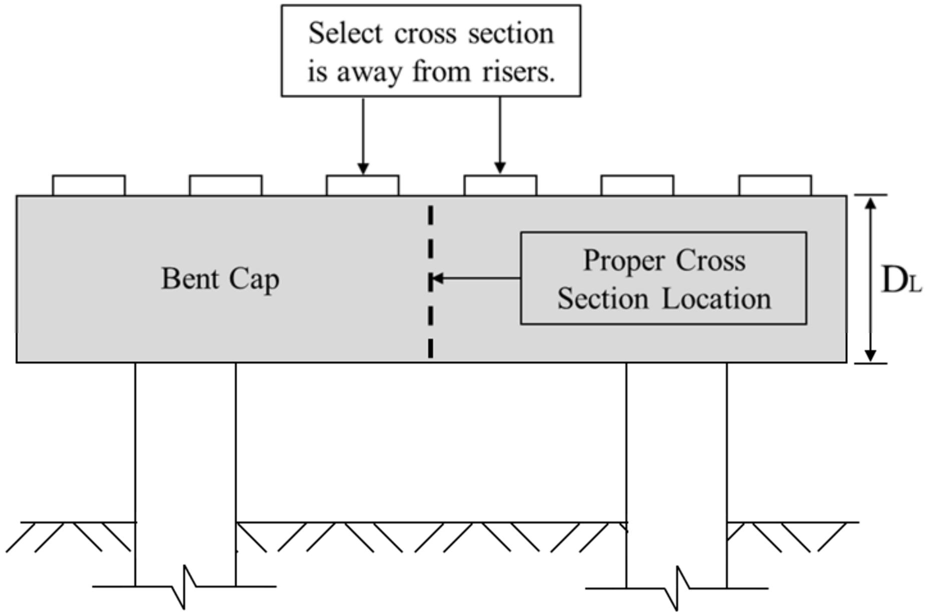


Figure 7-14: Proper cross section location on bent cap (D_L = least dimension) (n.t.s.)

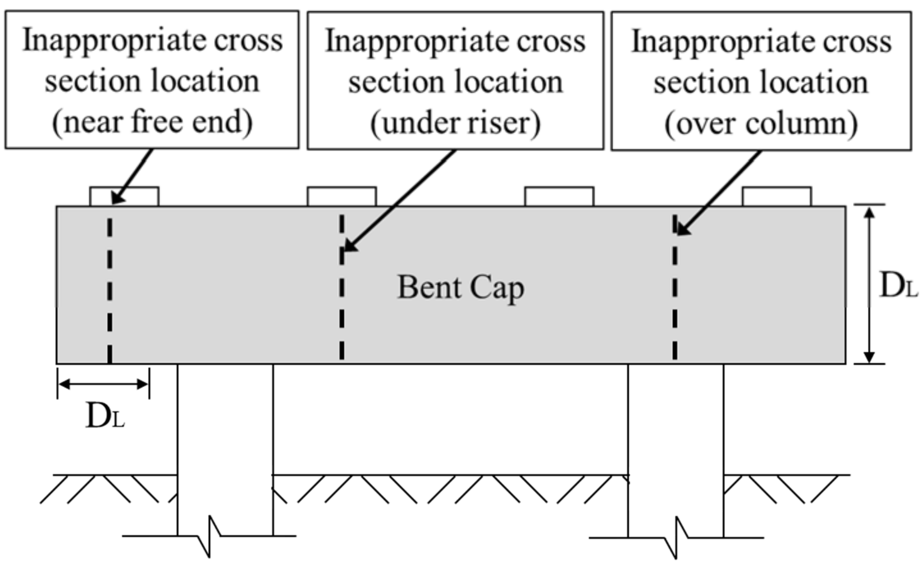


Figure 7-15: Inappropriate cross section locations on bent cap (n.t.s.)

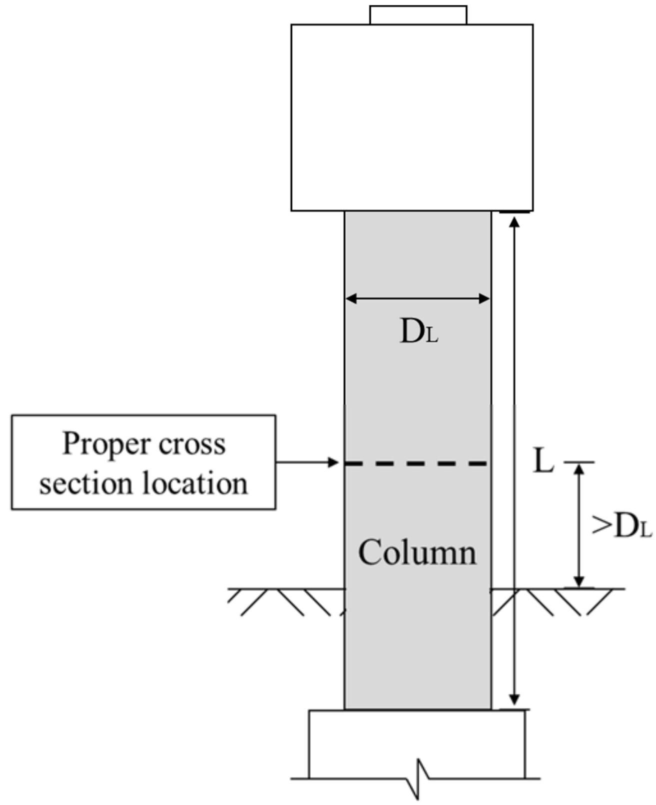


Figure 7-16: Proper cross section location on column (n.t.s.)

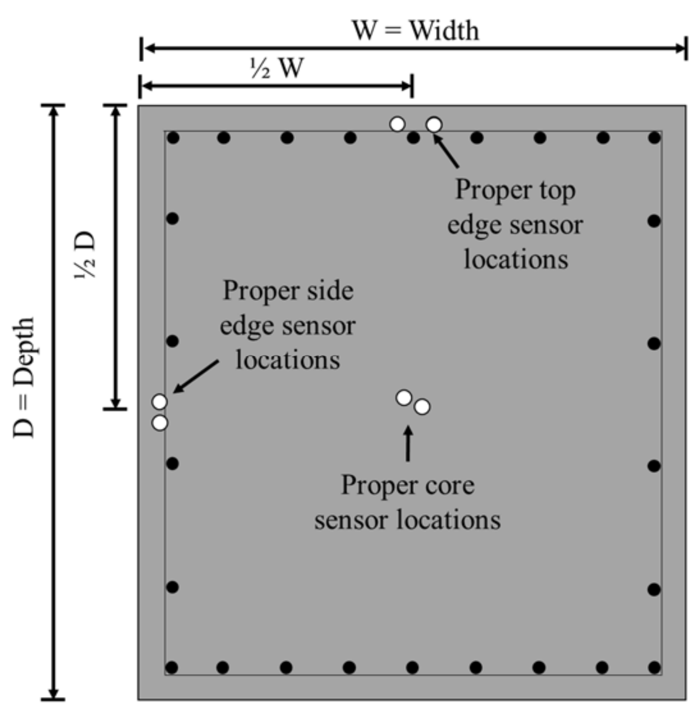


Figure 7-17: Proper sensor placement on rectangular cross section

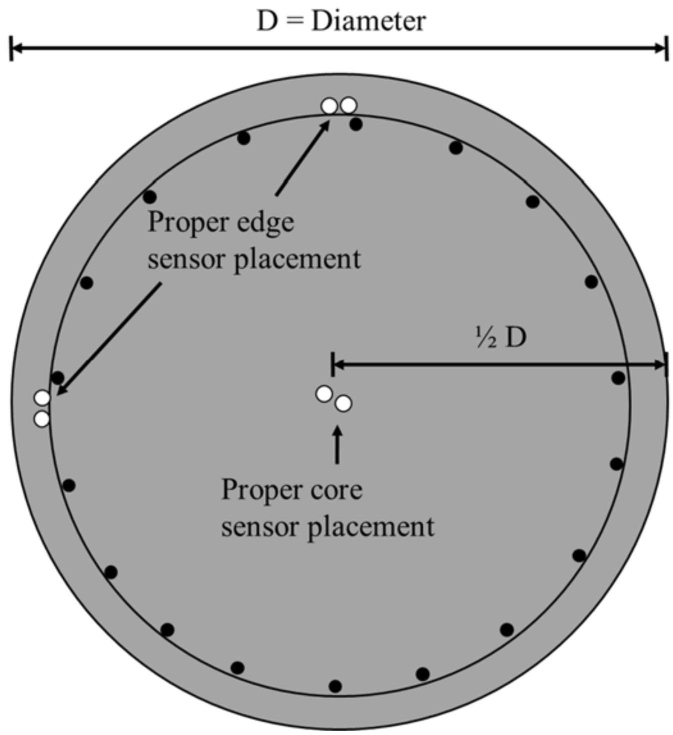


Figure 7-18: Proper sensor placement on circular cross section

The cross section and sensor locations provided are intended to assist in illustrating where sensors should be placed to capture the maximum in-place concrete temperature and largest temperature difference within the element at the location of highest cracking risk (see Figure 2-2). The edges of the element (not corner) are expected to experience the highest degree of internal restraint, and it is at these locations that temperatures will be of interest. The mass concrete specification (presented in Appendix A) contains example diagrams of how to determine the least dimension of some special cases that may be encountered.

7.5.2.1 Comparison of "Rebar Edge" versus "Cover Edge" Sensors

It is important that edge sensors be installed as close to the edge of the element as possible to capture the true temperature difference between the edge and core of the concrete. Edge sensors installed too deeply will record higher temperatures and the resulting temperature difference will be less than the true temperature difference. However, attaching sensors directly to the reinforcing steel at the edge (rebar-edge sensor) is much more practical than trying to place sensors within the concrete cover zone (cover-edge sensor) (see Figure 4-5). Because of this it is recommended that sensors be installed at the rebar-edge location with the sensor as close to the concrete edge as possible, but a comparison of rebar-edge sensor data to cover-edge-sensor data is warranted.

Rebar-edge sensors and cover-edge sensors were installed at six of the seven field elements. Figure 7-19 and Figure 7-20 compare the difference in rebar-edge and cover-edge sensors for these locations. For five of the six elements, the concrete cover is approximately 2 inches, but for the Scottsboro pedestal the concrete cover is approximately 6 inches. This is evident when comparing the data. For all elements except the pedestal the maximum difference between the rebar-edge and cover-edge sensors is between 4°F to 6°F. However, the pedestal with a concrete cover three times as thick, has a maximum temperature difference of 14°F. For this reason, measures should be taken to ensure that sensors are placed 2 to 6 inches away from the concrete surface. If the concrete cover exceeds 6 inches, measures should be taken to place the edge sensors within 6 inches of the surface. This can be accomplished by installing additional bars or brackets that extend into the cover region to support the sensor.

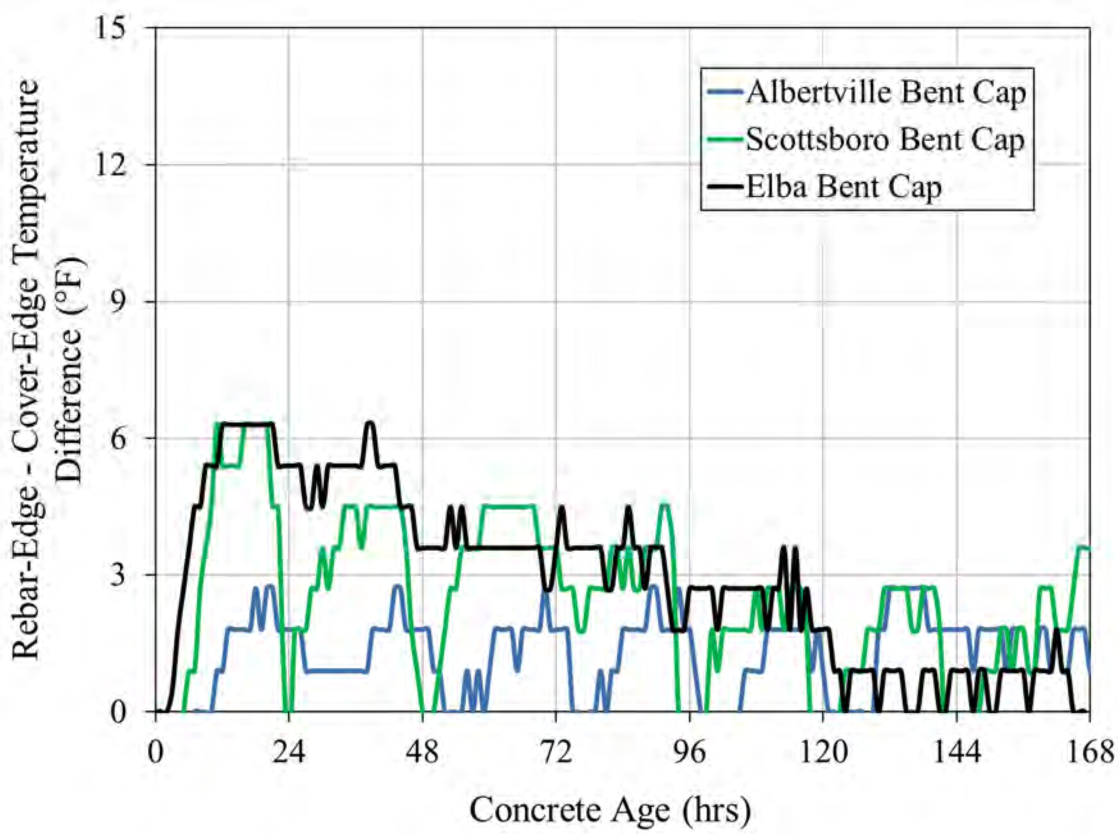


Figure 7-19: Rebar-edge – cover-edge temperature difference data (Albertville, Scottsboro, and Elba bent caps)

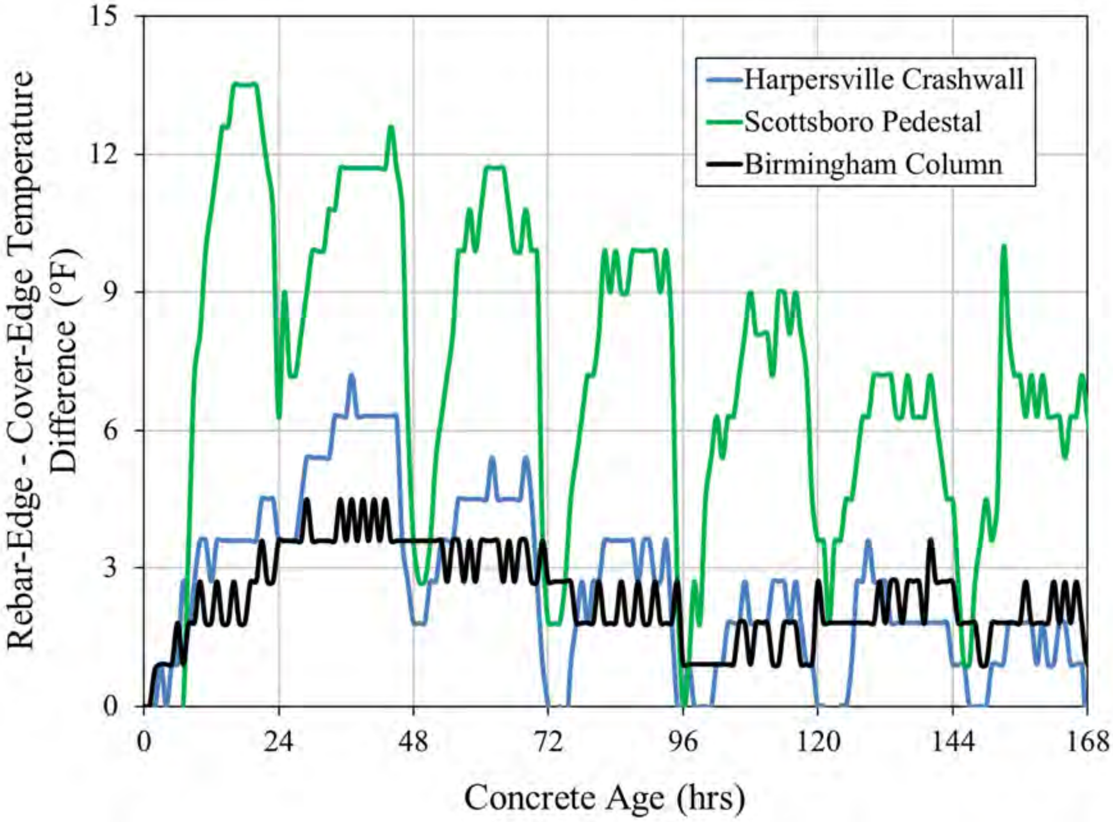


Figure 7-20: Rebar-edge – cover-edge temperature difference data (Harpersville crashwall, Scottsboro pedestal, and Birmingham column)

Chapter 8

Summary and Conclusions

8.1 Project Summary

The research and study described herein was performed to develop guidelines for an ALDOT mass concrete specification and provide ALDOT with a draft of said specification. In order to accomplish this objective, work was performed to establish a mass concrete size designation, concrete temperature limits, and temperature control plan requirements for use in ALDOT construction. A review of other state DOT mass concrete specifications, field instrumentation of potential ALDOT mass concrete members, ConcreteWorks analyses, and numerical modeling of temperature difference limits were performed in order to accomplish these tasks.

Seven ALDOT bridge elements were instrumented with temperature sensors to monitor their early-age and long-term temperature behavior. These seven elements were determined to be potential mass concrete elements, and temperature sensors were installed at the core and edges of each element to evaluate their maximum concrete temperatures and maximum concrete temperature differences during the week after placement. When possible, temperature data were collected for over a year to determine the long-term temperature profiles of these elements. Multiple field condition surveys were also conducted to inspect each element for potential signs of DEF and thermal cracking. These data were used to assess the least dimension designation, maximum concrete temperature limit, maximum concrete temperature difference limit, and current shrinkage and temperature reinforcement requirements for mass concrete.

This element data also helped assess the accuracy of ConcreteWorks. All seven elements were modeled in ConcreteWorks for seven days after placement, and the predicted core and edge temperatures were compared to the measured field data. To more accurately model each element, laboratory testing was done to determine each mixture's hydration parameters, concrete strength, and concrete CTE. Mixture proportions and cementing material properties were also input to improve ConcreteWorks' predictions. Comparing these data helped validate ConcreteWorks as an effective tool for use in mass concrete analysis for ALDOT.

A thorough analysis of the behavior of various ALDOT mass concrete elements was performed by modeling 480 mass concrete placement scenarios in ConcreteWorks. The following input variables were analyzed: placement date, time of day, placement location, cementing materials used, coarse aggregate type, and least dimension. The three output variables monitored were maximum concrete temperature, maximum concrete temperature difference, and maximum concrete cracking risk. The results of all analyses were statistically evaluated to determine the impact of input variables on each output variable. This aided in the determination of an appropriate least dimension for mass concrete.

To aid in the development of an age-dependent maximum concrete temperature limit, Bamforth's thermal cracking equation (2007) was modified to include time-dependent variables that better represent the increase of tensile strength of concrete with time. The time-dependent variables incorporated include concrete compressive strength, tensile strength, modulus of elasticity, and creep effects. By using properties of materials common in Alabama concrete, an age-dependent temperature difference limit was found, and it may be modified for any ALDOT concrete using different concrete CTE values.

To conclude this project, a mass concrete specification for use in ALDOT mass concrete construction was drafted and is found in Appendix A.

