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A case study of mass concrete construction for Midwest boarder bridges

Jacob Joseph Shaw
Iowa State University

A case study of mass concrete construction for Midwest boarder bridges

by

Jacob Joseph Shaw

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Program of Study Committee:
Charles T. Jahren, Co-Major Professor
Kejin Wang, Co-Major Professor
Fouad S. Fanous, Co-Major Professor
Jon M. Rouse

Iowa State University

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ABSTRACT

The construction of mass concrete elements poses several risks with regard to thermal damage, which results from the thermal characteristic and size of mass concrete elements. If the heat of hydration generated in mass concrete is not controlled or dissipated properly, thermal damage may result by means of thermal cracking and/or delayed ettringite formation.

The objectives of the present research are: (1) to have an understanding of how different mix proportions, construction, and environmental conditions affect the thermal development of mass concrete. (2) To identify and compare the similarities and differences of mass concrete specifications for different agencies throughout the United States. (3) To explore a tool for effectively analyzing temperature development and cracking potential of mass concrete. (4) Provide rational recommendations for improving current mass concrete specifications.

In this study, a literature and specification survey was conducted to determine typical construction practices, and to identify conditions that typically have the largest effect on the thermal development of mass concrete. A case study was developed for two typical Midwest boarder bridges, and was utilized to calibrate and verify ConcreteWorks. ConcreteWorks is a concrete thermal analysis software program, capable of predicting the early age thermal behavior of mass concrete. ConcreteWorks was then utilized to validate conditions that are believed to have the largest effect on the thermal development of mass concrete.

The research also provides two case studies of typical Midwest boarder bridges that may be used to calibrate other thermal analysis software programs. Through the sensitivity study, the research also verifies conditions that have the largest effect on the thermal development of mass concrete for Midwest boarder bridges.

The research determined: (1) How different mix proportion, construction, and environmental conditions affect the thermal development of mass concrete. (2) There is little consistency between mass concrete specifications between state agencies. (3) ConcreteWorks is capable of effectively analyzing the temperature development and cracking potential of mass concrete elements. (4) Performance based specifications should be utilized to allow contractors more flexibility in the design of the construction of mass concrete placements. Furthermore, additional specification requirements should be implemented for the thermal monitoring of mass concrete elements to assure accurate temperature readings.

This research contributes to the current knowledge of mass concrete by providing recommendations for specification improvement.

CHAPTER 1. INTRODUCTION

1.1 OVERVIEW

The chapter presents the industrial and technical problems that lead to the research, the goal and objectives of the research, the methodology utilized to complete the research, the significance of the research, and the scope of the research.

1.2 INDUSTRIAL AND TECHNICAL PROBLEMS

The construction of mass concrete placements is an inevitable consequence of the characteristics of Midwest boarder bridges. The proper design and construction of mass concrete placements is essential to ensure the durability and serviceability of the structure.

If the heat of hydration generated in a mass concrete element is not controlled, thermal cracking and or delay ettringite formation may cause damage to the element. Thermal cracking is the result of thermal gradients caused by the surface of the placement, or mass concrete element, dissipating the heat of hydration more rapidly than the center of the placement. The thermal gradients develop tensile stresses at the surface of the placement, if the stresses exceed the developed tensile strength of the concrete, the placement may experience cracking.

Ettringite formation is a normal aspect of the cement hydration processes. The formation of ettringite causes expansive pressure to develop in the concrete as a result of the increase in volume. The expansive pressures due to ettringite formation do not pose a threat to the durability of the concrete if the concrete is still plastic. However, if the concrete experiences extreme temperatures during hydration, the formation of ettringite ceases and the previously formed ettringite decomposes. During later stages, the ettringite may reform in the presence of water, causing expansive pressures in the hardened concrete. The expansive pressure in the hardened concrete cause tensile stresses to develop, which may cause the concrete to crack.

To avoid thermal damage to mass concrete elements, it is important to have a strong understanding of how various mix proportions, construction, and environmental parameters affect the thermal development of mass concrete placements. In general, it is understood how different conditions affect the thermal development, however there has been little research conducted on how different conditions affect the thermal development for specifically Midwest boarder bridge construction.

In addition, the design and planning of mass concrete construction often requires a thermal analysis of each placement, which must be calibrated and verified before application. Currently there are many software packages available capable of analyzing the thermal development of mass concrete placements including ConcreteWorks, 4C Temp&Stress, ANSYS, and many others. Additionally, case studies have been conducted to validate the effectiveness of thermal analysis software packages to analyze mass concrete placements. However, there is little research detailing the effectiveness of thermal analysis software programs to analyze the thermal behavior of mass concrete typical of Midwest boarder bridges construction.

1.3 GOALS AND OBJECTIVES OF THE RESEARCH

The goal of the research is to provide insight on the construction, design, and thermal analysis of typical Midwest boarder bridge mass concrete placements.

This thesis contains four different objectives. (1) Identify the similarities and differences of mass concrete specifications for different agencies throughout the United States. (2) Provide two case studies of typical Midwest boarder bridge mass concrete construction. (3) Evaluate the effectiveness of ConcreteWorks to analyze typical mass concrete placements of Midwest board bridges. (4) Confirm parameters identified from literate that have the largest effect on the thermal development of Midwest boarder bridge mass concrete placements.

1.4 METHODOLOGY

The first aspect of the research was to provide an overview of the mass concrete, and a summary of the current knowledge. The literature review provides a background on the basis of cement hydrations, current mass concrete practices, and the failure mechanisms and concerns of mass concrete. In addition to the literature review, a specification survey was conducted to identify typical requirements and practices for mass concrete construction. After completing a survey of current knowledge, an investigation was conducted to determine the typical characteristics of Midwest boarder bridges to be used to validate two case studies as being typical Midwest boarder bridges, and provide parameter ranges for the sensitivity study.

A case study was subsequently conducted on the WB I-80 Bridge over the Missouri River Bridge and the US 34 over the Missouri River Bridge. The purpose of the case study was to detail the actual construction practices of typical Midwest boarder bridges. The case studies also served as a reference for the calibration of ConcreteWorks, a mass concrete thermal analysis software package.

Following the initial calibration of ConcreteWorks, the software application was investigated further by means of a sensitivity analysis. The objective of the sensitivity analysis was to provide additional verification of ConcreteWorks by investigating parameters known to affect the thermal development of mass concrete. The sensitivity analysis was also conducted to provide insight for the design and construction of mass concrete elements by evaluating the general affect of various parameters on the thermal development of typical Midwest boarder bridge mass concrete elements.

1.5 SIGNIFICANCE OF THE RESEARCH

This reach provides additional knowledge to the field of mass concrete in the scope of Midwest boarder bridges. The research provides a detailed account of the construction of two Midwest boarder bridges that may serve as a reference for future research. In addition, the research demonstrates the ability of ConcreteWorks to analyze Midwest boarder bridges for future application of the software package on similar projects.

This research will allow the industry to more readily use ConcreteWorks to analyze mass concrete elements of Midwest boarder bridges, and make inferences on the accuracy of the respective results. The case study detailed in the following thesis may also serve as a baseline for the accuracy of future mass concrete analysis software packages.

1.6 THESIS SCOPE

Following the introduction chapter, this thesis is divided into eight additional chapters including the literature review, specification survey, typical Midwest boarder bridges, case study, ConcreteWorks calibration, sensitivity study, conclusions, and recommendations.

Chapter 2. Literature Review: An overall review of the subject matter of the research, containing a historical development of mass concrete, review of cement hydration process, review of aspects affecting mass concrete, a mass concrete overview, and mass concrete cracking mechanisms.

Chapter 3. Specification Survey: The methodology, results, and discussion of the specification survey.

Chapter 4. Typical Midwest Boarder Bridges: An overview of the typically characteristics of Midwest boarder bridges with regard to mass concrete construction.

Chapter 5. Case Study: A case study describing the construction of two Midwest boarder bridges.

Chapter 6. ConcreteWorks Calibration: The overview, input development, results, and discussion of the calibration of ConcreteWorks for typical Midwest boarder bridges.

Chapter 7. Sensitivity Study: The overview, input development, results, and discussion of the sensitivity study.

Chapter 8. Conclusions: A discussion of the results and the findings of the research.

Chapter 9. Recommendations: A discussion of how the results may be applied in practice and recommendations for future research.

Several appendices are included to provide additional information related to the thesis. APPENDIX A provides typical examples of the installation and layout of thermal sensors. APPENDIX B and APPENDIX C provide the thermal results of each individual placement for the WB I-80 over the Missouri River Bridge case study, and US 34 over the Missouri River Bridge case study respectively.

CHAPTER 2 LITERATURE REVIEW

2.1 INTODUCTION

The thermal development of mass concrete placements is largely dictated by conditions that are within the control of the design and contactor. An understanding of the mix proportion, environmental, and construction parameters that affect the development of mass concrete is crucial to provide a sustainable mass concrete structure.

The first section provides a brief history of mass concrete. The second section discusses the hydration of cement and how it affects mass concrete development. The third section of this chapter identifies and describes the aspects that greatly affect the thermal development and cracking potential of mass concrete. The third section of this chapter describes mass concrete generalities and failure mechanisms.

2.2 HISTORICAL DEVELOPMENT OF MASS CONCRETE

Before the twentieth century, mass concrete was of little concern due to the relatively small size of buildings and dams constructed at the time. Mass concrete also posed a minimal threat due to the rate at which concrete could be produced and placed. Before the introduction of mechanical mixing and placing devices, most of the work was conducted by hand. The reduced production and placement rate did not allow for the large continuous pours of today, reducing the threat of thermal damage. At this time, there were also limited standards for concrete ingredients, as well as a limited understanding of how a mix proportion affects the strength of concrete.

During the early part of the twentieth century, a need for large damns was developed for use in power production, irrigation, and water supply. Through most of the early twentieth century, only simple tools were used to place and mix the concrete. During this time there was still limited knowledge of the affect concrete mix proportion.

The construction of the Hoover dam began in the beginning of the 1930's, setting a new precedent for size, which required additional investigations into mass concrete construction and concrete mix proportion. The investigations generated substantial gains in the mass concrete knowledge pool, including the use of cooling pipes, low heat cements, curing temperature, and many others (ACI 207 1995).

From 1930-1970, large steps were made in the construction of mass concrete placements. At this time advanced, mixing and placing tools were implemented including cranes with large buckets and cableways. This time also brought with it more understanding of the effect of mix proportion, which provided more consistent concrete, resulting from more accurately measured ingredients. This era also introduced the use of chemical admixtures and supplementary cementitious materials (ACI 207 1995).

Since the 1970's, mass concrete construction has remained relatively unchanged. However, there has been an increased use of mix proportions with low cement contents, due to a better understanding of cement strength. Additionally, thermal monitoring of mass concrete placements has become relatively main stream in recent years to assure the durability of structures.

2.3 HYDRATION OF PORTLAND CEMENT

The hydration of Portland cement is the chemical reaction between cement and water that transforms concrete from a plastic to solid state. The heat of hydration developed from the chemical reaction and the rate at which it is generated is largely responsible for the success or failure of a mass concrete element. This section presents the process of cement hydration and how it influences mass concrete.

The process of cement hydration is typically described by five different stages, which are defined by the chemical reaction. Figure 2.1 shows the five different stages of cement hydration (mixing, dormancy, hardening, cooling, and densification) along with the amount of heat generated in each stage.

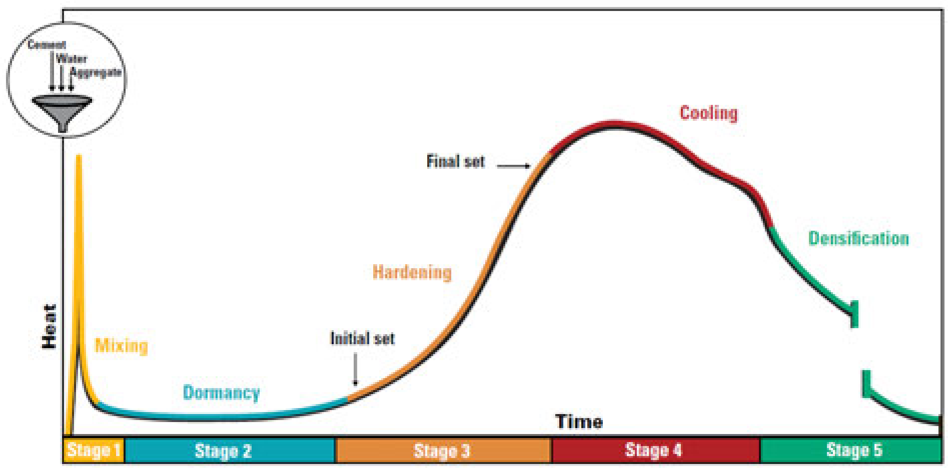


Figure 2.1 Heat generation of the stages of cement hydration (Taylor, Kosmatka, and Voigt 2007)

Cement is composed of three main compound groups; silicates, aluminates, and sulfates. The group silicate includes two compounds, alite (C_3S) and belite (C_2S). The two aluminate compounds found in cement are tricalcium aluminate (C_3A) and ferrite (C_4AF). Sulfate compounds ($C\underline{S}$) found in cement include dehydrate (gypsum), hemihydrate (plaster) and anhydrite.

Concrete has four major compounds that result from hydration reactions; calcium hydrate (C-S-H), calcium hydroxide (CH), ettringite (C-A-$\underline{S}$-H), and monosulfate (C-A-$\underline{S}$-H). The shorthand notation for the compounds is as follows:

$A=Al_2O_3$
$C = CaO$
$F=Fe_2O_3$
$H=H_2O$
$M=MgO$
$S=SiO_2$
$\underline{S}=SO_3$

The first stage of cement hydration is mixing, in which the process of hydration is initiated by the contact of the cement and water. The process begins with the rapid reaction of aluminate (C_3A) and water, which generates very large amounts of heat. This reaction is slowed by the formation of (C-A-$\underline{S}$-H), a reaction of aluminate (C_3A) and sulfate (C$\underline{S}$) that prevents water from coming into contact with the aluminate. Stage one defines the first few minutes of the hydration process.

Stage two is the dominate period, where the reactions are slowed and minimum heat is generated in the concrete. Dormancy lasts for a few hours and allows for proper construction of the concrete element. During stage two, the (C-A-$\underline{S}$-H) continues to develop and the silicates, alite (C_3S) and belite (C_2S), slowly react with the water to form calcium and hydroxyl ions.

Stage three is defined as the concrete hardening stage. The hardening of the concrete is the result of the formation of the calcium silicate hydrate (C-S-H) and calcium hydroxide (C-H). Calcium silicate hydrate and calcium hydroxide are the result of the calcium ions along with the alite and belite that were developed in the dormant stage reacting with water as shown by Equation 2.1 and Equation 2.2. Additionally, as the process of hardening begins to accelerate, the formation of ettringite (C-A-S-H) increases as shown by equation 2.3. The reactions of stage three are responsible for the vast majority of the overall heat of hydration generated in the concrete. The heat generated during stage three has the largest impact on mass concrete.

$$\text{(Alite) } 2C3S + 6H \rightarrow C\text{-}S\text{-}H + 3CH \qquad 2.1$$

$$\text{(Belite) } 2C2S + 4H \rightarrow C\text{-}S\text{-}H + CH \qquad 2.2$$

$$C3A + 3CS + 26H \rightarrow C3A\bullet 3CS\bullet H32 \text{ (Ettringite)} \qquad 2.3$$

The possibility of thermal cracking of mass concrete begins with stage three and continues through stage four. During stage four, the concrete develops both compressive and tensile strength. As the concrete begins to cool, the concrete begins to experience tensile stress. If the tensile stress surpasses the developed tensile strength, the concrete may experience cracking.

The fourth stage, cooling, is the result of silicate hydrate and calcium hydroxide building up in the concrete. The buildup of silicate hydrate and calcium hydroxide limits the contact between cement particles and the remaining water, slowing down the reactions. During stage four there may also be an increase in heat due to the formation of monosulfate (C-A-$\underline{S}$-H), resulting from the exhaustion of the sulfate in the concrete as shown by Equation 2.4.

$$2C3A + C3A\bullet 3CS\bullet H32 \rightarrow 4H\bullet 3(C3A\bullet CS\bullet H12) \text{ (Monosulfate)} \qquad 2.4$$

Stage five is the longest of all the stages, densification, in which the hydration reactions continue to slow. During this stage the concrete continues to cool and calcium silicate hydrate crystals continue to form until silicates are completely exhausted, continuing to increase the strength of the concrete. The probability of cracking remains a concern in stage five until the concrete has dissipated all of the remaining heat (Taylor, Kosmatka, and Voigt, 2007).

Figure 2.2 shows the rate of hydration of the primary compounds for type I cement. The figure shows that the silicate alite (C_3S) reacts much more rapidly in comparison to belite (C_2S). Additionally, the aluminate compound tricalcium aluminate (C_3A) hydrates more rapidly than ferrite (C_4AF).

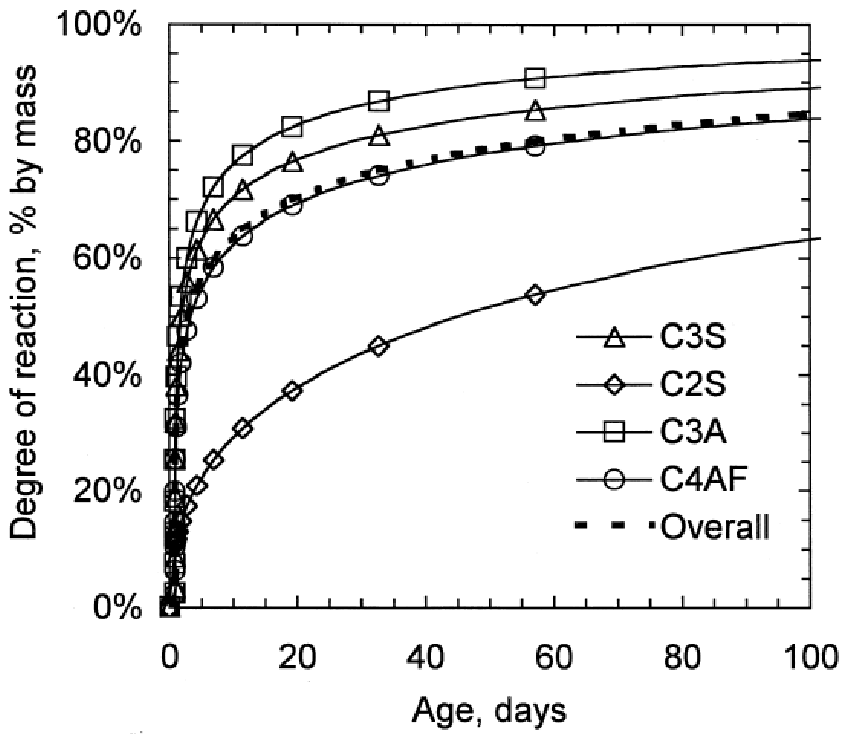


Figure 2.2 Hydration of principal compounds by % mass with age (Tennis and Jennings 2000).

2.4 CONCRETE MIX PROPORTION

The concrete mix proportion is the constituents that make up the concrete and the proportions that relate them. Constituents of the concrete mix proportion that dictate the thermal development of the mass concrete include the cement, water, aggregate, air , chemical admixture, and supplementary cementitious material content. The following describes how each mix element affects the properties of the concrete.

2.4.1 Cement Content

Cement content is the weight of cement per unit volume of concrete. Cement content has a large influence on the strength of the concrete and the amount of heat generated in the concrete. As more cement is added to a concrete mix, more heat of hydration is developed as a result of the additional chemical reactions. It is recommended that the minimum amount of cement content is used in the concrete mix to achieve the minimum strength requirements.

2.4.2 Fly Ash

Fly ash is classified as a pozzolans, which are non-cementitious materials. If pozzolans are finely ground and come into contact with both moisture and calcium hydroxide, the reaction will generate compounds similar to that of cementitious materials. Generally, there are two different types of fly ash applicable to mass concrete construction; class C and class F fly ash. Both class C and F fly are supplementary cementitious materials produced as a byproduct of energy production, generally coal burning power plants. The distinguishing characteristic between class C and F fly ash is the calcium content (CaO). Class C fly ash generally has higher calcium contents (10-30% CaO) while class F fly ash generally has lower calcium contents (less than 10% CaO). Class F fly ash is more suitable to environments where sulfate exposer is a concern (Kosmatka, Kerkhoff, and Panarese 2002).

Both class C and class F fly ash produce concrete with delayed setting times and higher ultimate strength at the completion of hydration. Both fly ash types may be used as substitution to cement to reduce the heat generated by hydration. Class F fly is generally used over class C fly ash in mass concrete applications, because class F fly ash generates less heat during hydration and is more suitable to sulfate exposer conditions. Class F fly ash commonly has cement substitution percentages of 15-50, while class C commonly has substitution percentages of 15-40 (Kosmatka, Kerkhoff, and Panarese 2002).

2.4.3 Ground Granulated Blast Furnace Slag

Ground granulated blast furnace slag, or GGBFS, is a byproduct of iron blast furnaces. Slag interacts with compounds found in cementitious materials and produces compounds similar to that of cementitious materials. Concrete mix proportions that include slag have reduced heat generation rates and generate less heat overall. Suitable substitution percentages for GGBFS in mass concrete applications range up to 80% of the total cementitious materials (Gajida and Vangeem 2002).

2.4.4 Aggregate

The aggregate gradation and proportions play a large role in the strength of concrete. Generally, the larger the coarse aggregate size a concrete mix utilizes the higher the strength. Similarly, increasing the course aggregate content also produces a higher strength. Larger aggregate size and higher contents of course aggregate reduce the void space in the concrete, allowing the aggregate to carry the load more efficiently, increasing the strength. As a result of the increase in concrete strength, less cement may be required. Reducing the amount of cement in the concrete will reduce the heat developed in the concrete.

2.4.5 Cement Types

There are five different types of cements as defined by ASTM C150; Type I (normal), Type II (moderate sulfate resistance), Type III (high early strength), Type IV (low heat of hydration), and Type V (high sulfate resistance). Type I and II cements are most commonly used in mass concrete construction due to relative low cost and availability. Figure 2.3 shows how the cement type affects the adiabatic temperature rise in mass concrete (ASTM 1999).

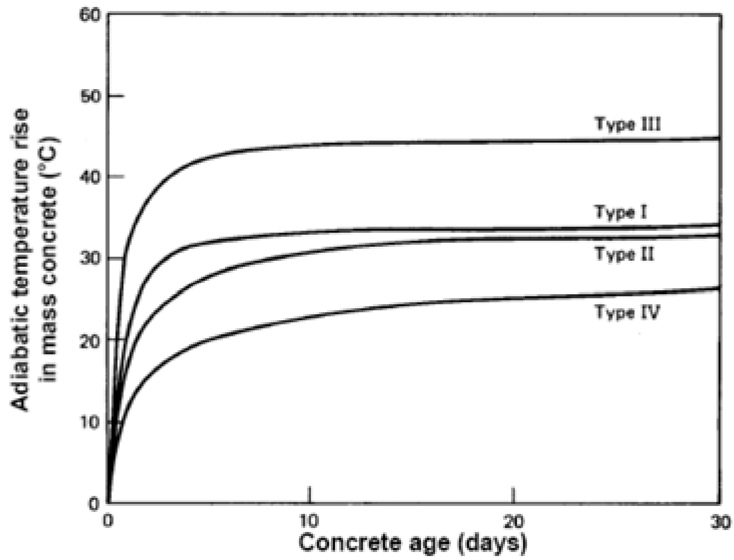


Figure 2.3 Adiabatic temperature rise in mass concrete for cement types (Mindess and Young 1981)

Type I cement is more suited for mass concrete application when it is blended with a supplementary cementitious material, such as slag or fly ash. Such examples include type IP and IPM, which include pozzolan in the cement blend. The percent substitution of pozzolan for IP and IPM cements is 15-40% and up to 15% by weight respectively. Additionally, type IS and ISM contain 25-70% and up to 25% slag substitution by weight respectively. These blended cements are capable of reducing the heat of hydration compared to the typical type I cement (ACI 2006).

Type II cement is also a common cement type used in mass concrete construction because it generates a low heat of hydration. Type II cements contribute to reduced heat of hydration because of the limits placed on the compounds (C_3A) and (C_3S) which, greatly contribute to the heat of hydration of the concrete.

Type III cements are generally not used in mass concrete applications due to an increased heat of hydration. Type IV and V cements are not commonly used because of the increased cost and lack of availability. Table 2.1 shows the chemical and the potential phase composition for cement types I, II, III, and V.

Table 2.1 Chemical and compound composition of cement types by % mass (Bhatty and Tennis 2008)

Cement Type	Value	Chemical Composition						Potential Phase Composition			
		SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	C_3S	C_2S	C_3A	C_4AF
Type I	Mean	20.17	5.07	2.66	63.23	2.51	3.26	56.9	14.8	8.9	8.2
	Maximum	21.8	6.1	3.6	65.2	4.5	4.4	65	21	12	11
	Minimum	19.0	3.9	2.0	61.5	0.8	2.0	45	6	6	6
Type II	Mean	20.85	4.62	3.32	63.66	1.98	2.91	56.5	17.1	6.7	10.1
	Maximum	22.5	5.5	4.4	65.6	4.5	4.0	68	25	8	13
	Minimum	20.0	3.8	2.6	61.3	0.6	2.1	48	8	4	8
Type III	Mean	20.38	4.84	2.86	63.33	2.21	3.60	56.2	16.2	7.8	8.8
	Maximum	22.1	7.3	4.2	64.9	4.3	4.9	66	27	12	13
	Minimum	18.6	3.4	1.3	61.6	0.8	2.6	48	8	2	4
Type V	Mean	21.61	3.80	3.87	63.85	2.18	2.34	57.7	18.4	3.5	11.8
	Maximum	22.8	4.8	5.8	65.2	4.5	2.8	64	27	5	18
	Minimum	20.3	3.3	3.2	62.3	0.8	2.0	47	12	0	10

2.5 ENVIRONMENTAL CONDITIONS

Environmental conditions play a large role in the thermal development of mass concrete, both directly and indirectly. Examples of the direct affect include the selection of formwork and insulation based on the environmental conditions. Construction of placements in cooler environments requires additional formwork compared to those in warmer climates to reduce the maximum temperature difference.

The ambient air temperature indirectly affects the thermal development of mass concrete through the fresh placement temperature of the concrete. The fresh placement temperature of concrete is largely affected by the environmental conditions that the concrete materials are exposed to prior to mixing.

2.5.1 Ambient Air Temperature

The ambient air temperature of a placement is a dominating aspect of the concrete boundary conditions. The rate of heat flow for thermal conduction, Fourier's law, is shown by Equation 2.1.

$$kA\frac{T}{d} \qquad 2.5$$

Where,

Q: the rate of heat flow through the material

k: the thermal conductivity of the material

A: the cross sectional area perpendicular to the heat flow

ΔT: the temperature difference across the section

d: the distance of the heat flow

As shown by Fourier's law, the rate of flow through the material is a function of the temperature difference across the section. The temperature difference is the difference of maximum temperature at the concrete core and the temperature at the surface of the concrete. The temperature at the surface is dependent on many variables including the ambient air temperature.

As the ambient air temperature increases, the surface temperature of the concrete increases and the rate of the heat flow is reduced. The rate of heat flow corresponds to the thermal gradient of the concrete. Similarly, if the ambient air temperature is decreased the rate of heat flow will increase, increasing the thermal gradient in the concrete.

2.6 CONSTRUCTION PARAMETERS

The utilization of the proper construction practices for mass concrete construction provides substantial benefits to the thermal development and cracking potential. Possible construction parameters that are within control of the contractors or designers include fresh placement temperature, formwork type, cooling pipe utilization, dimensional size of the placement, curing method, formwork removal time, insulation type, and subbase material.

2.6.1 Fresh Placement Temperature

Fresh placement temperature is the initial temperature of the concrete when it is placed in the formwork. Lowering the fresh placement temperature of the concrete at the time of placement is one of the most effective ways to reduce the heat development in the concrete and likelihood of cracking. ACI 207.2R-95 recommends the maximum fresh placement temperature be limited to the average summer temperature for the location and less than 100°F (ACI 207 1995).

The rate of hydration reactions as the concrete begins to harden is dependent upon the temperature of the concrete. As the temperature of the concrete increases, the hydration reactions are accelerated, increasing the rate of heat generation, developing a higher peak temperature in the concrete. Figure 2.4 shows the effect of fresh placement temperature and time with adiabatic temperature rise of mass concrete. Reducing the fresh placement temperature of a concrete placement is often an applicable method to reduce the likelihood of cracking in mass concrete.

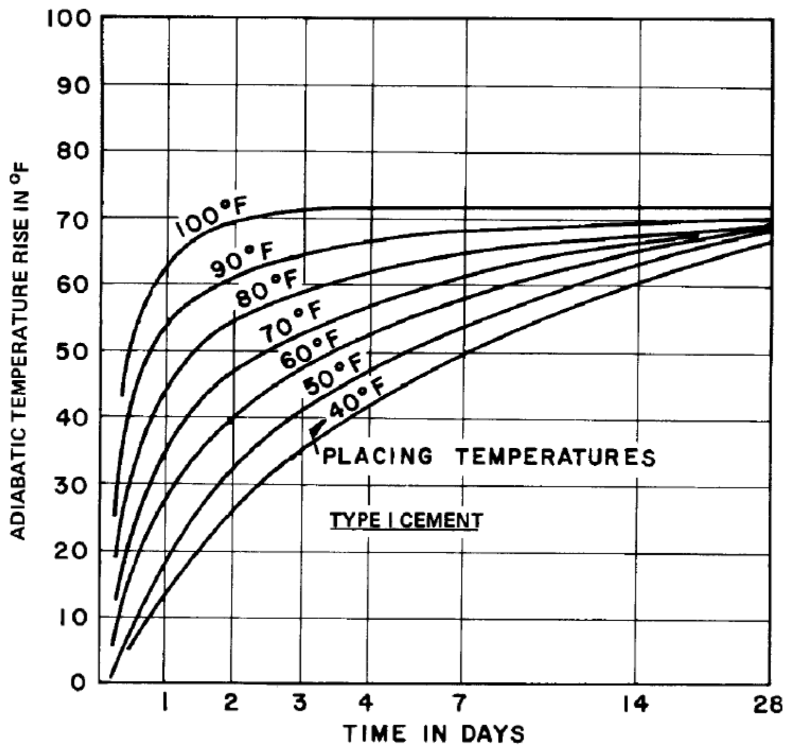


Figure 2.4 Adiabatic temperature rise of mass concrete for 376 lb/yd^3 of type I cement with fresh placement temperature and time (ACI 207 1995)

2.6.1.1 Precooling

Precooling is the process of cooling the concrete before it is placed in the formwork, reducing the fresh placement temperature. There are various methods to precool concrete with varying effectiveness and cost. Methods include:

- Using cooled water or substitute water with a portion of ice
- Shading aggregate stockpiles
- Spraying of aggregate stockpiles for cooling by evaporation
- Nitrogen cooling of concrete
- Construct placements when the ambient temperatures are reduced (nights, early mornings, or cooler season)

2.6.2 Formwork

The material that is used to form the concrete placement plays a large role in the thermal development of the placement. Generally, there are three types of formwork used in mass concrete construction; wood, steel, and soil. The effect of the formwork material is dependent on the R value of the material. The R value is a unit of measurement to describe the thermal resistance or insulating value of a material. The United States customary unit for R value is $h \cdot ft^2 \cdot °F/Btu$.

Steel formwork provides minimal thermal insulation beyond providing an outside air foil, because of the relatively negligible thermal resistance of steel. Wood formwork does supply thermal insulation, with typical plywood having an R value 1.25 (typical formwork plywood is ¾ inch and would provide a total R value of 0.94), compared to extruded polystyrene, which has an R value of 4.0 (Hurd 2005).

Soil forming of mass concrete is an applicable and economical alternative to forming concrete in certain applications. Soil is capable of providing substantial thermal resistance to the placement as a result of the relatively high thermal resistance, and large thickness of the soil surrounding the placement. Soil forming is common in footings that are relatively shallow compared to the respected width and length. Additionally, large drilled shafts may be considered soil formed mass concrete. Soil has the capacity to act as an advantageous insulating material depending on the type of soil, depth of the soil, moisture content, and soil temperature.

2.6.3 Cooling Pipes

The use of cooling pipes is a postcooling application used to reduce the peak temperature and minimize thermal gradients in the concrete. Additionally, cooling pipes may be used in accelerated construction to reduce the formwork cycle time. Cooling pipes utilize a cool liquid, generally water, to remove excess heat by circulating the liquid throughout the placement by means of piping. The piping material is generally, PVC (polyvinyl chloride), PEX (cross-linked polyethylene), aluminum, or steel. Sources of cool water include, nearby rivers or streams, local wells, municipal water supplies, or supplied by trucks from nearby sources. Generally the water is circulated through the placement by means of a diesel, gas, or electric water pump. Figure 2.5 shows an example of a PVC cooling pipe system used on a mass concrete footing.

Figure 2.5 PVC post cooling system

The correct cooling pipe system design is crucial to properly reduce the peak temperature and thermal stress. The development of a cooling pipe system includes the design of the following parameters: pipe material, cooling liquid temperature, piping spacing, cooling liquid flow rate, pipe diameter, and the pipe loop length. If the system is not correctly designed, localized stressed may develop around the pipes.

2.6.4 Dimensional Size

The dimensional size of a mass concrete placement is a term used to describe all of the dimensions of the placement. The least dimension of a placement, generally the depth, is the dimension most commonly used to define mass concrete. The least dimension is an important parameter when designing mass concrete placements, as conduction is a function of the distance from core of the placement to the surface. Furthermore, width, length, surface area, and volume also contribute to the thermal development of mass concrete elements.

2.6.5 Curing Methods

Common practices for the curing of mass concrete include curing compounds, plastic films, and wet curing. The preferred method of curing mass concrete is wet curing. Wet curing of concrete assures complete hydration of the concrete at the surface. The process of wet curing also helps to insulate the concrete. Wet curing is typically only applicable to the top surface of the placement due the difficult of installing and maintaining moisture on vertical and bottom surfaces.

Curing compounds provides little benefit to mass concrete beyond providing proper strength and durability. Plastic film curing provides benefits to mass concrete by helping to prevent moisture loss from the concrete, as well as providing thermal insulation to the concrete. Plastic film is the most common curing method because of the effectiveness and relative ease of installation. Leaving formwork on is one way to cure concrete, and is also the most practical in many applications. However, formwork provides minimal benefits with respect to curing, and may not help to prevent thermal cracking.

2.6.6 Form Removal Time

The form removal time is the time when the formwork is removed from the placement. In many applications, the form removal time correlates to the insulation removal time, as the majority of insulation is either attached to the formwork or wraps the formwork, and for constructability reasons is generally removed at the same time. In applications that require accelerated construction, formwork may be removed to be recycled, and the insulation may be reapplied.

When the formwork is removed, the thermal insulated provided to the concrete surface is subsequently removed. Following the removal of the thermal insulation, the surface will cool to the ambient temperature. To ensure the concrete will not crack, the formwork should remain on the placement until the difference between the ambient temperature and the concrete core temperature is small enough to prevent cracking.

2.6.7 Insulation

In mass concrete construction, the majority of placements utilize insulation to reduce the thermal gradients within the placement. There are typically two types of insulation utilized in mass concrete construction, rigid foam insulation (extruded polystyrene) or insulation blankets. Rigid foam insulation is typically used on the interior of the formwork, and insulation blankets wrap the exterior of the formwork as shown by Figure 2.6. The R value of the insulation used in mass concrete construction range from 2.5 - 10, depending on the conditions.