8.2 Research Conclusions

- ConcreteWorks is a sufficiently accurate tool for predicting temperatures and cracking risks in ALDOT mass concrete projects if representative hydration parameters and mixture properties are used.
- The new maximum temperature limits from ACI 201.2R (2016) may be sufficient for use in ALDOT construction. No signs of DEF were observed in any elements where the maximum concrete temperature exceeded 158°F.
- The 35°F concrete temperature difference limit is over-conservative—particularly for concrete containing limestone coarse aggregate— and an age-dependent temperature difference limit should be used to more appropriately reflect the development of concrete tensile strength with age.
- The element least dimension most greatly influences the temperature difference and thermal cracking risk. The coarse aggregate type has the second greatest effect on the thermal cracking risk, whereas it has little impact on the maximum concrete temperature difference.
- To best mitigate thermal cracking in ALDOT mass concrete, limestone coarse aggregate should be used.
- To best mitigate DEF in ALDOT mass concrete, recommended amounts of SCMs should be used.
- To best mitigate DEF and thermal cracking in ALDOT mass concrete, concrete should be placed at the lowest possible placement temperature.
- In ALDOT construction, a maximum concrete temperature limit of 185°F should be used when the minimum SCM limits from ACI 201.2R (2016) are met or exceeded. A maximum

temperature limit of 158°F should be used in all other cases to minimize the risk of DEF occurring.

- In ALDOT construction, the least dimension that designates an element as mass concrete should be based on the coarse aggregate type and cementing materials used as shown in Figure 7-1. A least dimension of 7 feet should be used when a concrete mixture satisfies the SCM requirements of Table 6-6 and limestone coarse aggregate is used. A least dimension of 5 feet should be used when the concrete mixture satisfies the SCM requirements of Table 6-6 and any coarse aggregate other than limestone is used. A least dimension of 4 ft should be used to designate as mass concrete in all other cases.
- For ALDOT construction, the maximum concrete temperature difference limit used should be based on information known about the concrete's CTE. Refer to Table 8-1 for a summary of the three temperature difference limit options that may be implemented based on the information known about the concrete CTE.

Table 8-1: Tiered allowable temperature difference limit for use in a mass concrete specification

Tier	Requirement	Specification
I	CTE of project concrete is tested according to AASHTO T336	Use ***known*** CTE value to calculate ***age-dependent ΔT limit*** in accordance with Table 8-2 and modified with Equation 8-1
II	Coarse aggregate type of project concrete is known, but concrete CTE has not been tested	Use ***default*** CTE value from Table 8-3 to calculate ***age-dependent ΔT limit*** in accordance with Table 8-2 and modified with Equation 8-1
III	Coarse aggregate type of project concrete is unknown	Use maximum ΔT limit of ***35 °F***

$$\Delta T_{max,modified}(t) = F_{CTE} \times \Delta T_{max}(t) \qquad \textbf{Equation 8-1}$$

Where,

$\Delta T_{max,modified}(t)$ = modified allowable maximum temperature limit (°F),

F_{CTE} = concrete CTE modification factor in Equation 8-2 (unitless), and

$\Delta T_{max}(t)$ = allowable maximum temperature limit as a function of concrete age (°F) (listed in Table 8-2).

$$F_{CTE} = \frac{6.95 \times 10^{-6} \text{ in./in./°F}}{CTE_{measured}} \qquad \textbf{Equation 8-2}$$

Where,

F_{CTE} = concrete CTE modification factor (unitless), and

$CTE_{measured}$ = measured concrete CTE (in./in./°F).

Table 8-2: Maximum allowable temperature differences – River gravel concrete

Concrete Age, *t* (hours)	Maximum Allowable Temperature Difference, $\Delta T_{max}(t)$ (°F)
12	36
24	40
36	42
48	44
60	45
72	46
84	46
96	47
108	47
120	48
132	48
144	49
156	49
168	49

Table 8-3: Default CTE values for concrete made with typical Alabama coarse aggregate types (Schindler et al. 2010)

Coarse Aggregate Type	Concrete CTE (in./in./°F)
River Gravel	6.95×10^{-6}
Limestone	5.52×10^{-6}
Granite	5.60×10^{-6}

- For all ALDOT mass concrete elements temperature monitoring sensors should be installed at the core, the middle of the top surface, and the middle of the side surface of the element to monitor concrete temperatures for 7 days after placement. All edge sensors should be installed no more than 6 inches from the concrete surface.

- The maximum spacing of any shrinkage and temperature steel in ALDOT mass concrete elements should not exceed 12.0 in. as defined by AASHTO LRFD (2016) Section 5.10.8 (see Table 2-6).
- Insufficient amounts of transverse temperature and shrinkage control reinforcement were provided in the Scottsboro bridge pedestal. All mass concrete elements (including pedestals) should be designed to have temperature and shrinkage reinforcement in accordance with AASHTO LRFD (2016) Section 5.10.8 that requires a minimum reinforcement from 0.11 to 0.60 in.2/ft to control crack widths due to early-age and long-term shrinkage and temperature effects. This minimum shrinkage and temperature reinforcement must be evenly distributed around the perimeter near surfaces exposed to daily temperature changes and in structural mass concrete.

8.3 RESEARCH RECOMMENDATIONS

- Conduct research into using the maturity method to determine an age-dependent maximum allowable temperature difference. The maturity method is used to account for the effect of in-place temperatures on concrete properties. Use of the maturity method could result in a less conservative, but more accurate and efficient, temperature difference limit. The FHWA and Rhode Island DOT currently require the use of maturity meters to monitor mass concrete placements.
- Education about thermal cracking and DEF in mass concrete should take place before requiring contractors and ALDOT employees to implement temperature control requirements from a specification. Construction personnel should comprehend the purpose of lowering maximum concrete temperatures and temperature differences in order for a temperature control plan to be most effective.
- Change the current ALDOT supplementary cementitious material requirements to allow the use of at least 25% Class F fly ash, 35% Class C fly ash, or 35% slag cement in concrete mixtures.

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Appendix A

DRAFT ALDOT Mass Concrete Specification

Section 5XX
Mass Concrete

5XX.01 Description.

The work covered in this Section consists of means to define mass concrete and requirements for maximum allowable temperatures, temperature monitoring, and the Thermal Control Plan. The requirements of this Section shall apply to any concrete member that meets the definition of mass concrete as defined in Section 5XX.02. A Thermal Control Plan to control and monitor concrete temperatures is required for all mass concrete.

5XX.02 Designation of Mass Concrete.

Mass concrete size designation shall be defined as shown in Figure A.1. Any member containing the minimum amount of supplementary cementing materials (SCM) (as defined in Table 1) and limestone coarse aggregate shall be considered mass concrete when the least dimension equals or exceeds 7.0 ft. Any member containing the minimum amount of SCM (as defined in Table 1) and any other type of coarse aggregate shall be considered mass concrete when the least dimension equals or exceeds 5.0 ft. Any member not containing the minimum amount of SCM (as defined in Table 1) or the coarse aggregate type is unknown shall be considered mass concrete when the least dimension equals or exceeds 4.0 ft. Figure A.2 through Figure A.5 define the least dimension in some atypical scenarios.

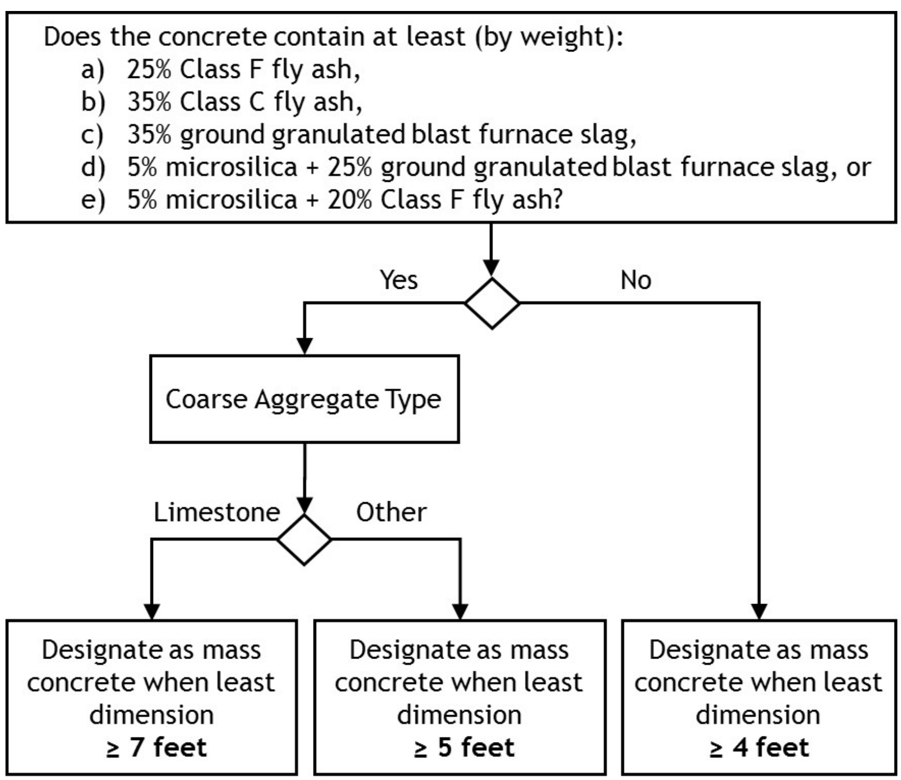
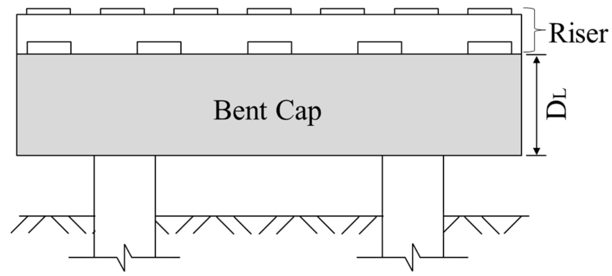


Figure A.1: Mass Concrete Size Designation Based on Concrete Constituents



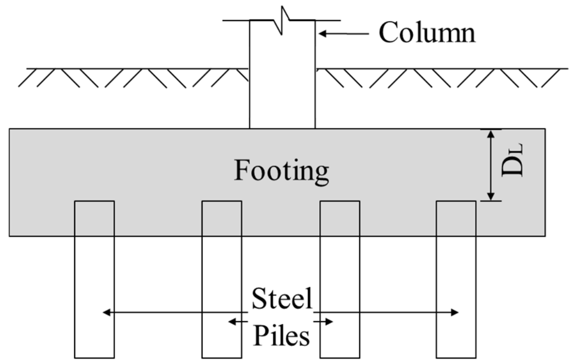
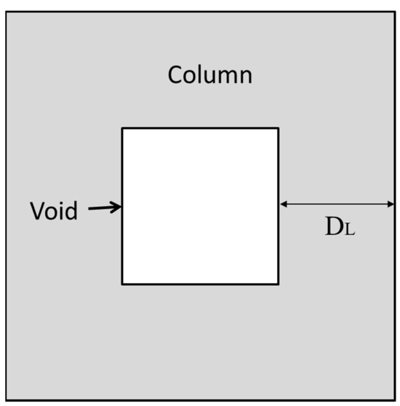


Figure A.2: Stepped bent cap least dimension (D_L)

Figure A.3: Footing with steel piles least dimension (D_L)



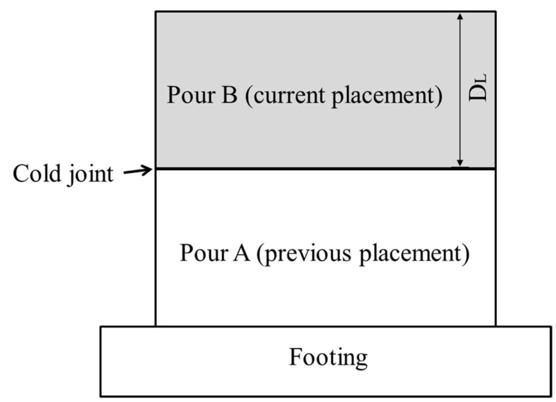


Figure A.4: Column with void least dimension (D_L)

Figure A.5: Least dimension of multiple placements (D_L)

5XX.03 Material Selection.

Table 1 shall be used to determine the substitution rates of Class C fly ash, Class F fly ash, Ground Granulated Blast Furnace Slag, and Microsilica in mass concrete. The use of supplementary cementing materials (SCMs) in mass concrete mixtures is highly recommended at or above the amounts listed in Table 1 in the row that allows a maximum concrete temperature limit of 185°F. High-early-strength (AASHTO M25 Type III) cement shall not be used in mass concrete construction.

Limits listed in ALDOT Subarticle 501.02(c) for the maximum substitution rates of Class C fly ash, Class F fly ash, ground granulated blast furnace slag, and microsilica shall not apply to mass concrete.

The following note shall be added to the bottom of the Master Proportions Table in ALDOT Article 501.02:

"f. Type III portland cement shall not be used in any element with a least dimension equal or greater to 4.0 ft, unless approved in writing by the Engineer."

5XX.04 Concrete Temperature Limits for Mass Concrete.

Mass concrete shall conform to the following concrete temperature limits to minimize the risk of delayed ettringite formation (DEF) and thermal cracking:

(a) MAXIMUM CONCRETE TEMPERATURE LIMIT.

The maximum concrete temperature limit for mass concrete shall be either 160°F or 185°F in accordance with Table A.1.

Table A.1: Maximum Concrete Temperature Limit

Maximum Concrete Temperature Limit, $T_{c,\ max}$	Cementitious Materials Used
$T_{c,\ max}$ = 160°F	1. Plain portland cement 2. Portland cement in combination with supplementary cementing materials less than the proportions listed in the row below.
$T_{c,max}$ = 185°F	Use one of the following: 1. Portland cement in combination with the following proportions of supplementary cementing materials: a) Greater than or equal to 25 percent fly ash meeting the requirements of ASTM C618 for Class F fly ash b) Greater than or equal to 35 percent fly ash meeting the requirements of ASTM C618 for Class C fly ash c) Greater than or equal to 35 percent ground granulated blast furnace slag meeting the requirements of ASTM C989 d) Greater than or equal to 5 percent microsilica (meeting ASTM C1240) in combination with at least 25 percent ground granulated blast furnace slag e) Greater than or equal to 5 percent microsilica (meeting ASTM C1240) in combination with at least 20 percent Class F fly ash 2. An ASTM C595/C595M or ASTM C1157/C1157M blended hydraulic cement with the same pozzolan or ground granulated blast furnace slag content in Item 1

(b) MAXIMUM CONCRETE TEMPERATURE DIFFERENCE LIMIT.

The maximum concrete temperature difference limit for mass concrete shall satisfy the requirements of Tier I, II, or III as listed in Table A.2.

Table A.2: Maximum Temperature Difference Limit Tiered Specification

Tier	Requirement	Specification Limit
I	CTE of project concrete is tested according to AASHTO T336.	Use *known* CTE value to calculate *age-dependent* ***ΔT limit*** in accordance with Table A.3 and modified with Equations A.1 and A.2.
II	Coarse aggregate type of project concrete is known, but concrete CTE has not been tested.	Use *default* CTE value from Table A.4 to calculate *age-dependent* ***ΔT limit*** in accordance with Table A.3 and modified with Equations A.2 and A.3.
III	Coarse aggregate type of project concrete is unknown.	Use maximum *ΔT* limit of **35 °F**.

1. TIER 1 - COEFFICIENT OF THERMAL EXPANSION OF CONCRETE KNOWN.

The coefficient of thermal expansion shall be tested in accordance with AASHTO T 336 (2015). An age-dependent temperature difference limit shall be used when the coefficient of thermal expansion (CTE) of the project concrete has been tested. The age-dependent temperature limit shall be calculated by modifying the maximum temperature difference limit values in Table A.3 by the CTE correction factor by using Equations A.1 and A.2.

Table A.3: Age-Dependent Maximum Temperature Difference Limit for Concrete with River Gravel

Concrete Age, t (hours)	Maximum Temperature Difference Limit, $\Delta T_{max}(t)$ (°F)
12	32
24	35
36	37
48	38
60	39
72	40
84	41
96	41
108	42
120	42
132	43
144	43
156	43
168	43

$$F_{CTE} = \frac{6.95 \times 10^{-6} \text{ in./in./°F}}{CTE_{measured}} \qquad \text{Equation A.1}$$

Where,

F_{CTE} = concrete CTE modification factor (unitless), and

F_{CTE} = measured concrete CTE (in./in./°F).

$$\Delta T_{max,modified}(t) = F_{CTE} \times \Delta T_{max}(t) \qquad \text{Equation A.2}$$

Where,

$\Delta T_{max,modified}(t)$ = modified maximum temperature difference limit (°F),

F_{CTE} = concrete CTE modification factor (unitless), and

$\Delta T_{max}(t)$ = allowable maximum temperature difference limit as a function of concrete age (°F) (listed in Table A.3).

2. TIER 2 - COARSE AGGREGATE TYPE KNOWN.

Based on a default CTE for the concrete, an age-dependent temperature difference limit shall be used when the coarse aggregate type of the project concrete is known, but the CTE of the project concrete has not been tested. The default CTE shall be equal to the value in Table 4 depending on the corresponding coarse aggregate type. This age-dependent concrete temperature difference limit shall be calculated by modifying the maximum temperature difference limit values in Table A.3 by the CTE correction factor found in Equation A.3, then using Equation A.2. When river gravel coarse aggregate is used, the concrete CTE modification factor shall be equal to 1.0, and the age-dependent concrete temperature difference limit shall be equal to the values in Table A.3.

Table A.4: Default Concrete CTE for Other Typical Alabama Aggregates

Coarse Aggregate Type	Default Concrete CTE (in./in./°F)
Limestone	5.52×10^{-6}
Granite	5.60×10^{-6}

$$F_{CTE} = \frac{6.95 \times 10^{-6} \text{ in./in./°F}}{CTE_{default}} \qquad \text{Equation A.3}$$

Where,

F_{CTE} = concrete CTE modification factor (unitless),
$CTE_{default}$ = default concrete CTE (in./in./°F) (Table A.4).

3. TIER 3 - COARSE AGGREGATE TYPE UNKNOWN.

A temperature difference limit of 35°F shall be used when the coarse aggregate type of the project concrete is unknown.

5XX.05 Temperature Monitoring and Recording.

The temperature monitoring duration shall be for a minimum of 7 days after concrete placement, unless otherwise designated by the Engineer.

Temperatures shall be electronically recorded automatically by an approved recorded furnished by the Contractor and shall be capable of continuously recording a minimum of one reading per hour for the duration of the mass concrete temperature monitoring period. The sensors and recorder shall be accurate to within +/- 2°F (1°C) within the temperature range of 40°F (4°C) to 200°F (93°C).

For each mass concrete placement, two temperature sensors shall be installed at each of the following locations (for a total of six temperature sensors) on one cross section. The purpose for two sensors at each location is to have a backup reading. The surface sensors may be tied to the rebar cage, provided that the sensors are within 6 in. of the surface. Cross sections on which sensors are placed are typically on either horizontally or vertically oriented. Vertically oriented cross sections include bent caps and footings. Horizontally oriented cross sections include columns, drilled shafts, pedestals, and walls. Refer to Figures A.6 through A.10 for guidance on proper and inappropriate locations for cross sections to position temperature sensors.

A minimum of two temperature sensors shall be placed at each of the following locations:

- At the center of the cross section,
- No further than 6 in. from and at the midpoint of the edge of the cross section which is the shortest distance from the center, and
- No further than 6 in. from and at the midpoint of the edge of the cross section that is perpendicular to the other edge on which temperature sensors are placed.

Vertically oriented cross sections:

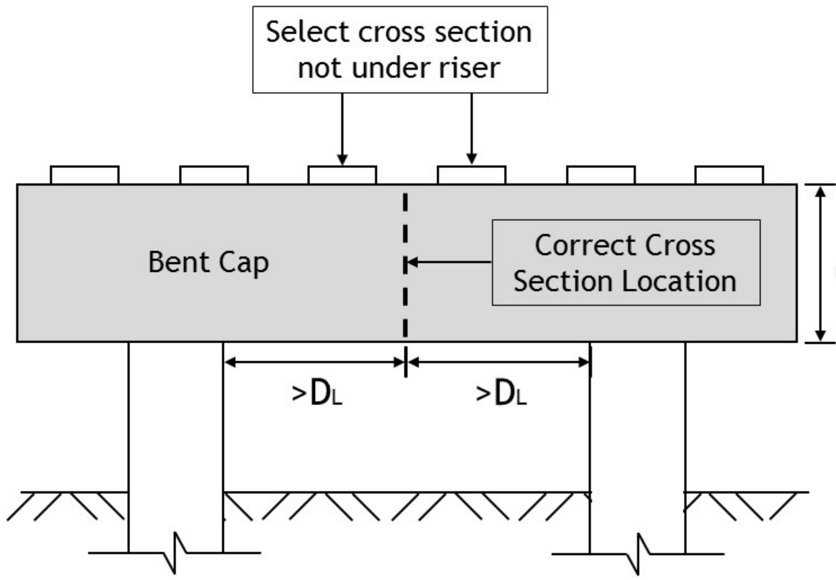


Figure A.6: Proper temperature sensor cross-section location within a bent cap (example of a vertically oriented cross section)

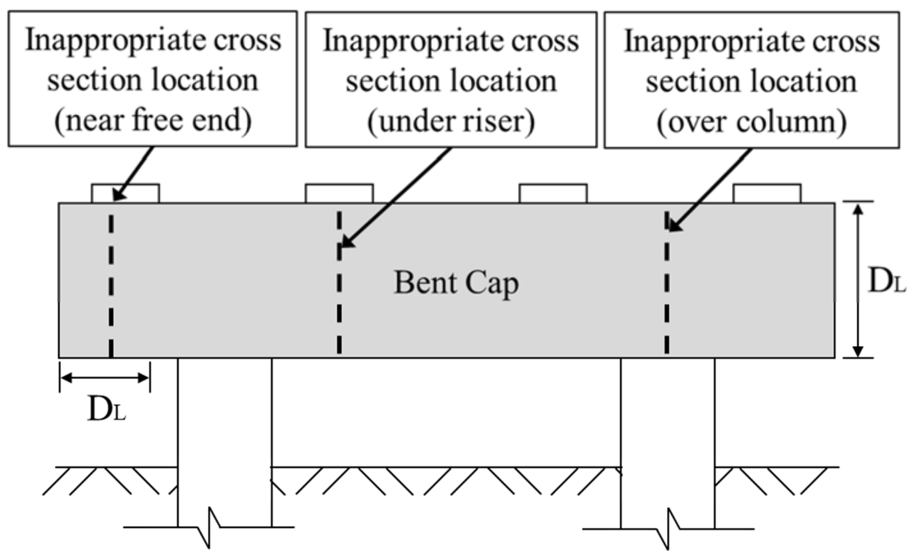


Figure A.7: Inappropriate temperature sensor cross section locations within a bent cap (D_L = least dimension) (example of a vertically oriented cross section)

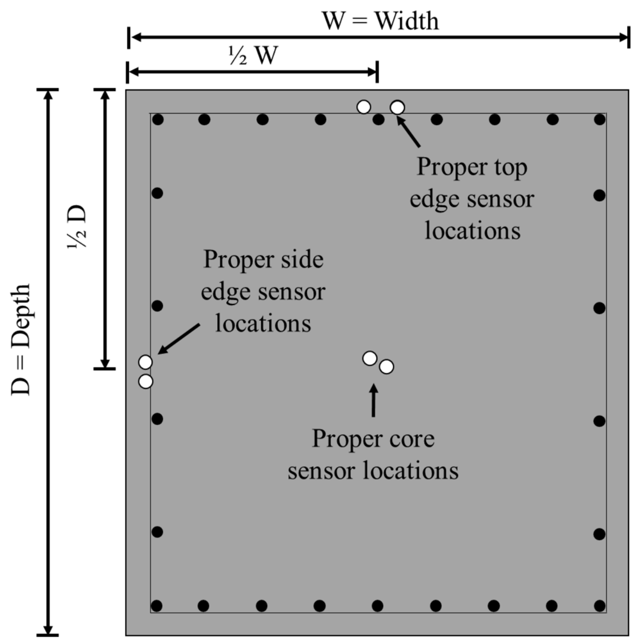


Figure A.8: Proper temperature sensor placement on a rectangular bent cap cross section (example of a vertically oriented cross section)

Horizontally oriented cross sections:

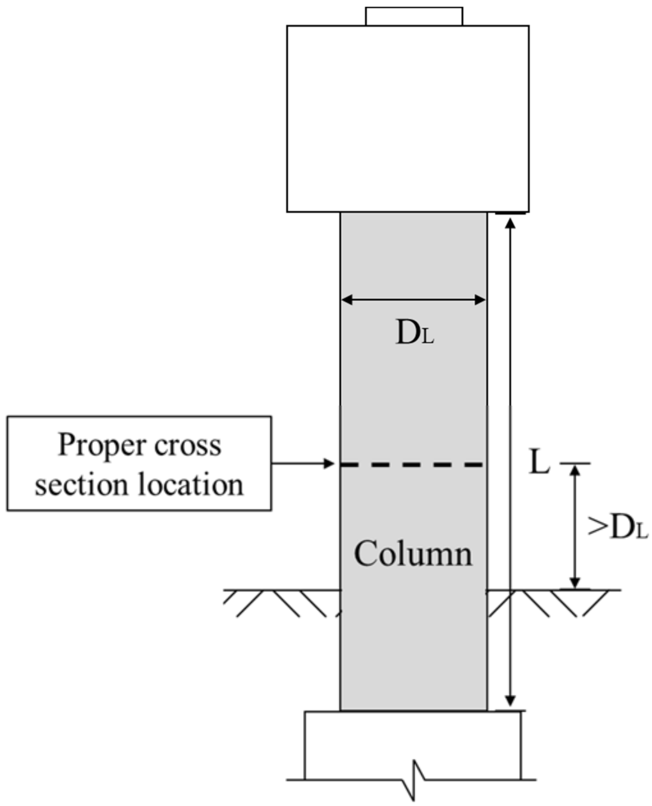


Figure A.9: Proper temperature sensor cross section locations within a column (D_L = least dimension) (example of a horizontally oriented cross section)

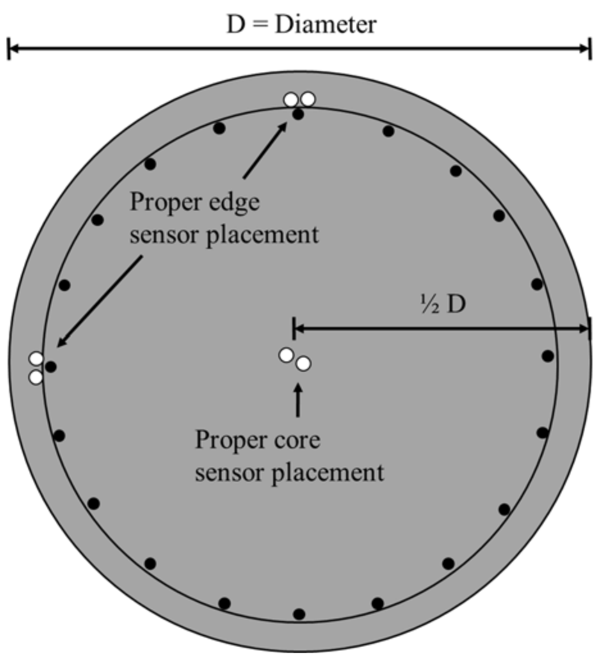


Figure A.10: Correct temperature sensor placement on a circular column cross section (example of a horizontally oriented cross section)

5XX.06 Thermal Control Plan.

The Contractor shall develop and submit a written Thermal Control Plan to the Engineer for approval describing the procedures that will be used during the 7-day period following concrete placement, so that the maximum concrete temperatures and maximum concrete temperature differences do not exceed the limits defined in Section 5XX.04.a and 5XX.04.b. The Thermal Control Plan shall list actions to take when any of the temperature limits set in Section 5XX.04 are exceeded or anticipated to be exceeded. The Thermal Control Plan shall be submitted at least 30 calendar days before the first intended mass concrete placement. The Contractor shall not place concrete covered by this specification until the Thermal Control Plan has received written approval from the engineer and equipment and materials necessary to facilitate the plan are on site and ready for use. The Contractor shall provide and install temperature sensing devices according to Section 5XX.05.

The Contractor shall notify the Engineer when the maximum concrete temperature comes within 15°F of the maximum temperature limit, and take corrective measures immediately to retard further increase in temperature. The Contractor must take actions to prevent the concrete exterior surfaces from cooling too quickly, such as using formwork insulation, or the contractor must prevent the core from getting too hot. The Contractor shall notify the Engineer when the maximum concrete temperature difference comes within 10°F of the maximum concrete temperature difference, and take corrective action to retard further increase in the temperature difference.

For all mass concrete construction, the TCP shall be developed by a Professional Engineer, licensed in the state of Alabama, who shall be competent in the modeling, design, and temperature control of mass concrete.

The Thermal Control Plan shall include the following:

1. Concrete mixture design showing composition, proportions, and sources for all components. If the concrete mixture proportions are changed, the TCP shall be updated.
2. Calculated or measured adiabatic temperature rise of the mass concrete element.
3. Maximum concrete temperature at time of placement.
4. Proposed methods to control concrete temperature at time of placement, such as pre-cooling of raw materials or concrete.
5. Proposed methods to control maximum concrete temperature during curing. A mechanical cooling system may be used to control the internal temperature of mass concrete during curing but shall be designed in conformance with the Thermal Control Plan. If a mechanical cooling system is used, the plans for the cooling system operation and final grouting after cooling shall be submitted to the Engineer for approval.
6. Calculated maximum concrete temperature during curing based on expected conditions and methods used to control temperature.
7. Proposed methods to control concrete temperature differences during curing. An example of this could be to include insulation for the forms and exposed portions of the concrete.
8. Calculated maximum temperature difference based on expected conditions and methods used to control temperature difference. Submit the thermal calculation model and/or computational software to the Engineer.
9. Information on the temperature sensing and recording equipment to be used and drawings of locations of temperature sensors within each placement.
10. Description of the format and frequency of providing temperature data.
11. List of corrective measures to be taken to reduce excessive temperatures and temperature differences, if they occur.
12. Description of curing methods and duration of curing.
13. Description of formwork removal procedures and method to ensure temperature difference at exposed surfaces will not exceed temperature difference limit.

Appendix B

Albertville Bent Cap

Appendix Table B-1: Concrete mixture proportions – Albertville bent cap

Item	Amount
Type I/II Cement (pcy)	427
Class F Fly Ash (pcy)	140
Water (pcy)	280
#57/67 Limestone Coarse Aggregate (pcy)	1900
#100 Sand (pcy)	1169
Air Entraining Admixture (oz.)	3.0
Water Reducer (oz.)	17.0
MR Water Reducer (oz.)	28.4
Retarder (oz.)	22.7
Accelerator (oz.)	90.2

Appendix Table B-2: Cement composition – Albertville bent cap

Chemical Analysis	
Item	**Result (%)**
SiO_2	20.23
Al_2O_3	3.94
Fe_2O_3	3.00
CaO	63.56
MgO	2.84
SO_3	2.79
LOI	2.36
Na_2O_{eq}	0.59
CO_2	0.30
C_3S	60.9
C_2S	12.0
C_3A	5.4
C_4AF	9.1
Physical Analysis	
Blaine Fineness (m^2/kg)	448.9

Appendix Table B-3: Fly ash composition – Albertville bent cap

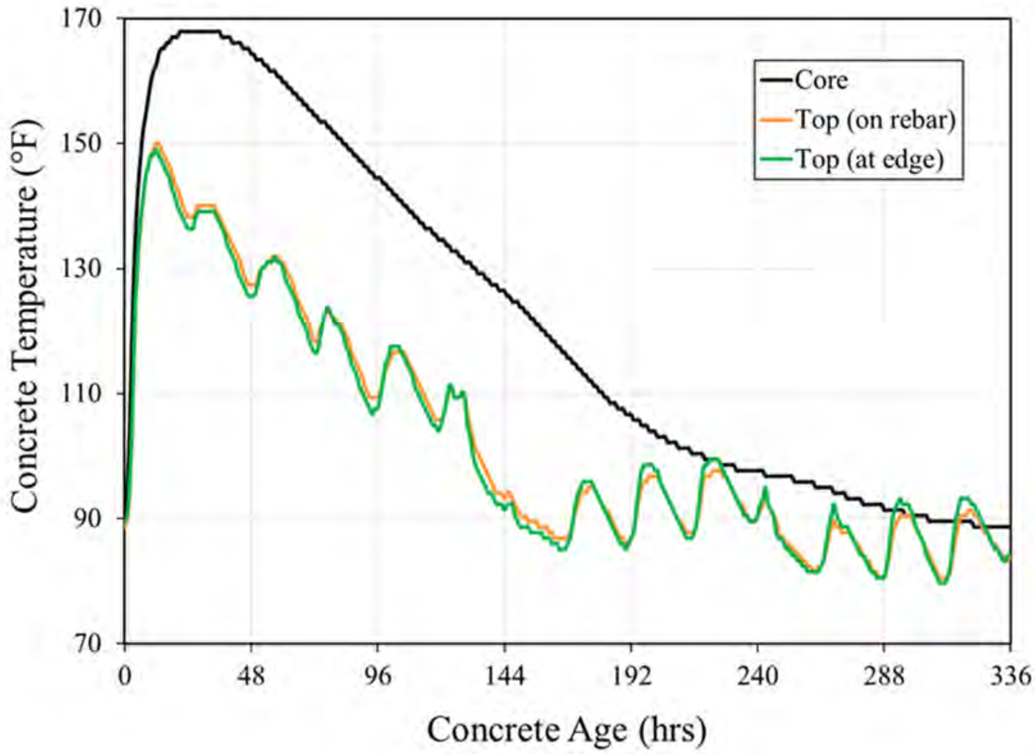
Chemical Analysis	
Item	**Result (%)**
SiO_2	20.23
Al_2O_3	3.94
Fe_2O_3	3.00
CaO	63.56
MgO	2.84
SO_3	2.79
Na_2O	0.59
K_2O	2.03

Appendix Table B-4: 7-day weather data – Albertville bent cap

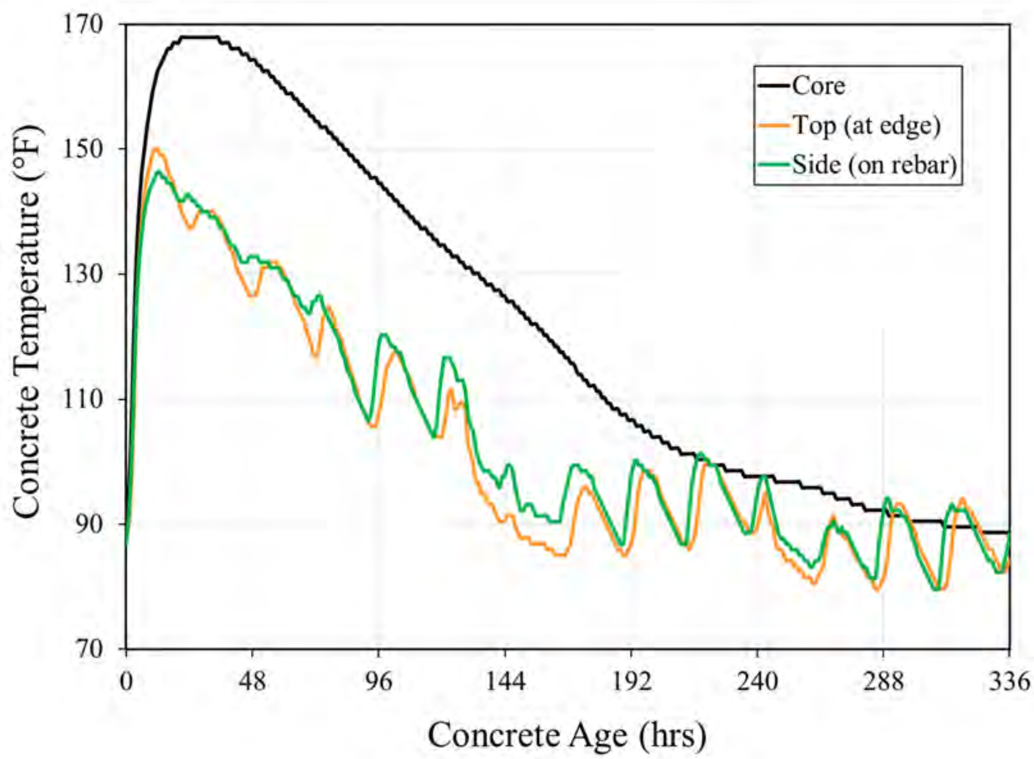
Date	Day	Temperature		Relative Humidity		Max Wind Speed	Max % Cloud Cover
		Min	Max	Min	Max		
7/31/2016	1	68	88	45	93	20	CLR
8/1/2016	2	64	91	41	100	17	SCT120
8/2/2016	3	64	91	31	100	9	CLR
8/3/2016	4	64	93	38	100	13	CLR
8/4/2016	5	70	97	41	100	9	BKN120
8/5/2016	6	70	95	44	100	23	OVC120
8/6/2016	7	68	79	82	100	13	OVC120

Appendix Table B-5: ALDOT test results of project concrete sample specimens – Albertville bent cap

Laboratory Test		Set #1	Set #2	Set #3
Strengths	5-day	4340	3920	3570
	7-day	3790	3800	4410
	28-day	4720	4410	3810
	28-day	4110	4280	3820
Slump (in.)		5.5	5.5	4.5
Air Content (%)		6	6	3.3
Temperature (°F)		85	85	84



Appendix Figure B-1: Cross-section #1 temperature data – Albertville bent cap



Appendix Figure B-2: Cross-section #2 temperature data – Albertville bent cap

Appendix C

Harpersville Crashwall

Appendix Table C-1: Concrete mixture proportions – Harpersville crashwall

Item	Amount
Type I/II Cement (pcy)	425
Class C Fly Ash (pcy)	110
Water (pcy)	267
#67 Limestone Coarse Aggregate (pcy)	1825
#100 Sand (pcy)	1319
Air Entraining Admixture (oz.)	1
Water Reducer (oz.)	21.4
MR Water Reducer (oz.)	32.1

Appendix Table C-2: Cement composition – Harpersville crashwall

Chemical Analysis	
Item	**Result (%)**
SiO_2	19.8
Al_2O_3	4.7
Fe_2O_3	3.1
CaO	62.8
MgO	3.2
SO_3	3.1
LOI	2.5
Na_2O_{eq}	0.41
CO_2	1.5
C_3S	53
C_2S	16
C_3A	7
C_4AF	9
Physical Analysis	
Blaine Fineness (m^2/kg)	387

Appendix Table C-3: Fly ash composition – Harpersville crashwall

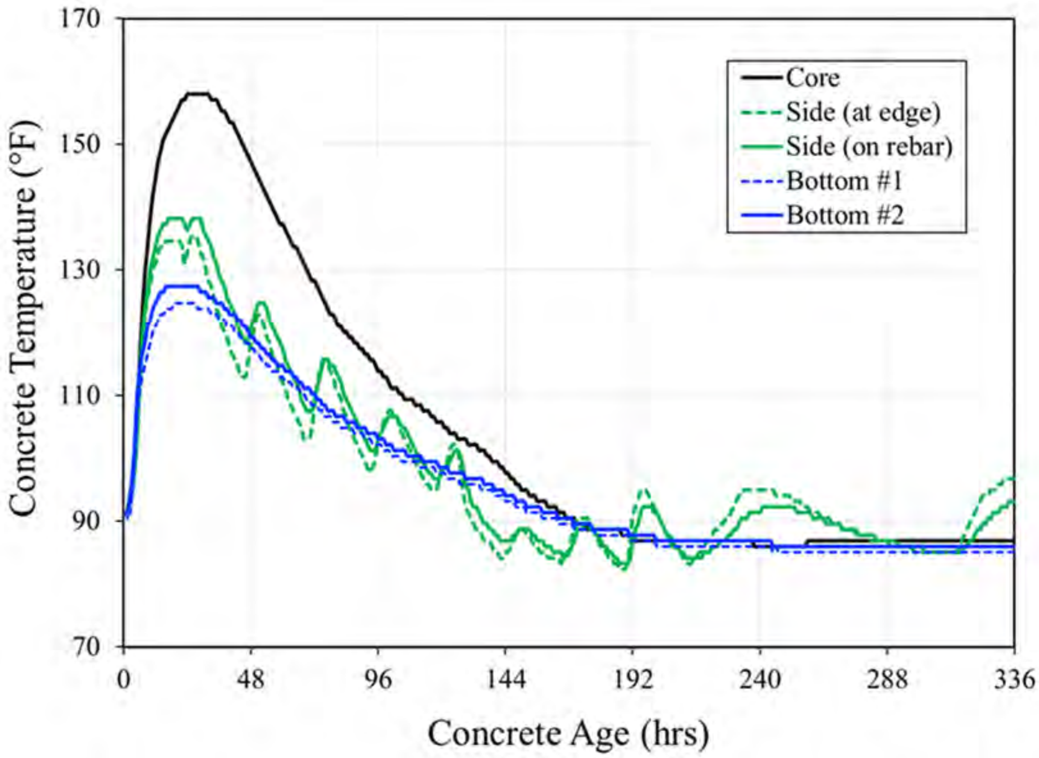
Chemical Analysis	
Item	**Result (%)**
SiO_2	40.68
Al_2O_3	19.74
Fe_2O_3	6.08
CaO	21.22
MgO	4.71
SO_3	1.44
Na_2O	1.39
K_2O	0.68