Figure 2.6 Insulation being installed on the outside of the formwork of a mass concrete placement

2.6.7 Subbase

The subbase of a mass concrete element is the material that the concrete is cast on. Mass concrete is generally placed on three distinct subbase materials, soil, stone, and concrete. The subbase material affects the development of the placement through conduction and restraint. Different subbase materials transfer heat at different rates, contributing to the dissipation of heat from the placement, and the subsequent thermal gradients. Additionally, different subbase materials provide varying levels of restraint for the concrete affecting the likelihood of cracking.

2.7 MASS CONCRETE

The fundamental difference between mass concrete and general concrete is the thermal characteristics. Mass concrete elements generate substantial thermal gradients between the core and the surface of the concrete, which are large enough to pose a considerable risk of thermal damage. Therefore, they are declared to be mass concrete and extra precautions are taken. Three circumstances contribute to cracking and reduce the durability of mass concrete elements; internal restraint, external restraint, and delayed ettringite formation. Proper understanding and design of mass concrete provides elements free of cracks and thermal damage.

2.7.1 Definition

ACI 207 defines mass concrete "as any volume of concrete with dimensions large enough to require that measures be taken to cope with the generation of heat from hydration of the cement and attendant volume change to minimize cracking" (ACI 207 2006).

Mass concrete is often defined by the least dimension of the placement, however a single least dimension definition does not cover all applications. Each placement has varying concrete mixes, environmental conditions, and construction conditions that affect the concrete thermal development, making a standard least dimension definition impractical. Accordingly, the definition of mass concrete varies widely between agencies and applications.

2.8 RESTRAINT AND THERMAL STRESS

Cracking in mass concrete is the result of restraint, which induces tensile stresses that exceed the relatively low tensile strength of the concrete. All mass concrete is restrained both internally by the element itself, and externally by the support system of the element.

2.8.1 Internal Restraint

When mass concrete is placed, the core of the concrete experiences large temperature increases due to the heat of hydration, and the inability of concrete to efficiently transfer heat to the surrounding environment. The increase in temperature causes the core of the concrete to expand due to thermal expansion. Due to the proximity to the surrounding environment, the surface of the concrete cools more rapidly compared to the core, causing the surface of the placement to contract relative to the core, due to thermal expansion. The respective volume changes in the concrete causes compressive forces to develop in the core, and tension forces to develop at the surface as shown by Figure 2.7. If the tensile stress in the concrete exceeds the developed tensile strength of the concrete, the concrete will experience thermal cracking.

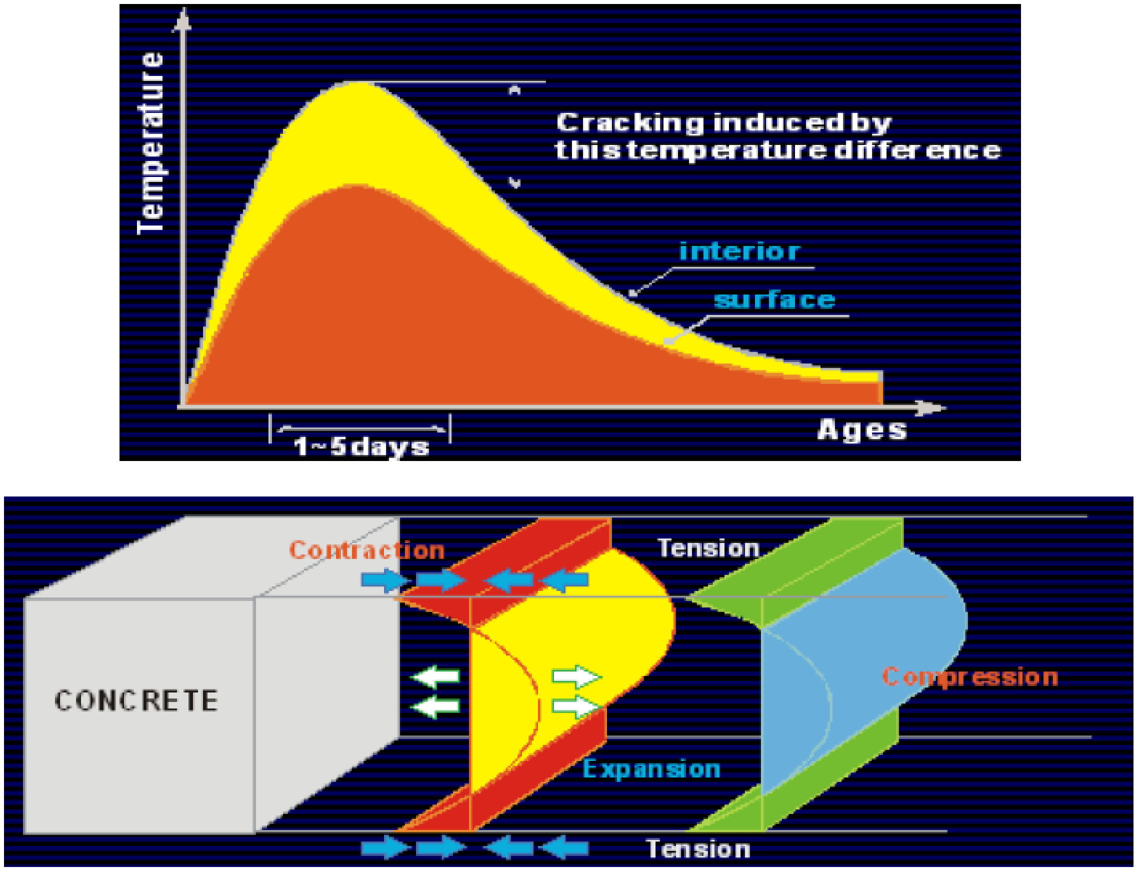


Figure 2.7 Internal restraint mechanism due to thermal gradients (Kim 2010)

2.8.2 External Restraint

External restraint is the result of the mass concrete support structure. After the concrete has reached its peak temperature the placement begins to cool, and subsequently contracts in volume. The contraction of the concrete is resisted by external restraints, such as the subbase, rigid support structure, or adjoining structure supporting the mass concrete element. Figure 2.8 shows how the volumetric changes of mass concrete are resisted by external restraint. Figure 2.9 shows a steel pile, which is an example of typical external restraint for mass concrete footings. The contracting volume of concrete will develop tensile stresses resulting from the resistance provided by the external restraint. If the tensile stresses exceed the developed tensile strength of the concrete, the placement will experience cracking.

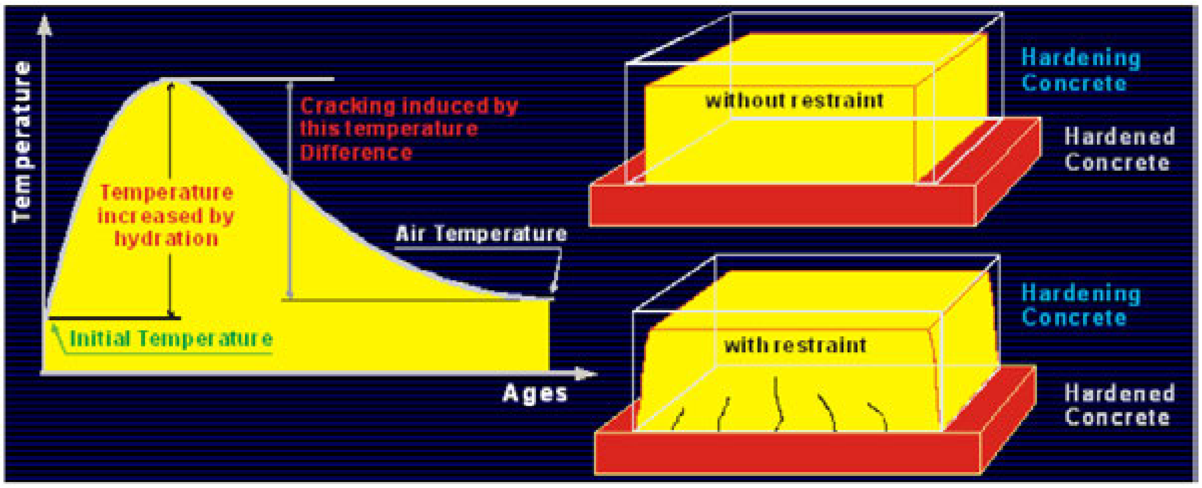


Figure 2.8 External restraint mechanism due to thermal gradients (Kim 2010)

Figure 2.9 Steel pile providing external restraint to a mass concrete footing

The tensile stress resulting from an external restraint depends on the degree of restraint. The degree of restraint depends on the relative dimensions, strength, and modulus of elasticity of the restraining material. ACI 207.2R defines the equation for the developed tensile strength at the centerline of the placement by Equation 2.6.

$$f_t = K_R \Delta_c E_c \tag{2.6}$$

Where:

f_t : tensile stress at any point on the centerline of the placement

K_R : degree of restraint expressed as a percentage defined by Figure 2.10

Δ_c : contraction of the concrete if there was no restraint

E_c : modulus of elasticity during the occurrence

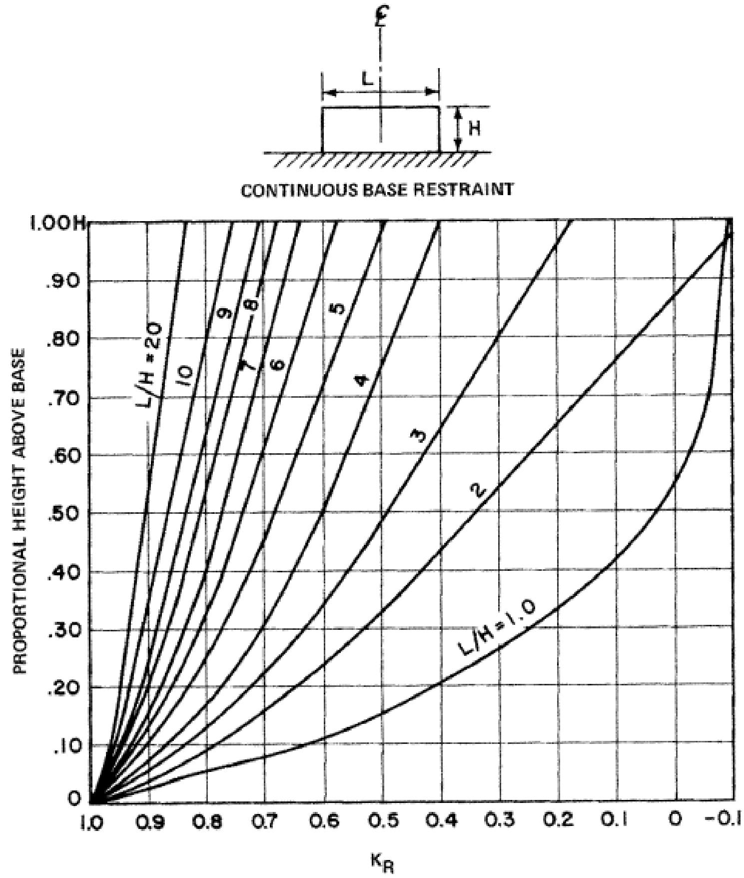


Figure 2.10 Degree of tensile strength at center section (ACI 207 1995)

ACI 207.2R-95 also states that the K_R needs to be adjusted to account for the stiffness of the restraint. Equation 2.7 defines the correct factor for the restraint stiffness.

$$\text{Correction Factor} = 1/[1 + A_g E_c/(A_F E_F)] \qquad 2.7$$

Where:

A_g : gross cross sectional area of the concrete

A_F : area of the plane surface of contact of the restraint

E_F : modulus of elasticity of the restraining element

2.8 DELAYED ETTRINGITE FORMATION

Delayed ettringite formation (DEF), also known as heat induced delayed expansion (HIDE), is the process in which ettringite forms in matured concrete causing expansive pressures. The exact cause of DEF is not fully known. It is understood that only certain concrete mixes are susceptible to delayed ettringite formation when they reach an extreme temperature. It has been shown that the use of fly ash and slag may help to reduce delayed ettringite formation. To prevent DEF specifications typically limit the maximum temperature of concrete to 160°F.

Mass concrete elements are capable of generating extreme temperatures during hydration. If the temperature of the concrete becomes excessive, ettringite that was previously formed in the concrete may begin to decompose, and further ettringite formation is stopped. This is the result of the constriction of the sulfate and aluminates in the calcium hydrate (C-S-H), preventing the formation of ettringite (C-A-$\underline{\text{S}}$-H). After the concrete has hardened, the calcium hydrate releases the confined sulfate, which may react with calcium monosulfoaluminate in the presence of water and form ettringite in the concrete paste. After a period of time, the accumulation of ettringite crystals in the concrete paste may build up and cause expansive pressures. If the pressures due to the expansive crystals become extreme, cracking between the aggregate and the paste may develop, as shown by Figure 2.11, reducing the durability of the concrete (Kosmatka, Kerkhoff, and Panarese 2002).

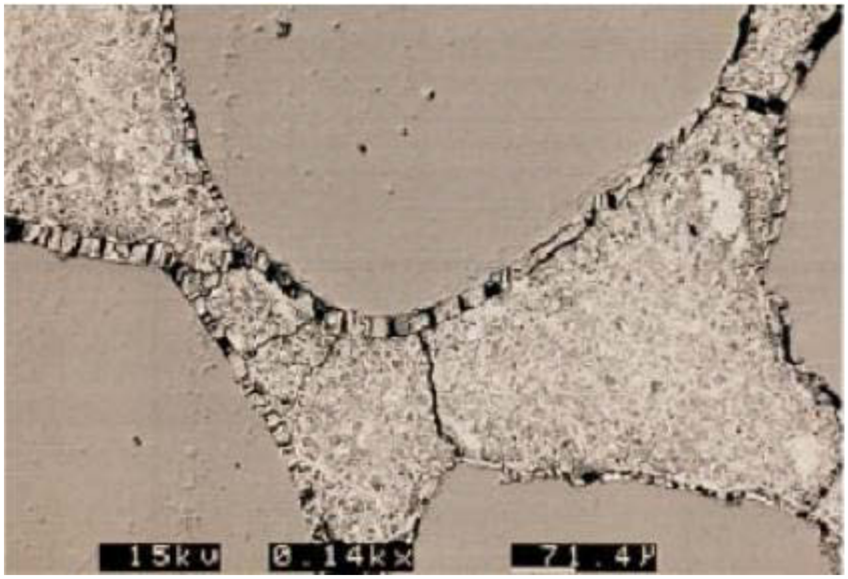


Figure 2.11 Delayed ettringite cracking between cement paste and aggregate (Kosmatka, Kerkhoff, and Panarese 2002)

2.9 THERMAL MONITORING

The thermal development of mass concrete is generally monitored and recorded to verify that specification requirements have been satisfied. Additionally, thermal monitoring is utilized to identify potential thermal issues, so that changes may be made to the placement conditions to prevent thermal damage. Temperature sensors are often used to monitor the maximum temperature, which correlates to DEF, and the minimum temperatures, to identify thermal gradients at different location in the placement.

Currently, there are many different thermal sensors available for mass concrete applications. Typically, sensors are applied in pairs to provide redundancy in case of sensor damage during construction. Additionally, sensors need to be placed in locations that provide protection from the worker, concrete consolidation tools, and the concrete as it is being placed. Figure 2.12 shows a thermal sensor installed on the rebar cage of a mass concrete footing.

Figure 2.12 Installed thermal monitoring sensor

2.10 SHRINKAGE

Shrinkage is an unavoidable effect of the hydration of concrete. As concrete shrinks, stresses are developed resulting from internal and external restraint. There are three general types of concrete shrinkage; drying shrinkage, chemical shrinkage, and autogenous shrinkage.

2.10.1 Drying Shrinkage

Drying shrinkage is the contraction of concrete due to a loss of moisture. When concrete is cured in an environment with minimal moisture, the concrete will experience drying shrinkage as the moister is transferred from the concrete to the surrounding environment. If a concrete element does not have interior or external restraint, and has uniform moisture loss, the element will uniformly contract with no stress development. Mass concrete generally has substantial internal and external restraint, causing stresses to develop as a result of drying shrinkage. Additionally, moisture contents in mass concrete placements vary throughout the placement. Generally, the surface of the element will have a reduced moisture content causing more shrinkage at the surface compared to the interior.

Drying shrinkage may produce additional tensile stresses at the surface of mass concrete placements increasing the likelihood of cracking. To combat drying shrinkage, concrete may be cured in an environment with excess moisture. Curing mass concrete with a plastic film wrap, or wet curing blanket provides sufficient moisture to reduce drying shrinkage. Figure 2.13 shows the effect of shrinking or swelling for different curing regimes. If concrete is cured with sufficient moisture, the placement may actually swell to a small degree, resulting from gain in moisture. The swelling of the concrete causes compressive stresses at the surface of the concrete, reducing the likelihood of cracking. If possible, mass concrete should be wet or moist curing to combat drying shrinkage (AÏtcin 1999).

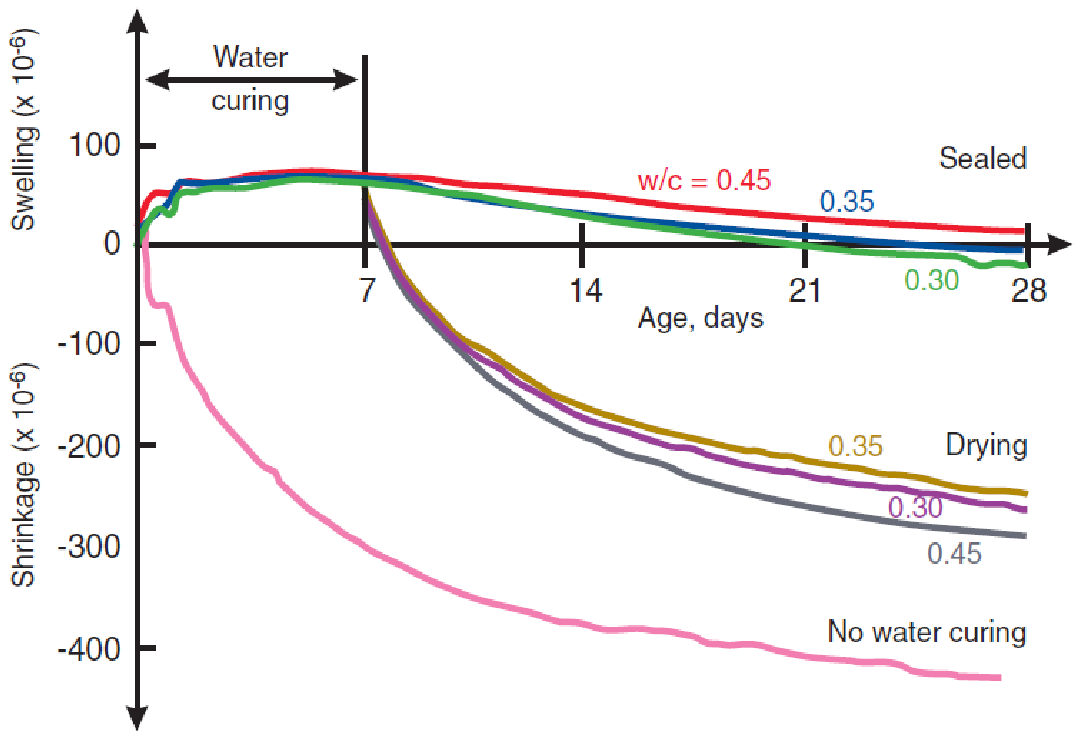


Figure 2.13 Length change of concrete due to different curing methods (AÏtcin 1999)

2.10.2 Chemical Shrinkage

Chemical shrinkage is a reduction in the absolute volume of concrete resulting from the hydration of cement. During hydration, cement and water chemically react to produce a product that has a decreased absolute volume compared to the absolute volume of the cement and water prior to hydration. When the concrete begins to set and harden, the volume change is resisted by the concrete, causing stress and voids to develop in the concrete. Stresses in the concrete provided by chemical shrinkage are developed during all stages of hydration and may contribute to the cracking of mass concrete (Kosmatka et al, 2002)

2.10.3 Autogenous Shrinkage

Autogenous shrinkage is the visible isotropic shrinkage resulting from chemical shrinkage. Autogenous shrinkage results from a lack of water in the concrete, and is generally believed to be caused by capillary depression resulting from chemical shrinkage (AÏtcin 1999). In typical concrete mixes, autogenous shrinkage contributes very little to the overall shrinkage of the concrete. In mixes with lower water to cement ratios, autogenous shrinkage may contribute substantially to the overall shrinkage of the concrete. Autogenous shrinkage may cause additional stresses to develop in the concrete, increasing the likelihood of cracking. To reduce the effects of autogenous shrinkage, it is recommended to wet cure mass concrete placements to assure the concrete has ample moisture. Figure 2.14 shows the volume changes resulting from autogenous and chemical shrinkage.

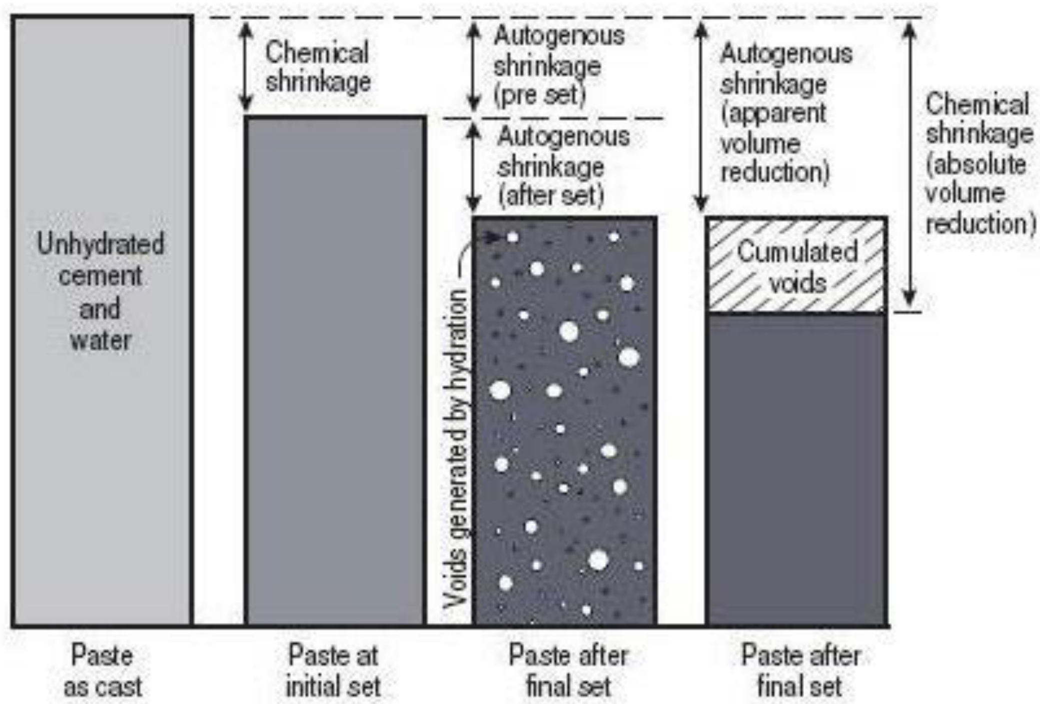


Figure 2.14 Autogenous and chemical shrinkage of concrete from paste to final set

2.11 CREEP

Creep is the permanent deformation of a material that increases with time, even though the stress or load is constant, as shown by Figure 2.15. Not to be confused with elastic deformation where the element returns to the original length when the load is removed or plastic deformation where the deformation occurs immediately after the stress is applied. Creep is dependent on the modulus of elasticity of the concrete, the magnitude of the stress, and length of time the stresses are applied.

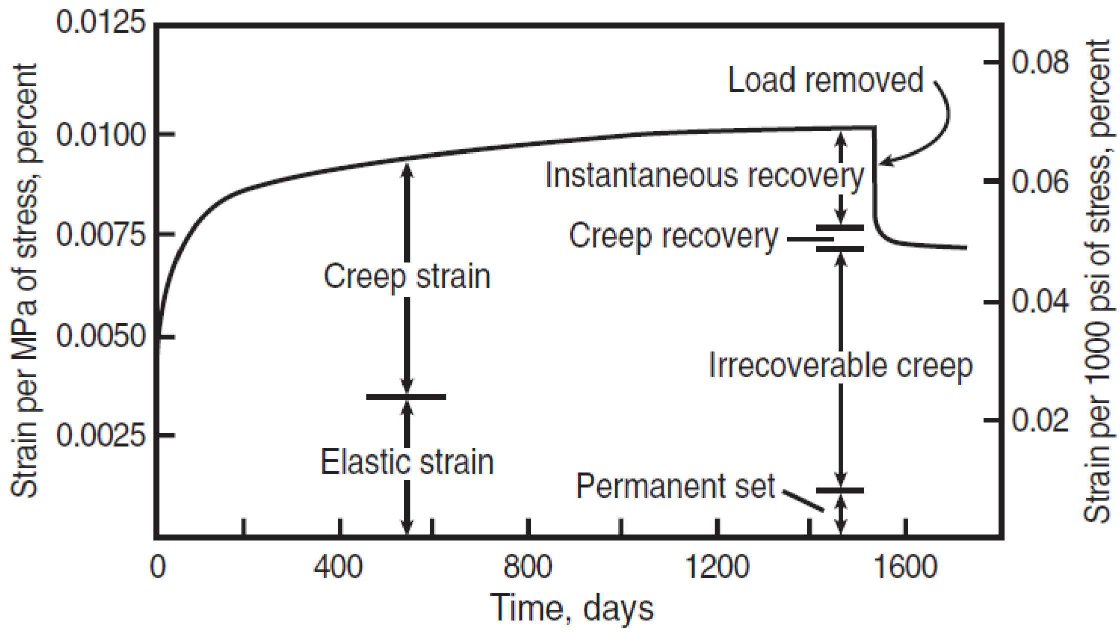


Figure 2.15 Comparison of elastic and creep strains over time

Creep has the capability to reduce the tensile strains and stresses resulting from drying shrinkage, chemical shrinkage, autogenous shrinkage, internal restraint, and external restraint at early ages (Altoubat 2001). The effectiveness of creep to reduce the strains and stresses in a mass concrete element depends on the period of time over which the element is loaded. Since creep is time dependent, the more gradually the load is applied the more affect creep will have. Creep provides minimal relief to stresses and strains resulting from temperature gradients, as such stresses are applied rapidly (ACI 1995).

2.12 CRACKING

All mass concrete cracks due to thermal gradients, drying shrinkage, autogenous shrinkage, and loads. The objective for mass concrete elements is to minimize crack size to ensure the durability and esthetics of the element. If large cracks develop at the concrete surface, water and sulfates may permeate into the concrete causing mechanical and chemical degradation. ACI 207 recommends a maximum crack width of 0.009 inches to assure sufficient durability. ACI 224R suggests the following equation for estimating the crack width based on the tension stress in the steel:

$$w = 0.10\, f_s \sqrt[3]{d_c A}\, 10^{-3} \tag{2.8}$$

Where:

w: maximum crack width at the surface (in)

d_c: cover to the center of the bar (in)

A: average effective concrete area around a reinforcing bar (2*dc* x spacing) (in^2)

f_s: calculated steel stress (ksi)

2.12.1 Crack Repair

Cracks in mass concrete structures may be repaired if the cracks are relatively small and pose a limited risk of decreased durability. Typically, cracks resulting from thermal gradients in mass concrete placement may be injected with epoxy to seal the concrete.

Cracks resulting from DEF or HIDE are more difficult to repair, as the cracks develop gradually due to increasing stress developments over a long period of time. If elements damaged by DEF are simply repaired with epoxy, the stresses inside the concrete remain, and continue to increase with time, resulting in more cracking at a later date. To properly repair placements with DEF thermal damage, the stresses in the element must be reduced. One option to decrease the stresses in the concrete and prevent future cracking is to provide relief cuts in the placement. Repairing DEF thermal damage is a very difficult and costly process (ACI 364).

CHAPTER 3. SPECIFICATION SURVEY

3.1 INTRODUCTION

Mass concrete specification requirements throughout the United States vary greatly between agencies. The goal of the specification survey is to identify current trends in mass concrete requirements in the United States. Aspects of the mass concrete specification that will be surveyed include the definition of mass concrete, concrete mix portion requirements, thermal control requirements, construction requirements, design requirements, and additional special requirements.

The first section of this chapter describes the methodology that was used to complete the specification survey. The second section of the chapter describes the results of the survey. The final section of the chapter is a discussion of the sensitivity survey results.

3.2 METHODOLOGY

The specification survey was completed by investigating the mass concrete specification of the 51 state highway agencies, including the District of Columbia and two federal agencies. The first stage of the survey involved searching the internet for current standard specifications and additional special provisions of the state agencies in an effort to independently identify specifications. Following the initial internet search, state highway agencies that did not appear to have a mass concrete specification were contacted by telephone in a further effort to determine if the agency has a supplemental or developmental mass concrete specification that was not posted on the internet.

If an agency is listed as not having an identified specification, it does not mean the agency does not have a specification, rather that a specification was not identified in the search process. If a specification was not identified it means either the agency did not respond, the agency was unable to identify the specification, or the agency did not have a specification. Furthermore, agencies with minimal mass concrete specifications were excluded from the survey for lack of scope. For an example, the standard specification identifies only that mass concrete shall use type II cement.

3.3 RESULTS

In total, thirteen different mass concrete specifications were identified including standard specifications, special provisions, special notes, developmental specifications, and structural design guidelines as shown by Table 3.1. Similarly, mass concrete specifications of forty agencies were unable to be identified. The type, reference, and year for the identified specifications are listed in Table 3.2.

Table 3.1 Agencies with and without identified mass concrete specifications

Agencies With Specification	Agencies Without Specification	
Arkansas DOT	FHWA	Missouri DOT
California DOT	NAVFAC	Montana DOT
Florida DOT	Alabama DOT	Nebraska DOT
Idaho DOT	Alaska DOT	Nevada DOT
Illinois DOT	Arizona DOT	New Hampshire DOT
Iowa DOT	Colorado DOT	New Mexico DOT
Kentucky DOT	Connecticut DOT	North Carolina DOT
New Jersey DOT	Delaware DOT	North Dakota DOT
New York DOT	District of Columbia DOT	Ohio DOT
Rhode Island DOT	Georgia DOT	Oklahoma DOT
South Carolina DOT	Hawaii DOT	Oregon DOT
Texas DOT	Indiana DOT	Pennsylvania DOT
West Virginia DOT	Kansas DOT	South Dakota DOT
	Louisiana DOT	Tennessee DOT
	Maine DOT	Utah DOT
	Maryland DOT	Vermont DOT
	Massachusetts DOT	Virginia DOT
	Michigan DOT	Washington DOT
	Minnesota DOT	Wisconsin DOT
	Mississippi DOT	Wyoming DOT

Table 3.2 State agency specification reference

Agency	Specification type	Reference
Arkansas DOT	Standard specification	AHTD 2003
California DOT	Standard specification	California DOT 2010
Florida DOT	Standard specification Structural design guidelines	Florida DOT 2010 Florida DOT 2006
Idaho DOT	Standard specification	Idaho DOT 2004
Illinois DOT	Special provision	Illinois DOT 2012
Iowa DOT	Developmental Specification	Iowa DOT 2010
Kentucky DOT	Special note	Kentucky Transportation Cabinet 2008
New Jersey DOT	Standard specification	New Jersey DOT 2007
New York DOT	Special provision	New York State DOT 2012
Rhode Island DOT	Standard specification	Rhode Island 2010
South Carolina DOT	Standard specification	South Carolina DOT 2007
Texas DOT	Standard specification	Texas DOT 2004
West Virginia DOT	Special provision	West Virginia DOT 2006

3.3.1 Mass Concrete Definition

The definition of mass concrete designates which concrete elements must be designed and constructed in accordance with the specified mass concrete requirements. The definition of mass concrete often varies with the element type, dependent on if the placement is a drilled shaft, footing, substructure, or superstructure.

The definition of mass concrete is usually related to the dimensional size of the placement. Generally mass concrete is defined by the least dimension of the concrete pour, or the smallest dimension in all directions of the placement. Additionally, mass concrete may also defined by the volume of placement, the surface area of the placement, or a ratio of the dimensions. If an agency wishes to have additional control over which placements are deemed mass concrete, elements may be designated on a case-by-case basis.

Table 3.3 indicates the definition of mass concrete provided by the specifications identified in the survey. The definitions vary greatly between agencies, with least dimensions varying from 3'-5'. Additionally, the definition of mass concrete pertains to varying element types from only footings to all concrete placements. A common trend of the specifications is to define mass concrete differently for cast in place concrete piers, piles, or shafts. Similarly, five specifications identify mass concrete by designating it on the plans, allowing the agency to define mass concrete on a case-by-case basis depending on the situation.

Table 3.3 Mass concrete definition by agency

Agency	Definition
Arkansas DOT	NA
California DOT	Cast in place concrete piles with a diameter greater than 8', other definitions are reserved.
Florida DOT	Concrete with a least dimension of 3' and the volume to surface area of the concrete exceeds one 1'. Drilled shafts with a diameter greater than 6'.
Idaho DOT	Footings thicker than 4'.
Illinois DOT	Least dimension of 5' for drilled shafts, foundations, footings, substructures, or superstructures.
Iowa DOT	Least dimension of footings greater than 5', or other concrete placements with a least dimension of 4', excluding drilled shafts.
Kentucky DOT	Least plan dimension 5' or greater, excluding drilled shafts.
New Jersey DOT	As defined on the plans.
New York DOT	NA
Rhode Island DOT	Concrete dimensions in 3 directions is 5' or more.
South Carolina DOT	Concrete has dimensions of 5' or greater in 3 directions. For circular sections a diameter of 6' or greater and a length of 5' or greater, excluding driller shafts and foundation seals.
Texas DOT	Least dimension of 5' or greater, or as designated on the plans.
West Virginia DOT	Least dimension of 4' for footings, pier shafts, arms, and caps, excluding drilled caissons and tremie seals.

NA- not available

3.3.2 Temperature Restrictions

Specifications typically provide temperature restrictions to control thermal damage from delayed ettringite formation and thermal gradients. The temperature restrictions provided by agencies with an identified mass concrete specification are shown in Table 3.4.

Table 3.4 Temperature restrictions by agency

Agency	Maximum Temperature (°F)	Maximum Temperature Difference (°F)
Arkansas DOT	NA	36
California DOT	160	To be determined to prevent cracking due to heat of hydration
Florida DOT	180	35
Idaho DOT	NA	35
Illinois DOT	150	35, up to 50 if approved
Iowa DOT	160	20 (0-24 hrs) 30 (24-48 hrs) 40 (48-72 hrs) 50 (>72 hrs)
Kentucky DOT	160	35
New Jersey DOT	160	35
New York DOT	NA	35
Rhode Island DOT	NA	70
South Carolina DOT	160	35
Texas DOT	160	35
West Virginia DOT	160	35

NA - not available

Maximum temperature restrictions are specified to prevent delayed ettringite formation in the concrete. Of the agencies with an identified mass concrete specification, the maximum allowable temperature in the placement ranges from 150-180°F.

Maximum temperature differentials are specified to control the thermal damage to internal restraint. The majority of the specifications identified limited the maximum temperature difference to 35°F. The California DOT standard specification takes a performance-based approach allowing the contractor to submit maximum temperature differentials that prevent "cracking due to heat of hydration." The Iowa DOT developmental specification for mass concrete uses a gradient approach to define the maximum temperature differential. Over the first four days after the completion of the pour, the maximum temperature difference is allowed to increase 10°F for each day after placement, ranging from 20-50°F. The gradient approach allows the contractor to take advantage of the increase in concrete strength over time.