Appendix Table C-4: 7-day weather data – Harpersville crashwall

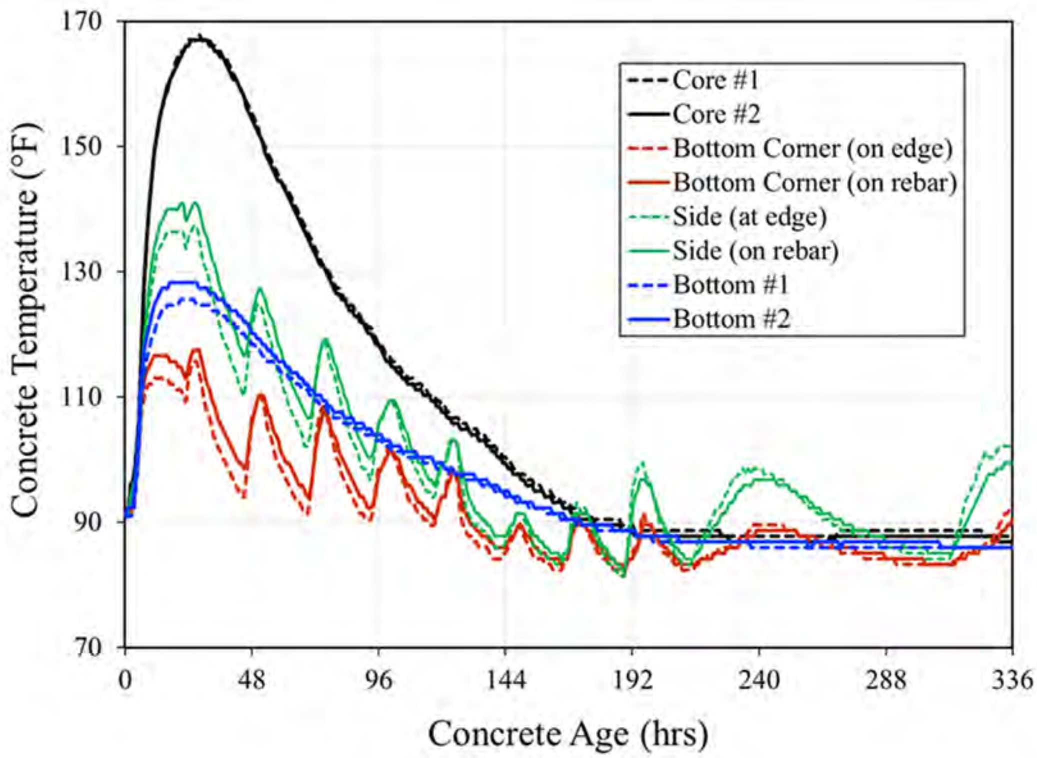
Date	Day	Temperature		Relative Humidity		Max Wind Speed	Max % Cloud Cover
		Min	Max	Min	Max		
8/24/2016	1	70	88	55	100	8	OVC016
8/25/2016	2	63	83	38	90	10	CLR
8/26/2016	3	61	82	43	84	11	CLR
8/27/2016	4	63	86	45	90	8	BKN050
8/28/2016	5	69	88	55	92	13	OVC090
8/29/2016	6	71	84	67	93	9	OVC120
8/30/2016	7	70	79	72	100	9	OVC110

Appendix Table C-5: ALDOT test results of project concrete sample specimens – Harpersville crashwall

Laboratory Test		Set #1	Set #2
Strengths	7-day	3390	2760
	28-day	5240	3860
	28-day	5220	3940
Slump (in.)		3.0	3.0
Air Content (%)		3.9	3.7
Temperature (°F)		82	83



Appendix Figure C-1: Cross-section #1 temperature data – Harpersville crashwall



Appendix Figure C-2: Cross-section #2 temperature data – Harpersville crashwall

Appendix D

Scottsboro Pedestal

Appendix Table D-1: Concrete mixture proportions – Scottsboro pedestal

Item	Amount
Type I/II Cement (pcy)	496
Class F Fly Ash (pcy)	124
Water (pcy)	295
#67 Limestone Coarse Aggregate (pcy)	1870
#100 Sand (pcy)	1111
Air Entraining Admixture (oz.)	3.0
Water Reducer (oz.)	18.6
MR Water Reducer (oz.)	31.0
Retarder (oz.)	18.6
Accelerator (oz.)	99.2

Appendix Table D-2: Cement composition – Scottsboro pedestal

Chemical Analysis	
Item	**Result (%)**
SiO_2	20.38
Al_2O_3	4.13
Fe_2O_3	3.15
CaO	63.26
MgO	2.79
SO_3	2.83
LOI	2.50
Na_2O_{eq}	0.59
CO_2	1.02
C_3S	57.0
C_2S	15.4
C_3A	5.6
C_4AF	9.6
Physical Analysis	
Blaine Fineness (m^2/kg)	387

Appendix Table D-3: Fly ash composition – Scottsboro pedestal

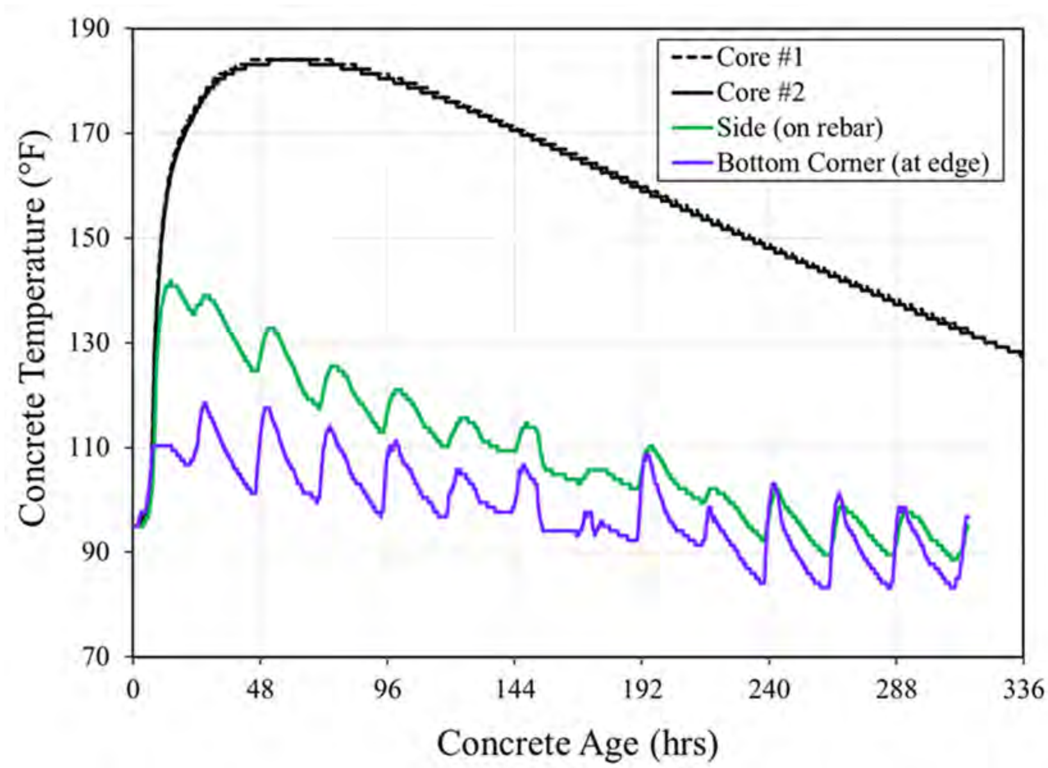
Chemical Analysis	
Item	**Result (%)**
SiO_2	47.4
Al_2O_3	19.3
Fe_2O_3	16.9
CaO	7.4
MgO	1.2
SO_3	2.67
Na_2O	0.66
K_2O	2.11

Appendix Table D-4: 7-day weather data – Scottsboro pedestal

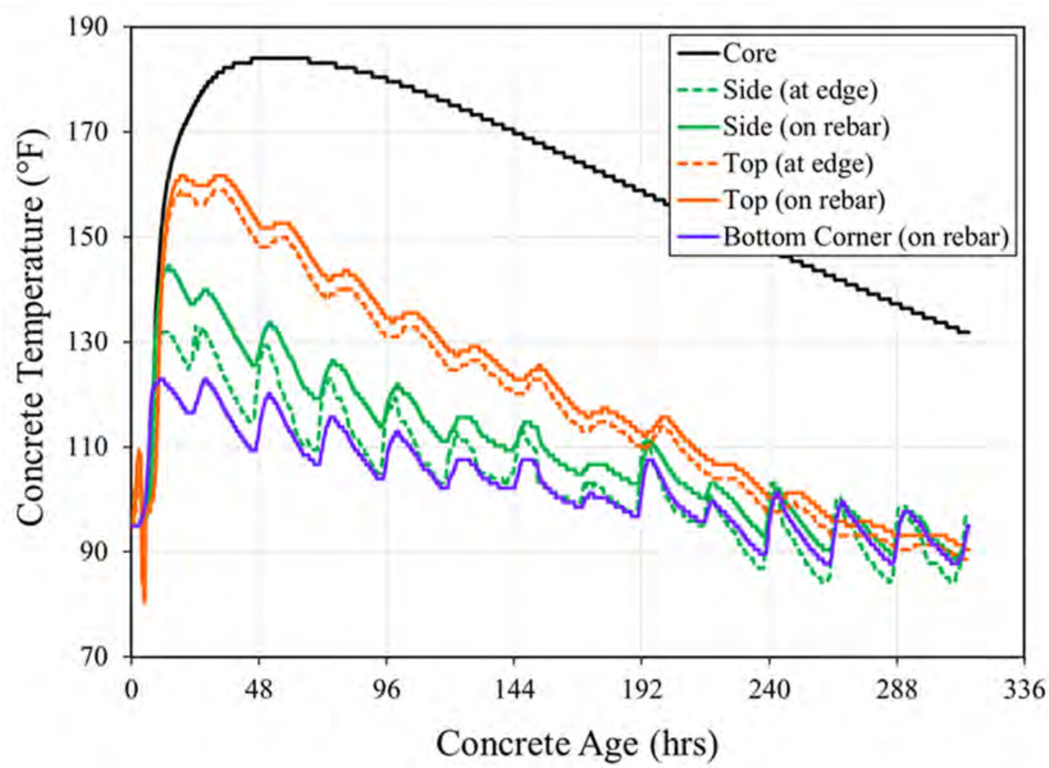
Date	Day	Temperature		Relative Humidity		Max Wind Speed	Max % Cloud Cover
		Min	Max	Min	Max		
9/3/2016	1	72	91	44	100	10	OVC080
9/4/2016	2	68	91	41	100	9	OVC004
9/5/2016	3	68	90	52	100	9	OVC050
9/6/2016	4	68	88	48	100	8	BKN050
9/7/2016	5	66	88	45	100	9	OVC004
9/8/2016	6	66	84	59	100	7	OVC100
9/9/2016	7	68	88	52	100	14	OVC110

Appendix Table D-5: ALDOT test results of project concrete sample specimens – Scottsboro pedestal

Laboratory Test		Set #1	Set #2	Set #3
Strengths	5-day	4340	3920	3570
	7-day	3790	3800	4410
	28-day	4720	4410	3810
	28-day	4110	4280	3820
Slump (in.)		5.5	5.5	4.5
Air Content (%)		6	6	3.3
Temperature (°F)		85	85	84



Appendix Figure D-1: Cross-section #1 temperature data – Scottsboro pedestal



Appendix Figure D-2: Cross-section #2 temperature data – Scottsboro pedestal

Appendix E

Scottsboro Bent Cap

Appendix Table E-1: Concrete mixture proportions – Scottsboro bent cap

Item	Amount
Type I/II Cement (pcy)	496
Class F Fly Ash (pcy)	124
Water (pcy)	295
#67 Limestone Coarse Aggregate (pcy)	1870
#100 Sand (pcy)	1111
Air Entraining Admixture (oz.)	3.0
Water Reducer (oz.)	18.6
MR Water Reducer (oz.)	31.0
Retarder (oz.)	18.6
Accelerator (oz.)	99.2

Appendix Table E-2: Cement composition – Scottsboro bent cap

Chemical Analysis	
Item	**Result (%)**
SiO_2	20.38
Al_2O_3	4.13
Fe_2O_3	3.15
CaO	63.26
MgO	2.79
SO_3	2.83
LOI	2.50
Na_2O_{eq}	0.59
CO_2	1.02
C_3S	57.0
C_2S	15.4
C_3A	5.6
C_4AF	9.6
Physical Analysis	
Blaine Fineness (m^2/kg)	387

Appendix Table E-3: Fly ash composition – Scottsboro bent cap

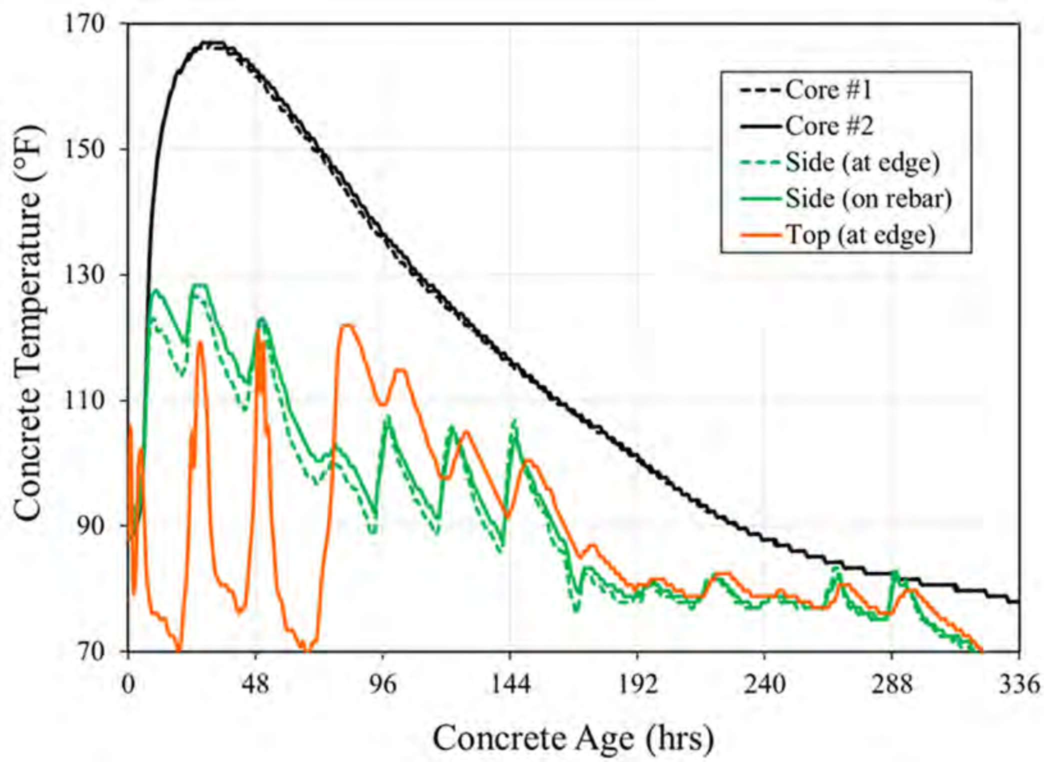
Chemical Analysis	
Item	**Result (%)**
SiO_2	47.4
Al_2O_3	19.3
Fe_2O_3	16.9
CaO	7.4
MgO	1.2
SO_3	2.67
Na_2O	0.66
K_2O	2.11

Appendix Table E-4: 7-day weather data – Scottsboro bent cap

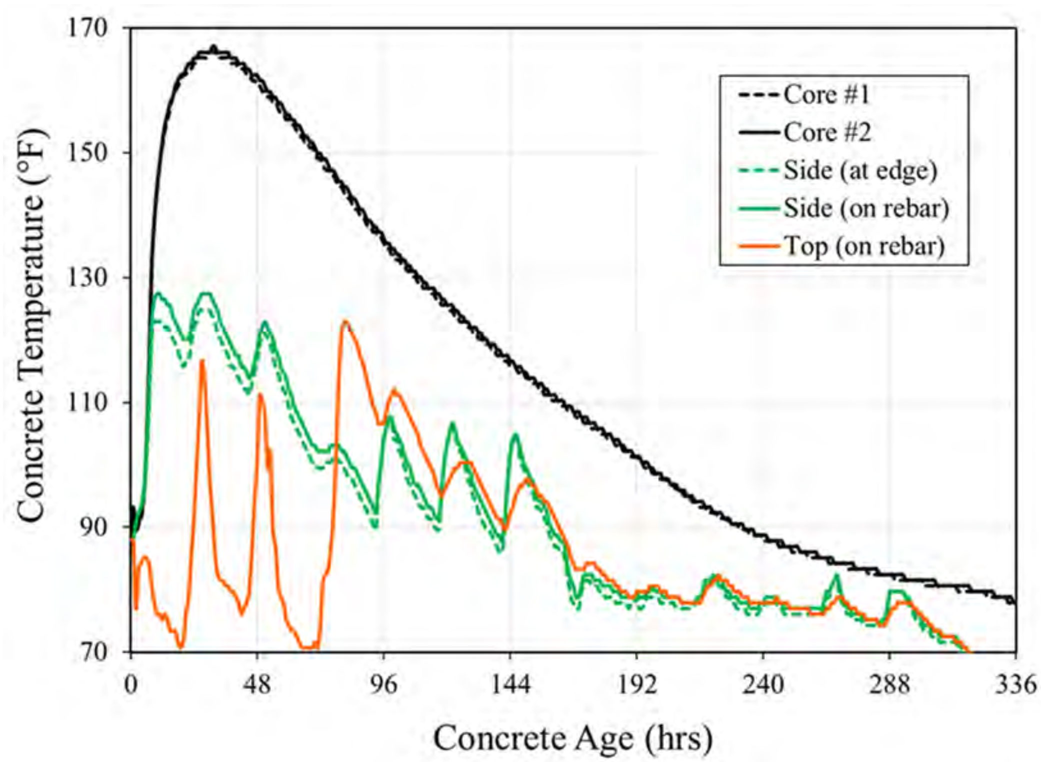
Date	Day	Temperature		Relative Humidity		Max Wind Speed	Max % Cloud Cover
		Min	Max	Min	Max		
9/18/2016	1	57	84	40	100	7	OVC004
9/19/2016	2	57	88	40	100	8	OVC085
9/20/2016	3	63	84	46	100	9	OVC045
9/21/2016	4	57	77	50	100	8	BKN120
9/22/2016	5	54	84	40	100	10	BKN120
9/23/2016	6	61	84	43	100	9	OVC120
9/24/2016	7	59	84	43	100	8	OVC110

Appendix Table E-5: ALDOT test results of project concrete sample specimens – Scottsboro bent cap

Laboratory Test		Set #1	Set #2
Strengths	3-day	3480	3150
	3-day	4080	3900
	28-day	5230	5000
	28-day	4520	4920
Slump (in.)		3.25	4.00
Air Content (%)		3.0	3.8
Temperature (°F)		82	84



Appendix Figure E-1: Cross-section #1 temperature data – Scottsboro bent cap



Appendix Figure E-2: Cross-section #2 temperature data – Scottsboro bent cap

Appendix F

Elba Bent Cap

Appendix Table F-1: Concrete mixture proportions – Elba bent cap

Item	Amount
Type I/II Cement (pcy)	440
Class F Fly Ash (pcy)	110
Water (pcy)	275
#57/67 River Gravel Coarse Aggregate (pcy)	1850
#100 Sand (pcy)	1250
Air Entraining Admixture (oz.)	1.0
MR Water Reducer (oz.)	22.0
Water Reducer Retarder (oz.)	5.5

Appendix Table F-2: Cement composition – Elba bent cap

Chemical Analysis	
Item	**Result (%)**
SiO_2	19.7
Al_2O_3	4.7
Fe_2O_3	3.1
CaO	62.9
MgO	2.9
SO_3	3.1
LOI	2.5
Na_2O_{eq}	0.40
CO_2	1.5
C_3S	54
C_2S	16
C_3A	7
C_4AF	9
Physical Analysis	
Blaine Fineness (m^2/kg)	387

Appendix Table F-3: Fly ash composition – Elba bent cap

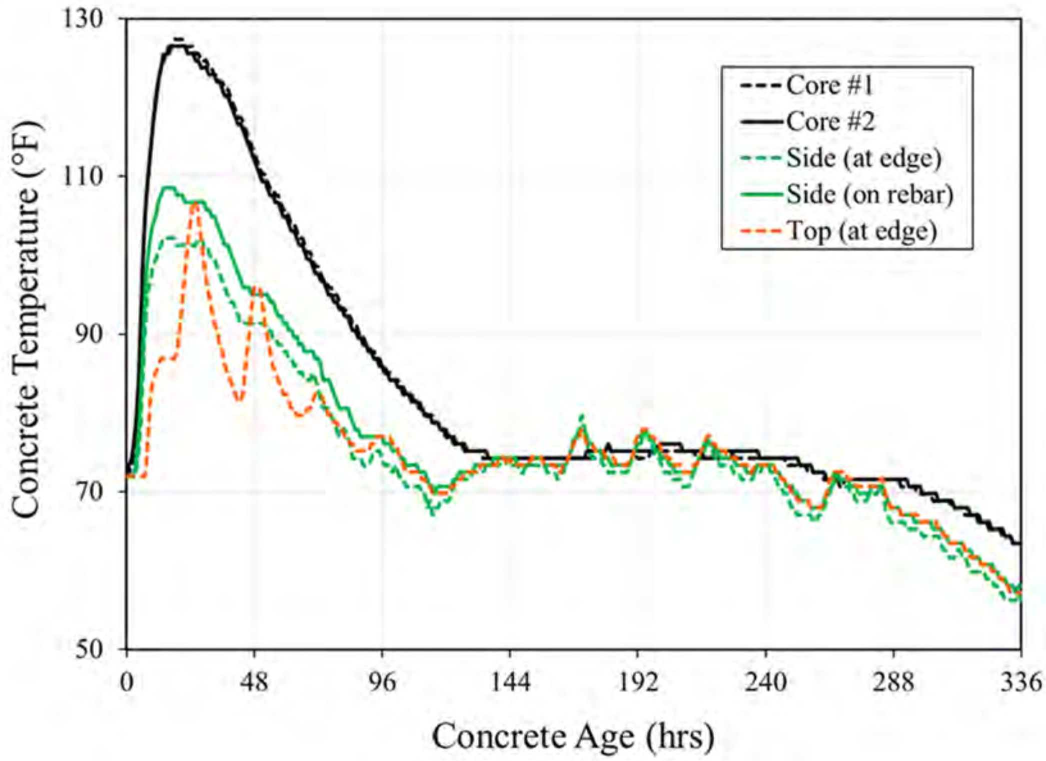
Chemical Analysis	
Item	**Result (%)**
SiO_2	44.87
Al_2O_3	21.06
Fe_2O_3	9.93
CaO	13.11
MgO	3.10
SO_3	1.38
Na_2O	1.36
K_2O	1.49

Appendix Table F-4: 7-day weather data – Elba bent cap

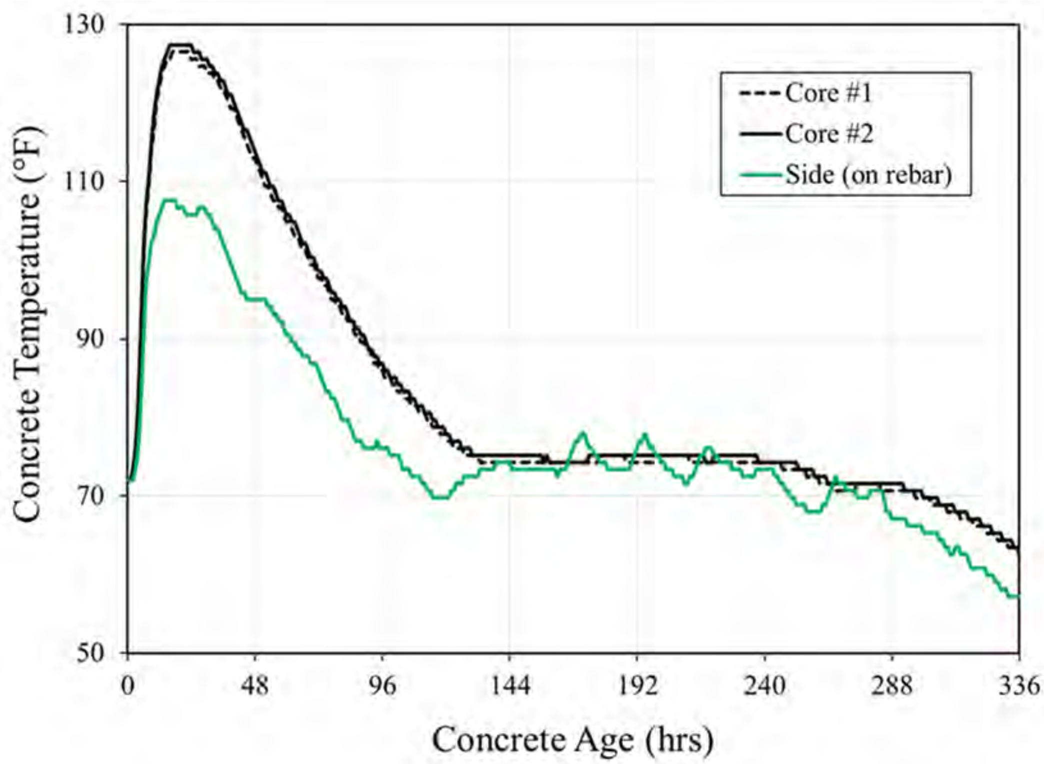
Date	Day	Temperature		Relative Humidity		Max Wind Speed	Max % Cloud Cover
		Min	Max	Min	Max		
12/18/2016	1	37	58	35	79	15	CLR
12/19/2016	2	33	60	22	82	8	CLR
12/20/2016	3	34	64	24	85	9	CLR
12/21/2016	4	54	70	62	90	10	OVC110
12/22/2016	5	66	72	76	90	7	OVC110
12/23/2016	6	66	79	79	90	16	OVC110
12/24/2016	7	73	79	74	90	17	OVC090

Appendix Table F-5: ALDOT test results of project concrete sample specimens – Elba bent cap

Laboratory Test		Set #1
Strengths	3-day	3460
	10-day	4160
	28-day	4510
	28-day	4740
Slump (in.)		3.50
Air Content (%)		4.0
Temperature (°F)		82



Appendix Figure F-1: Cross-section #1 temperature data – Elba bent cap



Appendix Figure F-2: Cross-section #2 temperature data – Elba bent cap

Appendix G

Birmingham Column

Appendix Table G-1: Concrete mixture proportions – Birmingham column

Item	Amount
Type I/II Cement (pcy)	480
Class C Fly Ash (pcy)	120
Water (pcy)	270
#67 Limestone Coarse Aggregate (pcy)	1910
#100 Sand (pcy)	1209
Air Entraining Admixture (oz.)	1.8
Water Reducer (oz.)	24
MR Water Reducer (oz.)	36

Appendix Table G-2: Cement composition – Birmingham column

Chemical Analysis	
Item	**Result (%)**
SiO_2	19.6
Al_2O_3	4.8
Fe_2O_3	3.1
CaO	63.0
MgO	3.1
SO_3	3.1
LOI	2.5
Na_2O_{eq}	0.37
CO_2	1.6
C_3S	54
C_2S	15
C_3A	7
C_4AF	9
Physical Analysis	
Blaine Fineness (m^2/kg)	388

Appendix Table G-3: Fly ash composition – Birmingham column

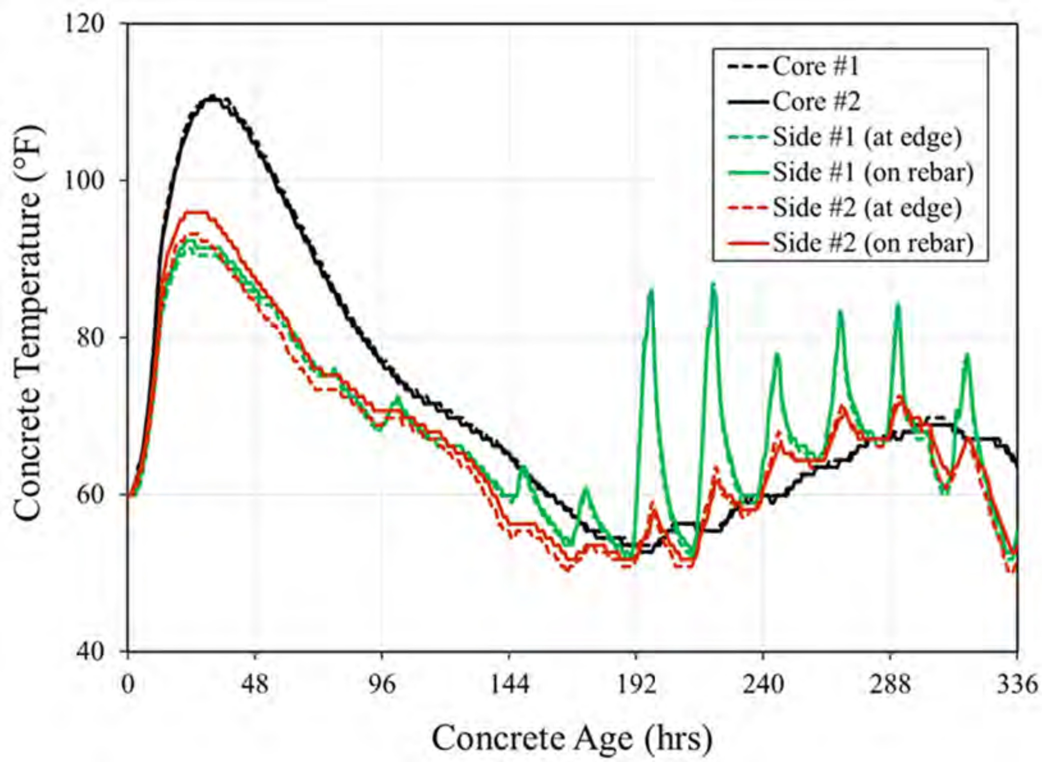
Chemical Analysis	
Item	**Result (%)**
SiO_2	39.9
Al_2O_3	18.9
Fe_2O_3	5.73
CaO	22.29
MgO	5.43
SO_3	1.41
Na_2O	1.56
K_2O	0.63

Appendix Table G-4: 7-day weather data – Birmingham column

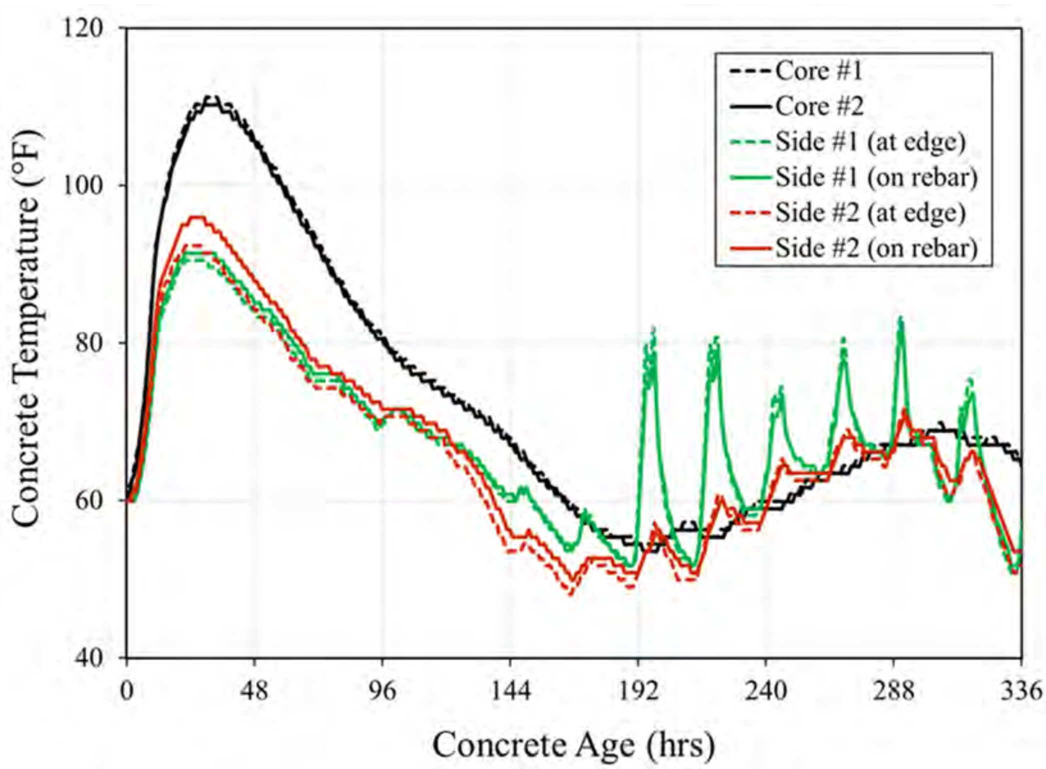
Date	Day	Temperature		Relative Humidity		Max Wind Speed	Max % Cloud Cover
		Min	Max	Min	Max		
1/21/2016	1	41	52	83	96	22	OVC050
1/22/2016	2	30	53	72	100	24	OVC095
1/23/2016	3	26	37	55	75	20	OVC028
1/24/2016	4	22	49	28	88	7	OVC250
1/25/2016	5	32	62	48	79	15	OVC250
1/26/2016	6	43	56	74	100	20	OVC120
1/27/2016	7	35	49	52	96	13	OVC010

Appendix Table G-5: ALDOT test results of project concrete sample specimens – Birmingham column

Laboratory Test		Set #1
Strengths	7-day	5060
	14-day	6430
	28-day	7580
	28-day	7800
Slump (in.)		1.50
Air Content (%)		2.5
Temperature (°F)		68



Appendix Figure G-1: Cross-section #1 temperature data – Birmingham column



Appendix Figure G-2: Cross-section #2 temperature data – Birmingham column

Appendix H

Brewton Bent Cap

Appendix Table H-1: Concrete mixture proportions – Brewton bent cap

Item	Amount
Type I/II Cement (pcy)	427
Class C Fly Ash (pcy)	107
Water (pcy)	267
#67 Limestone Coarse Aggregate (pcy)	1852
#100 Sand (pcy)	1196
Air Entraining Admixture (oz.)	1
Water Reducer (oz.)	21
MR Water Reducer (oz.)	27

Appendix Table H-2: Cement composition – Brewton bent cap

Chemical Analysis	
Item	**Result (%)**
SiO_2	19.40
Al_2O_3	5.10
Fe_2O_3	3.80
CaO	64.2
MgO	0.80
SO_3	3.10
LOI	2.10
Na_2O_{eq}	0.54
CO_2	0.70
C_3S	63
C_2S	7
C_3A	7
C_4AF	12
Physical Analysis	
Blaine Fineness (m^2/kg)	405

Appendix Table H-3: Fly ash composition – Brewton bent cap

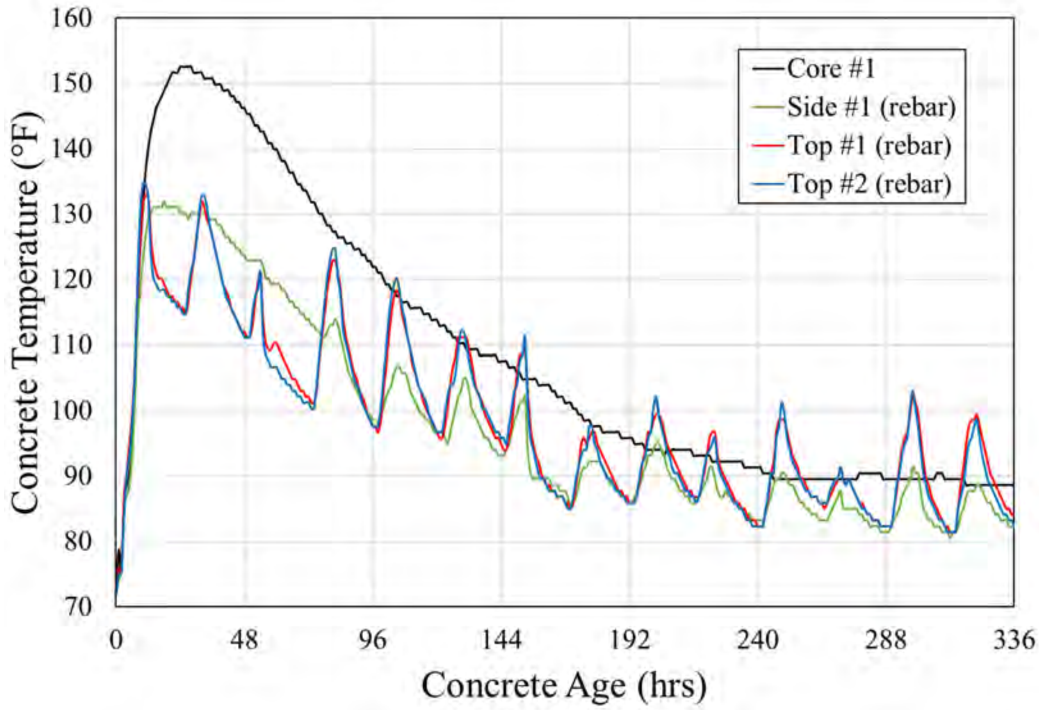
Chemical Analysis	
Item	**Result (%)**
SiO_2	48.87
Al_2O_3	21.28
Fe_2O_3	13.58
CaO	5.83
MgO	0.89
SO_3	2.38
Na_2O	0.71
K_2O	2.41

Appendix Table H-4: 7-day weather data – Brewton bent cap

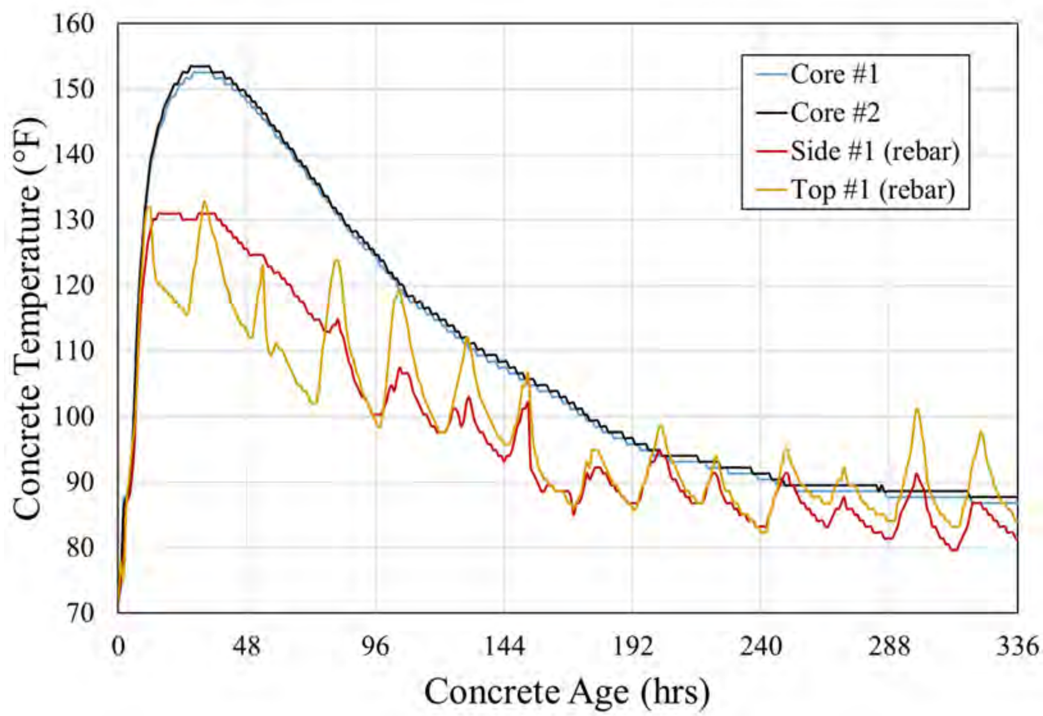
Date	Day	Temperature		Relative Humidity		Max Wind Speed	Max % Cloud Cover
		Min	Max	Min	Max		
7/15/2016	1	76	95	46	100	13	BKN070
7/16/2016	2	75	95	47	100	13	OVC028
7/17/2016	3	76	94	56	100	14	OVC050
7/18/2016	4	75	100	39	100	9	BKN050
7/19/2016	5	76	96	43	100	14	BKN010
7/20/2016	6	76	100	42	100	13	BKN075
7/21/2016	7	76	100	45	100	14	OVC028

Appendix Table H-5: ALDOT test results of project concrete sample specimens – Brewton bent cap

Laboratory Test		Set #1
Strengths	3-day	3255
	7-day	3980
	14-day	4295
	28-day	4865
	28-day	4755
Slump (in.)		3.00
Air Content (%)		4.0
Temperature (°F)		81



Appendix Figure H-1: Cross-section #1 temperature data – Brewton bent cap



Appendix Figure H-2: Cross-section #1 temperature data – Brewton bent cap

Appendix I

Example Calculations Using Bamforth's Method

Contained in this appendix are the crack prediction calculations for the Scottsboro pedestal using Bamforth's method.