3.3.3 Mix Proportion Requirements

Specifications may limit the mix proportion of the concrete to control the strength, durability, and heat generation from the hydration of the concrete. Table 3.5 and Table 3.6 show the specification requirements for allowable cement types, cement content, compressive strength, and supplementary cementitious material substitution for agencies identified as having a specification.

Table 3.5 Cement and compressive strength restriction by agency

Agency	Allowable Cement Types	Cement Content	Compressive Strength
Arkansas DOT	II or I if approved	NA	3500psi-90 day, 3000psi-28 day
California DOT	NA	NA	NA
Florida DOT	NA	NA	NA
Idaho DOT	NA	NA	NA
Illinois DOT	NA	Minimum Portland cement content of 330lb/cy	NA
Iowa DOT	I/II, IP, or IS	Minimum cement content of 560 lb/cy	NA
Kentucky DOT	NA	NA	NA
New Jersey DOT	NA	NA	NA
New York DOT	Type II cement only	Total cementitious content of 300kg/m^3 (506 lb/cy)	21MPa(3046 psi)-56 day
Rhode Island DOT	NA	NA	NA
South Carolina DOT	NA	NA	NA
Texas DOT	NA	NA	NA
West Virginia DOT	NA	NA	NA

NA - not available

Table 3.6 Supplementary cementitious material substitution by agency

Agency	Supplementary Cementitious Material Substitution
Arkansas DOT	70
California DOT	NA
Florida DOT	Fly ash substitution of cement by weight 18-50%, slag substitution 50%-70%.
Idaho DOT	NA
Illinois DOT	Maximum cement substitution for fly ash 40%, GGBFS 65%.
Iowa DOT	Total cement substitution of 50% for fly ash and slag, class C fly ash limited to 20%.
Kentucky DOT	Substitution of GGBFS up to 50% of cement content, total fly ash and slag substitution of 50%, with a maximum fly ash substitution of 20%.
New Jersey DOT	NA
New York DOT	Class F fly ash 20-50% substitution of cementitious materials.
Rhode Island DOT	NA
South Carolina DOT	NA
Texas DOT	NA
West Virginia DOT	Total slag and fly ash substitution of 50%, maximum fly ash substitution of 25%, and maximum slag substitution of 50%.

NA - not available

The specification survey shows that many agencies do not have mix proportion restrictions specifically for mass concrete. Additionally, there is little commonality between agencies in regard to mix proportion requirements.

3.3.4 Construction

Specification requirement for the construction of mass concrete placements are difficult to establish because of the wide range of element types, locations, and thermal concerns. Construction practices that may be reasonable for an element with a large risk of thermal damage may not be reasonable for a simple placement with little concern of thermal damage. Therefore, typically only the fresh placement temperature of a placement is restricted for the construction of mass concrete elements.

Table 3.7 shows the restrictions on fresh placement temperature for mass concrete construction. The results show that many agencies do not place additional restrictions on the fresh placement temperature for mass concrete. Additionally, there is little commonality in fresh placement temperature restrictions between agencies. The range of maximum fresh placement temperature is 60-90°F for agencies with identified specifications.

Table 3.7 Fresh placement temperature by agency

Agency	Fresh Placement Temperature Range (°F)
Arkansas DOT	Maximum temperature 75
California DOT	NA
Florida DOT	NA
Idaho DOT	NA
Illinois DOT	40-90
Iowa DOT	40-70
Kentucky DOT	Maximum temperature 60
New Jersey DOT	NA
New York DOT	NA
Rhode Island DOT	NA
South Carolina DOT	Maximum temperature 80
Texas DOT	50-75
West Virginia DOT	NA

NA - not available

3.3.5 Thermal Control Verification

Thermal control verification is the process of verifying that the thermal control requirements of the placement are met. Generally, mass concrete placements are monitored during construction to ensure that temperature restrictions are not violated, or in danger of being violated. Pours are monitored through the use of temperature sensors installed in locations that provide the maximum and minimum temperatures of the placement. These temperatures provide the maximum temperature and maximum temperature difference to verify the thermal requirements.

Proper sensor location is crucial to accurately gage the thermal stresses in the placement. If sensors are improperly installed, the temperature reading may have significant error, providing misleading results. Additionally, the surface sensors may comprise the durability and cosmetic appearance of the concrete if installed to close to the surface. To capture accurate results, sensors must be installed in the proper location in the placement. Table 3.8 shows the sensor location requirements and the surface cover requirements for sensors placed near the surface.

The survey shows that many agencies do not directly specify the sensor location or the required cover for surface sensors. Additionally, there is little uniformity in the sensor location or surface sensor concrete cover requirements among agencies that have identified specification requirements.

Table 3.8 Sensor locations and cover by agency

Agency	Sensor Locations	Surface Sensor Cover
Arkansas DOT	Contractor developed, agency approved.	NA
California DOT	Calculated hottest location, 2 outer faces, 2 corners, top surface.	NA
Florida DOT	Contractor developed, agency approved.	NA
Idaho DOT	NA	NA
Illinois DOT	Contractor developed, agency approved. Additionally the ambient air temperature and entrance/exit of cooling water.	1-3"
Iowa DOT	Center of the placement, midpoint of side closest to the center, midpoint of top surface, corner of the placement furthest from the center, and ambient air temperature.	2" minimum
Kentucky DOT	2 at separate locations near the geometric center, 2 at the center of the exterior face with the longest distance from the interior sensors, and that has the least sun exposer.	1"
New Jersey DOT	As close as possible to the center, and at the exposed surface.	NA
New York DOT	Center of the placement, base of the mass, the surface of the mass, center of the exterior face that is the shortest distance from the center of the mass.	NA
Rhode Island DOT	Designated by the engineer.	NA
South Carolina DOT	Contractor developed, agency approved.	NA
Texas DOT	NA	NA
West Virginia DOT	Hottest location, on at least two outer faces, two corners, and top surfaces.	NA

NA - not available

Thermal control completion time denotes the time when the contractor ceases the monitoring of the concrete and thermal protective procedures. At completion, the threat of thermal damage without outside intervention has been reduced to an acceptable level. Table 3.9 shows the thermal control completion time by agency.

The survey shows that the majority of specifications require that the maximum temperature in the placement to be within the maximum temperature differential requirement of the ambient air temperature. This requirement allows the formwork and insulation to be removed from the placement, without increasing the risk of thermal damage. Additionally, this requirement will typically force the placement to reach a maximum temperature and begin to cool.

Table 3.9 Thermal control completion time by agency

Agency	Time of Thermal Completion
Arkansas DOT	At least 7 days.
California DOT	Maximum internal temperature is falling, difference between core temperature and ambient temperature is within the ambient air temperature for 3 consecutive days, and no adjacent mass concrete element to be poured.
Florida DOT	The maximum temperature differential begins to decrease, and the core temperature is within 35°F of the ambient air temperature.
Idaho DOT	7 days.
Illinois DOT	After the maximum temperature is reached, post-cooling is no longer required, and the maximum temperature differential does not exceed 35°F.
Iowa DOT	Maximum temperature difference is within 50°F of the average ambient temperature of the previous seven days.
Kentucky DOT	Temperature at the center is within 35°F of the average ambient air temperature of the past 7 days.
New Jersey DOT	15 days, or until the interior concrete temperature is within 35°F of the lowest ambient temperature.
New York DOT	Maximum temperature differential is reached and begins to decrease.
Rhode Island DOT	NA
South Carolina DOT	2 weeks, or until the interior concrete temperature is within 35°F of the lowest ambient temperature.
Texas DOT	4 days.
West Virginia DOT	Maximum temperature differential is reached and decreasing, and the maximum temperature is within the maximum allowable temperature differential of the ambient air temperature.

NA - not available

3.4 DISCUSSION

The results show that there are very large differences between mass concrete specifications for each agency. There is little consensus between agencies on what aspect of mass concrete mix proportion, construction, and thermal control need to be specified. Aspects that are specified by all agencies generally still have large discrepancies in requirements.

CHAPTER 4. TYPICAL MIDWEST BOARDER BRIDGES

4.1 INTRODUCTION

To validate the subsequent case studies as being typical of Midwest board bridges, it is necessary to identify the commonalities of boarder bridges built in the Midwest. This chapter includes the process of determining the characteristics of Midwest boarder bridges, the identified physical bridge characteristics, construction procedures, concrete properties, and environmental conditions.

4.2 METHODOLOGY

To provide a starting point to investigate the typical characteristics of Midwest boarder bridges, the WB I-80 over the Missouri River Bridge and US 34 over the Missouri River Bridge were first investigated. From the initial investigation, a base line was develop for bridge characteristics, construction practices, and concrete properties, and environmental conditions. Subsequently, additional bridges designed by the Iowa DOT were investigated to provide a wider range of bridge characteristics. Finally, six construction companies and design firms who are commonly involved in the design and construction of Midwest boarder bridges were contracted and interviewed to provide a range of bridge characteristics that encompass all Midwest boarder bridges.

4.3 BRIDGE CHARACTERISTICS

Midwest boarder bridges are typically very large in size resulting from the width of the rivers they span, largely the Mississippi and Missouri Rivers and their tributaries. Additionally, the dimensions of Midwest boarder bridge elements are largely dictated by the main span length, which are generally large to allow for the navigation of barges. The following sections describe the typical element dimensions found on Midwest boarder bridges.

4.3.1 Footings

From the investigation, it was determined that the footing on Midwest boarder bridges ranged in size from elements not typically considered to be mass concrete (assuming a least dimension of four feet) to placements with a least dimension of over 12 feet. Additionally, the length and width of footings also varied greatly between bridges, depending on the number of footings per pier and location of the footing. It was determined that the general width and lengths of footings ranged from less than 12 feet to over 160 feet. The location of the footing relative to the main span has the largest impact on the dimension of the footing, with the smallest footings located near the abutments, and the largest footings located at the main span.

4.3.2 Stems and Columns

Similar to footings, the dimensions of stems and columns are dependent on the number of stems and columns per pier and the relative location on the bridge. The investigation concluded that the least dimension of columns and stems ranged from elements not considered to be mass concrete (assuming a least dimension of four feet), to a least dimension of over 24 feet. In general, the length and width of stems and columns have minimal impact on the thermal development, as they are typically substantially larger than the least dimension and may be assumed to be infinitely large for analysis purposes.

4.3.3 Caps

From the investigation, it was determined that the least dimension of caps for Midwest boarder bridges typically range in size from dimensions typically not considered to be mass concrete (assuming a least dimension of four feet), to a least dimension of over 20 feet. It was observed that the largest caps were located near the main span.

4.4 CONSTRUCTION

Due to the similar conditions of Midwest boarder bridges, the construction practices are related. The general construction practices of Midwest boarder bridges include the following.

- Use of wood and steel formwork
- Thermal monitoring of elements with thermal sensors
- Use of insulation to reduce thermal gradients
- Utilization of cooling pipe systems on large placements

4.5 CONCRETE PROPERTIES

Due to the relative cost and benefits of supplementary cementitious material with regard to the thermal development of mass concrete, fly ash and GGBFS are typically utilized in the mix design of concrete for Midwest boarder bridges.

4.6 ENVIRONMENTAL CONDITIONS

Due to the relatively fast pace construction requirements of boarder bridges, construction typically continues year round, requiring construction in extreme weather conditions. The Midwest has a unique climate where temperature can vary from over 110°F to less than -40°F. The extreme climate conditions requires mass concrete to be constructed in very hot and cold climates, separating Midwest boarder bridge construction from mass concrete construction in other geographic locations.

CHAPTER 5. CASE STUDY

5.1 INTRODUCTION

The purpose of this chapter is to provide a description of conditions under which the WB I-80 over the Missouri River Bridge, and the US 34 over the Missouri River Bridge were constructed and verify that they are typical examples of Midwest boarder bridges. The first two sections of this chapter will provide a general overview of the WB I-80 and US 34 Bridges respectively. The following sections will describe the conditions under which the bridges were constructed, the mix proportion used, and the environmental conditions.

5.2 WB I-80 OVER THE MISSOURI RIVER BRIDGE OVERVIEW

The WB I-80 over the Missouri Bridge is a 2477' 10" by 84' continuous welded girder bridge. The bridge spans the Missouri River connecting Omaha, NE and Council Bluffs, IA. The bridge consisted of 27 different mass concrete elements as, defined by the Iowa DOT mass concrete developmental specification (DS-09047).

The mass concrete elements were constructed from August 2008 through August 2009. Elements defined as mass concrete included footings, stems, columns, and pier caps. The elements had a range of sizes varying from a least dimension of 4' to 10.5'.

The construction of the mass concrete elements was completed by two separate contractors, Jensen Construction Company of Des Moines, IA and Cramer & Association, Inc. of Grimes, IA. CTL Group of Skokie, Illinois was engaged by Jensen Construction Company to be the consultant for the construction of the mass concrete elements.

5.3 US 34 OVER THE MISSOURI RIVER BRIDGE OVERVIEW

The US 34 over the Missouri River Bridge is a 3276' 1" by 86' 3" continuous welded girder bridge with pretensioned prestressed concrete beam approaches. The bridge crosses the Missouri River south of Omaha, NE and Council Bluffs, IA. The bridge began construction in 2010 and is scheduled for completion in 2014.

The bridge has several mass concrete elements as defined by the Iowa DOT mass concrete developmental specification (DS-09047). The elements include footings, columns, and caps that were constructed with and without cooling pipes. The elements have least dimensions ranging in size from 5.5' to 6.5'.

The construction of the mass concrete elements was completed by Jensen Construction Company. CTL Group was engaged by Jensen Construction Company to be the consultant for the construction of the mass concrete elements.

5.4 CONSTRUCTION

This section describes the general conditions in which the mass concrete elements on the WB I-80 Bridge and US 34Bridge were constructed. The exact conditions that the elements were constructed under are described in more depth by Chapter 6.

5.4.1 Footing Subbase and Support

Each footing has a supporting mechanism that transfers the load placed on the footing to the soil structure below. In addition to supporting the footing, the support structure also externally retains the footings. To support the footings on the WB I-80 Bridge, two techniques were used, steel bearing piles and drilled shafts. Piers 1-5, 7, and 10-11utilized HP 12 x 84 steel bearing piles to support the respective footings. The Pier 6 footing was supported by 48 inch diameter drilled shafts, and Piers 7 and 8 were supported by 72 inch diameter drilled shafts. Similarly on the US 34 Bridge, Piers 1-4 and 7-17 were supported by HP 14 x 89 steel bearing piles. Piers 5 and 6 were supported by 30 individual 48 inch diameter open-ended steel piling.

The subbase material that each footing is poured against depends on the location of the footing. Footings that are placed in or close to the river require a seal coat, which is a layer of concrete that is several feet thick, be cast below the footing to prevent the footing from being cast on water. Each footing that is placed on a seal coat is still restrained by the footing support structure, piling or drilled shafts, that extends through the seal coat, in addition to the seal coat.

Footings that were not cast on seal coats were typically placed on clay subbase, a typical soil condition along Midwest rivers. Alternatively, a layer of gravel was also placed on top of the clay subbase to provide a firm and dry casting surface in some instances. The WB I-80 Bridge footings were cast against a clay subbase, while the US 34 Bridge footings were cast against a crushed rock subbase, as shown by Figure 5.1 and Figure 5.2 respectively.

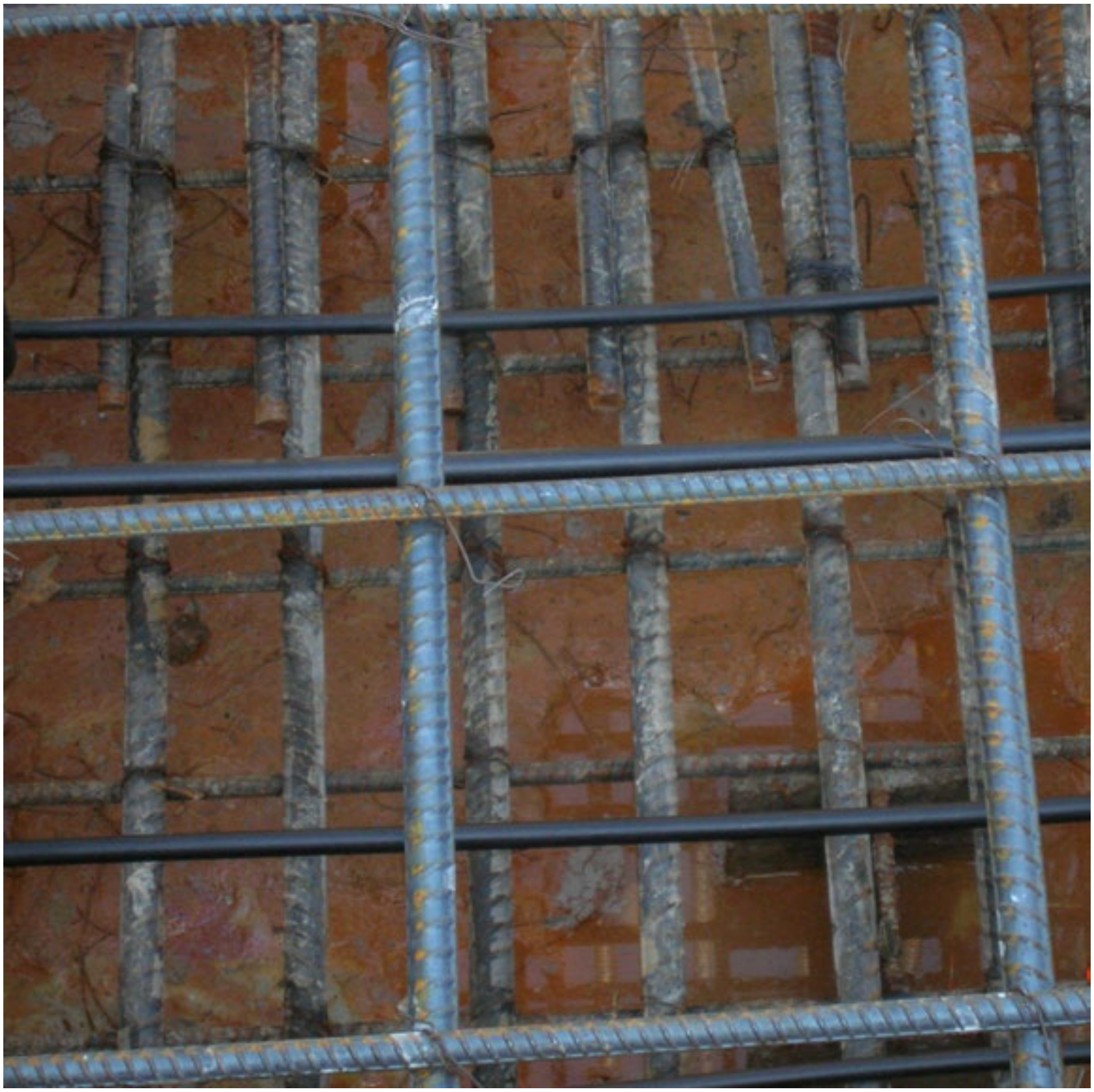


Figure 5.1 Clay subbase with steel bearing pile

Figure 5.2 Crushed rock subbase with steel bearing pile

5.4.2 Formwork Material

Two different formwork materials were used to form the placements on both the WB I-80 and US 34 Bridges, wood and steel. The choice of formwork material is dependent on the type of placement that is being formed. Generally, the placements that are shorter in height and are relatively simple shapes used wood formwork. The typical applications of the wooden formwork include simple footings, and the patching of steel formwork gaps, such as the bottom of pier caps. Steel formwork is typically used on larger placements that develop more hydraulic pressure, such as columns, stems, large footings, and large caps.

The wood formwork consists of three quarter inch plywood attached to two-by-four and two-by-six inch supporting members with nails. A typical example of the wood formwork used on both projects is shown by Figure 5.3. The steel formwork that was used on both projects consisted of yellow EFCO formwork. Figure 5.4 and Figure 5.5 show typical examples of the steel formwork used on both projects.

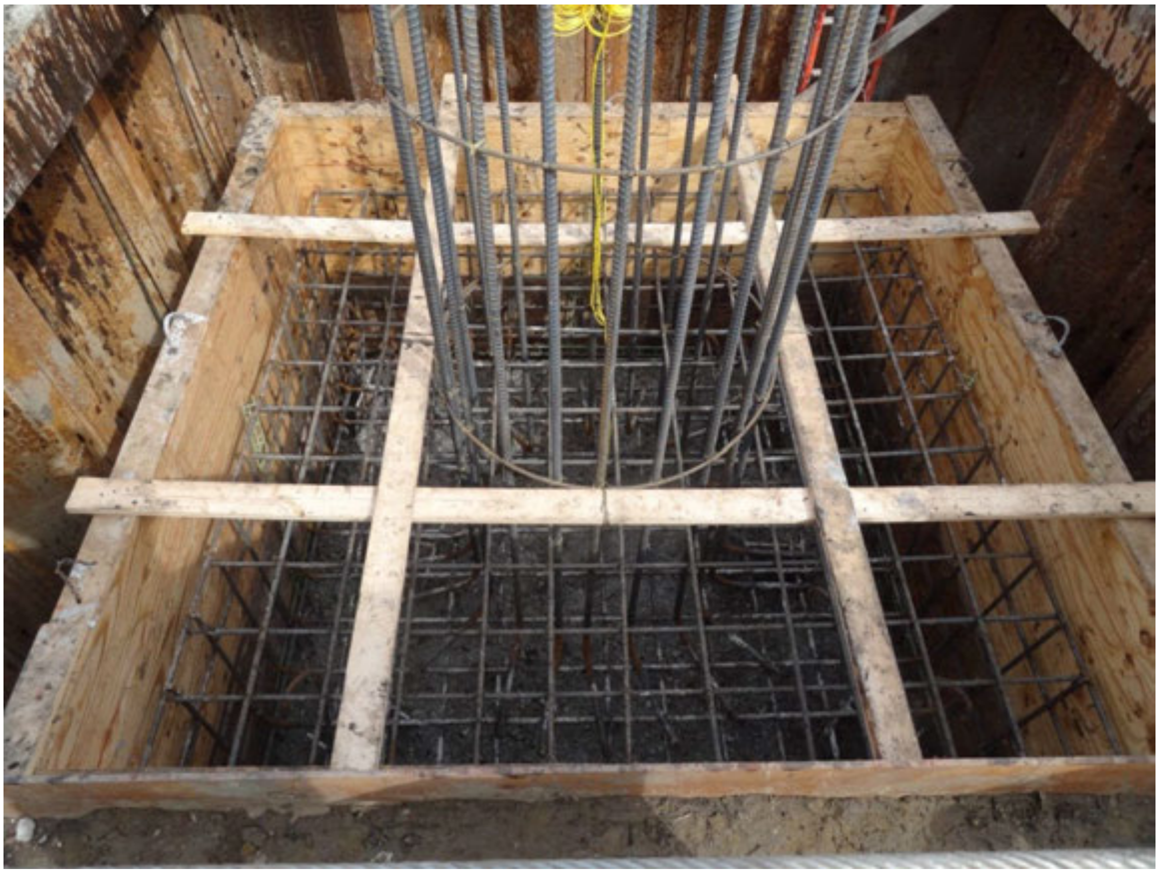


Figure 5.3 US 34 over the Missouri River Bridge Pier 3 footing

Figure 5.4 WB I-80 over the Missouri River Bridge column formwork

Figure 5.5 US 34 over the Missouri River Bridge column formwork

5.4.3 Pier Elements

To ease in the construction of the bridges, construction joints were installed in the piers at discrete locations. For both the WB I-80 and US 34 Bridges, the piers were typically poured in four sections, the footing, stem, column, and cap, as shown by Figure 5.6. The allowable locations for the construction joints were designated by the bridge designer.

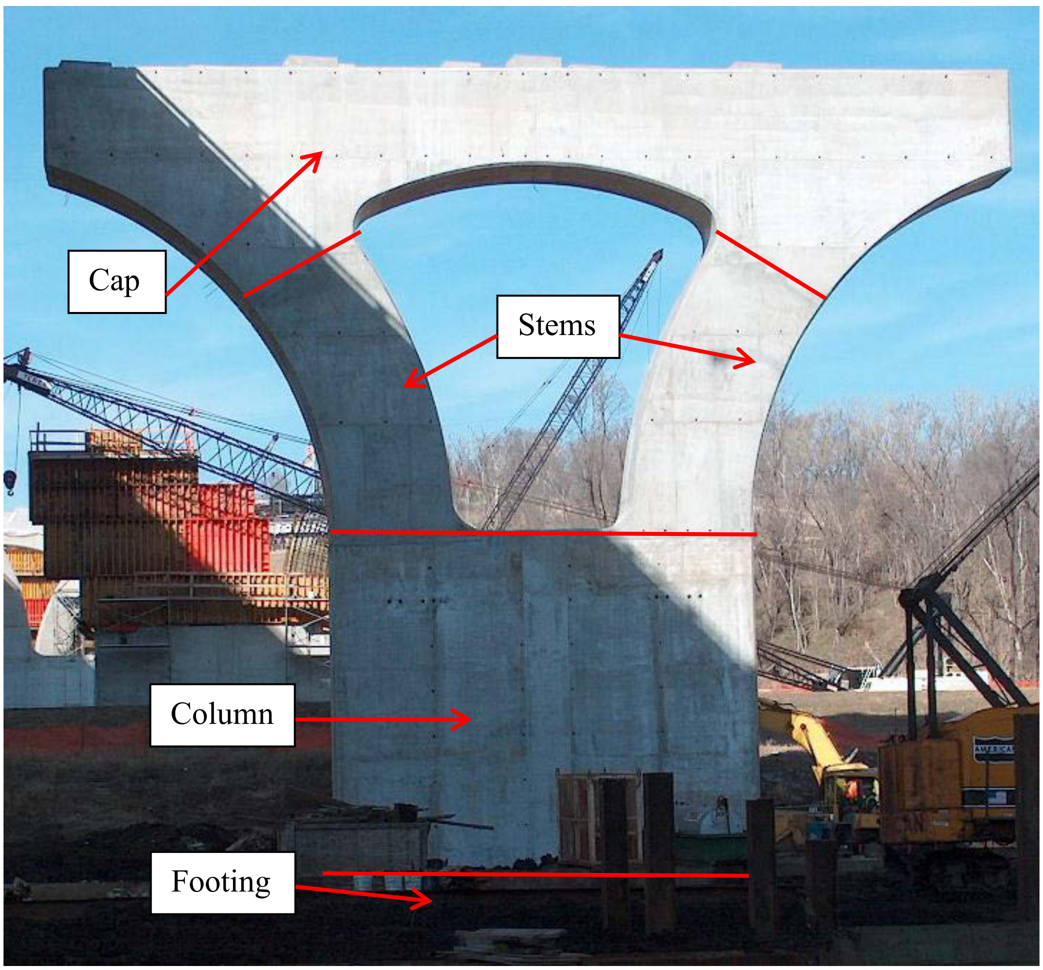


Figure 5.6 Typical bridge pier element sections

For small or simple elements, the number of pier elements was reduced for both bridges. The stem and column on Pier 1from the WB I-80 were combined into one pour due to the relatively small size of the stem and column. The US 34 Bridge utilized four separate footings and columns for Piers 1-3 and 8-17, which simplified the geometry and reduced the size of each element. As a result, the piers were poured in three sections, the footing, column, and cap.

5.4.4 Concrete Placement

The relative size of the concrete placements on the WB I-80 and US 34 Bridges required large amounts of concrete to be placed in a single unit. To complete the pours, two different methods were utilized, concrete hopper buckets and concrete pump trucks. Many factors that affect the placement method include the size of the placement, congestion of the pour site, height of the pour, and availability of equipment.

Concrete pump trucks allow the concrete to be placed at a lower height compared to hopper bucks in congested areas as shown by Figure 5.7, especially when equipped with an extended tremie pipe. Concrete hopper buckets were also utilized on placements with large depths by utilizing tremie pipes to reduce the drop height. A lower concrete placement height reduced the segregation of the concrete.

Figure 5.7 US 34 Bridge Pier 4 footing concrete placement

The use of concrete hopper buckets is often a less expensive alternative to concrete pump trucks for accessible placements with little congestions. Concrete hopper buckets are typically less expensive for contractors as they do not require renting additional equipment. Due to the size of concrete hopper buckets, the concrete is generally dropped above the top of the formwork. If a tremie pipe is not utilized, the application of concrete hopper buckets is limited to placements of relatively short depth to prevent concrete segregation. Figure 5.8 shows the use of concrete hopper bucket to pour a 5.5 foot deep foundation.

Figure 5.8 US 34 Bridge Pier 2 footing concrete placement

5.4.5 Consolidation

Consolidation of concrete is an essential step in the placement of mass concrete. If concrete is not properly consolidated, the concrete element will have substantial voids, reducing the overall strength and durability of the element. Consolidation on both the WB I-80 and US 34 Bridges utilized concrete vibrators with flexible shafts to internally vibrate the concrete. To assure that the concrete was adequately consolidated, the concrete was vibrated at each individual concrete placement layer. A typical example of the vibratory compactor used on both bridges is shown by Figure 5.9.

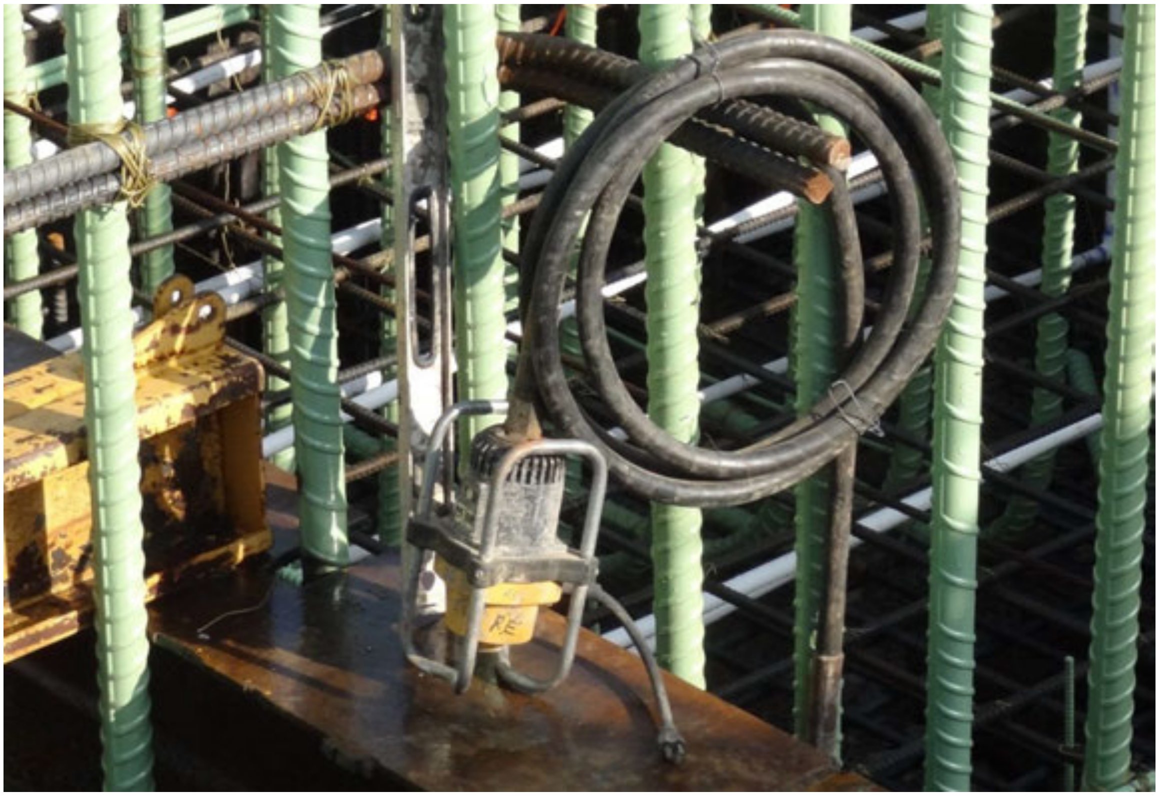


Figure 5.9 Jensen Construction Company flexible shaft vibratory compactor

5.4.6 Insulation

To control the maximum temperature difference of the mass concrete placements, all elements on the WB I-80 and US 34 Bridges were insulated. The typical insulating method on both bridges was to wrap the exterior of the formwork and the top of the placements with a black insulating blanket with a specified R value rating of 5.

The general practices for each placement was to use a single layer of insulating blankets on each surface of the placement, except for the bottom of the footings. Insulation was also used to cover any exposed steel protruding from the placement, generally rebar. As steel is an efficient heat transferring material, it is necessary to keep the rebar at relatively the same temperature as the concrete to prevent large thermal gradients from developing near the rebar.

In an attempt to efficiently control the thermal development of the placements, blankets were added and removed from the placement over the duration of the period of thermal control. During the construction of the WB I-80 Bridge, conditions arose that required additional insulating blankets be added to the placement to prevent exceeding the maximum temperature difference limits. In some instances, additional insulating blankets were added to all sides, but were typically limited to the top surface. During the construction of the WB I-80 Bridge, instances also arose that allowed for the unexpected early removal of insulating blankets. If the placement was not in danger of exceeding the specified maximum temperature difference limits, cooling blankets were occasionally removed to more rapidly dissipate the heat generated in the placement. Removal of some or all of the insulating blankets reduced the time in which the placement was under thermal control, allowing shorter formwork cycle times. The removal of insulating blankets was also utilized if the placement was in danger of exceeding the allowable maximum temperature of the placement.

The typical condition of the insulating blankets used on both bridges was that of used insulating blankets. Generally, the blankets had minor damage from previous use including many holes from being previously attached to formwork. Additionally, many blankets had small rips and tears.

To attach the insulation to wooden formwork, the insulation was typically nailed around the edges to secure the blanket in place. The blankets were attached to formwork to the degree required to withstand the weather conditions, but not attached to a degree that greatly prevented the movement of air between the formwork and the insulating blankets.

The blankets typically appeared to be sufficiently lapped at the joints between blankets so that one could assume the concrete unit was covered by a continuous layer. Figure 5.10 shows a typical situation where an insulating blanket attached to wood formwork.

The insulation blankets were generally attached to the exterior of the formwork before the placement of the concrete began. The top surface of the placement was covered with insulation blankets once the concrete had taken a set. The top surface was viewed as the most sensitive surface as there was no formwork to provide additional thermal resistance, therefore extra care was taken to assure that the blankets were properly lapped on the top surfaces.

Figure 5.10 Insulation attached to wood formed footings

As a result of the formwork shoring on certain footing of the WB I-80 Bridge, the sides of the footings were unable to be directly attached to the formwork. To provide additional rigidity to formwork, shores were installed to support the formwork walls by the coffer dam sheet pile walls as shown by Figure 5.11. As a result, the insulating blankets were unable to be attached directly to the formwork.

Figure 5.11 WB I-80 Bridge wood formed footing shoring

In an effort to provide thermal resistance to the sides of the placement, thermal blankets were applied on top of the shoring, bridging the gap between the top of the formwork and the cofferdam walls as shown by Figure 5.12. The insulating blankets were intended to prevent air flow along the sides of the footing and capture the heat of the placement in the void. The effectiveness of the insulating blanket installed on top of the shoring is unknown.