Results			
$A_{s,prov}$ (in^2/ft)	0.20	ρ_{prov}	0.0003
$A_{s,min(EA)}$ (in^2/ft)	2.25	$\rho_{crit(EA)}$	0.0042
$A_{s,min(LT)}$ (in^2/ft)	2.89	$\rho_{crit(LT)}$	0.0053
Crack spacing (in.)	65.24	**Crack risk**	1.12
Crack width (EA) (in.)	0.0020		
Crack width (LT) (in.)	0.0222		

Strength class	Strength (MPa)		
	f_{ck}	$f_{ck,cube}$	f_{cm}
C16/20	16	20	24
C20/25	20	25	28
C25/30	25	30	33
C30/37	30	37	38
C35/45	35	45	43
C40/50	40	50	48
C45/55	45	55	53
C50/60	50	60	58
C55/67	55	67	63
C60/75	60	75	68
C70/80	70	58	78
C80/90	80	95	88
C90/105	90	105	98
Selected:	**20**	**25**	**28**

Legend	
(green)	Input
(yellow)	Intermediate Calculation
(blue)	Result of interest
(orange)	Free strain
(dark green)	Restrained strain
(red)	Crack inducing strain

Risk and control of cracking due to continuous edge restraint						
Input parameters	**Symbol**	**Value**	**Unit**	**U.S. Units**		
Section and steel properties						
Section thickness	h	3048	mm	120	in	
Strength class	f_{ck} / $f_{ck,cube}$	20.68	Mpa	3000	psi	Design compressive strength (EG)
		C20/25	MPa			Strength class based on f'_c (EG)
Age at cracking	$t_{c(EA)}$	7	days			Time of max. temp. difference (EG)
Creep factor	K_1	0.65				K_1 = 0.65 if R is calculated; K_1 = 1 if R is assumed to be 0.5 (including creep to EN1992-1-1)
Sustained load factor	K_2	0.80				
Coefficient of thermal expansion of concrete	α_c	7.3	$\mu\varepsilon/^{o}C$	4.04	$\mu\varepsilon/^{o}F$	If aggregate is unknown use 12 $\mu\varepsilon$ / ^{o}C
Characteristic yield strength of reinforcement	f_{yk}	414	MPa	60	ksi	500 Mpa
Early age concrete properties						
Tensile strength at cracking	$f_{ctm}(t_{c(EA)})$	1.72	MPa	250	psi	Mean value of tensile strength $f_{ctm}(t_{c(EA)})$
Elastic modulus	$E_{cm}(t_{c(EA)})$	27.8	GPa	4032	ksi	Mean value of elastic modulus $E_{cm}(t_{c(EA)})$
Tensile strain capacity	$\varepsilon_{ctu(EA)}$	76	$\mu\varepsilon$			$\varepsilon_{ctu(EA)} = [\, f_{ctm}(t_{c(EA)}) / E_{cm}(t_{c(EA)}) \,] \times [K_2 / K_1]$
Early-age strain						
Temperature drop	T_1	17	^{o}C	30	°F	T_1 = Peak temperature - mean ambient temperature
Autogenous shrinkage	$\varepsilon_{ca(EA)}$	10	$\mu\varepsilon$			(EN1992-1-1) $\varepsilon_{ca(EA)} = 2.5\,(f_{ck} - 10) \times (1\text{-}\exp(-\,0.2\,t_c^{0.5})$
Early-age Free contraction	$\varepsilon_{free(EA)}$	131	$\mu\varepsilon$			$\varepsilon_{free(EA)} = T_1\,\alpha_c + \varepsilon_{ca(EA)}$
Restrained early-age strain and risk of cracking						
Restraint	R	0.80				*Use restraint calculator for walls or adjacent slabs; or historical data*
Early-age restrained strain	$\varepsilon_{r(EA)}$	68	$\mu\varepsilon$			$\varepsilon_{r(EA)} = R_1\,K_1\,(T_1\,\alpha_c + \varepsilon_{ca})$
Risk of early age cracking	*Risk*	**1.12**				Risk = $\varepsilon_{r(EA)}/\varepsilon_{ctu(EA)}/K_2$ - Low risk of early age cracking if $\varepsilon_{r(EA)}/\varepsilon_{ctu(EA)} < 1$
Early-age crack-inducing strain	$\varepsilon_{cr(EA)}$	30	$\mu\varepsilon$			$\varepsilon_{cr(EA)} = \varepsilon_{r(EA)} - 0.5\,\varepsilon_{ctu(EA)}$

Long term concrete properties						
Tensile strength	$f_{ctm(28)}$	2.21	MPa	321	psi	Mean 28-day value
Elastic modulus	$E_{cm(28)}$	30.0	GPa	4346	ksi	Mean 28-day value
Tensile strain capacity (sustained loading)	$\varepsilon_{ctu(LT)}$	91	με			$\varepsilon_{ctu(LT)} = [\, f_{ctm(28)} \,/\, E_{cm(28)} \,]\ x\ [K_2 \,/\, K_1]$
Long term strain (excluding early-age strain)						
Autogenous shrinkage (residual up to 28 days)	$\delta\varepsilon_{ca(LT)}$	6	με			$\delta\varepsilon_{ca(LT)} = \varepsilon_{ca(28)} - \varepsilon_{ca(ea)}$
Long term temperature change	T_2	67	°C	120	°F	*T_2 and ε_{cd} only apply when causing differential contraction or when the sections acting integrally are subject to external*
Drying shrinkage	ε_{cd}	224	με			
Long term free contraction	$\varepsilon_{free(LT)}$	715	με			$\varepsilon_{free(LT)} = \delta\varepsilon_{ca} + T_2\,\alpha_c + \varepsilon_{cd}$
Restrained long term strain						
Restraint to long term thermal strains	R_2	0.70				Restraint will reduce as $E_n \,/\, E_o$ approaches 1 in the long term
Restraint to drying shrinkage	R_3	0.70				
Long term restrained strain	$\boldsymbol{\varepsilon_{free(lt)}}$	325	με			$\varepsilon_{c(lt)} = K_1\,\{R_2\,T_2\,\alpha_c + R_3\,(\delta\varepsilon_{ca} + \varepsilon_{cd})\}$
Increase in tensile strain capacity	$\delta\varepsilon_{ctu}$	15	με			$\delta\varepsilon_{ctu} = \varepsilon_{ctu(28)} - \varepsilon_{ctu(ea)}$
Long term crack-inducing strain	$\boldsymbol{\varepsilon_{cr(lt)}}$	311	με			$\varepsilon_{c(lt)} = K_1\,\{R_2\,T_2\,\alpha_c + R_3\,(\delta\varepsilon_{ca} + \varepsilon_{cd})\} - \delta\varepsilon_{ctu}$
Total strain (early-age + long term)						
Free contraction	$\varepsilon_{r(total)}$	**846**	με			$\varepsilon_{free(total)} = \varepsilon_{free(ea)} + \varepsilon_{free(lt)}$
Restrained contraction	$\varepsilon_{r(total)}$	**394**	με			$\varepsilon_{r(total)} = \varepsilon_{r(ea)} + \varepsilon_{r(lt)}$
Crack-inducing strain	$\varepsilon_{cr(total)}$	**341**	με			$\varepsilon_{cr(total)} = \varepsilon_{cr(ea)} + \varepsilon_{cr(lt)}$

Reinforcement details						
Bar diameter	φ	12.7	mm	0.5	in.	
Bar spacing	s	304.8	mm	12	in.	
Cover	c	50.8	mm	2	in.	
Area of steel per face per m	$A_{s,prov}$	416	mm^2	**0.20**	**in^2/ft**	$A_{s,prov} = A_{bar} * 1000/s$
Cracking initiated at early age strain						
Minimum reinforcement for early-age cracking, $A_{s,min(EA)}$						
Steel ratio for early age cracking	$\rho_{crit(EA)}$	0.00416				$f_{ctm} / f_{yk} = \rho_{crit}$
Coefficient	k	0.75				k = 1.0 for $h \leq$ 300mm; k = 0.75 for $h \geq$ 800mm; intermediate values are interpolated
Coefficient	k_c	1				For pure tension $k_c = 1$
Surface zone used in calculating $A_{s,min}$	$h_{s,min}$	1143	mm	45	in.	$h_{s,min} = k\, k_c\, h/2$
Minimum area of steel per face per m	$A_{s,min(EA)}$	4756	mm^2/m	**2.25**	**in^2/ft**	$A_{s,min} = (h_{s,min} \times 1000)\, \rho_{crit(EA)}$ Highlighted if $A_s < A_{s,min}$
Minimum reinforcement requirement for late-life cracking only						
Steel ratio for late-life cracking	ρ_{crit}	0.0053				$f_{ctm} / f_{yk} = \rho_{crit}$
Minimum area of steel per face per m	$A_{s,min(LT)}$	6107	mm^2/m	**2.89**	**in^2/ft**	$A_{s,min} = (h_{s,min} \times 1000)\, \rho_{crit(LT)}$ Highlighted if $A_s < A_{s,min}$
Crack spacing and width						
Surface zone defining the effective area of concrete in tension, $A_{c,eff}$	$h_{e,ef}$	142.875	mm	5.63	in.	$h_{e,ef} = 2.5\,(c + \varphi/2)$ [NOTE: $h_{s,min}$ and $h_{e,ef}$ are not the same]
Steel ratio for estimating crack spacing	$\rho_{p,eff}$	0.00291				$\rho_{p,eff} = A_s / A_{c,eff} = A_s / (h_{e,ef} \times 1000)$
Coefficient for bond characteristics	k_1	0.8				EN1992-1-1 recommends k_1 = 0.8 but provides a factor of 0.7 where good bond cannot be guaranteed. Hence k_1 = 0.8/0.7 = 1.14
Crack spacing	$S_{r,max}$	1657	mm	**65.24**	**in.**	$S_{r,max} = 3.4c + 0.425\, k_1\, \varphi/\rho_{p,eff}$
Early age crack width	w_k	0.05	mm	**0.0020**	**in.**	$w_k = \varepsilon_{c(ea)}\, S_{r,max}$
Long term crack width	w_k	0.56	mm	**0.0222**	**in.**	$w_k = \varepsilon_{c(total)}\, S_{r,max}$

Appendix J

ConcreteWorks Analysis

Appendix Table J-1: 480 ConcreteWorks Statistical Runs' Results

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
1	100% PC	Limestone	Mobile	Jan 15 - 12 pm	54	4	35	117	Low
2	100% PC	Limestone	Mobile	Jan 15 - 12 pm	54	5	42	123	Low
3	100% PC	Limestone	Mobile	Jan 15 - 12 pm	54	6	47	128	Low
4	100% PC	Limestone	Mobile	Jan 15 - 12 pm	54	7	50	132	Low
5	100% PC	Limestone	Mobile	Jan 15 - 12 pm	54	8	54	134	Medium
6	100% PC	Limestone	Mobile	March 15 - 10 am	64.4	4	37	134	Low
7	100% PC	Limestone	Mobile	March 15 - 10 am	64.4	5	44	140	Low
8	100% PC	Limestone	Mobile	March 15 - 10 am	64.4	6	50	143	Medium
9	100% PC	Limestone	Mobile	March 15 - 10 am	64.4	7	53	146	Medium
10	100% PC	Limestone	Mobile	March 15 - 10 am	64.4	8	57	148	High
11	100% PC	Limestone	Mobile	May 15 - 8 am	73.6	4	35	148	Low
12	100% PC	Limestone	Mobile	May 15 - 8 am	73.6	5	43	152	Low
13	100% PC	Limestone	Mobile	May 15 - 8 am	73.6	6	48	155	Low
14	100% PC	Limestone	Mobile	May 15 - 8 am	73.6	7	51	157	Medium
15	100% PC	Limestone	Mobile	May 15 - 8 am	73.6	8	55	159	High
16	100% PC	Limestone	Mobile	July 15 - 8 am	80.6	4	35	157	Low
17	100% PC	Limestone	Mobile	July 15 - 8 am	80.6	5	42	161	Low
18	100% PC	Limestone	Mobile	July 15 - 8 am	80.6	6	47	163	Low
19	100% PC	Limestone	Mobile	July 15 - 8 am	80.6	7	50	165	Low
20	100% PC	Limestone	Mobile	July 15 - 8 am	80.6	8	54	166	Medium

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
21	100% PC	Limestone	Mobile	Sept 15 - 10 am	81.3	4	37	157	Low
22	100% PC	Limestone	Mobile	Sept 15 - 10 am	81.3	5	44	161	Low
23	100% PC	Limestone	Mobile	Sept 15 - 10 am	81.3	6	50	164	Low
24	100% PC	Limestone	Mobile	Sept 15 - 10 am	81.3	7	53	165	Medium
25	100% PC	Limestone	Mobile	Sept 15 - 10 am	81.3	8	58	167	High
26	100% PC	Limestone	Mobile	Nov 15 - 12 pm	66.4	4	37	135	Low
27	100% PC	Limestone	Mobile	Nov 15 - 12 pm	66.4	5	45	141	Low
28	100% PC	Limestone	Mobile	Nov 15 - 12 pm	66.4	6	50	145	Low
29	100% PC	Limestone	Mobile	Nov 15 - 12 pm	66.4	7	54	147	Medium
30	100% PC	Limestone	Mobile	Nov 15 - 12 pm	66.4	8	59	150	High
31	100% PC	Limestone	Huntsville	Jan 15 - 12 pm	41.4	4	30	99	Low
32	100% PC	Limestone	Huntsville	Jan 15 - 12 pm	41.4	5	36	105	Low
33	100% PC	Limestone	Huntsville	Jan 15 - 12 pm	41.4	6	42	111	Low
34	100% PC	Limestone	Huntsville	Jan 15 - 12 pm	41.4	7	46	115	Low
35	100% PC	Limestone	Huntsville	Jan 15 - 12 pm	41.4	8	50	118	Medium
36	100% PC	Limestone	Huntsville	March 15 - 10 am	55.9	4	36	122	Low
37	100% PC	Limestone	Huntsville	March 15 - 10 am	55.9	5	43	128	Low
38	100% PC	Limestone	Huntsville	March 15 - 10 am	55.9	6	48	132	Medium
39	100% PC	Limestone	Huntsville	March 15 - 10 am	55.9	7	52	135	Medium
40	100% PC	Limestone	Huntsville	March 15 - 10 am	55.9	8	56	138	High
41	100% PC	Limestone	Huntsville	May 15 - 8 am	67.5	4	34	140	Low
42	100% PC	Limestone	Huntsville	May 15 - 8 am	67.5	5	42	145	Low
43	100% PC	Limestone	Huntsville	May 15 - 8 am	67.5	6	47	148	Low
44	100% PC	Limestone	Huntsville	May 15 - 8 am	67.5	7	50	150	Medium
45	100% PC	Limestone	Huntsville	May 15 - 8 am	67.5	8	54	152	High
46	100% PC	Limestone	Huntsville	July 15 - 8 am	78.6	4	34	155	Low
47	100% PC	Limestone	Huntsville	July 15 - 8 am	78.6	5	42	159	Low
48	100% PC	Limestone	Huntsville	July 15 - 8 am	78.6	6	47	161	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
49	100% PC	Limestone	Huntsville	July 15 - 8 am	78.6	7	50	163	Low
50	100% PC	Limestone	Huntsville	July 15 - 8 am	78.6	8	54	164	Medium
51	100% PC	Limestone	Huntsville	Sept 15 - 10 am	75.9	4	36	150	Low
52	100% PC	Limestone	Huntsville	Sept 15 - 10 am	75.9	5	44	154	Low
53	100% PC	Limestone	Huntsville	Sept 15 - 10 am	75.9	6	49	157	Low
54	100% PC	Limestone	Huntsville	Sept 15 - 10 am	75.9	7	53	159	Medium
55	100% PC	Limestone	Huntsville	Sept 15 - 10 am	75.9	8	57	161	High
56	100% PC	Limestone	Huntsville	Nov 15 - 12 pm	60.1	4	36	126	Low
57	100% PC	Limestone	Huntsville	Nov 15 - 12 pm	60.1	5	44	132	Low
58	100% PC	Limestone	Huntsville	Nov 15 - 12 pm	60.1	6	49	136	Low
59	100% PC	Limestone	Huntsville	Nov 15 - 12 pm	60.1	7	55	139	Low
60	100% PC	Limestone	Huntsville	Nov 15 - 12 pm	60.1	8	59	142	Medium
61	100% PC	River Gravel	Mobile	Jan 15 - 12 pm	54	4	34	121	Low
62	100% PC	River Gravel	Mobile	Jan 15 - 12 pm	54	5	43	128	Low
63	100% PC	River Gravel	Mobile	Jan 15 - 12 pm	54	6	49	133	Medium
64	100% PC	River Gravel	Mobile	Jan 15 - 12 pm	54	7	52	137	High
65	100% PC	River Gravel	Mobile	Jan 15 - 12 pm	54	8	56	140	Very High
66	100% PC	River Gravel	Mobile	March 15 - 10 am	64.4	4	36	139	Low
67	100% PC	River Gravel	Mobile	March 15 - 10 am	64.4	5	45	145	Medium
68	100% PC	River Gravel	Mobile	March 15 - 10 am	64.4	6	51	149	High
69	100% PC	River Gravel	Mobile	March 15 - 10 am	64.4	7	55	152	Very High
70	100% PC	River Gravel	Mobile	March 15 - 10 am	64.4	8	59	154	Very High
71	100% PC	River Gravel	Mobile	May 15 - 8 am	73.6	4	37	153	Low
72	100% PC	River Gravel	Mobile	May 15 - 8 am	73.6	5	43	158	High
73	100% PC	River Gravel	Mobile	May 15 - 8 am	73.6	6	49	161	Very High
74	100% PC	River Gravel	Mobile	May 15 - 8 am	73.6	7	53	163	Very High
75	100% PC	River Gravel	Mobile	May 15 - 8 am	73.6	8	56	165	Very High
76	100% PC	River Gravel	Mobile	July 15 - 8 am	80.6	4	36	162	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
77	100% PC	River Gravel	Mobile	July 15 - 8 am	80.6	5	41	166	Medium
78	100% PC	River Gravel	Mobile	July 15 - 8 am	80.6	6	48	169	High
79	100% PC	River Gravel	Mobile	July 15 - 8 am	80.6	7	52	171	High
80	100% PC	River Gravel	Mobile	July 15 - 8 am	80.6	8	55	173	Very High
81	100% PC	River Gravel	Mobile	Sept 15 - 10 am	81.3	4	38	162	Low
82	100% PC	River Gravel	Mobile	Sept 15 - 10 am	81.3	5	44	167	High
83	100% PC	River Gravel	Mobile	Sept 15 - 10 am	81.3	6	51	169	Very High
84	100% PC	River Gravel	Mobile	Sept 15 - 10 am	81.3	7	55	171	Very High
85	100% PC	River Gravel	Mobile	Sept 15 - 10 am	81.3	8	59	173	Very High
86	100% PC	River Gravel	Mobile	Nov 15 - 12 pm	66.4	4	36	140	Low
87	100% PC	River Gravel	Mobile	Nov 15 - 12 pm	66.4	5	45	146	Low
88	100% PC	River Gravel	Mobile	Nov 15 - 12 pm	66.4	6	51	150	High
89	100% PC	River Gravel	Mobile	Nov 15 - 12 pm	66.4	7	55	153	Very High
90	100% PC	River Gravel	Mobile	Nov 15 - 12 pm	66.4	8	60	156	Very High
91	100% PC	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	4	31	102	Low
92	100% PC	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	5	37	109	Low
93	100% PC	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	6	42	115	Medium
94	100% PC	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	7	48	120	High
95	100% PC	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	8	52	123	Very High
96	100% PC	River Gravel	Huntsville	March 15 - 10 am	55.9	4	35	127	Low
97	100% PC	River Gravel	Huntsville	March 15 - 10 am	55.9	5	44	133	Low
98	100% PC	River Gravel	Huntsville	March 15 - 10 am	55.9	6	50	138	Very High
99	100% PC	River Gravel	Huntsville	March 15 - 10 am	55.9	7	53	141	Very High
100	100% PC	River Gravel	Huntsville	March 15 - 10 am	55.9	8	58	144	Very High
101	100% PC	River Gravel	Huntsville	May 15 - 8 am	67.5	4	33	146	Low
102	100% PC	River Gravel	Huntsville	May 15 - 8 am	67.5	5	42	150	Medium
103	100% PC	River Gravel	Huntsville	May 15 - 8 am	67.5	6	48	154	High
104	100% PC	River Gravel	Huntsville	May 15 - 8 am	67.5	7	52	156	Very High

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
105	100% PC	River Gravel	Huntsville	May 15 - 8 am	67.5	8	55	158	Very High
106	100% PC	River Gravel	Huntsville	July 15 - 8 am	78.6	4	36	160	Low
107	100% PC	River Gravel	Huntsville	July 15 - 8 am	78.6	5	42	164	Medium
108	100% PC	River Gravel	Huntsville	July 15 - 8 am	78.6	6	48	167	High
109	100% PC	River Gravel	Huntsville	July 15 - 8 am	78.6	7	52	169	Very High
110	100% PC	River Gravel	Huntsville	July 15 - 8 am	78.6	8	56	171	Very High
111	100% PC	River Gravel	Huntsville	Sept 15 - 10 am	75.9	4	38	155	Low
112	100% PC	River Gravel	Huntsville	Sept 15 - 10 am	75.9	5	44	160	High
113	100% PC	River Gravel	Huntsville	Sept 15 - 10 am	75.9	6	51	163	Very High
114	100% PC	River Gravel	Huntsville	Sept 15 - 10 am	75.9	7	55	165	Very High
115	100% PC	River Gravel	Huntsville	Sept 15 - 10 am	75.9	8	59	167	Very High
116	100% PC	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	4	36	130	Low
117	100% PC	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	5	44	137	Low
118	100% PC	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	6	51	141	Medium
119	100% PC	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	7	55	145	High
120	100% PC	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	8	61	148	Very High
121	70% PC + 30% FFA	Limestone	Mobile	Jan 15 - 12 pm	54	4	27	102	Low
122	70% PC + 30% FFA	Limestone	Mobile	Jan 15 - 12 pm	54	5	32	108	Low
123	70% PC + 30% FFA	Limestone	Mobile	Jan 15 - 12 pm	54	6	36	112	Low
124	70% PC + 30% FFA	Limestone	Mobile	Jan 15 - 12 pm	54	7	40	116	Low
125	70% PC + 30% FFA	Limestone	Mobile	Jan 15 - 12 pm	54	8	43	119	Low
126	70% PC + 30% FFA	Limestone	Mobile	March 15 - 10 am	64.4	4	30	119	Low
127	70% PC + 30% FFA	Limestone	Mobile	March 15 - 10 am	64.4	5	36	124	Low
128	70% PC + 30% FFA	Limestone	Mobile	March 15 - 10 am	64.4	6	40	128	Low
129	70% PC + 30% FFA	Limestone	Mobile	March 15 - 10 am	64.4	7	44	131	Low
130	70% PC + 30% FFA	Limestone	Mobile	March 15 - 10 am	64.4	8	48	134	Low
131	70% PC + 30% FFA	Limestone	Mobile	May 15 - 8 am	73.6	4	29	133	Low
132	70% PC + 30% FFA	Limestone	Mobile	May 15 - 8 am	73.6	5	35	137	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
133	70% PC + 30% FFA	Limestone	Mobile	May 15 - 8 am	73.6	6	39	141	Low
134	70% PC + 30% FFA	Limestone	Mobile	May 15 - 8 am	73.6	7	42	143	Low
135	70% PC + 30% FFA	Limestone	Mobile	May 15 - 8 am	73.6	8	46	145	Low
136	70% PC + 30% FFA	Limestone	Mobile	July 15 - 8 am	80.6	4	28	142	Low
137	70% PC + 30% FFA	Limestone	Mobile	July 15 - 8 am	80.6	5	35	146	Low
138	70% PC + 30% FFA	Limestone	Mobile	July 15 - 8 am	80.6	6	39	150	Low
139	70% PC + 30% FFA	Limestone	Mobile	July 15 - 8 am	80.6	7	42	152	Low
140	70% PC + 30% FFA	Limestone	Mobile	July 15 - 8 am	80.6	8	46	154	Low
141	70% PC + 30% FFA	Limestone	Mobile	Sept 15 - 10 am	81.3	4	30	141	Low
142	70% PC + 30% FFA	Limestone	Mobile	Sept 15 - 10 am	81.3	5	37	146	Low
143	70% PC + 30% FFA	Limestone	Mobile	Sept 15 - 10 am	81.3	6	41	149	Low
144	70% PC + 30% FFA	Limestone	Mobile	Sept 15 - 10 am	81.3	7	45	151	Low
145	70% PC + 30% FFA	Limestone	Mobile	Sept 15 - 10 am	81.3	8	49	153	Low
146	70% PC + 30% FFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	4	30	120	Low
147	70% PC + 30% FFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	5	36	126	Low
148	70% PC + 30% FFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	6	41	130	Low
149	70% PC + 30% FFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	7	45	133	Low
150	70% PC + 30% FFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	8	49	135	Low
151	70% PC + 30% FFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	4	22	84	Low
152	70% PC + 30% FFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	5	27	90	Low
153	70% PC + 30% FFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	6	32	95	Low
154	70% PC + 30% FFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	7	36	99	Low
155	70% PC + 30% FFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	8	40	103	Low
156	70% PC + 30% FFA	Limestone	Huntsville	March 15 - 10 am	55.9	4	29	108	Low
157	70% PC + 30% FFA	Limestone	Huntsville	March 15 - 10 am	55.9	5	34	113	Low
158	70% PC + 30% FFA	Limestone	Huntsville	March 15 - 10 am	55.9	6	38	117	Low
159	70% PC + 30% FFA	Limestone	Huntsville	March 15 - 10 am	55.9	7	42	120	Low
160	70% PC + 30% FFA	Limestone	Huntsville	March 15 - 10 am	55.9	8	45	123	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
161	70% PC + 30% FFA	Limestone	Huntsville	May 15 - 8 am	67.5	4	28	126	Low
162	70% PC + 30% FFA	Limestone	Huntsville	May 15 - 8 am	67.5	5	33	130	Low
163	70% PC + 30% FFA	Limestone	Huntsville	May 15 - 8 am	67.5	6	37	134	Low
164	70% PC + 30% FFA	Limestone	Huntsville	May 15 - 8 am	67.5	7	41	136	Low
165	70% PC + 30% FFA	Limestone	Huntsville	May 15 - 8 am	67.5	8	44	138	Low
166	70% PC + 30% FFA	Limestone	Huntsville	July 15 - 8 am	78.6	4	28	140	Low
167	70% PC + 30% FFA	Limestone	Huntsville	July 15 - 8 am	78.6	5	34	144	Low
168	70% PC + 30% FFA	Limestone	Huntsville	July 15 - 8 am	78.6	6	38	147	Low
169	70% PC + 30% FFA	Limestone	Huntsville	July 15 - 8 am	78.6	7	42	150	Low
170	70% PC + 30% FFA	Limestone	Huntsville	July 15 - 8 am	78.6	8	46	151	Low
171	70% PC + 30% FFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	4	30	135	Low
172	70% PC + 30% FFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	5	36	140	Low
173	70% PC + 30% FFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	6	40	143	Low
174	70% PC + 30% FFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	7	45	146	Low
175	70% PC + 30% FFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	8	49	148	Low
176	70% PC + 30% FFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	4	29	111	Low
177	70% PC + 30% FFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	5	35	116	Low
178	70% PC + 30% FFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	6	41	121	Low
179	70% PC + 30% FFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	7	45	124	Low
180	70% PC + 30% FFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	8	49	127	Low
181	70% PC + 30% FFA	River Gravel	Mobile	Jan 15 - 12 pm	54	4	27	104	Low
182	70% PC + 30% FFA	River Gravel	Mobile	Jan 15 - 12 pm	54	5	33	111	Low
183	70% PC + 30% FFA	River Gravel	Mobile	Jan 15 - 12 pm	54	6	37	116	Low
184	70% PC + 30% FFA	River Gravel	Mobile	Jan 15 - 12 pm	54	7	41	120	Low
185	70% PC + 30% FFA	River Gravel	Mobile	Jan 15 - 12 pm	54	8	45	123	Medium
186	70% PC + 30% FFA	River Gravel	Mobile	March 15 - 10 am	64.4	4	30	122	Low
187	70% PC + 30% FFA	River Gravel	Mobile	March 15 - 10 am	64.4	5	36	128	Low
188	70% PC + 30% FFA	River Gravel	Mobile	March 15 - 10 am	64.4	6	41	132	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
189	70% PC + 30% FFA	River Gravel	Mobile	March 15 - 10 am	64.4	7	45	136	Medium
190	70% PC + 30% FFA	River Gravel	Mobile	March 15 - 10 am	64.4	8	49	138	High
191	70% PC + 30% FFA	River Gravel	Mobile	May 15 - 8 am	73.6	4	28	136	Low
192	70% PC + 30% FFA	River Gravel	Mobile	May 15 - 8 am	73.6	5	35	141	Low
193	70% PC + 30% FFA	River Gravel	Mobile	May 15 - 8 am	73.6	6	40	145	Low
194	70% PC + 30% FFA	River Gravel	Mobile	May 15 - 8 am	73.6	7	43	148	Medium
195	70% PC + 30% FFA	River Gravel	Mobile	May 15 - 8 am	73.6	8	48	150	High
196	70% PC + 30% FFA	River Gravel	Mobile	July 15 - 8 am	80.6	4	28	146	Low
197	70% PC + 30% FFA	River Gravel	Mobile	July 15 - 8 am	80.6	5	35	151	Low
198	70% PC + 30% FFA	River Gravel	Mobile	July 15 - 8 am	80.6	6	40	154	Low
199	70% PC + 30% FFA	River Gravel	Mobile	July 15 - 8 am	80.6	7	43	157	Medium
200	70% PC + 30% FFA	River Gravel	Mobile	July 15 - 8 am	80.6	8	47	159	High
201	70% PC + 30% FFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	4	30	146	Low
202	70% PC + 30% FFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	5	37	150	Low
203	70% PC + 30% FFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	6	43	154	Low
204	70% PC + 30% FFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	7	46	157	Medium
205	70% PC + 30% FFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	8	51	159	High
206	70% PC + 30% FFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	4	30	123	Low
207	70% PC + 30% FFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	5	36	129	Low
208	70% PC + 30% FFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	6	41	134	Low
209	70% PC + 30% FFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	7	46	137	Medium
210	70% PC + 30% FFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	8	51	140	High
211	70% PC + 30% FFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	4	22	86	Low
212	70% PC + 30% FFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	5	27	92	Low
213	70% PC + 30% FFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	6	33	98	Low
214	70% PC + 30% FFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	7	37	103	Low
215	70% PC + 30% FFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	8	41	106	Low
216	70% PC + 30% FFA	River Gravel	Huntsville	March 15 - 10 am	55.9	4	28	110	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
217	70% PC + 30% FFA	River Gravel	Huntsville	March 15 - 10 am	55.9	5	34	116	Low
218	70% PC + 30% FFA	River Gravel	Huntsville	March 15 - 10 am	55.9	6	39	121	Medium
219	70% PC + 30% FFA	River Gravel	Huntsville	March 15 - 10 am	55.9	7	43	124	Medium
220	70% PC + 30% FFA	River Gravel	Huntsville	March 15 - 10 am	55.9	8	47	127	High
221	70% PC + 30% FFA	River Gravel	Huntsville	May 15 - 8 am	67.5	4	28	129	Low
222	70% PC + 30% FFA	River Gravel	Huntsville	May 15 - 8 am	67.5	5	34	134	Low
223	70% PC + 30% FFA	River Gravel	Huntsville	May 15 - 8 am	67.5	6	39	138	Low
224	70% PC + 30% FFA	River Gravel	Huntsville	May 15 - 8 am	67.5	7	42	141	Medium
225	70% PC + 30% FFA	River Gravel	Huntsville	May 15 - 8 am	67.5	8	46	143	High
226	70% PC + 30% FFA	River Gravel	Huntsville	July 15 - 8 am	78.6	4	28	144	Low
227	70% PC + 30% FFA	River Gravel	Huntsville	July 15 - 8 am	78.6	5	35	148	Low
228	70% PC + 30% FFA	River Gravel	Huntsville	July 15 - 8 am	78.6	6	40	152	Low
229	70% PC + 30% FFA	River Gravel	Huntsville	July 15 - 8 am	78.6	7	43	154	Medium
230	70% PC + 30% FFA	River Gravel	Huntsville	July 15 - 8 am	78.6	8	47	156	High
231	70% PC + 30% FFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	4	29	138	Low
232	70% PC + 30% FFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	5	36	143	Low
233	70% PC + 30% FFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	6	42	147	Low
234	70% PC + 30% FFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	7	45	150	Medium
235	70% PC + 30% FFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	8	50	153	High
236	70% PC + 30% FFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	4	29	113	Low
237	70% PC + 30% FFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	5	35	119	Low
238	70% PC + 30% FFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	6	40	124	Low
239	70% PC + 30% FFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	7	46	128	Low
240	70% PC + 30% FFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	8	50	132	High
241	50% PC + 50% Slag	Limestone	Mobile	Jan 15 - 12 pm	54	4	25	99	Low
242	50% PC + 50% Slag	Limestone	Mobile	Jan 15 - 12 pm	54	5	30	106	Low
243	50% PC + 50% Slag	Limestone	Mobile	Jan 15 - 12 pm	54	6	36	112	Low
244	50% PC + 50% Slag	Limestone	Mobile	Jan 15 - 12 pm	54	7	41	117	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
245	50% PC + 50% Slag	Limestone	Mobile	Jan 15 - 12 pm	54	8	46	122	Low
246	50% PC + 50% Slag	Limestone	Mobile	March 15 - 10 am	64.4	4	30	119	Low
247	50% PC + 50% Slag	Limestone	Mobile	March 15 - 10 am	64.4	5	36	125	Low
248	50% PC + 50% Slag	Limestone	Mobile	March 15 - 10 am	64.4	6	42	131	Low
249	50% PC + 50% Slag	Limestone	Mobile	March 15 - 10 am	64.4	7	47	135	Low
250	50% PC + 50% Slag	Limestone	Mobile	March 15 - 10 am	64.4	8	52	139	Medium
251	50% PC + 50% Slag	Limestone	Mobile	May 15 - 8 am	73.6	4	30	134	Low
252	50% PC + 50% Slag	Limestone	Mobile	May 15 - 8 am	73.6	5	37	141	Low
253	50% PC + 50% Slag	Limestone	Mobile	May 15 - 8 am	73.6	6	42	146	Low
254	50% PC + 50% Slag	Limestone	Mobile	May 15 - 8 am	73.6	7	47	150	Low
255	50% PC + 50% Slag	Limestone	Mobile	May 15 - 8 am	73.6	8	52	153	Medium
256	50% PC + 50% Slag	Limestone	Mobile	July 15 - 8 am	80.6	4	30	145	Low
257	50% PC + 50% Slag	Limestone	Mobile	July 15 - 8 am	80.6	5	38	151	Low
258	50% PC + 50% Slag	Limestone	Mobile	July 15 - 8 am	80.6	6	43	156	Low
259	50% PC + 50% Slag	Limestone	Mobile	July 15 - 8 am	80.6	7	48	160	Low
260	50% PC + 50% Slag	Limestone	Mobile	July 15 - 8 am	80.6	8	52	163	Medium
261	50% PC + 50% Slag	Limestone	Mobile	Sept 15 - 10 am	81.3	4	32	144	Low
262	50% PC + 50% Slag	Limestone	Mobile	Sept 15 - 10 am	81.3	5	40	151	Low
263	50% PC + 50% Slag	Limestone	Mobile	Sept 15 - 10 am	81.3	6	46	156	Low
264	50% PC + 50% Slag	Limestone	Mobile	Sept 15 - 10 am	81.3	7	51	160	Low
265	50% PC + 50% Slag	Limestone	Mobile	Sept 15 - 10 am	81.3	8	56	163	Medium
266	50% PC + 50% Slag	Limestone	Mobile	Nov 15 - 12 pm	66.4	4	30	120	Low
267	50% PC + 50% Slag	Limestone	Mobile	Nov 15 - 12 pm	66.4	5	36	127	Low
268	50% PC + 50% Slag	Limestone	Mobile	Nov 15 - 12 pm	66.4	6	43	132	Low
269	50% PC + 50% Slag	Limestone	Mobile	Nov 15 - 12 pm	66.4	7	49	137	Low
270	50% PC + 50% Slag	Limestone	Mobile	Nov 15 - 12 pm	66.4	8	53	141	Medium
271	50% PC + 50% Slag	Limestone	Huntsville	Jan 15 - 12 pm	41.4	4	19	80	Low
272	50% PC + 50% Slag	Limestone	Huntsville	Jan 15 - 12 pm	41.4	5	25	86	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
273	50% PC + 50% Slag	Limestone	Huntsville	Jan 15 - 12 pm	41.4	6	30	92	Low
274	50% PC + 50% Slag	Limestone	Huntsville	Jan 15 - 12 pm	41.4	7	35	98	Low
275	50% PC + 50% Slag	Limestone	Huntsville	Jan 15 - 12 pm	41.4	8	39	102	Low
276	50% PC + 50% Slag	Limestone	Huntsville	March 15 - 10 am	55.9	4	27	105	Low
277	50% PC + 50% Slag	Limestone	Huntsville	March 15 - 10 am	55.9	5	33	112	Low
278	50% PC + 50% Slag	Limestone	Huntsville	March 15 - 10 am	55.9	6	39	117	Low
279	50% PC + 50% Slag	Limestone	Huntsville	March 15 - 10 am	55.9	7	44	122	Low
280	50% PC + 50% Slag	Limestone	Huntsville	March 15 - 10 am	55.9	8	49	127	Medium
281	50% PC + 50% Slag	Limestone	Huntsville	May 15 - 8 am	67.5	4	29	126	Low
282	50% PC + 50% Slag	Limestone	Huntsville	May 15 - 8 am	67.5	5	34	132	Low
283	50% PC + 50% Slag	Limestone	Huntsville	May 15 - 8 am	67.5	6	40	137	Low
284	50% PC + 50% Slag	Limestone	Huntsville	May 15 - 8 am	67.5	7	45	142	Low
285	50% PC + 50% Slag	Limestone	Huntsville	May 15 - 8 am	67.5	8	49	145	Medium
286	50% PC + 50% Slag	Limestone	Huntsville	July 15 - 8 am	78.6	4	30	143	Low
287	50% PC + 50% Slag	Limestone	Huntsville	July 15 - 8 am	78.6	5	37	149	Low
288	50% PC + 50% Slag	Limestone	Huntsville	July 15 - 8 am	78.6	6	43	153	Low
289	50% PC + 50% Slag	Limestone	Huntsville	July 15 - 8 am	78.6	7	48	157	Low
290	50% PC + 50% Slag	Limestone	Huntsville	July 15 - 8 am	78.6	8	52	160	High
291	50% PC + 50% Slag	Limestone	Huntsville	Sept 15 - 10 am	75.9	4	31	136	Low
292	50% PC + 50% Slag	Limestone	Huntsville	Sept 15 - 10 am	75.9	5	38	143	Low
293	50% PC + 50% Slag	Limestone	Huntsville	Sept 15 - 10 am	75.9	6	45	148	Low
294	50% PC + 50% Slag	Limestone	Huntsville	Sept 15 - 10 am	75.9	7	50	152	Low
295	50% PC + 50% Slag	Limestone	Huntsville	Sept 15 - 10 am	75.9	8	54	156	High
296	50% PC + 50% Slag	Limestone	Huntsville	Nov 15 - 12 pm	60.1	4	28	109	Low
297	50% PC + 50% Slag	Limestone	Huntsville	Nov 15 - 12 pm	60.1	5	35	116	Low
298	50% PC + 50% Slag	Limestone	Huntsville	Nov 15 - 12 pm	60.1	6	42	122	Low
299	50% PC + 50% Slag	Limestone	Huntsville	Nov 15 - 12 pm	60.1	7	47	127	Low
300	50% PC + 50% Slag	Limestone	Huntsville	Nov 15 - 12 pm	60.1	8	52	131	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
301	50% PC + 50% Slag	River Gravel	Mobile	Jan 15 - 12 pm	54	4	25	101	Low
302	50% PC + 50% Slag	River Gravel	Mobile	Jan 15 - 12 pm	54	5	31	109	Low
303	50% PC + 50% Slag	River Gravel	Mobile	Jan 15 - 12 pm	54	6	36	115	Low
304	50% PC + 50% Slag	River Gravel	Mobile	Jan 15 - 12 pm	54	7	42	121	Low
305	50% PC + 50% Slag	River Gravel	Mobile	Jan 15 - 12 pm	54	8	47	126	Medium
306	50% PC + 50% Slag	River Gravel	Mobile	March 15 - 10 am	64.4	4	30	122	Low
307	50% PC + 50% Slag	River Gravel	Mobile	March 15 - 10 am	64.4	5	37	129	Low
308	50% PC + 50% Slag	River Gravel	Mobile	March 15 - 10 am	64.4	6	43	135	Medium
309	50% PC + 50% Slag	River Gravel	Mobile	March 15 - 10 am	64.4	7	49	140	High
310	50% PC + 50% Slag	River Gravel	Mobile	March 15 - 10 am	64.4	8	54	144	Very High
311	50% PC + 50% Slag	River Gravel	Mobile	May 15 - 8 am	73.6	4	30	138	Low
312	50% PC + 50% Slag	River Gravel	Mobile	May 15 - 8 am	73.6	5	38	145	Low
313	50% PC + 50% Slag	River Gravel	Mobile	May 15 - 8 am	73.6	6	44	150	Medium
314	50% PC + 50% Slag	River Gravel	Mobile	May 15 - 8 am	73.6	7	49	155	High
315	50% PC + 50% Slag	River Gravel	Mobile	May 15 - 8 am	73.6	8	54	159	Very High
316	50% PC + 50% Slag	River Gravel	Mobile	July 15 - 8 am	80.6	4	30	149	Low
317	50% PC + 50% Slag	River Gravel	Mobile	July 15 - 8 am	80.6	5	38	156	Low
318	50% PC + 50% Slag	River Gravel	Mobile	July 15 - 8 am	80.6	6	44	161	Medium
319	50% PC + 50% Slag	River Gravel	Mobile	July 15 - 8 am	80.6	7	49	165	High
320	50% PC + 50% Slag	River Gravel	Mobile	July 15 - 8 am	80.6	8	54	168	Very High
321	50% PC + 50% Slag	River Gravel	Mobile	Sept 15 - 10 am	81.3	4	32	148	Low
322	50% PC + 50% Slag	River Gravel	Mobile	Sept 15 - 10 am	81.3	5	40	155	Low
323	50% PC + 50% Slag	River Gravel	Mobile	Sept 15 - 10 am	81.3	6	47	161	High
324	50% PC + 50% Slag	River Gravel	Mobile	Sept 15 - 10 am	81.3	7	52	165	Very High
325	50% PC + 50% Slag	River Gravel	Mobile	Sept 15 - 10 am	81.3	8	58	168	Very High
326	50% PC + 50% Slag	River Gravel	Mobile	Nov 15 - 12 pm	66.4	4	30	123	Low
327	50% PC + 50% Slag	River Gravel	Mobile	Nov 15 - 12 pm	66.4	5	37	130	Low
328	50% PC + 50% Slag	River Gravel	Mobile	Nov 15 - 12 pm	66.4	6	43	137	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
329	50% PC + 50% Slag	River Gravel	Mobile	Nov 15 - 12 pm	66.4	7	50	142	Medium
330	50% PC + 50% Slag	River Gravel	Mobile	Nov 15 - 12 pm	66.4	8	55	146	Very High
331	50% PC + 50% Slag	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	4	20	81	Low
332	50% PC + 50% Slag	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	5	25	89	Low
333	50% PC + 50% Slag	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	6	30	95	Low
334	50% PC + 50% Slag	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	7	36	101	Low
335	50% PC + 50% Slag	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	8	41	106	Low
336	50% PC + 50% Slag	River Gravel	Huntsville	March 15 - 10 am	55.9	4	27	108	Low
337	50% PC + 50% Slag	River Gravel	Huntsville	March 15 - 10 am	55.9	5	34	115	Low
338	50% PC + 50% Slag	River Gravel	Huntsville	March 15 - 10 am	55.9	6	39	121	Medium
339	50% PC + 50% Slag	River Gravel	Huntsville	March 15 - 10 am	55.9	7	45	127	High
340	50% PC + 50% Slag	River Gravel	Huntsville	March 15 - 10 am	55.9	8	50	131	Very High
341	50% PC + 50% Slag	River Gravel	Huntsville	May 15 - 8 am	67.5	4	29	130	Low
342	50% PC + 50% Slag	River Gravel	Huntsville	May 15 - 8 am	67.5	5	35	136	Low
343	50% PC + 50% Slag	River Gravel	Huntsville	May 15 - 8 am	67.5	6	41	142	Medium
344	50% PC + 50% Slag	River Gravel	Huntsville	May 15 - 8 am	67.5	7	47	146	High
345	50% PC + 50% Slag	River Gravel	Huntsville	May 15 - 8 am	67.5	8	51	150	Very High
346	50% PC + 50% Slag	River Gravel	Huntsville	July 15 - 8 am	78.6	4	30	147	Low
347	50% PC + 50% Slag	River Gravel	Huntsville	July 15 - 8 am	78.6	5	38	153	Low
348	50% PC + 50% Slag	River Gravel	Huntsville	July 15 - 8 am	78.6	6	44	158	Medium
349	50% PC + 50% Slag	River Gravel	Huntsville	July 15 - 8 am	78.6	7	49	162	High
350	50% PC + 50% Slag	River Gravel	Huntsville	July 15 - 8 am	78.6	8	54	166	Very High
351	50% PC + 50% Slag	River Gravel	Huntsville	Sept 15 - 10 am	75.9	4	31	140	Low
352	50% PC + 50% Slag	River Gravel	Huntsville	Sept 15 - 10 am	75.9	5	39	147	Low
353	50% PC + 50% Slag	River Gravel	Huntsville	Sept 15 - 10 am	75.9	6	45	153	Medium
354	50% PC + 50% Slag	River Gravel	Huntsville	Sept 15 - 10 am	75.9	7	51	157	High
355	50% PC + 50% Slag	River Gravel	Huntsville	Sept 15 - 10 am	75.9	8	57	161	Very High
356	50% PC + 50% Slag	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	4	28	111	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
357	50% PC + 50% Slag	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	5	35	119	Low
358	50% PC + 50% Slag	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	6	42	126	Low
359	50% PC + 50% Slag	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	7	48	131	Medium
360	50% PC + 50% Slag	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	8	54	136	High
361	70% PC + 30% CFA	Limestone	Mobile	Jan 15 - 12 pm	54	4	28	105	Low
362	70% PC + 30% CFA	Limestone	Mobile	Jan 15 - 12 pm	54	5	34	112	Low
363	70% PC + 30% CFA	Limestone	Mobile	Jan 15 - 12 pm	54	6	39	117	Low
364	70% PC + 30% CFA	Limestone	Mobile	Jan 15 - 12 pm	54	7	43	121	Low
365	70% PC + 30% CFA	Limestone	Mobile	Jan 15 - 12 pm	54	8	47	125	Low
366	70% PC + 30% CFA	Limestone	Mobile	March 15 - 10 am	64.4	4	32	124	Low
367	70% PC + 30% CFA	Limestone	Mobile	March 15 - 10 am	64.4	5	38	129	Low
368	70% PC + 30% CFA	Limestone	Mobile	March 15 - 10 am	64.4	6	43	134	Low
369	70% PC + 30% CFA	Limestone	Mobile	March 15 - 10 am	64.4	7	48	137	Low
370	70% PC + 30% CFA	Limestone	Mobile	March 15 - 10 am	64.4	8	52	140	Medium
371	70% PC + 30% CFA	Limestone	Mobile	May 15 - 8 am	73.6	4	31	138	Low
372	70% PC + 30% CFA	Limestone	Mobile	May 15 - 8 am	73.6	5	38	143	Low
373	70% PC + 30% CFA	Limestone	Mobile	May 15 - 8 am	73.6	6	42	147	Low
374	70% PC + 30% CFA	Limestone	Mobile	May 15 - 8 am	73.6	7	46	149	Low
375	70% PC + 30% CFA	Limestone	Mobile	May 15 - 8 am	73.6	8	50	151	Medium
376	70% PC + 30% CFA	Limestone	Mobile	July 15 - 8 am	80.6	4	31	147	Low
377	70% PC + 30% CFA	Limestone	Mobile	July 15 - 8 am	80.6	5	38	152	Low
378	70% PC + 30% CFA	Limestone	Mobile	July 15 - 8 am	80.6	6	42	155	Low
379	70% PC + 30% CFA	Limestone	Mobile	July 15 - 8 am	80.6	7	46	158	Low
380	70% PC + 30% CFA	Limestone	Mobile	July 15 - 8 am	80.6	8	50	160	Medium
381	70% PC + 30% CFA	Limestone	Mobile	Sept 15 - 10 am	81.3	4	33	147	Low
382	70% PC + 30% CFA	Limestone	Mobile	Sept 15 - 10 am	81.3	5	40	152	Low
383	70% PC + 30% CFA	Limestone	Mobile	Sept 15 - 10 am	81.3	6	45	155	Low
384	70% PC + 30% CFA	Limestone	Mobile	Sept 15 - 10 am	81.3	7	49	158	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
385	70% PC + 30% CFA	Limestone	Mobile	Sept 15 - 10 am	81.3	8	53	160	Medium
386	70% PC + 30% CFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	4	32	124	Low
387	70% PC + 30% CFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	5	38	130	Low
388	70% PC + 30% CFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	6	44	135	Low
389	70% PC + 30% CFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	7	49	139	Low
390	70% PC + 30% CFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	8	53	141	Low
391	70% PC + 30% CFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	4	23	87	Low
392	70% PC + 30% CFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	5	29	94	Low
393	70% PC + 30% CFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	6	34	99	Low
394	70% PC + 30% CFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	7	39	104	Low
395	70% PC + 30% CFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	8	43	108	Low
396	70% PC + 30% CFA	Limestone	Huntsville	March 15 - 10 am	55.9	4	30	111	Low
397	70% PC + 30% CFA	Limestone	Huntsville	March 15 - 10 am	55.9	5	36	117	Low
398	70% PC + 30% CFA	Limestone	Huntsville	March 15 - 10 am	55.9	6	41	122	Low
399	70% PC + 30% CFA	Limestone	Huntsville	March 15 - 10 am	55.9	7	46	126	Low
400	70% PC + 30% CFA	Limestone	Huntsville	March 15 - 10 am	55.9	8	49	129	Medium
401	70% PC + 30% CFA	Limestone	Huntsville	May 15 - 8 am	67.5	4	30	130	Low
402	70% PC + 30% CFA	Limestone	Huntsville	May 15 - 8 am	67.5	5	36	135	Low
403	70% PC + 30% CFA	Limestone	Huntsville	May 15 - 8 am	67.5	6	40	139	Low
404	70% PC + 30% CFA	Limestone	Huntsville	May 15 - 8 am	67.5	7	45	142	Low
405	70% PC + 30% CFA	Limestone	Huntsville	May 15 - 8 am	67.5	8	48	144	Low
406	70% PC + 30% CFA	Limestone	Huntsville	July 15 - 8 am	78.6	4	31	145	Low
407	70% PC + 30% CFA	Limestone	Huntsville	July 15 - 8 am	78.6	5	37	150	Low
408	70% PC + 30% CFA	Limestone	Huntsville	July 15 - 8 am	78.6	6	42	153	Low
409	70% PC + 30% CFA	Limestone	Huntsville	July 15 - 8 am	78.6	7	46	156	Low
410	70% PC + 30% CFA	Limestone	Huntsville	July 15 - 8 am	78.6	8	50	158	Medium
411	70% PC + 30% CFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	4	32	139	Low
412	70% PC + 30% CFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	5	39	145	Low