Figure 5.12 Shored formwork insulating blanket

Elevated placements on both bridges occasionally utilized cat walks to aid in the assembly of the formwork. As the catwalks are connected to exterior surfaces of the formwork, it is difficult to attach the insulation blankets directly to the formwork. To provide insulation to the placement, the blankets were wrapped around the catwalks, capturing a layer of air in between the insulation blankets and the formwork as shown by Figure 5.13

Figure 5.13 Elevated placement with insulating blankets wrapped around the catwalks

Placements that were formed with steel were insulated similarly to that of wood formwork. The main difference is that steel formwork on both bridges required that the insulation blankets be tied to the formwork. The insulating blankets were tied with simple tie wire onto the formwork struts. A typical example of insulating attached to steel formwork for both bridges is shown in Figure 5.14.

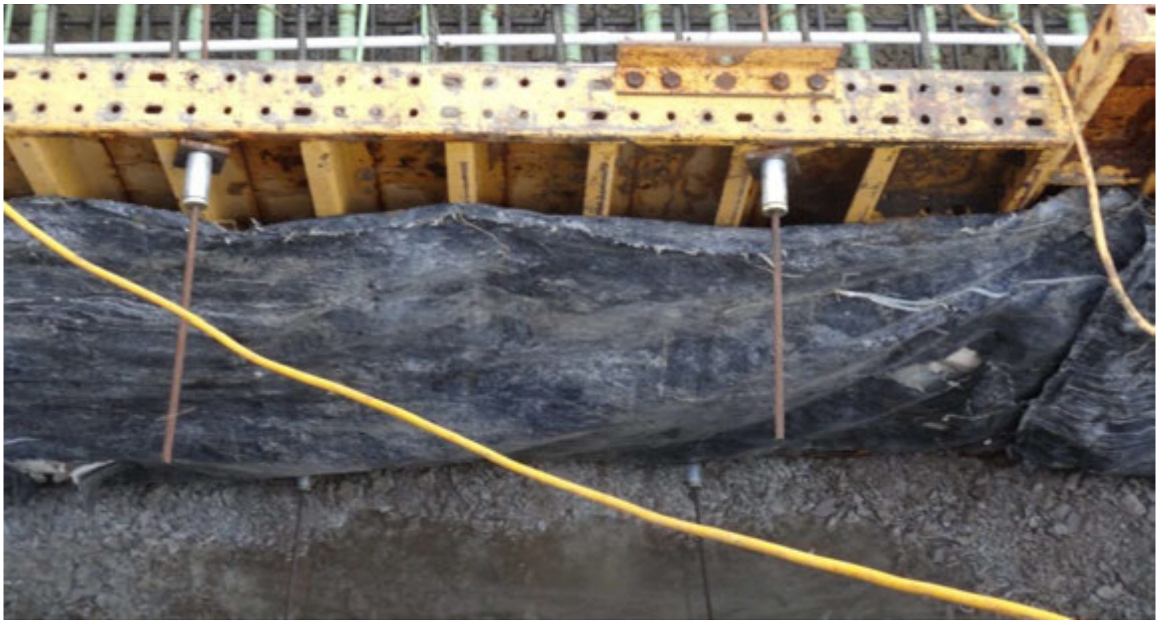


Figure 5.14 Steel formed footing with insulating blanket

5.4.7 Cooling Pipes

Cooling pipes were utilized on both bridges to control the thermal development of placements with relatively large dimensions. Cooling pipes were also occasional used to minimize the time in which the placement was required to remain under thermal control, to reduce the formwork cycle time.

The water required for the cooling pipe systems for both bridges was supplied by the adjacent Missouri River or contactor dug wells. The water was pumped to the placement and through the cooling pipes by means of diesel, gas, or electric powered water pumps. The water pump configuration utilized on the US 34 Bridge is shown in Figure 5.15.

Figure 5.15 US 34 Bridge cooling pipe system water supply pump

To reach the required placements, the water had to be pumped over long distances in some instances. The large distances required the use of a large water pump that could overcome the head loss developed by both the elevation differential between the river and the placement, as well as the pipe friction. In the case of the US 34 Bridge, the water had to be pumped over 400 feet horizontally and over 50 feet vertically to supply the cooling pipe system for the Pier 4 cap.

The water was pumped through piping approximately 4”-8” in diameter from the water pump until the piping reached the placement. As the water approaches the placement, the piping splits at a manifold to allow for the use of multiple cooling pipe systems, which also allows the following piping to be of reduced size to increase the pressure, as shown by Figure 5.16.

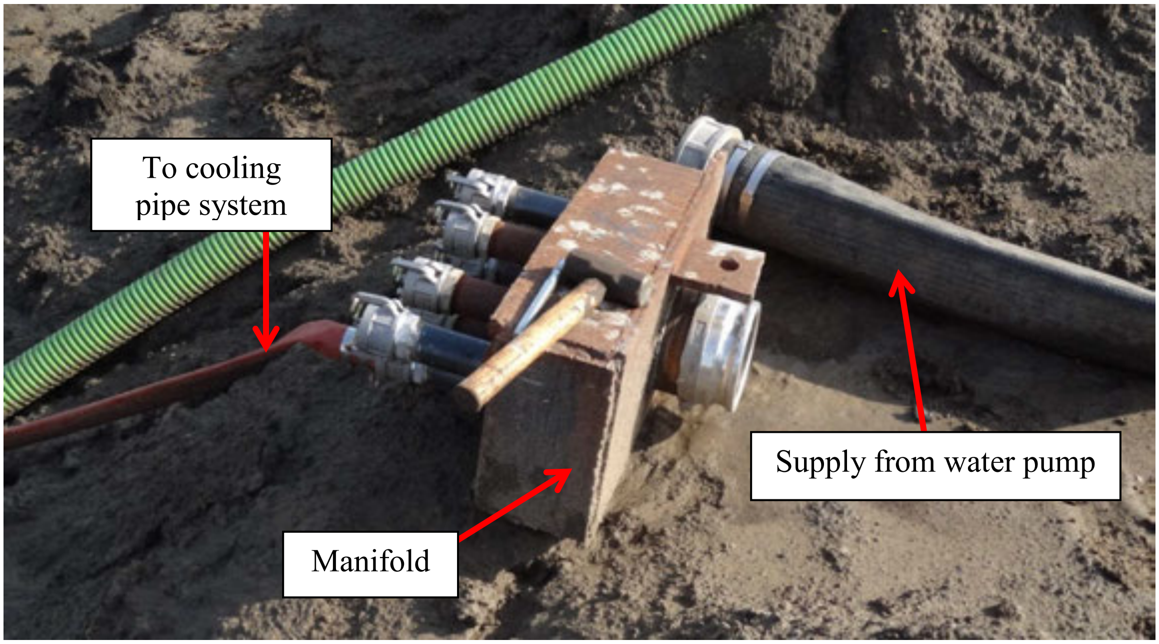


Figure 5.16 Cooling pipe system supply line manifold

As the piping reaches the placement, the water is pumped through an additional manifold. Typical examples of the manifolds used on the WB I-80 and US 34 Bridges are shown by Figure 5.17 and Figure 5.18 respectively. The manifold allows each separate loop of the cooling pipe system to be supplied by the single supply line. The manifold also allows the contractor to adjust the flow rate of water through each loop of the system.

Figure 5.17 WB I-80 Bridge cooling pipe system manifold

Figure 5.18 US 34 Bridge cooling pipe system manifold

Each cooling pipe system consisted of several loops that pumped the water through the placement. Each loop was typically spaced in both the vertical and horizontal directions by two to three feet. Additionally, the material utilized to construct the loops inside the placement varied between the two projects. The WB I-80 Bridge utilized ¾ inch PEX (cross-linked polyethylene) piping as shown by Figure 5.19 as well as ¾ inch PVC (polyvinyl chloride) piping. The US 34 Bridge utilized 1 inch PVC piping as shown by Figure 5.20. The PEX piping on the WB I-80 Bridge was attached to the rebar with cable ties, and the PVC piping on the US 34 Bridge was attached to the rebar with tie wire and cable ties.

Figure 5.19 PEX cooling pipes being installed on a WB I 80 bridge footing

Figure 5.20 Installed PVC piping on US 34 Bridge footing

Once the water was pumped through the circulation loop in the placement, the water was pumped out of the placement to different locations. Depending on the element, the water leaving the placement was either pumped directly back to the river, or was drained into the cofferdam. The water that was drained into the cofferdam would be subsequently pumped out by the cofferdam dewatering pumps. Since the cooling pipes utilized an open system, the systems were not pressure tested. To verify that there were no leaks in the cooling pipe system, water was run through the entire system before concrete placement began, and the system was checked for leaks.

To avoid thermally shocking the placement with the cooling pipes, the circulation of water began immediately after the completion of the pour. Additionally, once the circulation of the water through the placement was stopped, the circulation of the water was never restarted. Therefore the circulation of water was generally continued until the threat of thermal damage to the placement was completely past.

The temperature of the water circulating through the placement was measured as the water entered and exited the placement. The temperature of the water pumped from the adjacent river was approximately equal to that of the average ambient air temperature at the time the placement was poured. The temperature of the water supplied by contactor dug wells was approximately 15°F lower than the average air temperature in the summer. The difference in the water temperature between the entrance and exit locations was typically 1-3°F. The flow rate through each loop was adjusted, by means of the manifold, to maintain an acceptable temperature difference of the water entering and exiting the placement. The contactors estimated the flow rate through each loop to be approximately 10 gal/minute.

Following the completion of the thermal control requirements, the cooling pipes were cutoff at the surface of the placement and pumped full of high strength grout.

5.4.8 Thermal Monitoring

In accordance with the Iowa DOT mass concrete developmental specification, each placement on both bridges defined as mass concrete were monitored through the use of thermal sensors. To monitor the thermal development of the placements, two different thermal sensor models were utilized. The WB I-80 Bridge utilized both intelliRock

Temperature Loggers and iButton model DS 1921thermal sensors. The US 34 Bridge utilized only the intelliRock Temperature Loggers.

The location of the thermal sensors varied between both the project and the element type. During the construction of the WB I-80 Bridge, each placement, including all footings, stems, columns, and caps, utilized three discrete sensor locations to monitor the thermal development of the elements. The location of the sensors included the side surface, the top surface and the center of the placement. The location of the sensors are defined as follows: side surface – the center of the surface of the side closest to the geometric center of the placement, top surface – the center of the top (unformed) surface of the placement, center – the geometric center of the placement. In addition to the primary sensors, each location utilized a redundant thermal sensor, in case the primary sensors failed.

Similarly to the WB I-80 Bridge, the US 34 Bridge utilized three discrete sensor locations on many of the elements. However, some elements utilized only two sensor locations, resulting from the geometry of the placement. Since the threat of thermal cracking is the result of large temperature change over relatively short distances, it was determined to be unnecessary to monitor the thermal development at surfaces that were relatively long distances from the geometric center of the placement. Therefore, the columns and other elements with extreme dimension proportions utilized only two sensors. In addition, all placements utilized thermal sensors to monitor the current ambient conditions. Placements that utilized cooling pipes also monitored the temperature of the water entering and exiting the placement.

The Iowa DOT development specification for mass concrete requires that the minimum concrete cover for each sensor to be two inches, however the specification does not state a maximum amount of concrete cover (Iowa DOT 2010). As a result of the specification, the concrete cover for surface sensors varied greatly from element to element.

In general practice, the sensor measuring the surface temperature of the placement was located on the interior side of the rebar nearest the surface. The sensor was placed on the interior of the rebar in an effort to prevent damage to the sensor during concrete placement. Due to the structural rebar layout for each placement, and fabrication errors in the rebar construction and placement, the distance from the sensor and the surface varied greatly, as shown by Figure 5.21 and Figure 5.22. In addition, it was commonly observed that additional concrete was cast above the required height on many footings, in some cases exceeding 6 inches, greatly affecting the sensor concrete cover.

Figure 5.23 shows the rebar concrete cover for a footing, with the red chalk line representing the finish pour height.

Figure 5.21 Distance between formwork and outermost rebar/thermal sensor location – large distance

Figure 5.22 Distance between formwork and outermost rebar/thermal sensor location – small distance

Figure 5.23 Typical rebar cover for mass concrete footing

To determine the location of each sensor, typically no measuring devices were used. Generally, the sensors were approximately placed at their intended locations, which may provide noticeable errors in the thermal monitoring.

Three different methods were used to attach the thermal sensors and their respective wires to the rebar cage, tie wire, cable ties, and electrical tape. Care was taken in the installation of the sensors and wires to prevent damage during concrete placement including supporting the wires and sensors with additional rebar, attaching the sensors and wires to the underside of the rebar, and avoiding slack in the wires. The images in APPENDIX A show the installation of the thermal sensors and typical layout of installed thermal sensors.

Each wire was marked before installation to allow the thermal readings to be assigned to the respective sensor locations. It was common practice to test each thermal sensor after installation, prior to the placement of the concrete, to identify sensors that may have been damaged.

The thermal data was recorded in one hour intervals. Additionally, the data was remotely monitored, by visually checking the thermal readings, to assure that the placement was not in threat of thermal damage during the duration of the thermal control period. Upon the completion of the thermal control period, the data was submitted to the Iowa DOT as part of the required field reports.

5.4.9 Formwork Removal

To prevent thermal damage to the placement, formwork was typically retained on the placement until the time of thermal control expired. The Iowa DOT mass concrete development specification requires that the thermal control of each placement must be maintained until the interior temperature of the placement is within 50°F of the average ambient air temperature. Formwork was also commonly left on the placement beyond the time required by the thermal control requirements until it was required for use on another placement. It was viewed as an inconvenience to store the formwork on the jobsite rather than leave it on the placement until required.

The range of formwork removal times, as recorded by the contactors, ranged from 91-347 hours for both the WB I-80 Bridge and the US 34 Bridge. The large variance is the result of different thermal control requirements due to the varying complexity levels of each placement, as well as varying formwork cycle rates.

5.7 CONCRETE MIX PORPORTION

Both bridges utilized the same mix proportion. The concrete mix proportion along with the material and mechanical properties are described in detail in Chapter 6.

5.6 ENVIRONMENTAL CONDITIONS

Between the mass construction of the WB I-80 Bridge and the US 34 Bridge, the full range of environmental conditions in the Omaha, Nebraska area was experienced. The environmental conditions under which each element was placed is described in detail in Chapter 6.

CHAPTER 6. CONCRETEWORKS CALIBRATION

6.1 INTRODUCTION

Before one uses an analysis software program, it is crucial to understand how the software works and the accuracy and limitations of the program. To verify that ConcreteWorks is capable of accurately predicting the thermal development of mass concrete for Midwest boarder bridges, a case study was conducted. The case study was conducted by comparing the actual recorded temperature data from two separate projects, the WB I-80 over the Missouri River Bridge and the US 34 over the Missouri River Bridge, to a thermal analysis results developed from ConcreteWorks.

Case studies of ConcreteWorks have been previously conducted on bridges and other structures by Riding, Poole, and Meeks (Poole and Riding 2009) (Meeks 2011). Case studies of Midwest boarder bridges have also been conducted utilizing other mass concrete thermal analysis software programs by Li (Li 2012). While other case studies have shown the ability of ConcreteWorks to model general mass concrete structures, the following case study will focus on the effectiveness of ConcreteWorks to model Midwest river bridges.

All Midwest board bridges have general similarities with regard to the bridge characteristics, construction practices, concrete properties, and environmental conditions, as discussed in Chapter 4. To assure that ConcreteWorks is capable of being successfully utilized on Midwest boarder bridge projects, it is desirable to complete a case study applicable to the typical conditions.

The first section of the chapter describes the software package ConcreteWorks that was utilized to complete the case study. The second section pertains to the WB I-80 over the Missouri River Bridge case study, and includes the development of the inputs, the case study results, and the case study discussion. The final section of this chapter pertains to the US 34 over the Missouri River Bridge case study, and contains the development of the inputs, the case study results, and the case study discussion.

6.2 CONCRETEWORKS SOFTWARE

ConcreteWorks is a software package developed at the Concrete Durability Center at the University of Texas. ConcreteWorks was designed to assist with concrete mix proportioning, thermal analysis, and chloride diffusion service life evaluation of concrete. ConcreteWorks may also be used to analyze the early age thermal development and cracking potential of mass concrete, and assist in the design of mass concrete placements (Riding 2007). For this case study, only the thermal development of mass concrete will be investigated, due to the software limitations in predicting the cracking potential of mass concrete.

ConcreteWorks utilizes the finite difference method to analyze the thermal develop of mass concrete elements. To complete the thermal analysis of mass concrete, the software package considers the material constituents, mix proportion, geometry, formwork type, and environmental conditions. (Folliard 2007).

The process by which ConcreteWorks evaluates the thermal development of mass concrete placements is described by the flow chart shown in Figure 6.1.

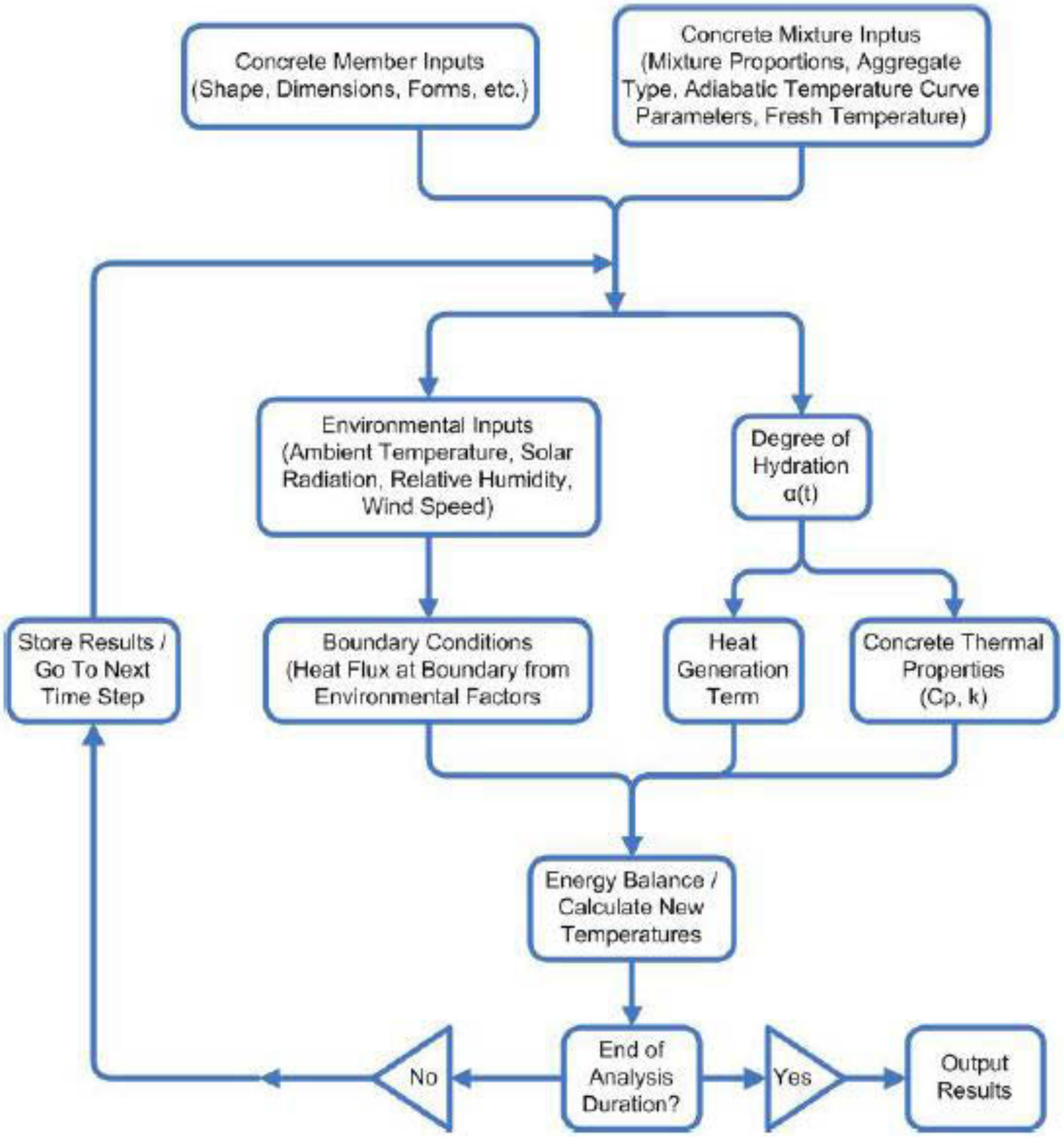


Figure 6.1 ConcreteWorks temperature prediction flowchart (Riding 2007)

6.2.1 Fundamentals of Temperature Prediction

In addition, the thermal energy generated from the process of cement hydration must also be considered. The change in the thermal energy in the placement, resulting from the transfer and generation of thermal energy, causes a change in the temperature of the concrete with time (Riding 2007). Therefore, to analyze the thermal development of mass concrete placements, conduction, convection, evaporation cooling, radiation, irradiation, and cement hydration must be considered to quantify the thermal energy entering (E_{in}), exiting (E_{out}), and being generated ($E_{generated}$) in the volume.

To approximate the thermal development of mass concrete placements, the finite element or finite difference methods may be utilized. In this work, the finite difference method was employed to numerically evaluate the variation in temperature inside a mass concrete structure. In this method, a volume of concrete is divided into sufficient small volumes with constant thermal properties, as shown by Figure 6.2. For each volume, the energy conservation law is used to relate the temperature at each corner of this volume to the physical properties of the volume. The relations for each individual volume were then assembled using equilibrium among adjacent volumes to express the behavior of the entire structure.

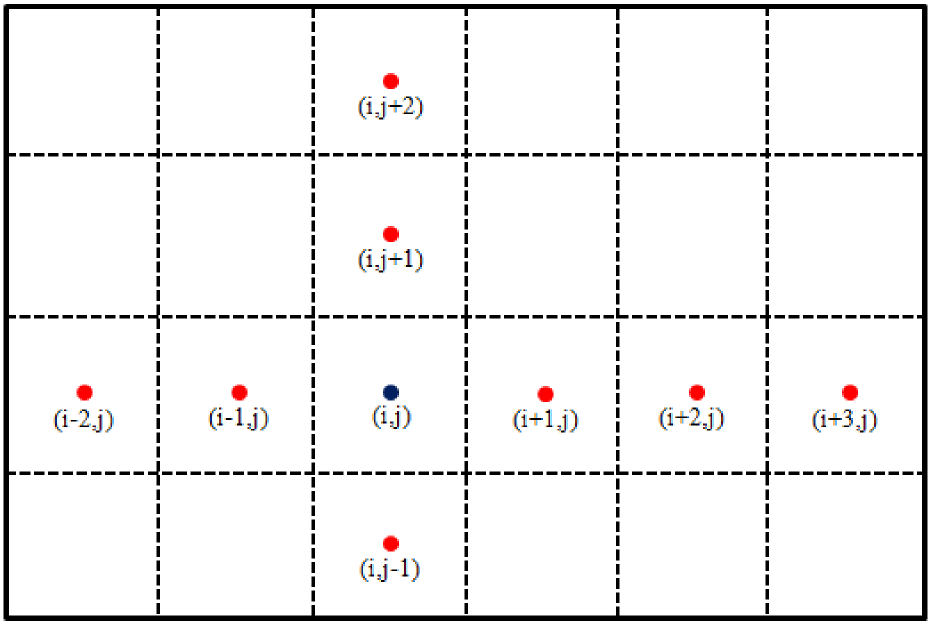


Figure 6.2 Two dimensional finite difference model

To analyze the temperature of a volume at discrete points with time, the law of conservation of energy is utilized. Figure 6.3 illustrates one element that experiences a change in thermal energy. The law of conservation of energy states that the thermal energy entering an volume (E_{in}), plus the thermal energy generated in the volume ($E_{generated}$) due to cement hydration must be equal to the change in the stored energy in the volume (ΔU), plus the energy exiting the volume (E_{out}), i.e.;

$$E_{in} + E_{generated} = \Delta U + E_{out} \quad 6.1$$

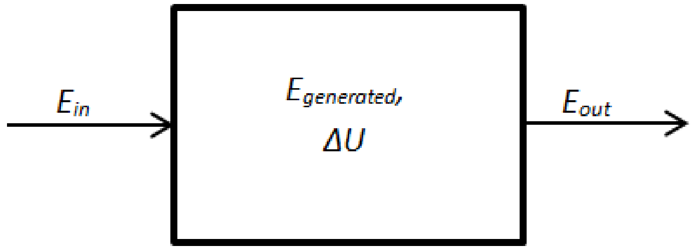


Figure 6.3 Law of conservation of energy

To further explain the relation shown in Equation 6.1, let us consider heat transfer through a volume with convection and one dimensional conduction as shown by Figure 6.4. Notice that Figure 6.4 represents a volume that is subjected to convection at the exposed surfaces of the element (top, bottom, front, and back surfaces). Additionally, the element is subjected to heat generation for the entire volume, and conduction on the surfaces in contact with the other elements (left and right surfaces). For the conditions provided in Figure 6.4, an equation may be developed to describe an element with heat generation throughout the volume, conduction in one dimension, and convection at the exposed surfaces as shown in Equation 6.2. Notice convection and conduction cannot occur on the same surfaces. From Equation 6.2, the term $q_xA\ dt$ pertains to the heat conducted at surface x, $QA\ dx\ dt$ is the heat generated in the volume, $c(\rho A\ dx)\ dt$ is the change in the stored energy, $q_{x+dx}\ A\ dt$ is the heat conducted at surface $x+dx$, and $q_hP\ dx\ dt$ is the heat transferred by convection.

$$q_xA\ dt + QA\ dx\ dt = c(\rho A\ dx)\ dt + q_{x+dx}\ A\ dt + q_hP\ dx\ dt \qquad 6.2$$

Where: q_x = the heat conducted into the volume at the surface x

q_{x+dx} = the heat conducted out of the volume at the surface $x + dx$

t = time

Q = the internal heat generated per unit time and unit volume

A = the cross-sectional area perpendicular to heat flow

c = the specific heat

ρ = the mass density

q_h = heat flow by convection

P = the perimeter of the areas that experience convection i.e.; $(2dy + 2dz)$

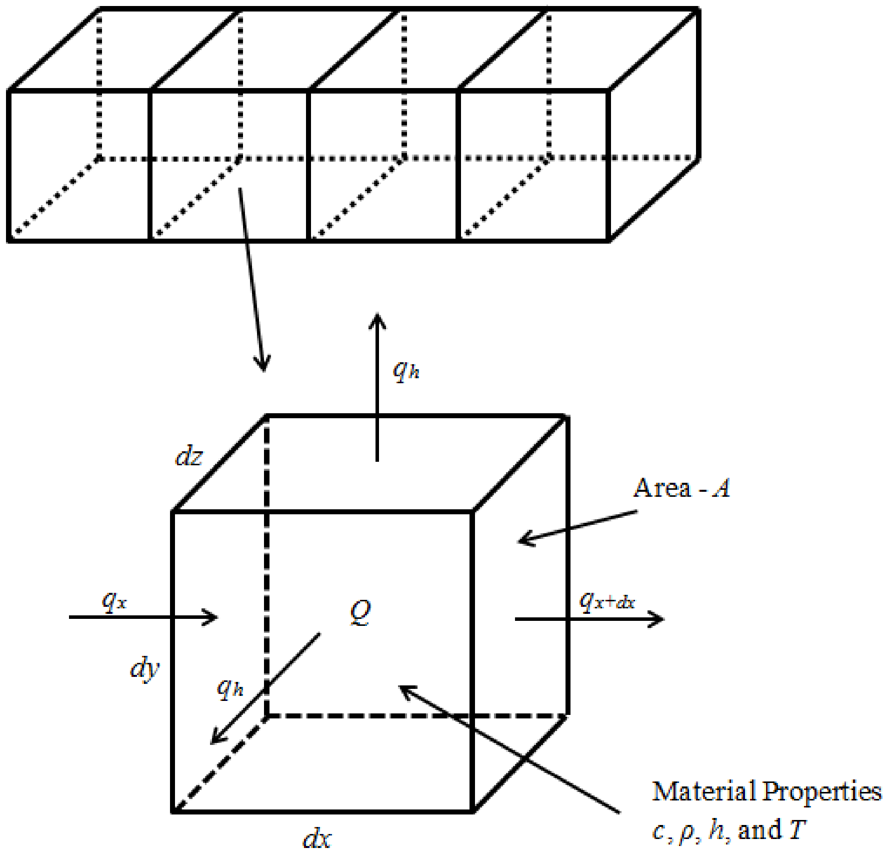


Figure 6.4 Heat transfer through a volume with convection and one dimensional conduction

Through the use of Fourier's Law of Heat Conduction, the heat conducted through the surfaces of the volume may be evaluated as shown by Equation 6.3 and 6.4. Additionally, through the use of the Taylor series expansion, shown by Equation 6.5, with the retention of only higher order terms, the heat conducted through the surface of the element at surface *x+dx* may be simplified to Equation 6.6 (Logan 2011).

$$q_x = -K_{xx}\frac{dT}{dx} \qquad 6.3$$

$$q_{x+dx} = -K_{xx}\frac{dT}{dx}\bigg|_{x+dx} \qquad 6.4$$

Where: K_{xx} = the thermal conductivity in the x direction

T = temperature

$\frac{dT}{dx}$ = the temperature gradient in the x direction

$$f_{x+dx} = f_x + \frac{df}{dx}dx + \frac{d^2f}{dx^2}\frac{dx^2}{2} + \cdots \quad 6.5$$

$$q_{x+dx} = -\left[K_{xx}\frac{dT}{dx} + \frac{d}{dx}\left(K_{xx}\frac{dT}{dx}\right)dx\right] \quad 6.6$$

By substituting Equations 3 and 5 into Equation 2, dividing the equation by $A\ dx\ dt$ and simplifying, the one-dimensional heat transfer equation may be described as shown by Equation 6.7.

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial T}{\partial x}\right) + Q = \rho c\frac{\partial T}{\partial t} \quad 6.7$$

To solve for the temperature at discrete locations in the volume, the minimum potential energy method is utilized. The minimum potential energy equation is shown by Equation 6.8, where π is the total potential energy, U is the energy stored in the volume, and the Ω terms are the potential energy applied to the volume.

$$\pi = U + \Omega_Q + \Omega_q + \Omega_h \quad 6.8$$

For one-dimensional heat transfer, the minimum potential energy equation terms may be described as shown in Equations 6.9-6.12. The Ω_Q, Ω_g, Ω_h terms describe the internal heat generation, heat conduction, and heat convection of the volume respectively. In the following equation, g denotes the temperature gradient between two discrete points, shown by Equation 6.13. Additionally, $(T\text{-}T\infty)$ denotes the difference in temperature between the volume and the surrounding conditions.

$$U = \frac{1}{2}\iiint[K_{xx}(\mathrm{g})^2]dV \quad 6.9$$

$$\Omega_Q = -\iiint QT\, dV \qquad 6.10$$

$$\Omega_q = -\iint qT\ dS \qquad 6.11$$

$$\Omega_h = \frac{1}{2}\iint h(T - T_\infty)^2 dS \qquad 6.12$$

$$\{g\} = \left\{\frac{dT}{dx}\right\} \qquad 6.13$$

The principal of minimum potential energy states that an equilibrium exists when t_i defines a structure state when π_p is equal to zero. In other word, the concept of minimum potential energy is to determine a minimum stationary value of the total potential energy π_p (Logan 2011). To solve for the temperature at each node, the derivative of the minimum potential energy equation is taken with respect to the temperature at each discrete point, shown by Equation 6.14, and is subsequently set to zero, shown by Equation 6.15.

$$\delta\pi_p = \frac{\partial\pi_p}{\partial t_1}\delta t_1 + \frac{\partial\pi_p}{\partial t_2}\delta t_2 + \frac{\partial\pi_p}{\partial t_3}\delta t_3 + \cdots + \frac{\partial\pi_p}{\partial t_n}\delta t_n \qquad 6.14$$

$$\frac{\partial\pi}{\partial\{t_i\}} = 0 \qquad 6.15$$

The partial derivative of Equation 6.15 yields a set of equations that relates the structural conductivity matrix $[k]$, and nodal temperature matrix $\{t\}$, to the nodal force matrix $\{f\}$, as shown by Equation 6.16. The nodal forces are generated by the thermal loadings that an element is subjected to.

$$[k]\{t\} = \{f\} \qquad 6.16$$

Equation 6.16, for each of the finite difference volumes, is then assembled to represent the behavior of the entire structure. Next, one must provide the boundary conditions, i.e., the temperature surrounding the exposed surfaces.

The above described process can be expanded to three-dimensional heat conduction, similarly to the above one-dimensional heat conduction process. The three-dimensional conduction heat transfer equation is given by Equation 6.17.

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial T}{\partial x}\right)+\frac{\partial}{\partial x}\left(K_{yy}\frac{\partial T}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_{zz}\frac{\partial T}{\partial z}\right)+Q=\rho c\frac{\partial T}{\partial t} \qquad 6.17$$

6.3 WB I-80 OVER THE MISSOURI RIVER BRIDGE

6.3.1 Overview

The WB I-80 over the Missouri River Bridge consisted of 27 different mass concrete elements, as defined by the Iowa DOT mass concrete developmental specification (DS-09047). The case study consisted of the analysis of 21 elements mass concrete elements. Six elements were unable to be analyzed because cooling pipes were utilized, and ConcreteWorks cannot analyze placement with cooling pipes, or the provided thermal data was not sufficient to provide accurate results or a valid comparison.

6.3.2 Inputs Overview

The construction of the WB I-80 over the Missouri River Bridge was completed prior to the start of this research. The thermal data from the construction of the mass concrete elements was provided by the Iowa DOT. The data included the name of the element, placement date, placement start time, placement completion time, and whether post cooling was utilized. Additionally, the thermal data provides hourly temperature readings of the air temperature, center temperature of the placement, top surface temperature of the placement, and side surface temperature of the placement.

To identify the concrete mix proportion, construction practices, and environmental conditions in which the elements were placed, a survey of information was conducted. The surveyed documents included examination of the bridge plans, thermal data from the bridge construction, pictures of the construction, thermal control plans, and mix designs. Additionally, to identify how the placements were constructed, interviews were conducted with personnel who worked on the projected, including contractors, project managers, contractor field engineers, and Iowa DOT inspectors.

From the documents and interviews, a general understanding of the concrete mix proportion, construction parameters, and environmental conditions of each placement was developed. The input parameters used to complete the thermal analysis were developed to model the actual conditions as accurately as possible with the information provided.

The development and values of the inputs used to complete the case study in ConcreteWorks is discussed in the following sections. The inputs are divided into three sections; concrete mix proportion, construction parameters, and environmental conditions.

6.3.3 Concrete Mix Proportion Inputs

The concrete mix proportion inputs include mixture proportion inputs, material properties, and mechanical properties as defined by ConcreteWorks.

6.3.3.1 Mixture Proportion Inputs

Each placement on the project utilized the same concrete mix proportion. The concrete mix proportion inputs were developed based on the mix proportion provided by the Ready Mixed Concrete Co. of the Lyman-Richey Corporation, shown in Table 6.1.

Table 6.1 Ready Mixed Concrete Co. mix design for WB I-80 over the Missouri River Bridge

IOWA MASS CONCRETE (5000 psi) w/SLAG

Cement, IPF	420	lbs.	2.28
Slab GGBFS	207	lbs.	1.13
Water (263#)	0.42	lb./lb.	4.21
Class V Sand-Gravel	1586	lbs.	9.70
#557 Limestone	1322	lbs	7.93
Air Content	6.5	%	1.75
Water Reducer	3 oz./	100#	.00
High Range Water Reducer	4-8 oz./	100#	.00
			27.00

Cement type IPF is a blended cement that contains approximately 75% type I cement and 25% class F fly ash by weight. The largest factor of fly ash affecting the heat generation of concrete is the lime or CaO content. Class F fly ash is generally defined as having a CaO percentage of less than ten percent. The percentage of CaO used in the case study is 8.7 percent, which is the value provided by Headwaters Resources, one of the main suppliers of fly ash in Iowa (Headwaters Resources 2005).

Slag is available in three different grades; 80, 100, and 120, which identify the rate of strength gain with grade 80 being the lowest. Grade 80 slag is not commonly used in general concrete construction. ConcreteWorks assumes a slag grade of 120, which is a reasonable assumption for the project. Additionally, the water reducing agents are assumed to be type F naphthalene high-range water reducer. The concrete mix proportion inputs used for all of the mass concrete elements on the WB I-80 over the Missouri River Bridge as used in ConcreteWorks are listed in Table 6.2.

Table 6.2 Mixture proportion inputs WB I-80 over the Missouri River Bridge

Input	Units	Value
Cement content	lb/yd^3	315
Water content	lb/yd^3	264
Course aggregate content	lb/yd^3	1322
Fine aggregate content	lb/yd^3	1586
Air content	%	6.5
Class F fly ash	lb/yd^3	105
Class F fly ash CaO	%	8.7
Grade 120 slag	lb/yd^3	207
Chemical admixture input	-	Water reducer*

*Napthalene high-range water reducer (type F)

6.3.3.2 Material Property Inputs

The material properties of the concrete are dependent on the mix proportion of the concrete, therefore all the mass concrete elements have the same material properties. The Bogue calculated values were provided by the Ash Grove Cement Company's Louisville, Nebraska plant for type I/II cement. The values were calculated by A.S.T.M test method C114 and represent the average values for cement produced between May 1st and May 31st of 2010. The values as imputed into ConcreteWorks are listed in Table 6.3.

Table 6.3 Ash Grove Cement Company type I/II cement Bogue calculated values (Ash Grove Cement Company 2010)

Compound	Value (%)
C3S	59.73
C2S	13.25
C3A	6.05
C4Af	9.46
Free CaO	0.9
SO3	3
MgO	2.97
Na2O	0.13
K2O	0.63

The coarse aggregate type is listed in the Ready Mixed Concrete Co. mix design as limestone. The fine aggregate type is siliceous river sand, which is the most commonly used fine aggregate type in the area. Typical Iowa concrete has a coefficient of thermal expansion in the range of 4.1-7.3 (10^{-6}/°F) (Wang 2008). The analysis utilized the value of a 4.1.

Table 6.4 shows the material properties used to model all of the elements from the WB I-80 over the Missouri River Bridge. The values that are denoted as ConcreteWorks default values are believed to accurately represent the actual material properties. Additionally, the cement hydration properties were not altered from the ConcreteWorks default values.

Table 6.4 Material property inputs for WB I-80 over the Missouri River Bridge

Input	Value
Cement Type	Type I/II
Blaine	371.5 m2/kg[a]
Tons CO2	0.9[a]
Bogue Calculated Values	Ash Grove I/II[b]
Coase Aggreagate Type	Limestone
Fine Aggregate Type	Siliceous River Sand
CTE	$4.1*10^{-6}$
Concrete k	1.6 BTU/hr-ft/°F[a]
Combined Aggregate Cp	0.20 BTU/lb/°F[a]

[a]denotes ConcreteWorks default value

[b]denotes values provided in Table 6.3

6.3.3.3 Mechanical Property Inputs

The mechanical properties were assumed to be the same for all the elements on WB I-80 over the Missouri River Bridge. The mechanical property inputs for ConcreteWorks include the maturity function, equivalent age elastic modulus inputs, equivalent age splitting tensile strength inputs, and early age creep parameters. This case study utilizes the ConcreteWorks default values for all inputs expect for the maturity function.

The maturity was defined using the logarithmic Nurse-Saul strength method. The Nurse-Saul logarithmic equation is shown by equation 4.1.

$$Sm = a + b \log(M) \qquad 4.1$$

Where:

Sm = is the strength of the concrete

a = strength for the maturity index $M = 1$

b = slope of the line

M = maturity index

The Nurse-Saul equation relates the concrete compressive strength with the average maturity index of the concrete. The logarithmic equation provides a simplistic relationship for strength and maturity by utilizing a straight line to represent the maturity function on a logarithmic scale (Carino and Lew 2001).

The constants, a and b, used to model all of the elements for the WB I-80 over the Missouri River Bridge was taken as the average value of a and b calculated from the thermal results for each individual placement. The constants for each individual placement were determined from the thermal, maturity, and strength development data, and are shown in Table 6.5. These values were averaged to determine the values used in each analysis, a = -9,609.7 psi and b = 3,450.1 psi/°F/hr.

Table 6.5 Calculated Nurse-Saul constants for each placement for WB I-80 over the Missouri River Bridge

Pier	Element	a (psi)	b (psi/°F/hr)
1	Footing	-9691.2	3462.8
1	Stem/Column	-5371.1	2077.6
1	Cap	-5038	1947.3
2	Footing	-8205.5	3030.6
2	Stem	-10908	3850.9
2	Column	-6658.9	2496
2	Cap	-8536.5	3135.6
3	Footing	-11894	4148.2
3	Stem	-11806	4153.6
3	Column	-9140.3	3272.6
3	Cap	-9197.9	3345.1
4	Footing	-9072.1	3311.2
4	Stem	-11089	3928.2
4	Column	-8592.8	3169.6
4	Cap	-8381.3	3076.1
5	Footing	-11324	3956.2
5	Stem	-12101	4166
5	Column	-9024.1	3308.1
5	Cap	-9462.4	3438.9
6	Footing	-11989	4223.7
6	Column	-12213	4253.5

6.3.4 Constriction Parameter Inputs

The construction parameter inputs include the general inputs, shape inputs, dimension inputs, and construction inputs.

6.3.4.1 General Inputs

The category of general input includes units, placement date, placement time, analysis setup, state, and city. The general convention for units in the United States is English units. The location of all the placements is taken as Omaha, NE, where the bridge was actually constructed. The placement dates and times were provided by the contractors with the thermal data for each placement, and are listed in Table 6.6. Placement start times that do not fall on the hour are rounded up to the nearest hour, as required by ConcreteWorks.

Table 6.6 Placement date and time for each element of the WB I-80 over the Missouri River Bridge

Pier	Element	Date	Placement Start Time
1	Footing	10/20/08	9:15 AM
1	Stem/Column	12/4/08	9:45 AM
1	Cap	1/23/09	10:30 AM
2	Footing	11/19/08	10:30 AM
2	Stem	1/9/09	12:00 PM
2	Column	2/18/09	8:30 AM
2	Cap	3/20/09	9:00 AM
3	Footing	10/30/08	3:30 PM
3	Stem	11/21/08	9:45 AM
3	Column	1/23/09	9:00 AM
3	Cap	2/25/09	10:00 AM
4	Footing	11/4/08	12:45 PM
4	Stem	12/10/08	9:00 AM
4	Column	3/5/09	8:00 AM
4	Cap	3/20/09	9:00 AM
5	Footing	2/3/09	12:30 PM
5	Stem	2/17/09	9:30 AM
5	Column	3/31/09	8:00 AM
5	Cap	5/5/09	8:00 AM
6	Footing	11/4/08	7:00 AM
6	Column	1/6/09	8:30 AM

6.3.4.2 Shape Inputs

ConcreteWorks provides six different shape options for mass concrete elements including rectangular column, rectangular footing, partially submerged rectangular footing, rectangular bent cap, T-shaped bent cap, and circular columns. To model the elements, all placements defined as footings were imputed as rectangular footings, columns and stems were imputed as rectangular columns, and caps were imputed as rectangular bent caps.

6.3.4.3 Dimension Inputs

The dimensional size of each element as provide by the final design plans of the bridge are listed in Table 6.7. For rectangular columns and rectangular bent caps, ConcreteWorks assumes that the elements are infinitely long and does not allow for the input of the element length. ConcreteWorks also allows for elements that are submerged in water or soil formed. The WB I-80 over the Missouri River Bridge did not have elements that were soil formed or submerged in water. Footings may be analyzed as 2-D or 3-D to account for the length of the elements; our models utilized the 2-D analysis, and assumed the footings were infinitely long.

Table 6.7 Dimensions of elements for the WB I-80 over the Missouri River Bridge

Pier	Element	Depth (ft)	Width (ft)	Length (ft)
1	Footing	4.5	12	43
1	Stem/Column	4	7	-
1	Cap	4	8.25	-
2	Footing	5	15	43
2	Stem	5	19	-
2	Column	5	11	-
2	Cap	5	8.25	-
3	Footing	7.25	27	43
3	Stem	6	16	-
3	Column	6	11	-
3	Cap	6	8.25	-
4	Footing	5	15	43
4	Stem	5	18	-
4	Column	5	11	-
4	Cap	5	8.25	-
5	Footing	6.5	19	43
5	Stem	5	20	-
5	Column	5	11	-
5	Cap	5	9.66	-
6	Footing	5.75	18	46
6	Column	8.33	11	-

6.3.4.4 Construction Inputs

The available construction inputs in ConcreteWorks include the concrete placement temperature, concrete age at form removal, formwork type, formwork color, blanket R value insulation, surrounding temperature, curing method, and subbase material.

The fresh placement temperatures for each placement were not recorded by the contractors. For this case study, the fresh placement temperature was taken to be the average of the initial thermal sensor readings, or the average concrete temperature at hour zero.

The concrete age at form removal was taken as the time from the start of the pouring, which was provided by the contractors, to the end of thermal monitoring, assumed to be the approximate time of form removal.

The type of formwork varied by placement and was not documented; it was assumed that all placements were formed using wood formwork. Similarly, the exact insulation used on each placement was not documented. The thermal control plans generally recommended the use of one insulating blanket with an R value of 2.5. It was assumed that all of the placements had an insulating blanket with an R value of 2.5.

The exact soil temperatures that the placements experienced were also not documented. It was assumed that the soil temperature for the footings was the average ambient air temperature during the 14 days the analysis was conducted over. The average ambient air temperature was provided by the National Weather Service historical data.

From interview with the contractors and the Iowa DOT inspectors, it was determined that none of the placements utilized any curing methods. Therefore, the analysis was conducted without curing for any placements.

From discussion with contractors and the Iowa DOT inspectors, it was determined that the footings were constructed on two different subbase conditions, clay and concrete. For footings that were conducted above the water table, no seal coat was needed and the footing was poured directly onto the clay-like material found in the river bed. Footings that were constructed below that water table required a concrete seal coat to slow water infiltration into the cofferdams. Therefore, the subbase material was determined by examining the plans and identifying if the bottom of the footings were above or below the water table. Stems, columns, and pier caps do not require subbase inputs, as they are assumed infinitely long.

Table 6.8 shows the construction inputs for the WB I-80 over the Missouri River Bridge. In addition to these parameters, the placements are assumed to have used wooden formwork and an insulating blanket with an R value of 2.5.

Table 6.8 Construction inputs for the WB I-80 over the Missouri River Bridge

Pier	Element type	Fresh Placement Temperature (°F)	Concrete Age at Form Removal (hr)	Soil Temperature (°F)	Subbase Material
1	Footing	60.8	198	56	Concrete
1	Stem/Column	55.1	94	-	-
1	Cap	53.9	101	-	-
2	Footing	63.8	289	41	Clay
2	Stem	57.8	236	-	-
2	Column	64.1	118	-	-
2	Cap	61.1	147	-	-
3	Footing	68.6	378	56	Clay
3	Stem	56.6	284	-	-
3	Column	56.8	156	-	-
3	Cap	66.2	166	-	-
4	Footing	68.6	193	66	Clay
4	Stem	56.6	323	-	-
4	Column	66.5	146	-	-
4	Cap	61.4	140	-	-
5	Footing	45.2	347	11	Clay
5	Stem	61.4	316	-	-
5	Column	67.7	148	-	-
5	Cap	67.7	153	-	-
6	Footing	71.6	373	66	Concrete
6	Column	60.5	346	-	-

6.3.5 Environmental Condition Inputs

The environmental condition inputs available in ConcreteWorks include the temperature, wind speed, percent cloud cover, relative humidity, and yearly temperature. To provide for a more accurate case study, the actual weather conditions for each placement were utilized in ConcreteWorks. The maximum and minimum daily temperatures were imputed as provided by the National Weather Service historical data archive for Omaha, Nebraska. All other weather data was set as the default.

6.3.6 Sensor Location Corrections

For each placement constructed, there were sensors installed at three locations, the center of the top surface, the center of the side surface closest to the center, and the center of the placement. The exact location of each sensor used during construction is unknown. It is assumed that the surface sensors were placed at the exact center of the respective surfaces with three inches of concrete cover, and the center sensor was installed at the exact center of the placement. These assumptions were developed for interviews with the contractors and the thermal control plans.

ConcreteWorks calculates the thermal properties of mass concrete placements at discrete points throughout the placement with time. The spacing of the discrete points in the depth and length direction is approximately 4-12 inches depending on the placement. To compare the analysis results generated by ConcreteWorks to the actual results, three points were utilized. The three points correspond to the assumed sensor locations used during construction. As the discrete temperature points do not exactly correspond with the assumed sensor locations, a linear approximation between the surrounding points is used to determine an effective temperature at the desired locations, as shown by Figure 6.5.

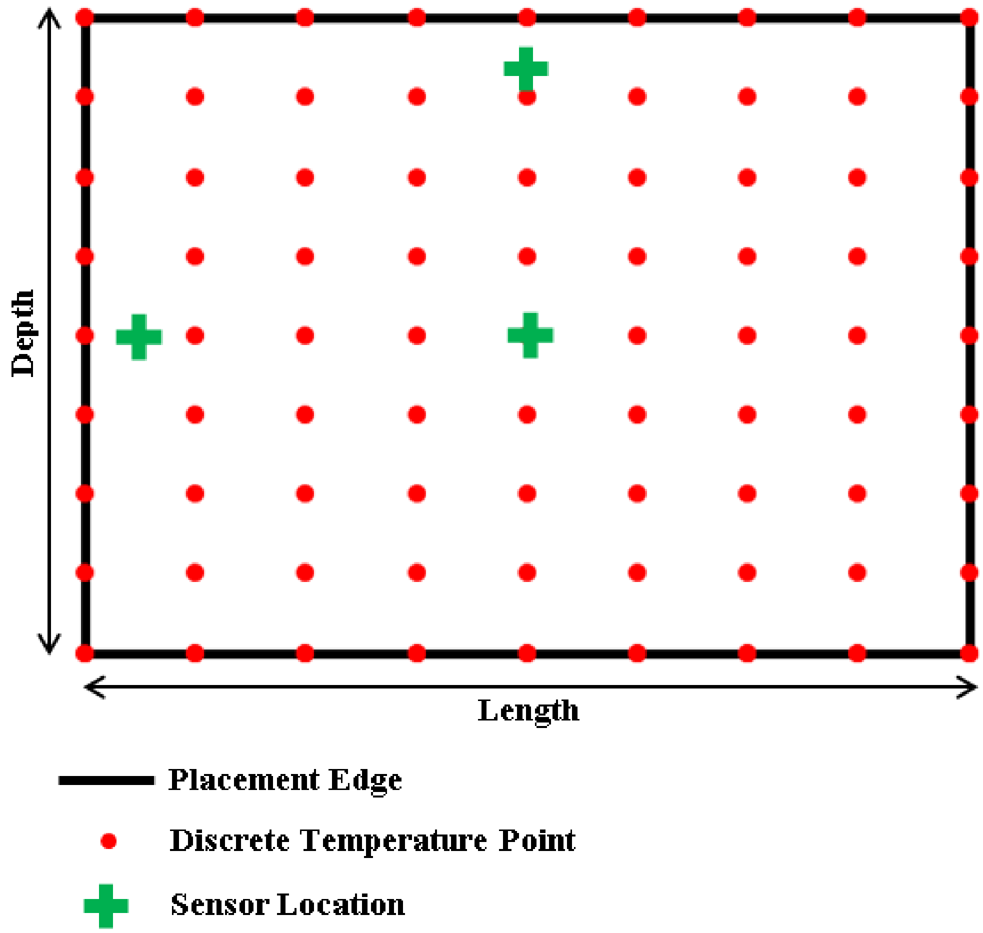


Figure 6.5 ConcreteWorks thermal analysis discrete temperature point layout

6.3.7 Results

The results of the WB I-80 over the Missouri River Bridge case study are listed in Table 6.9, Table 6.10, Table 6.11, and Table 6.12. The results are separated into separate tables for each placement type, footings, stems, columns, and caps, to show the accuracy of ConcreteWorks for each placement type. The table shows the maximum temperature and maximum temperature difference determined from the ConcreteWorks analysis compared to the actual recorded maximum temperature and maximum temperature difference. In addition, the error is also provided with negative error representing an underestimation by ConcreteWorks, and positive error representing an overestimation by ConcreteWorks.

Table 6.9 WB I-80 case study thermal results - footings

	Maximum Temperature (°F)			Maximum Temperature Difference (°F)		
Pier	Actual	ConcreteWorks	Error	Actual	ConcreteWorks	Error
1	131	119.2	-11.8	35.1	23.3	-11.8
2	134.6	126.5	-8.1	35.1	43.5	8.4
3	153.5	147.1	-6.4	59.4	56.3	-3.1
4	142	139.3	-2.7	38	47.8	9.8
5	136.4	101.5	-34.9	53.1	34.4	-18.7
6	156.2	144.6	-11.6	52.2	51.2	-1.0

Table 6.10 WB I-80 case study thermal results - stems

	Maximum Temperature (°F)			Maximum Temperature Difference (°F)		
Pier	Actual	ConcreteWorks	Error	Actual	ConcreteWorks	Error
1	97.7	92.6	-5.1	15.3	22.5	7.2
2	136.4	111.5	-24.9	31.5	40.4	8.9
3	139.1	119.8	-19.3	24.3	36.8	12.5
4	135.5	112	-23.5	40.5	39.8	-0.7
5	140.9	120.3	-20.6	43.2	39.7	-3.5

Table 6.11 WB I-80 case study thermal results - columns

	Maximum Temperature (°F)			Maximum Temperature Difference (°F)		
Pier	Actual	ConcreteWorks	Error	Actual	ConcreteWorks	Error
1	97.7	92.6	-5.1	15.3	22.5	7.2
2	126.5	121.1	-5.4	38.7	37	-1.7
3	128.3	112.9	-15.4	28.7	42.4	13.7
4	140.9	133.1	-7.8	50.4	34.9	-15.5
5	142	130.6	-11.4	39.5	34.3	-5.2
6	150.8	132.2	-18.6	50.4	48.8	-1.6

Table 6.12 WB I-80 case study thermal results - caps

	Maximum Temperature (°F)			Maximum Temperature Difference (°F)		
Pier	Actual	ConcreteWorks	Error	Actual	ConcreteWorks	Error
1	102.2	92.5	-9.7	24.7	22.5	-2.2
2	139.1	126.8	-12.3	49.5	28.5	-21.0
3	146.3	132	-14.3	54.9	44.9	-10.0
4	129.2	127.1	-2.1	37.8	28.5	-9.3
5	140	141.3	1.3	34.2	30.3	-3.9

APPENDIX B shows the results of each individual placement. The graphs show the comparison of the analysis results compared to the actual recorded data for the three discrete sensor locations with time.

6.3.8 Discussion

A statistical analysis of the maximum temperature error and maximum temperature difference error is provided in Table 6.13 and Table 6.14. The statistical analysis includes the range of errors, the error mean, and the standard deviation of the error for both the maximum temperature and the maximum temperature difference for each element type.

Table 6.13 Maximum temperature error statistical analysis of WB I-80 over the Missouri River case study

Element type	Minimum error (°F)	Maximum error (°F)	Error mean (°F)	Error standard deviation (°F)
Footings	-34.9	-2.7	-12.6	11.5
Stems	-24.9	-5.1	-18.7	7.9
Columns	-18.6	-5.1	-10.6	5.5
Caps	-14.3	1.3	-7.4	6.7

Table 6.14 Maximum temperature difference error statistical analysis of WB I-80 over the Missouri River case study

Element Type	Minimum Error (°F)	Maximum Error (°F)	Error Mean (°F)	Error Standard Deviation (°F)
Footings	-18.7	9.8	-2.7	11.1
Stems	-3.5	12.5	4.9	6.7
Columns	-15.5	13.7	-0.5	10.1
Caps	-21.0	-2.2	-9.3	7.4

The results show that under the conditions of this case study, ConcreteWorks underestimates the maximum temperature of a placement; the average error for the maximum temperature of all placements is 12.3°F. On average, ConcreteWorks underestimated the maximum temperature difference of a placement by 1.9°F for all placement types.

The results of the WB I-80 over the Missouri River case study show that ConcreteWorks is capable of predicting the maximum temperature and maximum temperature difference of mass concrete placements for Midwest boarder bridges to a reasonable degree. The WB I-80 case study was also able to confirm the ability of ConcreteWorks to accurately predict the temperature development of distinct points in a mass concrete element, as shown by the individual placement thermal results.

The error in the ConcreteWorks analysis can be largely attributed to assumed construction parameters. The lack of knowledge of the formwork type, insulation properties, and sensor locations, is likely to be largely responsible for the analysis errors. Additional errors in the top surface sensors for the footings and columns arises from the ConcreteWorks assumption that the top surfaces are wet cured, which is not applicable for this project.

6.4 US 34 OVER THE MISSOURI RIVER BRIDGE

6.4.1 Overview

The US 34 Bridge over the Missouri River has several mass concrete elements as defined by the Iowa DOT mass concrete developmental specification (DS-09047). The elements include footings, columns, and caps that were constructed with and without cooling pipes. Through the duration of this research, a total of 19 mass concrete elements have been completed. Of the 19 elements, four used cooling pipes. This case study will examine the 15 placements that did not use cooling pipes, as ConcreteWorks is not capable of analyzing mass concrete placements with cooling pipes. The elements have a least dimension ranging in size from 5.5' to 6.5'. Many of the elements have similar dimensions as several piers have four footings, columns, and caps with the same dimensions. In total, six of the elements are footing, eight are columns, and one is a pier cap.

6.4.2 Inputs Overview

While the construction of the WB I-80 over the Missouri River Bridge was not constructed during the duration of this research, the US 34 over the Missouri River Bridge was partially constructed during the duration of this research. The inputs for the case study were largely developed from first hand observations of the construction of the elements. Other sources of information for the development of the inputs included final bridge design plans, thermal control plans, field data reports, and interviews with the project superintendent.

6.4.3 Concrete Mix Proportion Inputs

The US 34 over the Missouri River Bridge utilized the same concrete mix proportion that was utilized on the WB I 80 over the Missouri River Bridge. As the mix proportion is the same, it is assumed that the material and mechanical properties of the concrete will be similar. For that reason all inputs for the mix proportion, material properties, and mechanical properties that were used to model the previous case study will be used to model the US 34 over the Missouri River Bridge case study.

6.4.4 Construction Parameter Inputs

The largest difference between the two case studies is the construction parameters. The US 34 case study has firsthand reports of the actual construction conditions.

6.4.4.1 General Inputs

The category of general inputs includes units, placement date, placement time, analysis setup, state, and city. The location of the placements on the US 34 case study is taken as Omaha, NE, which is approximately 15 miles north of the actual bridge location. The placement date and start time was supplied by the contactor in the thermal data field report, and is shown in Table 6.15. Placement start times that do not fall on the hour are rounded up to the nearest hour, as required by ConcreteWorks.

Table 6.15 Placement date and time for each element of the US 34 over the Missouri River Bridge

Pier	Element	Placement Date	Placement Start Time
2	Footing - A	3/8/2012	2:00pm
2	Footing - B	3/8/2012	2:00pm
2	Footing - C	3/2/2012	2:00pm
2	Footing - D	3/2/2012	2:00pm
2	Column - A	3/21/2012	9:15am
2	Column - B	3/21/2012	9:15am
2	Column - C	3/12/2012	10:00am
2	Column - D	3/12/2012	10:00am
2	Cap	4/5/2012	2:00pm
3	Footing - C	4/11/2012	2:00pm
3	Footing - D	4/11/2012	2:00pm
3	Column - A	5/3/2012	9:00am
3	Column - B	5/3/2012	9:00am
3	Column - C	4/25/2012	8:00am
3	Column - D	4/25/2012	8:00am

6.4.4.2 Shape Inputs

. To model the elements, all placements defined as footing were imputed as rectangular footings, columns were imputed as circular columns, and caps were imputed as rectangular bent caps.

6.4.4.3 Dimension Inputs

The dimensional size of each element was developed from the final bridge plans provided by the Iowa DOT. The dimensions of each placement required to run the ConcreteWorks analysis is provided in Table 6.16, with the column diameter defined as the width. ConcreteWorks assumes that columns and caps are infinitely long in comparison to the width and depth, therefore do not require a length input. None of the elements analyzed were submerged in water or soil formed. Similarly to the WB I-80, case study the footings are analyzed as two-dimensional elements.

Table 6.16 Dimensions of elements for the US 34 over the Missouri River Bridge

Pier	Element	Depth (ft)	Width (ft)	Length (ft)
2	Footing - A	5.5	12	12
2	Footing - B	5.5	12	12
2	Footing - C	5.5	12	12
2	Footing - D	5.5	12	12
2	Column - A	-	5.5	-
2	Column - B	-	5.5	-
2	Column - C	-	5.5	-
2	Column - D	-	5.5	-
2	Cap	5.5	5.75	-
3	Footing - C	6.5	15	22
3	Footing - D	6.5	15	22
3	Column - A	-	5.5	-
3	Column - B	-	5.5	-
3	Column - C	-	5.5	-
3	Column - D	-	5.5	-

6.4.4.4 Construction Inputs

The construction inputs available in ConcreteWorks include curing method, subbase material, insulating blanket R value, concrete fresh placement temperature, soil temperature, concrete age at formwork removal, formwork type, and formwork color.

It was observed and confirmed through interviews with the contractor that no curing methods were implemented on the placements of interest. No curing methods were defined in ConcreteWorks to complete the analysis.

The Pier 2 and Pier 3 were located outside of the river and did not require a seal coat. It was observed that the soil underlying the concrete was similar to that of clay covered with a layer of gravel. Since the available subbase material options are limited, the clay subbase material was utilized.

Each of the placements that were analyzed for the US 34 case study utilized one layer of insulating blankets attached to the exterior sides of the formwork and on the top of the placements. It was concluded through discussions with the contractors and inspections of the insulating blankets that the effective insulating R value of the blankets was approximately 2.5. To complete the analys,is it was assumed that all placements were covered on all sides with an insulating blanket with an R value of 2.5.

The concrete fresh placement temperature, soil temperature, concrete age at formwork removal, and formwork type as imputed into ConcreteWorks are listed in Table 6.17. The fresh placement temperature of the concrete utilized in the analysis was measured by the contractor at the time when the concrete arrival to the jobsite. The soil temperature used to model the footings was taken to be the average daily temperature over the time in which the placements were thermally monitored.

The time of formwork removal is approximately equal to the time at which the thermal monitoring of the placements ceased. Therefore, the concrete age at formwork removal was taken as the duration of time from the start of the pour to the final thermal reading of the concrete.

The formwork materials were observed and documented for the US 34 case study, unlike the WB I-80 case study. It was observed that the footings utilized wooden formwork and that the columns utilized steel formwork. The cap utilized steel formwork to form the sides of the placement and wood formwork for the bottom. It was also noted that all of the steel formwork was yellow in color.

Table 6.17 Construction inputs for the US 34 over the Missouri River Bridge

Pier	Element Type	Fresh Placement Temperature (°F)	Soil Temperature (°F)	Concrete Age at Form Removal (hr)	Formwork Material
2	Footing - A	55	46.3	157	Wood
2	Footing - B	55	46.3	157	Wood
2	Footing - C	55	38.6	182	Wood
2	Footing - D	55	38.6	230	Wood
2	Column - A	65	-	136	Steel
2	Column - B	65	-	136	Steel
2	Column - C	60	-	112	Steel
2	Column - D	60	-	156	Steel
2	Cap	68	-	138	Steel*
3	Footing - C	64	55	207	Wood
3	Footing - D	64	55	207	Wood
3	Column - A	72	-	91	Steel
3	Column - B	72	-	91	Steel
3	Column - C	72	-	115	Steel
3	Column - D	72	-	115	Steel

* denotes wood was used to form the bottom of the cap

6.4.5 Environmental Conditions Inputs

Similarly to the WB I-80 case study, the US 34 case study utilized the actual ambient air temperatures, as determined from the National Weather Service historical data archive for Omaha Nebraska, to complete the case study. Additionally, all other environmental conditions were left as the default values. The ConcreteWorks default values are calculated from the inputted start date and time of placement, and are based off the previous 30 years historical weather data.

6.4.6 Sensor Location Corrections

While first hand observations of the construction of the mass concrete placements from the US 34 bridge were conducted, the exact sensor locations were unable to be measured as a result of safety. It was observed that, in general, the sensors were placed similar to that described by the WB I-80 case study. To provide the most accurate data, the ConcreteWorks analysis results were adjusted to match the sensor locations described by the WB I-80 case study.

The thermal sensor location for the US 34 bridge varied compared to the WB I 80 bridge resulting from circular columns. The circular columns utilized two sensor locations, one at the center of the column, and one at the side surface. It is assumed that the center sensor will be placed at the exact center of the placement, and the side sensor will be located at an arbitrary location around the perimeter of the column with three inches of concrete cover. For the footings and the caps, it is assumed that one sensor is located at the exact geometric center of the placement, one sensor is located in the center of the top surface with three inches of concrete cover, and one sensor is located in the center of the side surface closest to the center with three inches of concrete cover.

6.4.7 Results

The results of the US 34 over the Missouri River Bridge case study are listed in Table 6.18, Table 6.19, and Table 6.20. The results are broken down into three tables, separating the results by element type. For each placement, the maximum temperature and maximum temperature difference is provided for the actual recorded data, and the ConcreteWorks analysis. In addition, the temperature errors are also listed. A negative error represents an underestimation by ConcreteWorks, and a positive error represents an overestimation.

Table 6.18 US 34 case study thermal results - footings

		Maximum Temperature (°F)			Maximum Temperature Difference (°F)		
Pier	Footing	Actual	ConcreteWorks	Error	Actual	ConcreteWorks	Error
2	A	127.4	118.7	-8.7	21.6	28.2	6.6
2	B	129.2	118.7	-10.5	28.8	28.2	-0.6
2	C	129.2	117.2	-12	41.4	33.3	-8.1
2	D	127.4	117.2	-10.2	30.6	33.2	2.6
3	C	143.6	139.3	-4.3	46.8	40.6	-6.2
3	D	147.2	139.3	-7.9	45	40.6	-4.4

Table 6.19 US 34 case study thermal results - columns

		Maximum Temperature (°F)			Maximum Temperature Difference (°F)		
Pier	Column	Actual	ConcreteWorks	Error	Actual	ConcreteWorks	Error
2	A	134.6	123	-11.6	23.4	37.8	14.4
2	B	138.2	123	-15.2	23.4	37.8	14.4
2	C	129.2	117.7	-11.5	18	32.7	14.7
2	D	134.6	117.7	-16.9	28.8	32.7	3.9
3	A	147.2	117.7	-29.5	28.8	32.7	3.9
3	B	149	117.7	-31.3	36	37.7	1.7
3	C	143.6	141.6	-2	16.2	43.3	27.1
3	D	150.8	141.6	-9.2	9	43.4	34.4

Table 6.20 US 34 case study thermal results - cap

	Maximum Temperature (°F)			Maximum Temperature Difference (°F)		
Pier	Actual	ConcreteWorks	Error	Actual	ConcreteWorks	Error
2	138.2	121.5	-16.7	36	47.2	11.2

A comparison of each individual placement with time is provided in APPENDIX C. The graphs show the comparison of the analysis results compared to the actual recorded data for the three discrete sensor locations with time.

6.4.8 Discussion

A statistical analysis of the temperature prediction error was developed for the maximum temperature and maximum temperature difference as shown in Table 6.21 and Table 6.22 respectively. The statistical analysis includes the range, mean, and standard deviation of the temperature prediction error. The statistical analysis is separated by element type. For the cap element type, the statistical analysis is arbitrary, as only one cap was analyzed.

Table 6.21 Maximum temperature error statistical analysis of US 34 over the Missouri River case study

Element Type	Minimum Error (°F)	Maximum Error (°F)	Error Mean (°F)	Error Standard Deviation (°F)
Footings	-12.0	-4.3	-8.9	2.7
Columns	-31.3	-2.0	-17.7	25.8
Caps	-16.7	-16.7	-16.7	-

Table 6.22 Maximum temperature difference error statistical analysis of US 34 over the Missouri River case study

Element Type	Minimum Error (°F)	Maximum Error (°F)	Error Mean (°F)	Error Standard Deviation (°F)
Footings	-8.1	6.6	-1.7	5.6
Columns	1.7	34.4	14.3	13.7
Caps	11.2	11.2	11.2	-

The results of the US 34 case study confirms that ConcreteWorks generally underestimates the maximum temperature of a placement for the given case study. On average for all placement types, the average maximum temperature error is 13.2°F.

Similar to the WB I-80 case study, ConcreteWorks both over and under estimates the maximum temperature difference compared to the actual field data. On average for all placements types, the average maximum temperature difference prediction error is 6.9°F.

The results of the US 34 over the Missouri River Bridge case study show that ConcreteWorks is capable of predicting the maximum temperature and maximum temperature difference of placements to a reasonable degree. Additionally, the results show that ConcreteWorks is capable of predicting the thermal development of placements at discrete locations with time to a reasonable degree. The results of the US 34 over the Missouri River Bridge case study confirm the results of the WB I-80 over the Missouri River Bridge case study.

CHAPTER 7. SENSITIVITY STUDY

7.1 INTRODUCTION

The early age development of mass concrete is affected by numerous mix proportion, construction, and environmental factors. To properly design and construct a mass concrete element, it is necessary to have an understanding of how each parameter affects the development of the placement.

The purpose of this chapter is to investigate parameters believed to have the largest effect on the development of mass concrete placements, typical of Midwest boarder bridges. The parameters were selected through a literature review of common practices used in the United States to reduce the risk of thermal damage. ConcreteWorks was utilized to explore thermal effects of the selected parameters. The parameters that were investigated in this study and the classification of each are shown in Table 7.1.

Table 7.1 Sensitivity parameter list and classification

Parameter Group	Parameter
Construction	Dimensional size
	Fresh placement temperature
	Curing method
	Forming method
	Formwork removal time
	Subbase
	Sensor Location
Environmental	Ambient air temperature
Mix Proportion	Cement content
	Fly ash substitution
	GGBFS substitution
	Combined class F fly ash and GGBFS substitution

The first section of this chapter describes the baseline inputs that were used to complete the sensitivity study. The second section of the chapter provides the results for each of the parameters. The final section of the chapter discusses the results of the sensitivity study.

7.2 BASELINE INPUTS

The mix proportion, construction, and environmental conditions affect the development of mass concrete placements differently. To capture a characteristic response to a change in a selected parameter, typical baseline conditions were selected in an attempt to model a standard mass concrete placement found on a Midwest boarder bridge. To assure realistic inputs, an element was selected from the WB I-80 over the Missouri River Bridge project. The pier 3 footing was selected to be a reasonable representation of an average mass concrete placement.