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
413	70% PC + 30% CFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	6	44	149	Low
414	70% PC + 30% CFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	7	49	152	Low
415	70% PC + 30% CFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	8	53	154	Medium
416	70% PC + 30% CFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	4	31	114	Low
417	70% PC + 30% CFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	5	37	121	Low
418	70% PC + 30% CFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	6	44	126	Low
419	70% PC + 30% CFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	7	49	130	Low
420	70% PC + 30% CFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	8	52	133	Low
421	70% PC + 30% CFA	River Gravel	Mobile	Jan 15 - 12 pm	54	4	29	108	Low
422	70% PC + 30% CFA	River Gravel	Mobile	Jan 15 - 12 pm	54	5	35	116	Low
423	70% PC + 30% CFA	River Gravel	Mobile	Jan 15 - 12 pm	54	6	40	122	Low
424	70% PC + 30% CFA	River Gravel	Mobile	Jan 15 - 12 pm	54	7	45	126	Medium
425	70% PC + 30% CFA	River Gravel	Mobile	Jan 15 - 12 pm	54	8	49	130	High
426	70% PC + 30% CFA	River Gravel	Mobile	March 15 - 10 am	64.4	4	32	128	Low
427	70% PC + 30% CFA	River Gravel	Mobile	March 15 - 10 am	64.4	5	40	134	Low
428	70% PC + 30% CFA	River Gravel	Mobile	March 15 - 10 am	64.4	6	45	139	Medium
429	70% PC + 30% CFA	River Gravel	Mobile	March 15 - 10 am	64.4	7	49	143	High
430	70% PC + 30% CFA	River Gravel	Mobile	March 15 - 10 am	64.4	8	54	146	Very High
431	70% PC + 30% CFA	River Gravel	Mobile	May 15 - 8 am	73.6	4	31	142	Low
432	70% PC + 30% CFA	River Gravel	Mobile	May 15 - 8 am	73.6	5	39	148	Low
433	70% PC + 30% CFA	River Gravel	Mobile	May 15 - 8 am	73.6	6	44	152	Medium
434	70% PC + 30% CFA	River Gravel	Mobile	May 15 - 8 am	73.6	7	48	155	High
435	70% PC + 30% CFA	River Gravel	Mobile	May 15 - 8 am	73.6	8	52	158	Very High
436	70% PC + 30% CFA	River Gravel	Mobile	July 15 - 8 am	80.6	4	30	152	Low
437	70% PC + 30% CFA	River Gravel	Mobile	July 15 - 8 am	80.6	5	38	157	Low
438	70% PC + 30% CFA	River Gravel	Mobile	July 15 - 8 am	80.6	6	44	161	Medium
439	70% PC + 30% CFA	River Gravel	Mobile	July 15 - 8 am	80.6	7	48	164	High
440	70% PC + 30% CFA	River Gravel	Mobile	July 15 - 8 am	80.6	8	52	166	Very High

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
441	70% PC + 30% CFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	4	32	152	Low
442	70% PC + 30% CFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	5	41	157	Low
443	70% PC + 30% CFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	6	47	161	Medium
444	70% PC + 30% CFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	7	50	164	High
445	70% PC + 30% CFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	8	55	166	Very High
446	70% PC + 30% CFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	4	32	128	Low
447	70% PC + 30% CFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	5	39	135	Low
448	70% PC + 30% CFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	6	45	140	Low
449	70% PC + 30% CFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	7	50	144	Medium
450	70% PC + 30% CFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	8	55	147	High
451	70% PC + 30% CFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	4	23	89	Low
452	70% PC + 30% CFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	5	29	97	Low
453	70% PC + 30% CFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	6	35	103	Low
454	70% PC + 30% CFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	7	40	108	Low
455	70% PC + 30% CFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	8	45	113	Medium
456	70% PC + 30% CFA	River Gravel	Huntsville	March 15 - 10 am	55.9	4	31	115	Low
457	70% PC + 30% CFA	River Gravel	Huntsville	March 15 - 10 am	55.9	5	37	122	Low
458	70% PC + 30% CFA	River Gravel	Huntsville	March 15 - 10 am	55.9	6	42	127	Medium
459	70% PC + 30% CFA	River Gravel	Huntsville	March 15 - 10 am	55.9	7	47	131	High
460	70% PC + 30% CFA	River Gravel	Huntsville	March 15 - 10 am	55.9	8	52	134	Very High
461	70% PC + 30% CFA	River Gravel	Huntsville	May 15 - 8 am	67.5	4	30	134	Low
462	70% PC + 30% CFA	River Gravel	Huntsville	May 15 - 8 am	67.5	5	37	140	Low
463	70% PC + 30% CFA	River Gravel	Huntsville	May 15 - 8 am	67.5	6	42	145	Medium
464	70% PC + 30% CFA	River Gravel	Huntsville	May 15 - 8 am	67.5	7	46	148	High
465	70% PC + 30% CFA	River Gravel	Huntsville	May 15 - 8 am	67.5	8	51	151	Very High
466	70% PC + 30% CFA	River Gravel	Huntsville	July 15 - 8 am	78.6	4	31	150	Low
467	70% PC + 30% CFA	River Gravel	Huntsville	July 15 - 8 am	78.6	5	38	155	Low
468	70% PC + 30% CFA	River Gravel	Huntsville	July 15 - 8 am	78.6	6	44	159	Medium

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Least Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
469	70% PC + 30% CFA	River Gravel	Huntsville	July 15 - 8 am	78.6	7	47	162	High
470	70% PC + 30% CFA	River Gravel	Huntsville	July 15 - 8 am	78.6	8	52	164	Very High
471	70% PC + 30% CFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	4	34	144	Low
472	70% PC + 30% CFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	5	40	150	Low
473	70% PC + 30% CFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	6	46	154	Medium
474	70% PC + 30% CFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	7	50	157	High
475	70% PC + 30% CFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	8	55	160	Very High
476	70% PC + 30% CFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	4	31	118	Low
477	70% PC + 30% CFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	5	38	125	Low
478	70% PC + 30% CFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	6	44	131	Low
479	70% PC + 30% CFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	7	50	135	Medium
480	70% PC + 30% CFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	8	55	139	High