The baseline inputs for this sensitivity study are similar to those utilized in the case study for the Pier 3 footing and are listed in Table 7.2 with additional values supplied in Table 7.3 and Table 7.4. The differences between the baseline conditions of the sensitivity study and the inputs used for the case study of the Pier 3 footing are the Nurse-Saul values for the concrete maturity and the sensor location corrections. For the sensitivity study, the Nurse-Saul values used were the values that were calculated from the data from the Pier 3 footing only, not the average value for all placements, as in the case study. Additionally, there were no corrections made for the sensor locations. The maximum temperature and maximum temperature difference in the placement is calculated from all discrete points in the placement. Therefore, the maximum temperature difference results are substantially higher than those from the case study, resulting from the minimum temperature occurring at the surface of the placement without concrete cover.

Table 7.2 Sensitivity study baseline inputs

Group	Input	Baseline Inputs
Member Type	Member Type	Mass Concrete
General	Placement Time	3:30 PM
	Placement Date	10/30/2008
	Life Cycle Duration	75 years
	Location	Omaha, NE
Shape	Shape	Rectangular Footing
Dimensions	Width	27'
	Length	43'
	Depth	7.25'
	Sides	NA
	Analysis	2D
Mix Proportion	Cement Content	315 lb/cy
	Water Content	264 lb/cy
	Coarse Aggregate	1322 lb/cy
	Fine Aggregate	1586 lb/cy
	Air Content	6.50%
	Class C Fly Ash	0 lb/cy
	Class F Fly Ash	105 lb/cy
	CaO%	8.70%
	GGBFS	207 lb/cy
	Admixture	Napthalene High Range Water Reducer
Material Properties	Cement Type	I/II
	Blaine	371.5m^2/kg
	Tons CO2/Tons Clinker	0.9
	Bogue Values	Ash Grove Type I/II[a]
	Coarse Aggregate	Limestone
	Fine Aggregate	Siliceous River Sand
	Hydration Calculation Properties	Default
	CTE	4.1*10^-6 /°F
	Concrete k	1.6 BTU/hr/ft/°F
	Aggregate Cp	0.2 BTU/lb/°F
Mechanical	Maturity Method	Nurse-Saul
	Nurse-Saul (a)	(-)11894 psi
	Nurse-Saul (b)	4148.2 psi/°F/Hr
	Elastic Modulus	Default
	Splitting Tensile Strength	Default
	Creep	Default
Construction	Fresh Placement Temperature	68.9 degrees F
	Form Removal Time	312 hours
	Forming Method	Wood
	Form Color	Natural Wood
	Blanket R Value	2.5
	Soil Temperature	49 degrees F
	Footing Subbase	Clay
Environment	All	Actual Max/Min for 10/30/08[b]
Corrosion Inputs	All	Default

a – denotes values listed in Table 7.3
b – denotes values listed in Table 7.4

Table 7.3 Ash Grove type I/II Bogue calculated values

Bogue Value	%
C3s	59.73
C2S	13.25
C3A	6.05
C4AF	9.46
Free CaO	0.9
SO3	3
MgO	2.97
Na2O	0.13
K2O	0.63

Table 7.4 Actual maximum and minimum temperature for 10/30/08-11/13/08

Date	Maximum (°F)	Minimum (°F)
10/30/2008	72	40
10/31/2008	70	39
11/1/2008	68	35
11/2/2008	76	48
11/3/2008	79	58
11/4/2008	74	57
11/5/2008	70	47
11/6/2008	49	36
11/7/2008	38	32
11/8/2008	34	28
11/9/2008	38	25
11/10/2008	36	26
11/11/2008	43	34
11/12/2008	39	34
11/13/2008	54	37

7.3 RESULTS

This section contains a description of the range for each parameter used in the sensitivity study and the results for each parameter.

7.3.1 Dimensional Size

The range of dimensions used in the study represents typical mass concrete element sizes. The sensitivity study looked at the effect of a change in depth, width, and length of a placement independently, holding the other dimensions constant. The list of placement dimensions analyzed in the sensitivity study, grouped by the dimension changed, is provided in Table 7.5.

Table 7.5 Dimensional size parameter ranges

Parameter Changed	Depth (ft)	Width (ft)	Length (ft)
Depth	5	27	43
	7.25*	27	43
	10	27	43
	15	27	43
	20	27	43
Width	7.25	10	43
	7.25	20	43
	7.25	27*	43
	7.25	30	43
	7.25	40	43
Length	7.25	27	20
	7.25	27	30
	7.25	27	40
	7.25	27	43*
	7.25	27	50

* denotes baseline conditions

The 14 day maximum temperature and maximum temperature difference as calculated by ConcreteWorks is shown in Table 7.6. The results show that typically an increase in the dimension of the placement increases both the maximum temperature and maximum temperature difference of the placement. However, there was no increase in either the maximum temperature or maximum temperature for an increase in width over 27 feet. Additionally, the length of the placement had no effect on the temperature development of the placement.

Table 7.6 Dimensional size sensitivity study results

Parameter Changed	Dimensional Size			Maximum Temperature (°F)	Maximum Temperature Difference (°F)
	Depth (ft)	Width (ft)	Length (ft)		
Depth	5	27	43	136	73
	7.25*	27	43	147	92
	10	27	43	154	108
	15	27	43	162	124
	20	27	43	166	131
Width	7.25	10	43	144	65
	7.25	20	43	147	89
	7.25	27*	43	147	92
	7.25	30	43	147	92
	7.25	40	43	147	92
Length	7.25	27	20	147	92
	7.25	27	30	147	92
	7.25	27	40	147	92
	7.25	27	43*	147	92
	7.25	27	50	147	92

*denotes baseline values

Since the width of the placement, larger than 27 feet, and the length, larger than 20 feet, is excessively large in comparison to the depth of the placement, the element is not affected by an increase in size. The results show that once a dimension reaches a length that is sufficiently larger than the other dimension, there is no affect from increasing said dimension on either the maximum temperature or maximum temperature difference in the placement. As the one dimension increases, the thermal results converge and the dimension may be assumed to be infinitely long. Typically, the depth of the placement is the smallest dimension and will have the largest effect on the thermal development; the width and length of the placement will typically play a lesser role in the thermal development of the placement.

7.3.2 Fresh Placement Temperature

The fresh placement temperature sensitivity study analyzed fresh placement temperatures that are commonly seen in mass concrete construction. A temperature of 40°F was selected as a minimum, which is typically the minimum temperature allowable by state agencies for general construction. A maximum temperature of 90°F was selected to represent the maximum fresh placement temperature, which is typically the maximum seen in general concrete construction. The sensitivity study examined the effect of fresh placement temperature in ten degree increments from 40-90°F.

The maximum temperature and maximum temperature difference results for the range of fresh placement temperatures, as analyzed by ConcreteWorks for the first 14 days after placement, are listed in Table 7.7. The results show that both the maximum temperature and maximum temperature difference increase with an increase in the fresh placement temperature. For the increase in fresh placement temperature from 40-90°F, the maximum temperature and maximum temperature difference increase by 55°F and 31°F respectively. For each degree increase in the fresh placement temperature, the maximum temperature and maximum temperature difference increased on average by 1.1°F and 0.62°F respectively.

Table 7.7 Fresh placement temperature sensitivity study results

Fresh Placement Temperature (°F)	Maximum Temperature (°F)	Maximum Temperature Difference (°F)
40	115	74
50	126	80
60	137	86
68.9*	147	92
70	148	93
80	159	99
90	170	105

* denotes baseline conditions

Fresh placement temperature directly affects the thermal development of a placement by providing initial heat to the placement. Additionally, the rate at which cement hydrates is affected by the temperature of the concrete; the warmer the concrete is, the faster the process of hydration. As the process of hydration is accelerated, heat is generated more rapidly, indirectly increasing the maximum temperature of the placement. Additionally, the increased hydration rate generates larger thermal gradients, resulting from the limited ability of the concrete to dissipate the generated heat in the placement to the surrounding environment.

7.3.3 Curing Method

The curing method sensitivity study considered five different curing methods used in mass concrete construction: no curing method, white curing compound, black plastic, clear plastic, and wet curing blanket. The results of the curing method sensitivity study are shown in Table 7.8, providing the maximum temperature and the maximum temperature difference, as provided by ConcreteWorks analysis.

Table 7.8 Curing method sensitivity study results

Curing Method	Maximum Temperature (°F)	Maximum Temperature Difference (°F)
None*	147	92
White Curing Compound	147	92
Black Plastic	147	92
Clear Plastic	147	92
Wet Curing Blanket	147	77

* denotes baseline condition

The results show that none of the five curing methods have an effect on the maximum temperature of the placement. In addition, no curing method, white curing compound, black plastic, and clear plastic also had no effect on the maximum temperature difference of the placement. Only the wet curing blanket had an effect on the thermal development, reducing the maximum temperature difference by 15°F compared to the other curing methods.

Curing method had no effect on the rate of hydration of the concrete or the temperature of the placement, and in turn had little effect on the maximum temperature of the placement. No curing method, white curing compound, black plastic, and clear plastic provide minimal, if any, insulating value to the exterior surface of the concrete, therefore have no effect on the maximum temperature difference of the placement. The process of wet curing concrete provides additional insulation to the surface of the concrete, resulting from both the blanket itself and the moisture on the surface concrete providing thermal resistance to the surface of the placement. The combined thermal insulating properties of the blanket and water provide a substantial reduction in the maximum temperature difference of the placement.

7.3.4 Forming Method

The forming method sensitivity study looked at the two most common formwork methods, wood and steel, used in mass concrete construction. In addition, the study also considered the effect of the color of steel formwork on the thermal development of mass concrete. The two colors examined include red and yellow formwork.

The 14 day maximum temperature and maximum temperature difference analysis results for the forming method sensitivity study are shown in Table 7.9. The results show that the formwork material and color had no effect on the maximum temperature of a placement. Additionally, steel formwork, both yellow and red colored, had an increased maximum temperature difference compared to that of wood formwork. Wood formwork had a maximum temperature difference 6°F less than that of steel formwork.

Table 7.9 Forming method sensitivity study results

Formwork Material	Formwork Color	Maximum Temperature (°F)	Maximum Temperature Difference (°F)
Natural Wood*	Natural Wood	147	92
Steel	Yellow	147	98
Steel	Red	147	98

* denotes baseline condition

The reduced maximum temperature difference resulting from the use of the wood formwork is largely the result of the thermal conductivity of wood compared to that of steel. Wood provides a larger insulating value and resistance to heat flow compared to steel, retaining more heat at the surface of the concrete, reducing the maximum temperature difference. The wood formwork does not provide enough insulation to increase the maximum temperature compared to that of steel formwork under these conditions.

7.3.5 Formwork Removal Time

The formwork removal time sensitivity study examined formwork removal times in the range of 48 hours to 336 hours in 24 hour increments. The minimum formwork removal time, 48 hours, was chosen to represent the earliest practical time that formwork could be removed in mass concrete construction. Typically, before 48 hours, the concrete does not have sufficient strength for the formwork to be removed. The upper bound of the formwork removal time is 336 hours, equivalent to 14 days, which is the maximum allowable analysis time for ConcreteWorks.

The maximum temperature and maximum temperature difference results for the formwork removal time sensitivity study are shown in Table 7.10. The results show that the maximum temperature was not affected by the formwork removal time, except for the 48 hour, which had a slightly reduced maximum temperature. The results show that the maximum temperature difference was greatly affected by the formwork removal time. For formwork removal times between 48-192 hours, the maximum temperature difference increased with an increase in formwork removal time. In addition, the results show that for formwork removal times of 216-336 hours, the maximum temperature difference decreased with an increase in formwork removal time. The largest maximum temperature difference under these conditions was during hours 192 and 216, with a maximum temperature difference of 113°F.

Table 7.10 Formwork removal time sensitivity study results

Formwork Removal Time (hr)	Maximum Temperature (°F)	Maximum Temperature Difference (°F)
48	145	101
72	147	104
96	147	106
120	147	108
144	147	110
168	147	112
192	147	113
216	147	113
240	147	109
264	147	100
288	147	96
312*	147	92
336	147	77

* denotes baseline condition

The results show that the formwork removal time has no effect on the maximum temperature of the placement except for a formwork removal times of 48 hours. Figure 7.1 shows that the maximum temperature occurs around 85 hours after the element was placed for these conditions. If the formwork removal is to reduce the maximum temperature of the placement it must be removed before the maximum temperature in the placement occurs.

The results show that the formwork removal times of 192 and 216 hours result in the largest maximum temperature difference. Figure 7.1 shows that for the conditions of this sensitivity study, the ambient air temperature noticeably dropped starting at approximately 144 hours after placement. The noticeable drop in the ambient air temperature largely accounts for the increased maximum temperature difference between hours 144 and 216.

The formwork removal time had a lesser effect on the maximum temperature difference of the placement in the time before 192 hours. Between the time of maximum temperature and 192 hours, the maximum temperature of the placement remains relatively constant, and does not noticeably change the maximum temperature difference of the placement.

As the formwork removal time for the placement is increased after 216 hours, the maximum temperature difference of the placement decreases. This is the result of the placement being allowed to gradually cool, shown by the decrease in the maximum temperature.

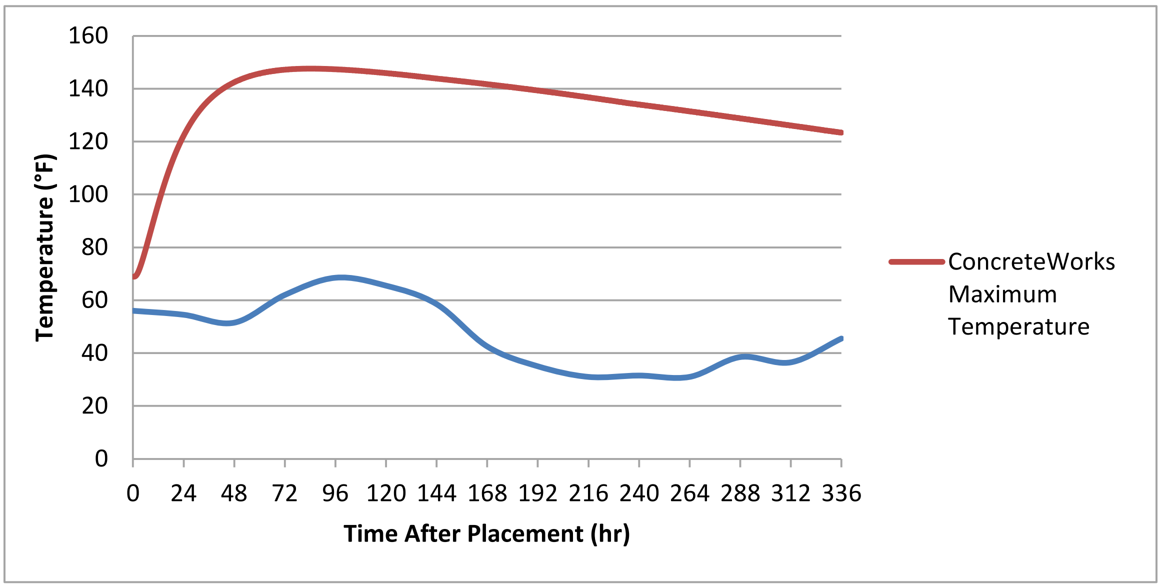


Figure 7.1 ConcreteWorks maximum temperature development and average ambient air temperature with time

7.3.6 Subbase Material

The subbase sensitivity study considered the effect of different subbase materials on the thermal development of mass concrete placements. The sensitivity study examined all subbase materials available in ConcreteWorks to model mass concrete footings. The various subbase materials and the maximum temperature and maximum temperature difference as calculated by ConcreteWorks are listed in Table 7.11.

The results show that the maximum temperature and the maximum temperature difference are both affected by the subbase material. Under the conditions of this sensitivity study, the maximum temperature of the placement ranged from 143-151°F, and the maximum temperature difference ranged from 72-105°F.

Table 7.11 Subbase material sensitivity study results

Subbase Material	Maximum Temperature (°F)	Maximum Temperature Difference (°F)
Clay*	147	92
Granite	144	81
Limestone	145	84
Marble	144	80
Quartzite	143	72
Sandstone	145	82
Sand	151	105
Top Soil	147	91
Concrete	144	79

* denotes baseline condition

The difference in the thermal development is attributed to the thermal properties of the subbase materials. The subbase material properties used by ConcreteWorks to model the placements are listed in Table 7.12. ConcreteWorks does not use a standard set of thermal properties for concrete subbase, but rather assumes the same thermal properties as the concrete being analyzed.

Table 7.12 Subbase material thermal properties (Riding 2007)

Subbase Material	Density (kg/m^3)	Thermal Conductivity (W/m/K)	Specific Heat (J/kg/K)
Clay	1460	1.3	880
Granite	2630	2.79	775
Limestone	2320	2.15	810
Marble	2680	2.8	830
Quartzite	2640	5.38	1105
Sandstone	2150	2.9	745
Sand	1515	0.27	800
Top Soil	2050	0.52	1840
Concrete*	2254	2.77	837

* thermal properties are determined from the concrete mix used in the sensitivity study

The results show that the thermal conductivity of the subbase has the largest effect on the thermal development of the placement. Figure 7.2 shows the maximum temperature results of the subbase sensitivity study with the corresponding thermal conductivity of each subbase. The results show that as the thermal conductivity decreases, the maximum temperature of the placement increases.

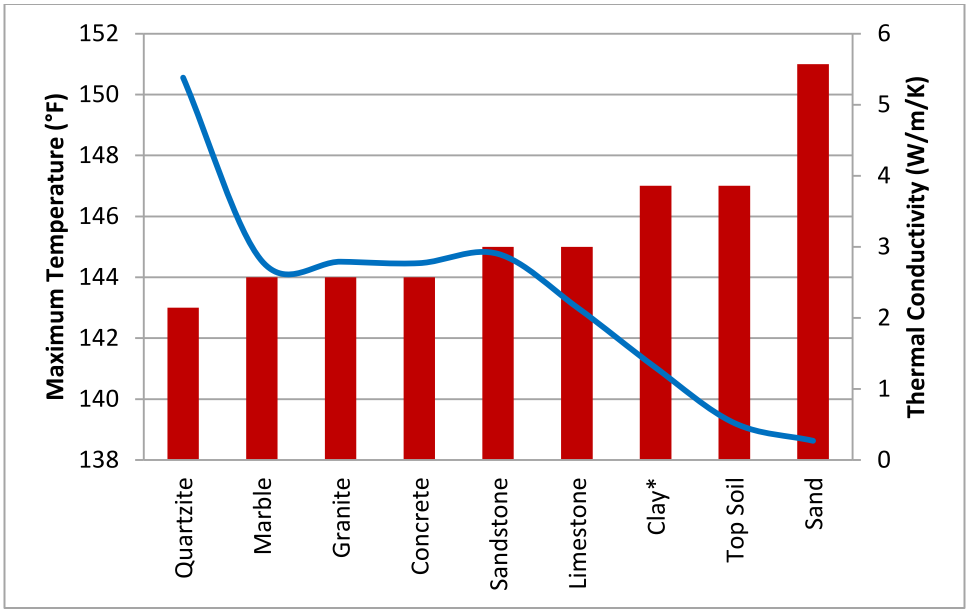


Figure 7.2 Placement temperature vs. subbase material thermal conductivity

7.3.7 Sensor Location

The sensor location sensitivity study was conducted to determine the effect of incorrect sensor placement on the thermal readings. The sensitivity study looked at the three typical sensor locations, center of the top surface, center of the side surface closest to the geometric center, and the geometric center of the placement. Each sensor location was examined to determine the effect of varying levels of error on the thermal readings.

The sensitivity study was conducted by examining the thermal development data of the Pier 3 footing as analyzed by ConcreteWorks. ConcreteWorks provides thermal data for the center cross-section of the placement at five minute time intervals for the entire duration of the thermal analysis. The sensitivity study considered the cross-section with the largest maximum temperature difference, which occurred at hour 336. The data is represented by a contour plot in Figure 7.3 to identify the general thermal gradient pattern of the placement.

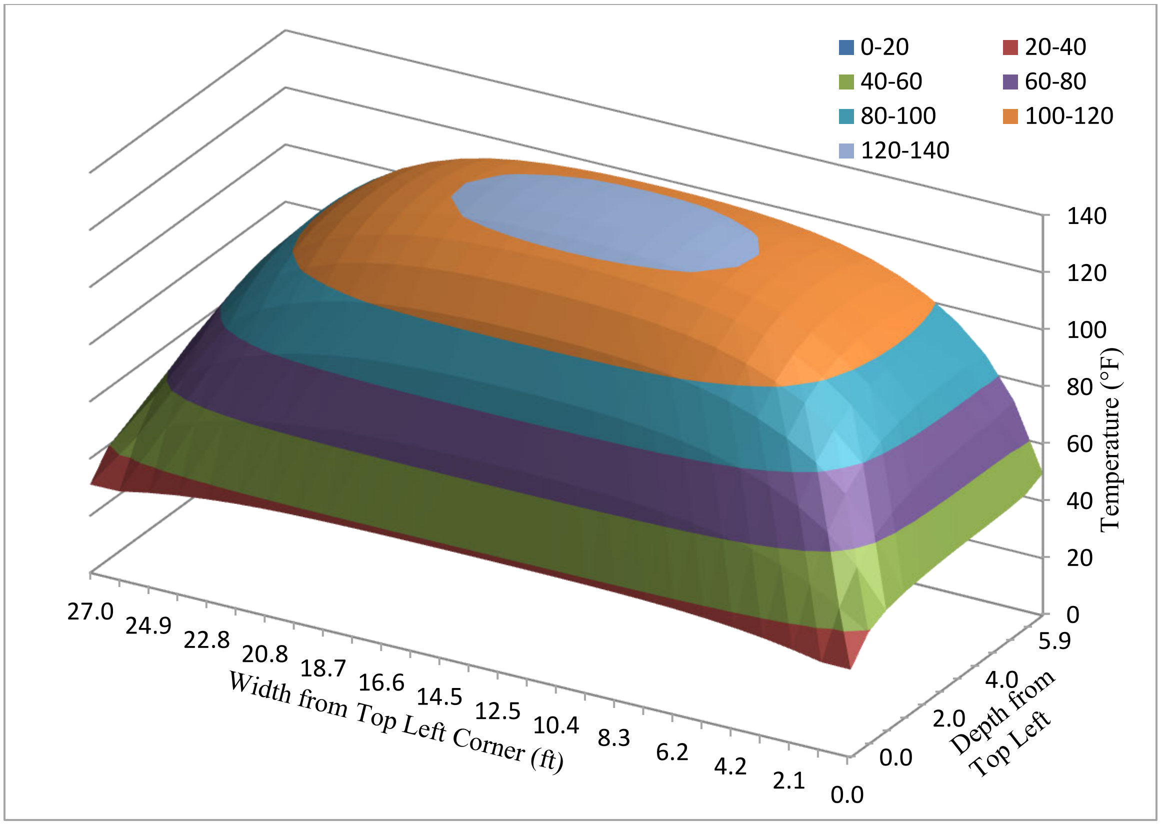


Figure 7.3 Pier 3 footing contour plot at time of maximum temperature difference

To examine the effects of incorrect sensor location, the cross-sectional thermal data was analyzed in the width direction at the center line of the depth for the side surface sensor, the center line of the width in the depth direction for the top surface sensor, and at the center line of the width and depth in the depth direction for the center sensor location. The locations and directions were chosen to have the largest impact with regard to sensor location error. The location of the thermal data utilized to evaluate the sensor location error is given in red in Figure 7.4.

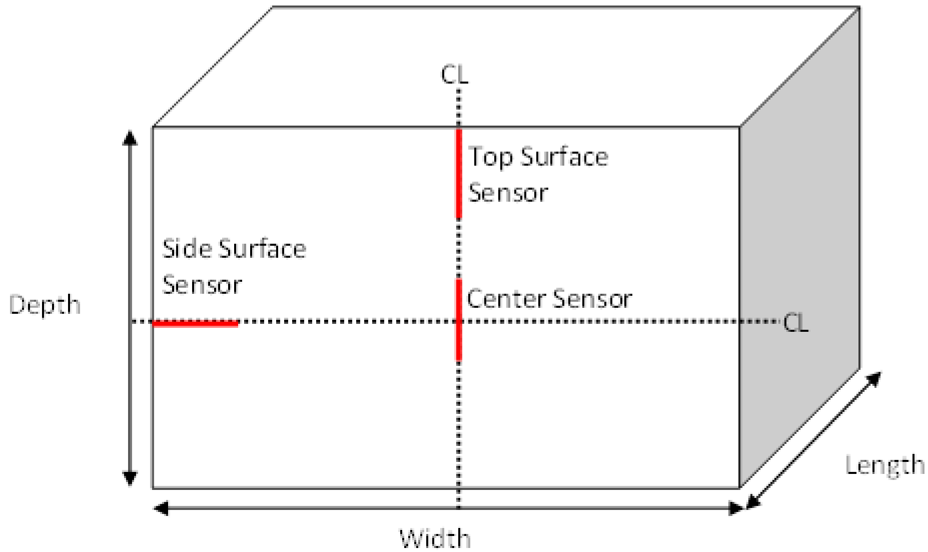


Figure 7.4 Top, side, and center sensor error locations

The baseline conditions for the top and side surface sensors were taken to be three inches in from the outside surface at the corresponding center line. Additionally, the baseline condition for the center sensor is taken to be the intersection of the width and depth center lines. These locations are typical in practice. It is assumed that if the sensors were placed at these locations, the thermal reading errors would be zero.

To evaluate the variance from the baseline conditions, the thermal data from the surface to 15 inches below the surface was utilized to quantify the thermal gradient for the top and side surface sensors. For the center sensor, 12 inches above and below the baseline condition was utilized to quantify the thermal gradient for the center sensor. The discrete thermal data points, falling in the respected ranges, were used to develop second degree polynomial equations for the thermal gradients at each sensor location. The graph of the thermal gradients for each sensor is provided in Figure 7.5, with zero representing the baseline condition. The graph represents sensor locations closer to the surface than the baseline condition as negative numbers, and locations closer to the center of the placement as positive numbers. Additionally, negative temperature errors represent temperature readings larger than that of the baseline conditions, and positive temperature errors represent temperature readings smaller than the baseline conditions.

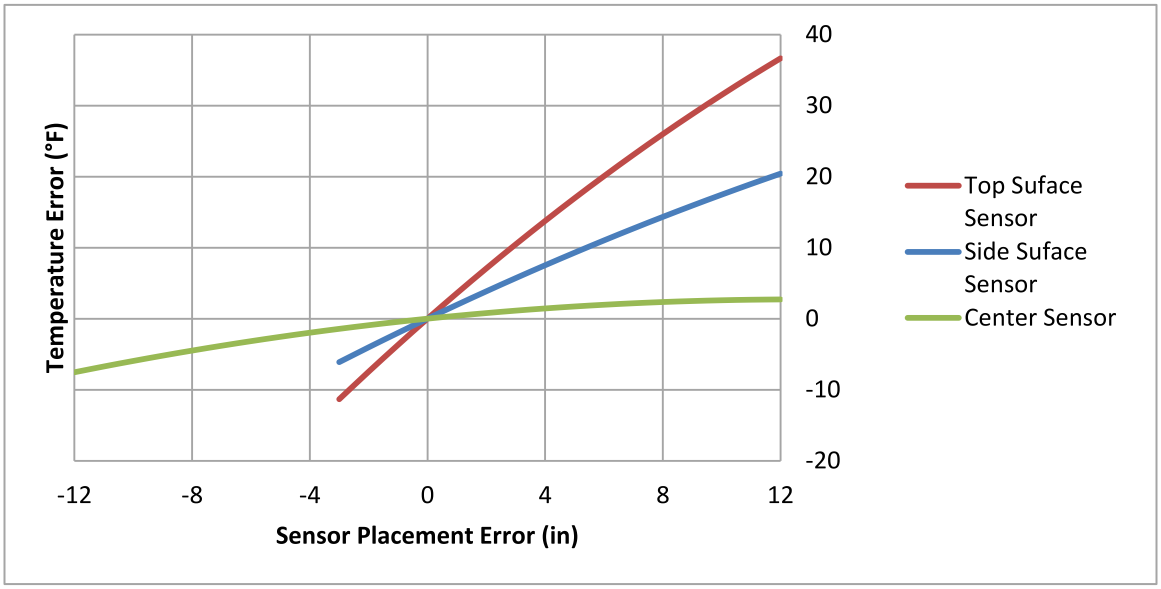


Figure 7.5 Temperature errors for sensor placement errors

The results show that all of the investigated sensor locations are affected by the location. All sensors show a decrease in the thermal reading temperature as the sensor location moves towards the surface, and an increase as the sensor location moves away from the surface of the placement. The increase in the center temperature error with positive sensor placement error is the result of the maximum temperature in the placement not occurring in the exact geometric center of the placement. Due to the relatively large insulating value of the subbase relative to the top surface insulation, the maximum temperature in the placement occurs slightly closer to the bottom of the footing than the top.

The results show that the center sensor has the least amount of temperature error for a given sensor placement error. Additionally, the top surface sensor temperature error is the most affected by a given error in sensor placement. Since the top surface sensor has the largest temperature error for a given sensor placement error, a table is provided to characterize the temperature error of the top surface sensor for a given sensor placement error. Table 7.13 shows how the temperature varies below the surface, along with the temperature error, using a baseline of three inches of concrete cover over the sensor.

Table 7.13 Top surface sensor temperature error by depth placement error

Actual Depth (in)	Depth Error (in)	Actual Temperature (°F)	Temperature Error (°F)
0	-3	38.3	-11.3
1	-2	42.2	-7.5
2	-1	46.0	-3.7
3	0	49.7	0.0
4	1	53.2	3.6
5	2	56.7	7.1
6	3	60.1	10.5
7	4	63.4	13.8
8	5	66.6	17.0
9	6	69.7	20.1
10	7	72.7	23.1
11	8	75.6	26.0
12	9	78.4	28.8
13	10	81.2	31.5
14	11	83.8	34.1
15	12	86.3	36.6

The results show that substantial temperature reading errors may occur if precautions are not taken to accurately locate the sensors in the placement. It is important to note that the maximum temperature of the placement is generally not located at the exact geometric center of the placement, resulting from the difference in the boundary conditions between the top and bottom surfaces of the placements, as shown by the temperature contour plot of Pier 3.

The greatly increased maximum temperature differences computed by ConcreteWorks compared to actual conditions may be largely attributed to the sensor locations. ConcreteWorks computes the maximum temperature difference from the absolute maximum and minimum temperature in the placement. Actual temperature recordings are at discrete locations with a certain amount of clear cover and placement error.

From the cross section data for the Pier 3 Footing, accounting for only three sensor locations with three inches of concrete cover without sensor placement error, the adjusted maximum temperature difference would be 67.9°F. The adjusted maximum temperature difference, as described above, is greatly reduced compared to that of the raw ConcreteWorks maximum temperature difference of 92°F.

7.3.8 Ambient Air Temperature

The ambient air temperature sensitivity study examines the effect of the surrounding ambient air temperature on the thermal development of mass concrete elements. The study examines the ambient temperature of two different placement dates, October 30, 2008 and July 30, 2008. These dates were selected to represent a warm ambient air temperature and a cool ambient air temperature. A winter date was not selected to prevent complications of the concrete freezing. October 30th represents a cool ambient air temperature, where freezing of the concrete is of minimal concern. July 30th is typically one of the warmest times of the year in the Midwest, and was selected to represent the warmest ambient air temperature conditions.

In lieu of using the ConcreteWorks default values for the corresponding placement dates, the actual historical weather data provided by the National Weather Service was imputed. This was done to give a more accurate representation of how real weather conditions affect the thermal development of mass concrete. The daily maximum and minimum temperatures for the day of placement and the 14 subsequent days for each placement are listed in Table 7.14 as inputted into ConcreteWorks.

Table 7.14 Ambient air temperature sensitivity study maximum and minimum temperature inputs

Date	Maximum (°F)	Minimum (°F)	Date	Maximum (°F)	Minimum (°F)
10/30/2008	72	40	7/30/2011	90	69
10/31/2008	70	39	7/31/2011	93	70
11/1/2008	68	35	8/1/2011	91	71
11/2/2008	76	48	8/2/2011	92	68
11/3/2008	79	58	8/3/2011	101	77
11/4/2008	74	57	8/4/2011	90	75
11/5/2008	70	47	8/5/2011	84	67
11/6/2008	49	36	8/6/2011	86	63
11/7/2008	38	32	8/7/2011	89	61
11/8/2008	34	28	8/8/2011	88	66
11/9/2008	38	25	8/9/2011	85	68
11/10/2008	36	26	8/10/2011	87	61
11/11/2008	43	34	8/11/2011	84	67
11/12/2008	39	34	8/12/2011	86	64
11/13/2008	54	37	8/13/2011	93	66

The maximum temperature and maximum temperature difference as calculated by ConcreteWorks for the two ambient air temperature conditions are listed in Table 7.15. The results show that the ambient air temperature has an effect on both the maximum temperature and the maximum temperature difference of the placement. The warmer ambient air temperature of 7/30/08 generated a larger maximum temperature and a reduced maximum temperature difference compared to that of the cooler ambient air temperature of 10/30/08.

Table 7.15 Ambient air temperature sensitivity study results

Placement Date	Maximum Temperature (°F)	Maximum Temperature Difference (°F)
10/30/2008*	147	92
7/30/2008	150	68

* denotes baseline condition

The ambient temperature and maximum temperature development with time, as calculated by ConcreteWorks, is shown in Figure 7.6. The figure shows how ConcreteWorks approximates the ambient air temperature surrounding the placement from the daily maximum and minimum temperautres. Additionally, the graph shows that the maximum temperature is reduced for the lower ambeint air temperature conditions of 10/30/08.

The maximum temperature and ambient air temperature curves show how the maximum temperature difference changes for each ambient air condition. At the time of formwork removal, 312 hours after placement, the surface of the placement will cool to the ambient air temperature. The maximum temperature difference will approach the difference of the maximum temperature and the ambient air temperature. The graph shows that although the element placed on 10/30/08 had a slightly reduced maximum temperature, the ambeient air temperature is greatly reduced compared to that of the placement poured on 7/30/08. The greatly reduced ambient air temperature causes an increase in the maximum temperature difference compared to the placement poured on 7/30/08.

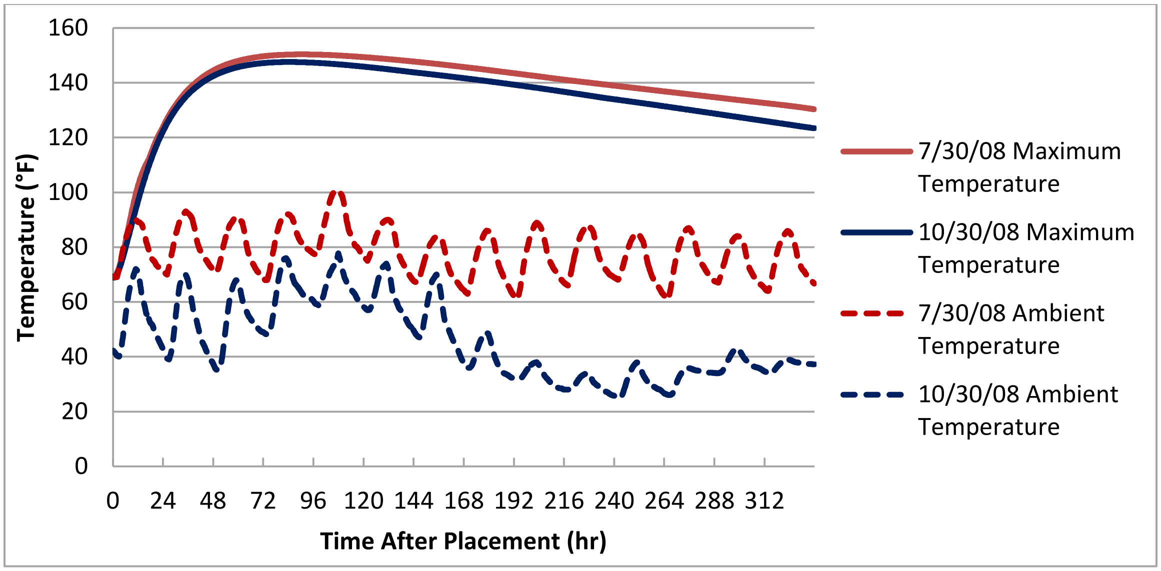


Figure 7.6 ConcreteWorks ambient air temperature and maximum temperature with time after placement

It is important to note that in this study, only the ambient air temperature was varied. In actual application, other parameters will also vary with the ambient air temperature including the fresh placement temperature and soil temperature. The changes in the additional parameters will alter the results in actual practice.

7.3.9 Cement Content

The cement content sensitivity study evaluated the effect of cement content in a concrete mix proportion on the thermal development of mass concrete. The study analyzed cementitious contents in increments of 100 lb/cy ranging from 527-827 lb/cy. Over the range of cementitious content, the class F fly ash and GGBFS contents were held to the baseline conditions of 105 and 207 lb/cy respectively. The change in cementitious content only affected the cement content as shown in Table 7.16.

Table 7.16 Cement content sensitivity study inputs

Total Cementitious Material (lb/cy)	Cement Content (lb/cy)	Class F Fly Ash (lb/cy)	GGBFS (lb/cy)
427	115	105	207
527	215	105	207
627	315	105	207
727	415	105	207
827	515	105	207

The results of the cement content sensitivity study are shown in Table 7.17. The results show that both the maximum temperature and the maximum temperature difference increased with an increase in cement content. For this study, each additional 100lb/cy of cement increased the maximum temperature and maximum temperature difference by approximately 9°F and 6°F respectively. Adding cement increases the heat in the placement due to the additional material undergoing hydration. The additional heat generated in the placement results in an increased maximum temperature, and subsequently an increased maximum temperature difference.

Table 7.17 Cement content sensitivity study results

Cementitious Content (lb/cy)	Maximum Temperature (°F)	Maximum Temperature Difference (°F)
527	136	85
627*	147	92
727	156	98
827	164	103

* denotes baseline condition

7.3.10 Fly Ash Substitution

The fly ash substitution sensitivity study looked at the effect substituting class F and class C fly ash for cement in a concrete mix proportion. The sensitivity study looked at the substitution of fly ash in 10% increments from 0-50% of the total cementitious content. The upper limit of 50% was set to represent typical mass concrete specifications. The total cementitious content of 627 lb/cy was selected to following the previous baselines. Table 7.18 and Table 7.19 show the inputs used to complete the class F fly ash and class C fly ash sensitivity study respectively. No GGBFS was used in the mix proportion in an effort to simply the study.

Table 7.18 Class F fly ash sensitivity study inputs

Class F Fly Ash Substitution (%)	Cement Content (lb/cy)	Class F Fly Ash Content (lb/cy)
0	627	0
10	564	63
20	502	125
30	439	188
40	376	251
50	314	313

Table 7.19 Class C fly ash sensitivity study inputs

Class C Fly Ash Substitution (%)	Cement Content (lb/cy)	Class C Fly Ash Content (lb/cy)
0	627	0
10	564	63
20	502	125
30	439	188
40	376	251
50	314	313

Table 7.20 and Table 7.21 show the results of the sensitivity study for both class F and C fly ash respectively. The results show that both the maximum temperature and maximum temperature difference decreased with the substitution of class F fly ash. Additionally, the substitution of class C fly reduced the maximum temperature of the placement, and the maximum temperature difference slightly.

Table 7.20 Class F fly ash sensitivity study results

Class F Fly Ash Substitution (%)	Maximum Temperature (°F)	Maximum Temperature Difference (°F)
0	154	89
10	148	86
20	142	83
30	136	80
40	131	76
50	125	73

Table 7.21 Class C fly ash sensitivity study results

Class C Fly Ash Substitution (%)	Maximum Temperature (°F)	Maximum Temperature Difference (°F)
0	154	89
10	152	88
20	150	88
30	150	88
40	145	87
50	142	87

Both class F and C fly ash generate less heat during hydration compared to cement. The chemical composition of class F fly ash allows for a larger reduction in the amount of heat generated during hydration compared to class C fly ash, resulting from a lower CaO percentage. Free lime content directly correlates to the amount of heat generated during hydration.

Class F fly ash substitution substantially reduced the maximum temperature in the placement resulting from the chemical composition. The large reduction in the maximum temperature subsequently led to a reduction in the maximum temperature difference. Class C fly ash substitution only slightly lowers the maximum temperature in the placement, which correlates to the minimal reduction in the maximum temperature difference.

7.3.11 GGBFS Substitution

The GGBFS sensitivity study explored the effect of the substitution of GGBFS on the thermal development of mass concrete placements. The sensitivity study utilized a total cementitious content of 627 lb/cy, following the previous baseline. The substitution percentage ranged from 0-50 percent in 10 percent increments. Table 7.22 identifies the inputs that were used to complete the sensitivity study. No fly ash was used in the mix proportion in an effort to simplify the study.

Table 7.22 GGBFS substitution sensitivity study inputs

GGBFS Substitution (%)	Cement Content (lb/cy)	GGBFS Content (lb/cy)
0	627	0
10	564	63
20	502	125
30	439	188
40	376	251
50	314	313

Table 7.23 shows the maximum temperature and the maximum temperature difference as calculated by ConcreteWorks for each GGBFS substitution percentage. The results show that increasing the substitution of GGBFS has minimal effect on the maximum

temperature of the placement, and slightly increases the maximum temperature difference of the placement.

Table 7.23 GGBFS substitution sensitivity study results

GGBFS Substitution (%)	Maximum Temperature (°F)	Maximum Temperature Difference (°F)
0	154	89
10	154	91
20	154	93
30	154	95
40	156	98
50	158	101

GGBFS delays the generation of heat in concrete. The delayed heat generation causes the maximum temperature in the placement to be reached at a later time compared to placements without GGBFS. Since the heat is developed later, the concrete has less time to dissipate the heat before the formwork is removed. Figure 7.7 shows that the placement with 50 percent GGBFS substitution will be warmer at the time of form removal compared to the placement without GGBFS, increasing the maximum temperature difference compared to the concrete without GGBFS. However, the results of the GGBFS sensitivity study are in conflict with current understanding of the effect of heat generation of concrete. It is generally believed that the substitution of GGBFS for cement typically reduces the overall heat generation and subsequent maximum temperature of mass concrete, which conflicts with the maximum temperature results of the GGBFS sensitivity study.

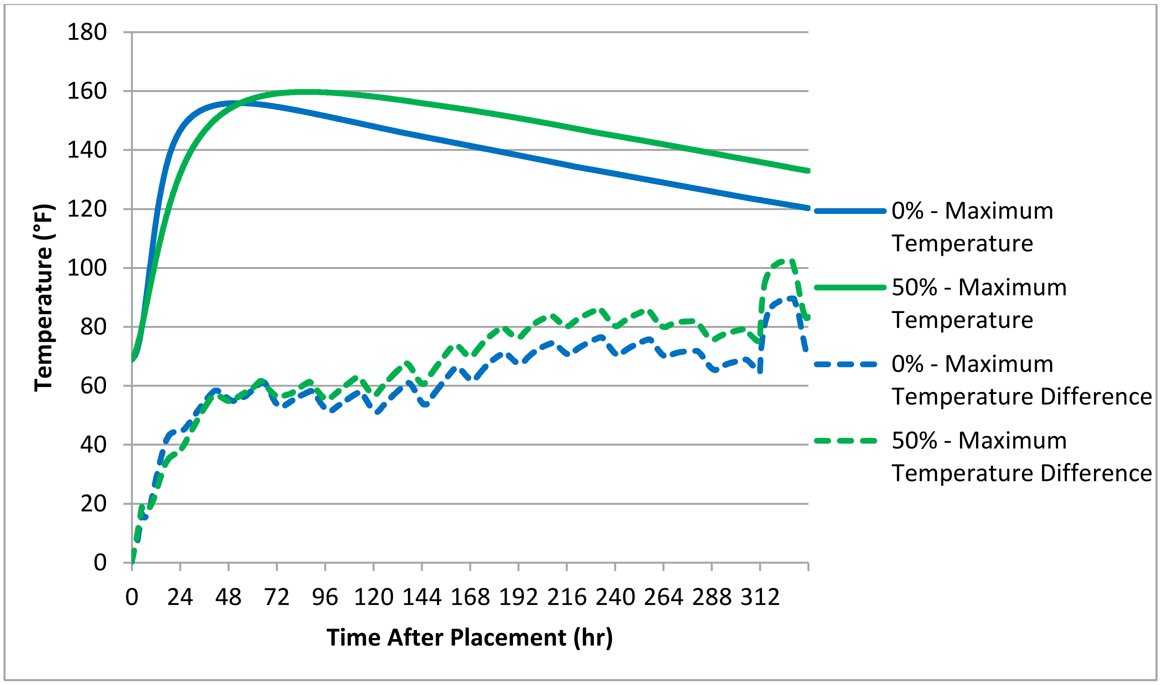


Figure 7.7 Maximum temperature and maximum temperature difference sensitivity study results for 0% and 50% GGBFS substitution

7.3.12 Combined Class F Fly Ash and GGBFS Substitution

Class F fly ash and GGBFS are commonly combined in mix proportions used in mass concrete. The sensitivity study looks at the thermal effect of the substitution of Class F fly ash and GGBFS at different ratios and total cement substitution percentages. The study looked at class F fly ash to GGBFS ratios from 0/100 for total cement substitution percentages ranging from 0 to 60 percent. The upper limit of 60 percent total cement substitution was selected to represent typical mass concrete specifications.

The inputs for the cement, class F fly ash, and GGBFS content used to complete the sensitivity study are shown in Table 7.24, Table 7.25, and Table 7.26 respectively. The tables are organized with each column representing a different total cement substitution percentage. Additionally, each row identifies a class F fly ash to GGBFS percentage, with the percentage of the cement substitution being fly ash in the leftmost column and GGBFS in the rightmost.

Table 7.24 Combined class F fly ash and GGBFS substitution - cement content (lb/cy) inputs

	Total Cement Substitution							
Fly Ash	0%	10%	20%	30%	40%	50%	60%	GGBFS
0%	627	564	502	439	376	314	251	100%
10%	627	564	502	439	376	314	251	90%
20%	627	564	502	439	376	314	251	80%
30%	627	564	502	439	376	314	251	70%
40%	627	564	502	439	376	314	251	60%
50%	627	564	502	439	376	314	251	50%
60%	627	564	502	439	376	314	251	40%
70%	627	564	502	439	376	314	251	30%
80%	627	564	502	439	376	314	251	20%
90%	627	564	502	439	376	314	251	10%
100%	627	564	502	439	376	314	251	0%

Table 7.25 Combined class F fly ash and GGBFS substitution – class F fly ash (lb/cy) inputs

	Total Cement Substitution							
Fly Ash	0%	10%	20%	30%	40%	50%	60%	GGBFS
0%	0	0	0	0	0	0	0	100%
10%	0	6	13	19	25	31	38	90%
20%	0	13	25	38	50	63	75	80%
30%	0	19	38	56	75	94	113	70%
40%	0	25	50	75	100	125	150	60%
50%	0	31	63	94	125	157	188	50%
60%	0	38	75	113	150	188	226	40%
70%	0	44	88	132	176	219	263	30%
80%	0	50	100	150	201	251	301	20%
90%	0	56	113	169	226	282	339	10%
100%	0	63	125	188	251	314	376	0%

Table 7.26 Combined class F fly ash and GGBFS substitution – GGBFS (lb/cy) inputs

	Total Cement Substitution							
Fly Ash	0%	10%	20%	30%	40%	50%	60%	GGBFS
0%	0	63	125	188	251	314	376	100%
10%	0	56	113	169	226	282	339	90%
20%	0	50	100	150	201	251	301	80%
30%	0	44	88	132	176	219	263	70%
40%	0	38	75	113	150	188	226	60%
50%	0	31	63	94	125	157	188	50%
60%	0	25	50	75	100	125	150	40%
70%	0	19	38	56	75	94	113	30%
80%	0	13	25	38	50	63	75	20%
90%	0	6	13	19	25	31	38	10%
100%	0	0	0	0	0	0	0	0%

The results of the sensitivity study are shown by Table 7.27 and Table 7.28. The results are organized in the same fashion as the inputs. Both the maximum temperature and the maximum temperature difference follow the same trend, the largest temperature is for 60 percent total cement substitution with 100 percent of the cement substitution being GGBFS. The minimum value also occurs at 60 percent total cement substitution, with 100 percent of the substitution being class F fly ash. Similarly to the class F fly ash and GGBFS substitution sensitivity study, class F fly ash reduces the maximum temperature and the maximum temperature difference, while GGBFS substitution increases both.

Table 7.27 Combined class F fly ash and GGBFS substitution results – maximum temperature (°F)

	Total Cement Substitution							
Fly Ash	0%	10%	20%	30%	40%	50%	60%	GGBFS
0%	154	154	154	154	156	158	162	100%
10%	154	153	153	153	153	155	158	90%
20%	154	153	152	151	151	151	153	80%
30%	154	152	150	149	148	148	149	70%
40%	154	152	149	147	145	145	145	60%
50%	154	151	148	145	143	141	140	50%
60%	154	150	147	144	141	138	136	40%
70%	154	150	146	142	138	135	132	30%
80%	154	149	145	140	136	132	128	20%
90%	154	149	143	138	133	128	124	10%
100%	154	148	142	136	131	125	120	0%

Table 7.28 Combined class F fly ash and GGBFS substitution results – maximum temperature difference (°F)

	Total Cement Substitution							
Fly Ash	0%	10%	20%	30%	40%	50%	60%	GGBFS
0%	89	91	93	95	98	101	106	100%
10%	89	90	92	94	96	99	103	90%
20%	89	90	91	92	94	96	99	80%
30%	89	89	90	91	92	93	95	70%
40%	89	89	89	89	89	90	91	60%
50%	89	88	88	87	87	87	88	50%
60%	89	88	87	86	85	85	84	40%
70%	89	87	86	84	83	82	81	30%
80%	89	87	85	83	81	79	77	20%
90%	89	86	84	81	79	76	74	10%
100%	89	86	83	80	76	73	70	0%

A graphical representation of the maximum temperature results is shown in Figure 7.8. In accordance, the maximum temperature difference follows the same trend as that shown for the maximum temperature.

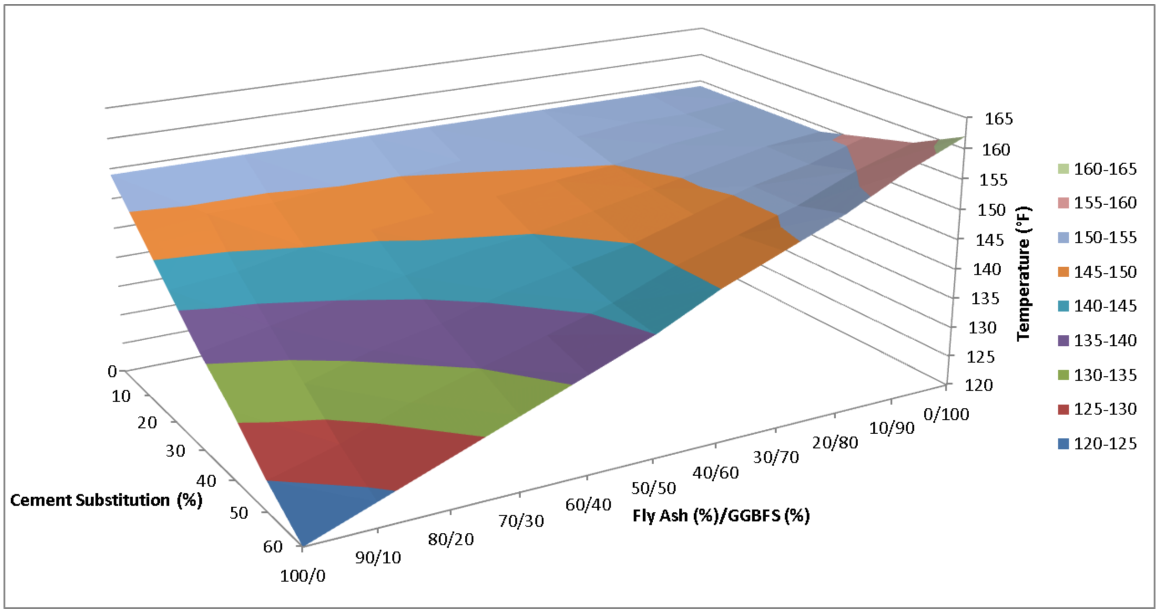


Figure 7.8 Combined class F fly ash and GGBFS substitution maximum temperature results

7.4 DISCUSSION

The results of the sensitivity study shows that all twelve of the parameters examined effect the thermal development of typical Midwest boarder bridge mass concrete placements. The parameters that have the largest effect on the maximum temperature, as shown by the results, include the depth of the placement, fresh placement temperature, cementitious content, and class F fly ash substitution. Additionally, parameters having the largest effect on the maximum temperature difference include dimensional size, fresh placement temperature, ambient air temperature, cementitious content, and class F fly ash substitution.

The results also show that the location of the thermal sensors plays a large role in maximum temperature and maximum temperature difference readings.

CHAPTER 8. CONCLUSIONS

8.1 OVERVIEW

This chapter provides an overview of the knowledge gained from the research. The conclusions are grouped into four categories including specification survey, case study, ConcreteWorks calibration, and sensitivity study. The final section of this chapter provides the connection of the conclusion with the goal of the research.

8.2 SPECIFICATION SURVEY

The specification survey identified 13 different mass concrete specifications, developmental specifications, or special provisions. The specification survey examined the following requirements of each agency:

- Definition of mass concrete – the dimensions for which a concrete element is considered mass concrete.
- Temperature restrictions – the maximum temperature and maximum temperature difference limits.
- Cement content requirements – the maximum or minimum required cement content for a mix proportion.
- Compressive strength requirement – the minimum requirement for the compressive strength of the concrete.
- Supplementary cementitious requirements – the allowable amount of different supplementary cementitious materials in a mix proportion.
- Fresh placement temperature limitations – the maximum and/or minimum allowable fresh placement temperature.

- Thermal sensor requirements – the requirements for the location and concrete cover for thermal sensors.
- Duration of thermal control – the circumstances under which thermal control of an element may be stopped.

8.3 CASE STUDY

Two case studies were verified as typical Midwest boarder bridges. The case study detailed the mix proportion, construction, and environmental conditions under which the two bridges were constructed. The following aspects were documented in the case study:

- Footing subbase and support
- Formwork material
- Pier elements
- Concrete placement
- Consolidation
- Insulation
- Cooling pipes
- Thermal monitoring
- Formwork removal
- Concrete mix proportion
- Environmental conditions

8.4 CONCRETEWORKS CALIBRATION

The ConcreteWorks calibration verified that ConcreteWorks is capable of predicting both the maximum and minimum temperature of typical Midwest boarder bridges. The results show that on average, ConcreteWorks underestimates the maximum temperature of a placement by 12.7° F, and overestimates the maximum temperature difference by 1.7°F.

8.5 SENSITIVITY STUDY

The sensitivity study validated parameters having the largest impact on the thermal development of mass concrete for Midwest boarder bridges. The parameters identified as having the largest impact are listed below.

1. Placement least dimension

   As the least dimension of a placement increases, both the maximum temperature and the maximum temperature difference increase.

2. Fresh placement temperature

   As the fresh placement temperature increases, both the maximum temperature and the maximum temperature difference increase.

3. Ambient air temperature

   Increased ambient air temperatures increase the maximum temperature, while reducing the maximum temperature difference.

4. Cement content

   As the cement content increases, both the maximum temperature and the maximum temperature difference increase.

5. Class F fly ash substitution.

   As the substitution percentage of class F fly ash increases, both the maximum temperature and the maximum temperature difference decrease.

8.6 SUMMARY OF CONCLUSIONS

The goal of the research was to provide insight on the construction, design, and thermal analysis of typical Midwest boarder bridge mass concrete placements. The conclusion shows that the goal of the research was attained.

CHAPTER 9. RECOMMENDATIONS

9.1 OVERVIEW

The following three sections of this chapter describe the immediate impact of the research, the long-term impact of the research, and recommendation for future research.

9.2 IMMEDIATE IMPACT

This research may be used as a reference to evaluate the thermal predictive capabilities of ConcreteWorks to analyze Midwest boarder bridges. The sensitivity study may also serve as a rough design reference for similar mass concrete placements during initial design.

9.3 LONG-TERM IMPACT

The research may serve as a case study for the calibration of current and future thermal analysis software programs. Additionally, the research also provides a bench mark for the level of accuracy for currently available or future mass concrete thermal analysis software programs.

9.4 RECOMMENDATIONS FOR FUTURE RESEARCH

A recommendation for future research is a cost analysis of different thermal control options for the construction of mass concrete. Additionally, it is recommended that a thermal analysis software program be developed to design cooling pipe systems for mass concrete placements. The final recommendation is to identify sensor locations and concrete cover requirements to properly evaluate the thermal development and cracking potential of mass concrete.

WORKS CITED

AHTD. (2003). Standard specification for highway construction, Arkansas Highway and Transportation Department, Little Rock, AR.

AÏtcin, P. (1999). Does concrete shrink or does it swell?, American Concrete Institute, Farmington, MI.

Altoubat, S, Lange, D. (2001). Creep, shrinkage, and cracking of restrained concrete at early ages, University of Illinois at Urbana-Champaign, Urbana, IL.

American Concrete Institute Committee 207 (ACI 207). (1995). Effect of restraint, volume chance, and reinforcement on cracking of mass concrete, American Concrete Institute, Farmington Hills, MI.

American Concrete Institute Committee 207 (ACI 207). (2006). Guide to mass concrete, American Concrete Institute, Farmington Hills, MI.

American Concrete Institute Committee 364 (ACI 364). (2006). Technical questions – ACI concrete knowledge center – alkali-aggregate reaction in mass concrete. American Concrete Institute, Farmington Hills, MI. http://www.concrete.org/FAQ /afmviewfaq.asp?faqid=46 (accessed April 5, 2012).

American Society for Testing and Materials (ASTM). (1999). ASTM C150 Standard specification for Portland cement. American Society for Testing and Materials, West Conshohocken, PA.

Ash Grove Cement Company. (2010). Type I/II cement Report, Ash Grove Cement Company, Louisville, NE.

Bhatty, J. and Tennis, P. (2008). U.S. and Canadian cement characteristics: 2004, Portland Cement Association, Skokie, IL.

California DOT. (2010). Standard specifications, California Department of Transportation , Sacramento, CA.

Carino,N.J., Lew, H.S. (2001). The maturity method: From theory to application, National Institute of Standards and Technology, Gaithersburg, MD.

Florida DOT. (2006). Structural design guidelines, Florida Department of Transportation, Tallahassee, FL.

Florida DOT. (2010). Standard specifications for road and bridge constrcution, Florida Department of Transportation, Tallahassee, FL.

Folliard, K., et.al. (2007). Prediction model for concrete behavior – final report, Center for Transportation Research, The University of Texas at Austin, Austin, TX.

Gajida, J., Vangeem, M. (2002). Controlling temperature in mass concrete, American Concrete Institute, Farmington, MI.

Headwaters Resources. (2005). Chemical comparison of fly ash and Portland cement, Headwaters Resources, South Jordan, UT.

Hurd, M. (2005). Formwork for concrete, American Concrete Institute, Farmington Hills, MI.

Idaho DOT. (2004). Standard specifications for highway construction, Idaho Transportation Department, Boise, ID.

Illinois DOT. (2012). Illinois special provision 2012 - 1020.15 Heat of Hydration Control for Concrete Structures, Illinois Department of Transportation, Springfield, IL.

Iowa DOT. (2010). DS-09047 Developmental specification for mass concrete – Control of heat of hydration , Iowa Department of Transportation, Ames, IA.

Kentucky Transportation Cabinet. (2008). Special note for structural mass concrete, Kentucky Transportation Cabinet, Frankfort, KY.

Kim, S. (2010). "Effect of heat generation from cement hydration on mass concrete placement." M.S. Thesis, Iowa State University, Ames, IA.

Kosmatka, S., Kerkhoff, B. and Panarese W. (2002). Design and control of concrete mixtures, 14th Ed., Portland Cement Association, Skokie, IL.

Meeks, C.F. (2011). "Implementation of ConcreteWorks software in Texas highway construction" M.S. Thesis, The University of Texas at Austin, Austin, TX.

Mindess, S. and Young, J. (1981). Concrete, Prentice-Hall Inc, Englewood Cliffs, NJ

Li, J. (2012). " Predicting early-age thermal behavior of mass concrete for bridge foundations." M.S. Thesis, Iowa State University, Ames, IA.

Logan, D (2011). A first course in the finite element method, Cengage Learning, Stamford, CT.

New Jersey DOT. (2007). Standard specifications for road and bridge construction, New Jersey Department of Transportation, Trenton, NJ.

New York State DOT. (2012). Concrete for structures class MP (mass placement), New York State Department of Transportation, Albany, NY.

Poole, J.L., Riding, K,A. 2009. Early age cracking: A case study in how materials modeling can improve concrete quality, American Concrete Institute, Farmington, MI.

Rhode Island DOT. (2010). Standard specifications for road and bridge construction, Rhode Island Department of Transportation, Providence, RI.

Riding, K. (2007). "Early age concrete thermal stress measurement and modeling." Ph.D Dissertation, The University of Texas at Austin, Austin, TX.

South Carolina DOT. (2007). Standard specifications for highway construction, South Carolina Department of Transportation, Columbia, SC.

Taylor, P., Kosmatka, S., and Voigt, G., et. al (2007). Integrated materials and construction practices for concrete pavement :A state-of-the-practice manual, National Concrete Pavement Technology Center/Center for Transportation Research and Education, Iowa State University, Ames, IA.

Tennis, P. and Jennings, H. (2000). A model for two types of calcium silicate hydrate in the microstructure of Portland cement pastes, Cement and Concrete Research, New York, NY.

Texas DOT. (2004). Standard specifications for construction and maintenance of highways, streets, and bridges, Texas Department of Transportation, Austin, TX.

Wand, K., Hu, J., and Zhi, G. (2008). Material thermal input for Iowa materials , Center of Transportation Research and Education, Iowa State University, Ames, IA.

West Virginia DOT. (2006). Special provision for section 601 – Structural , West Virginia Department of Transportation Division of Highways, Charleston, WV.

APPENDIX A INSTALLATION AND LAYOUT OF THERMAL SENSORS

Figure A.1 Installation of thermal sensors with cable ties and tie wire

Figure A.2 Top surface and center sensors installed with electrical tape

Figure A.3 Thermal sensor supported and protected with supplemental rebar

Figure A.4 Typical top surface and center sensor layout

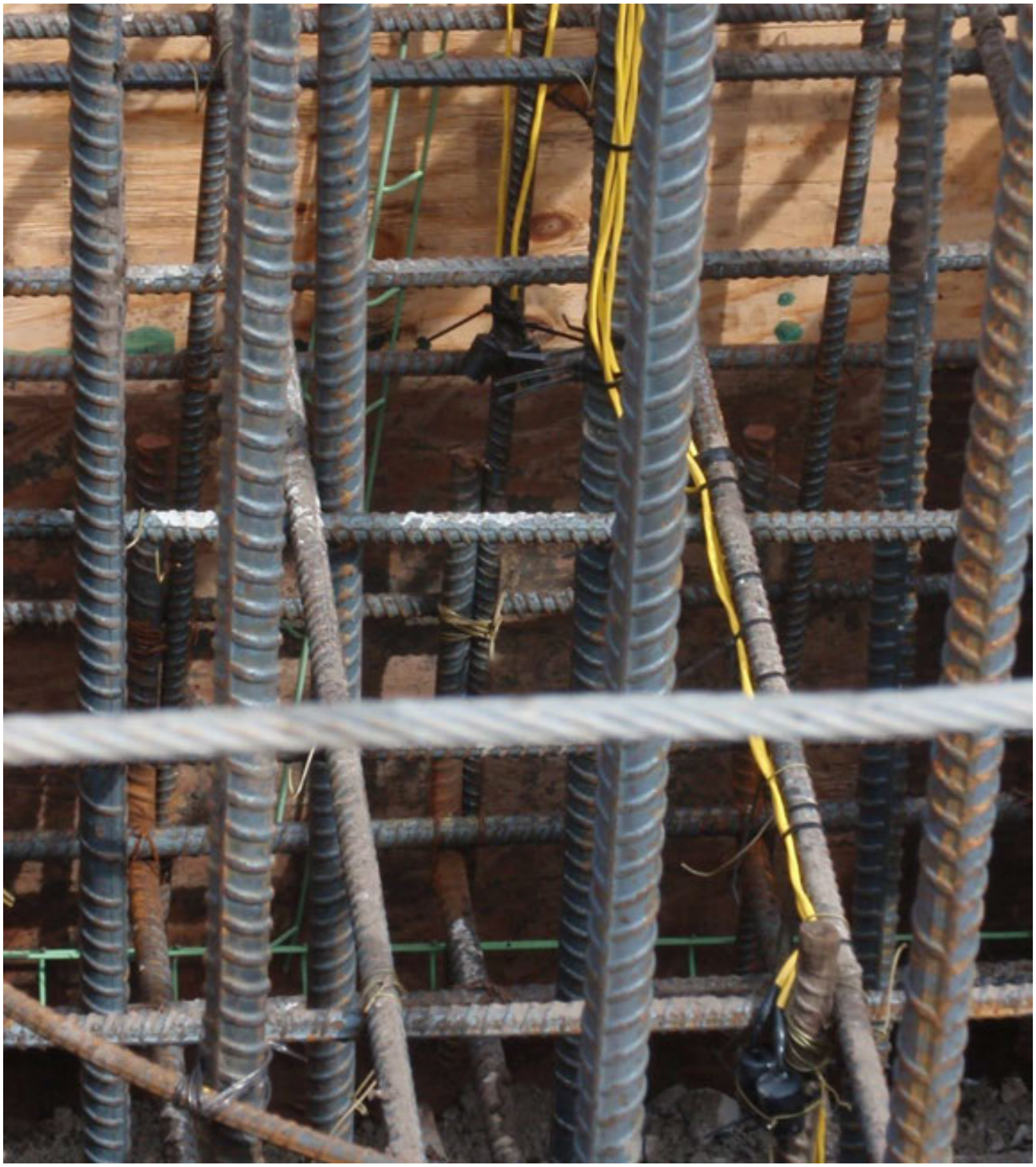


Figure A.5 Typical side surface and center sensor layout

Figure A.6 Verification of proper sensor function after installation

APPENDIX B WB I-80 CASE STUDY THERMAL RESULTS

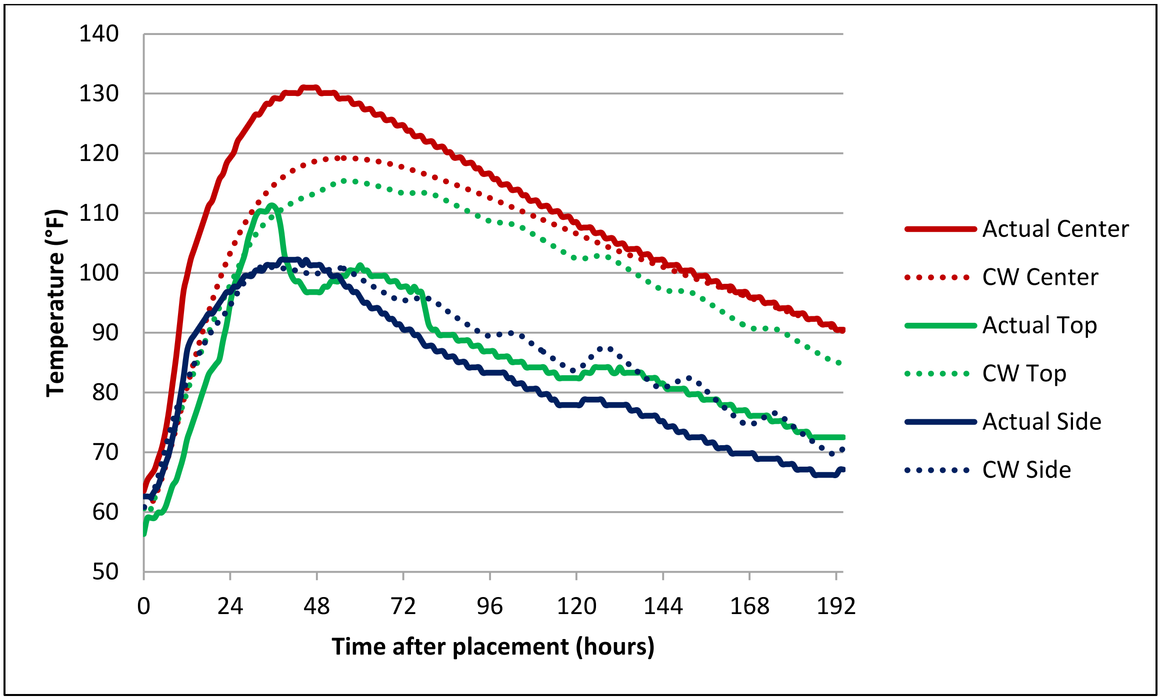


Figure B.1 WB I-80 case study thermal results – Pier 1 footing

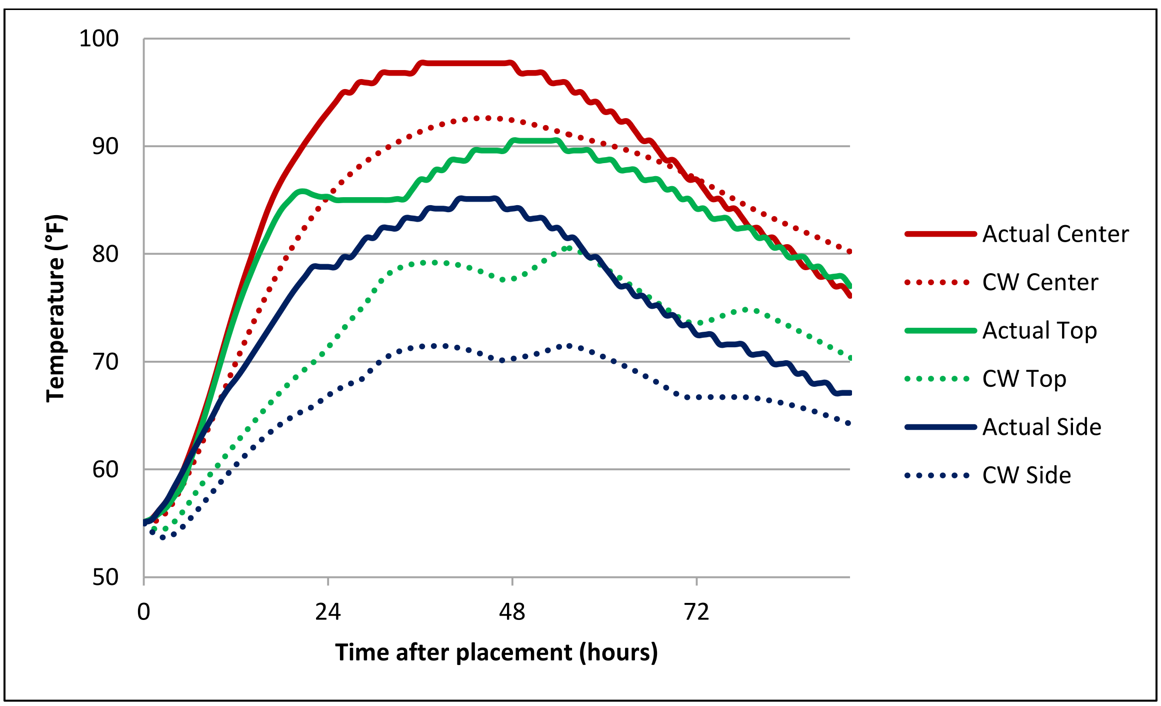


Figure B.2 WB I-80 case study thermal results – Pier 1 stem/column

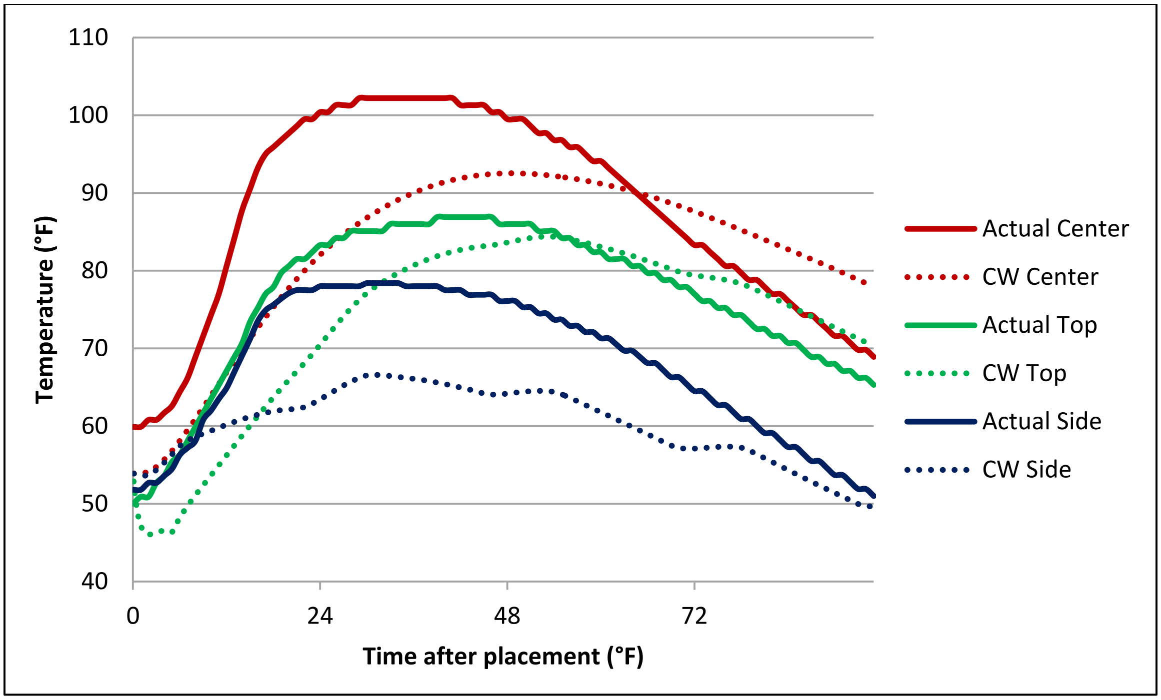


Figure B.3 WB I-80 case study thermal results – Pier 1 cap

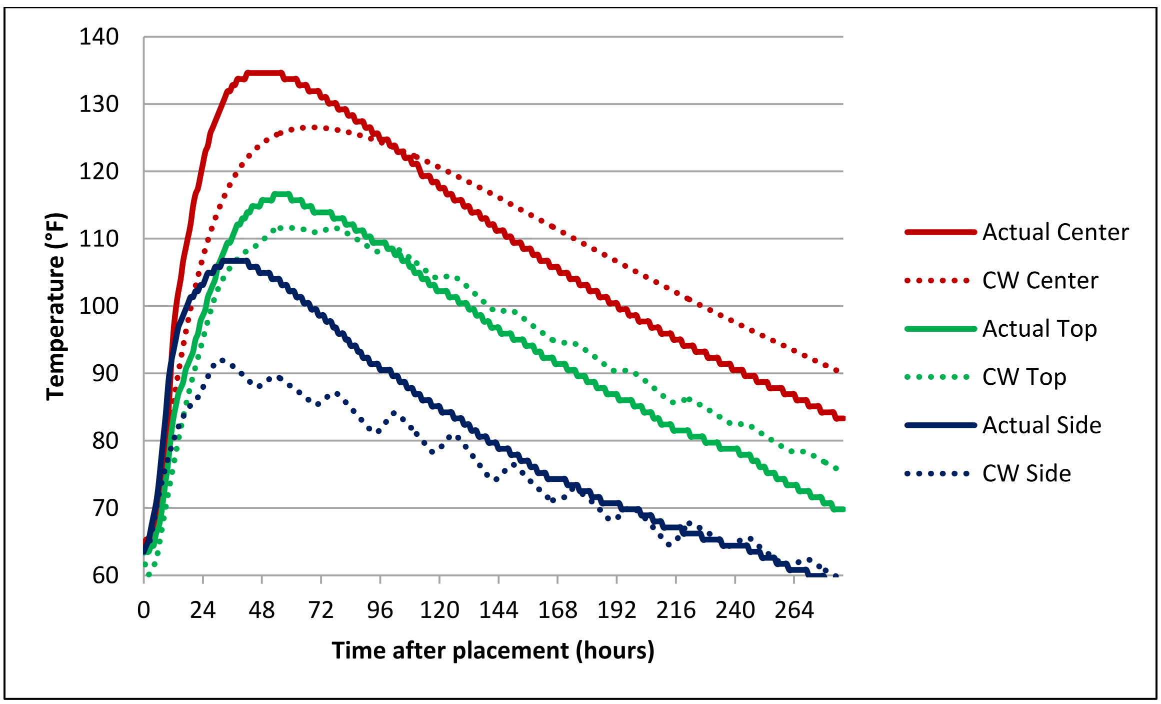


Figure B.4 WB I-80 case study thermal results – Pier 2 footing

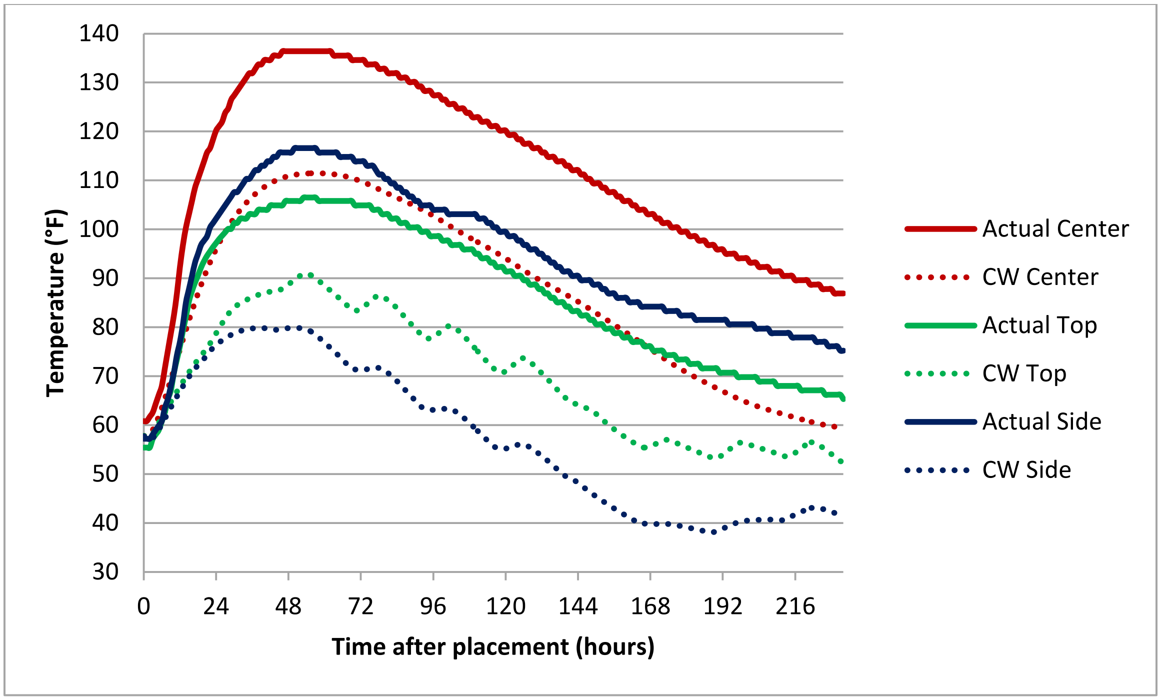


Figure B.5 WB I-80 case study thermal results – Pier 2 stem

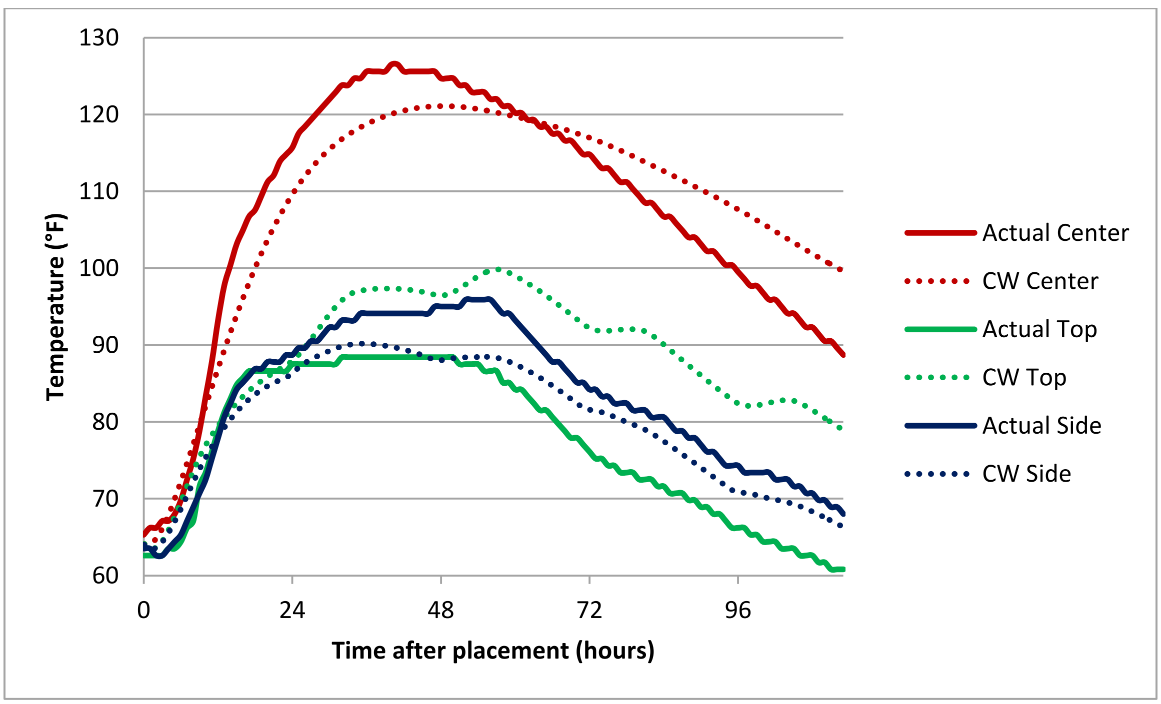


Figure B.6 WB I-80 case study thermal results – Pier 2 column

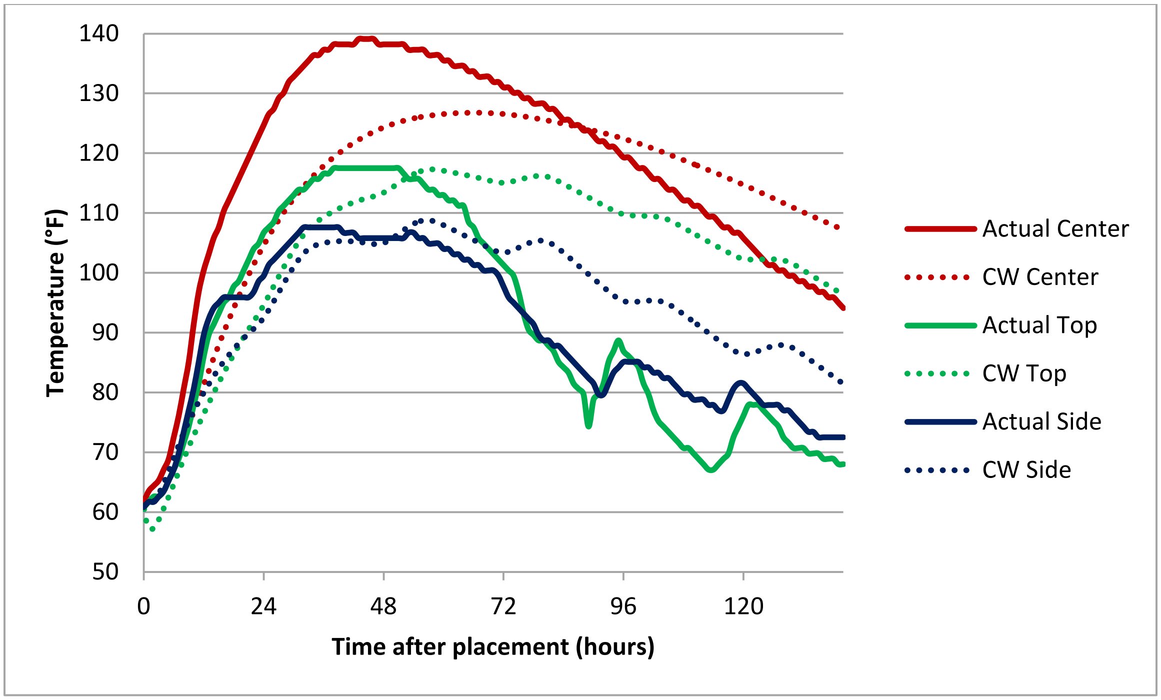


Figure B.7 WB I-80 case study thermal results – Pier 2 cap

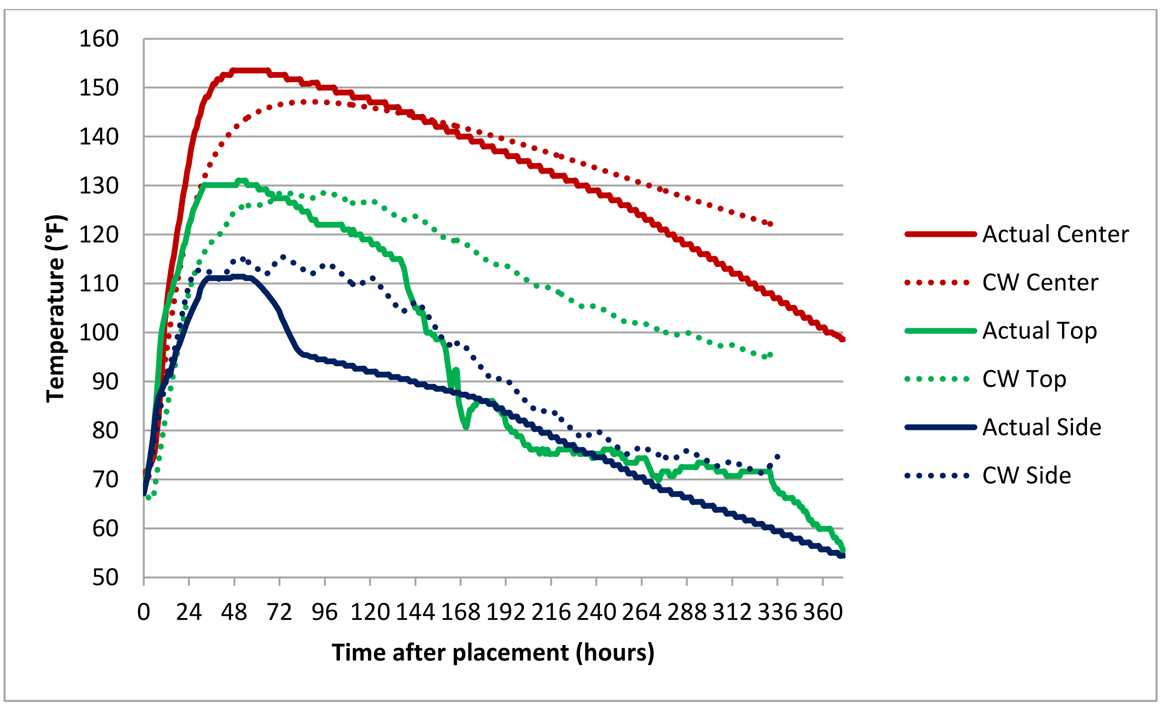


Figure B.8 WB I-80 case study thermal results – Pier 3 footing

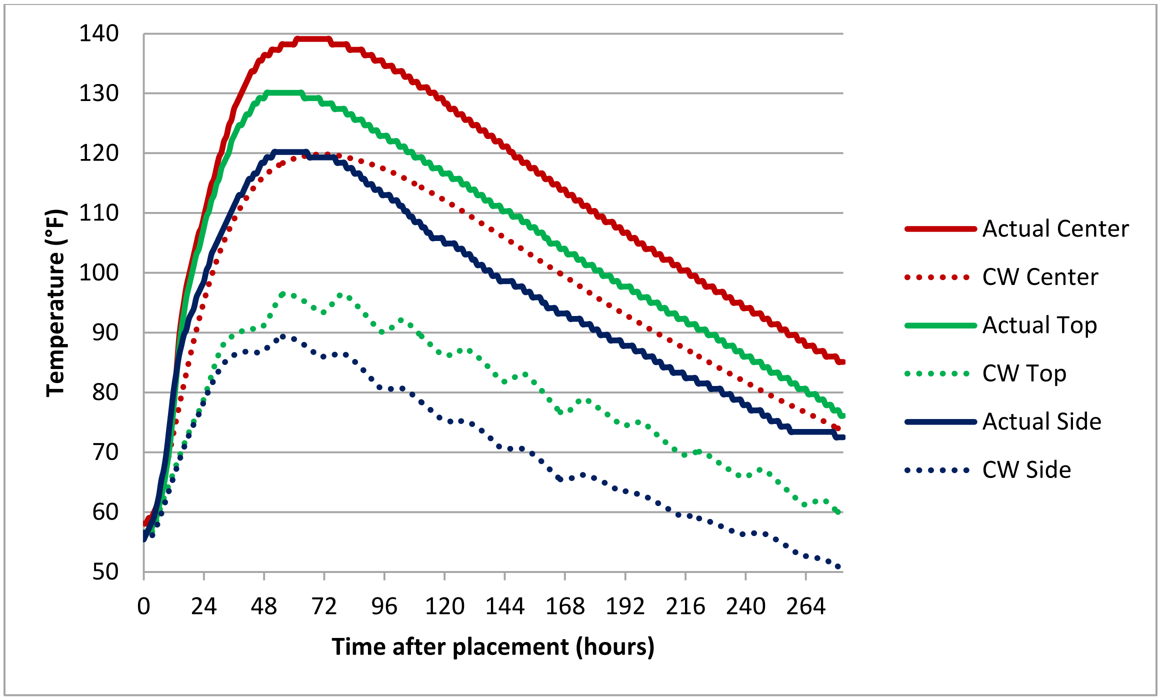


Figure B.9 WB I-80 case study thermal results – Pier 3 stem

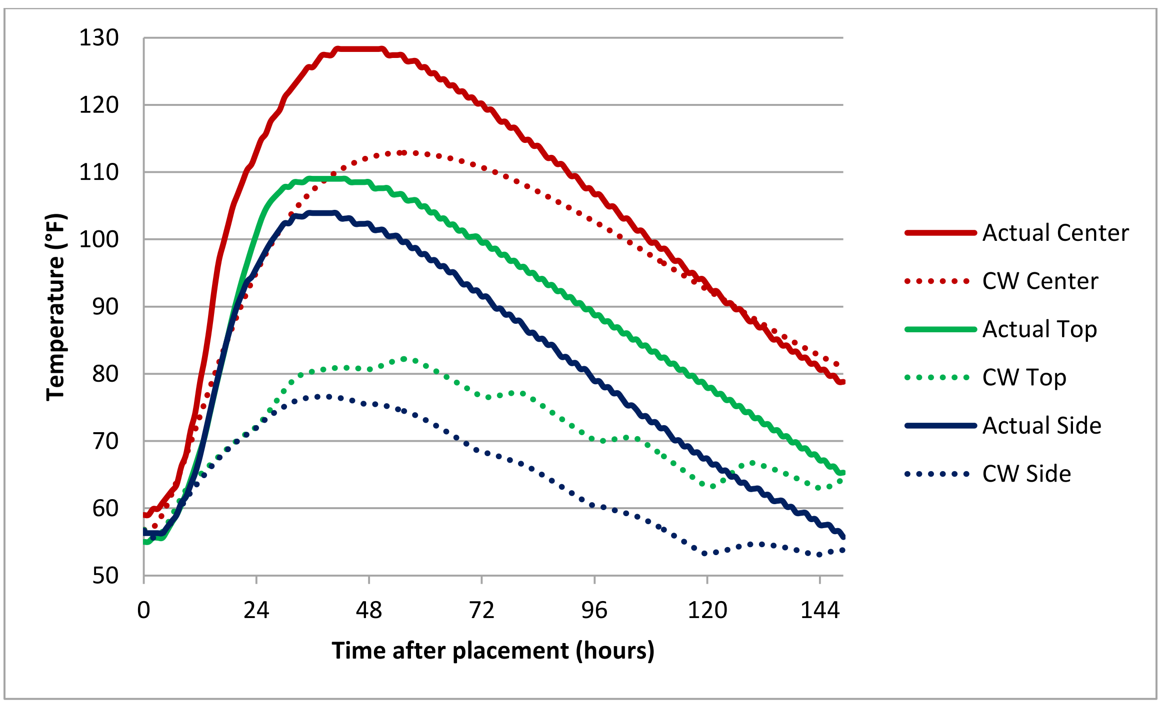


Figure B.10 WB I-80 case study thermal results – Pier 3 column

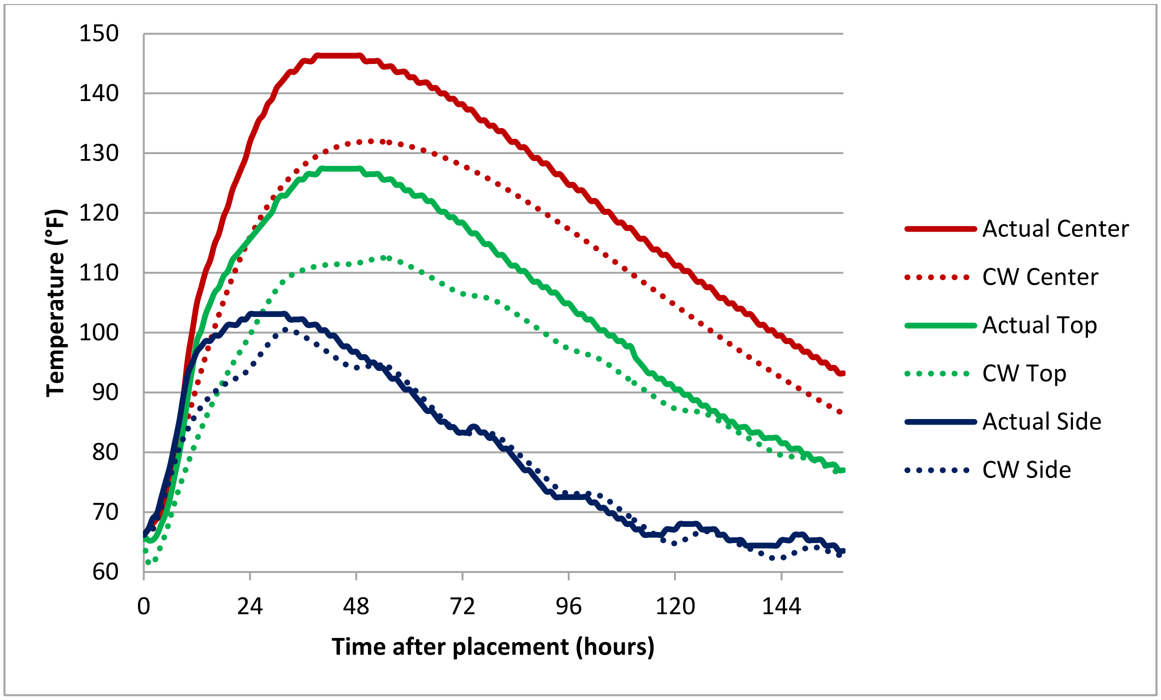
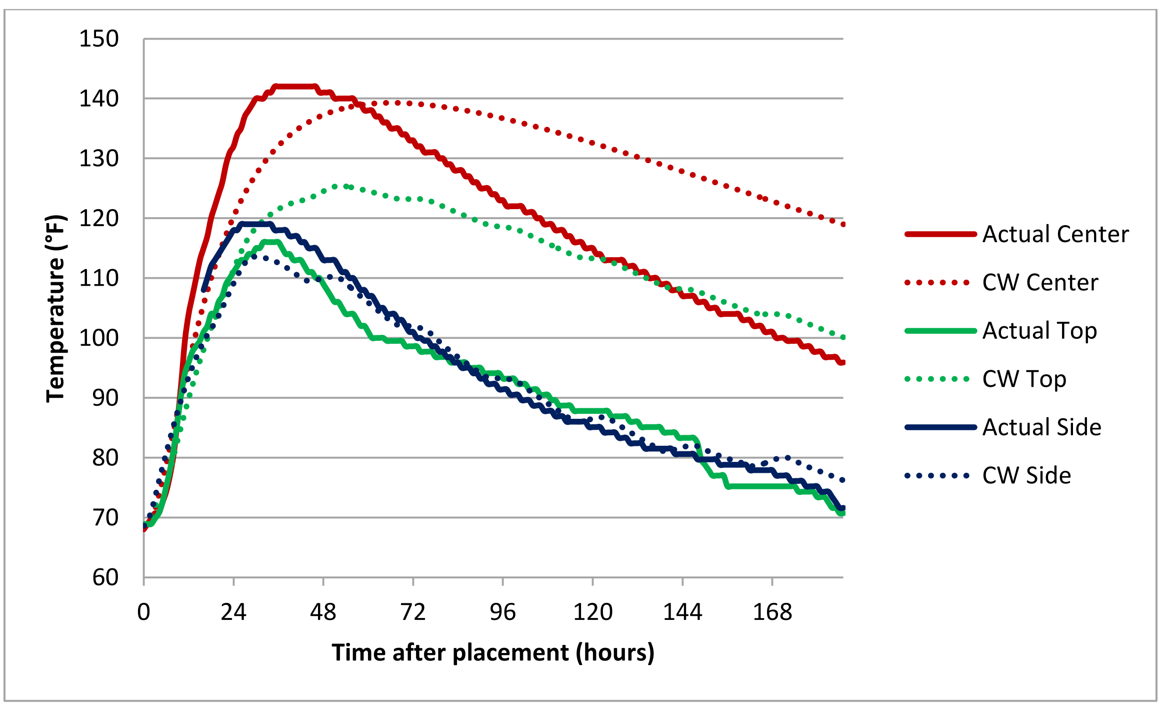


Figure B.11 WB I-80 case study thermal results – Pier 3 cap



*side sensor was not turned on until hours 16

Figure B.12 WB I-80 case study thermal results – Pier 4 footing

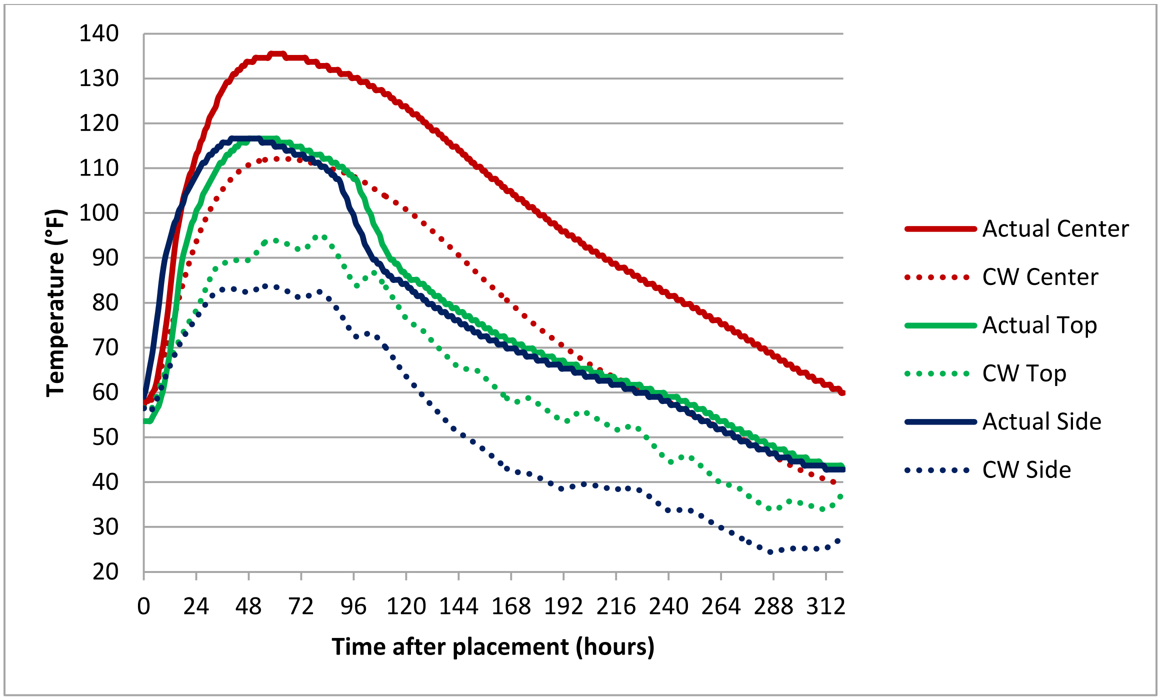


Figure B.13 WB I-80 case study thermal results – Pier 4 stem

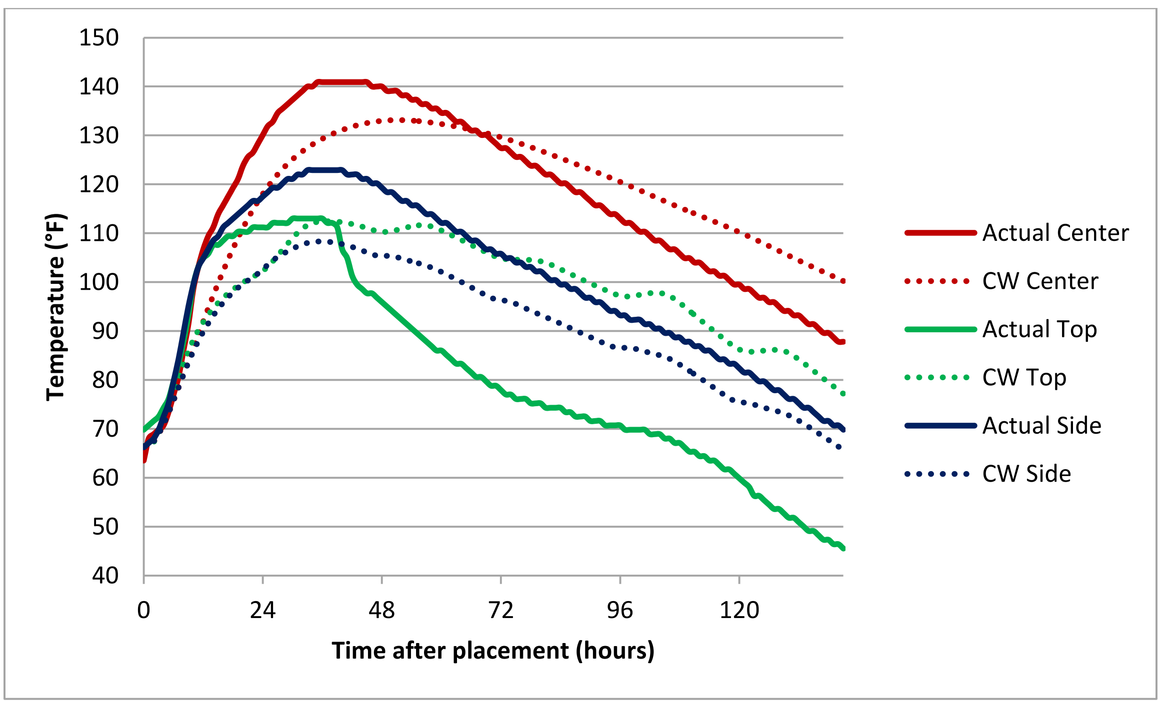


Figure B.14 WB I-80 case study thermal results – Pier 4 column

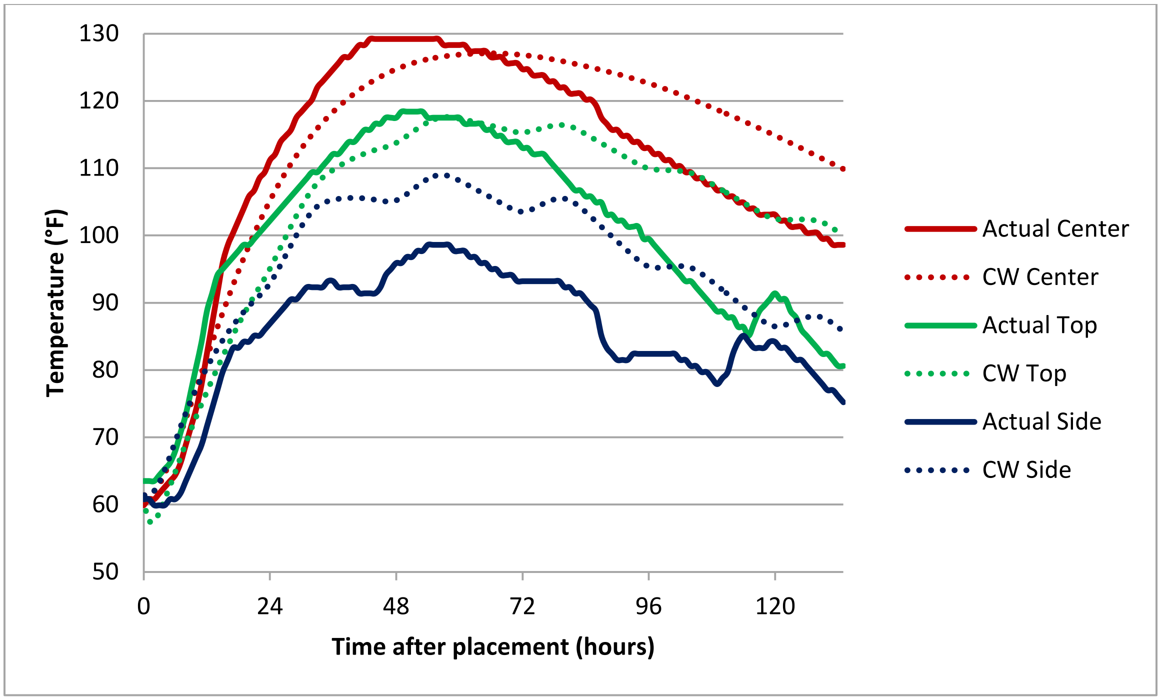


Figure B.15 WB I-80 case study thermal results – Pier 4 cap

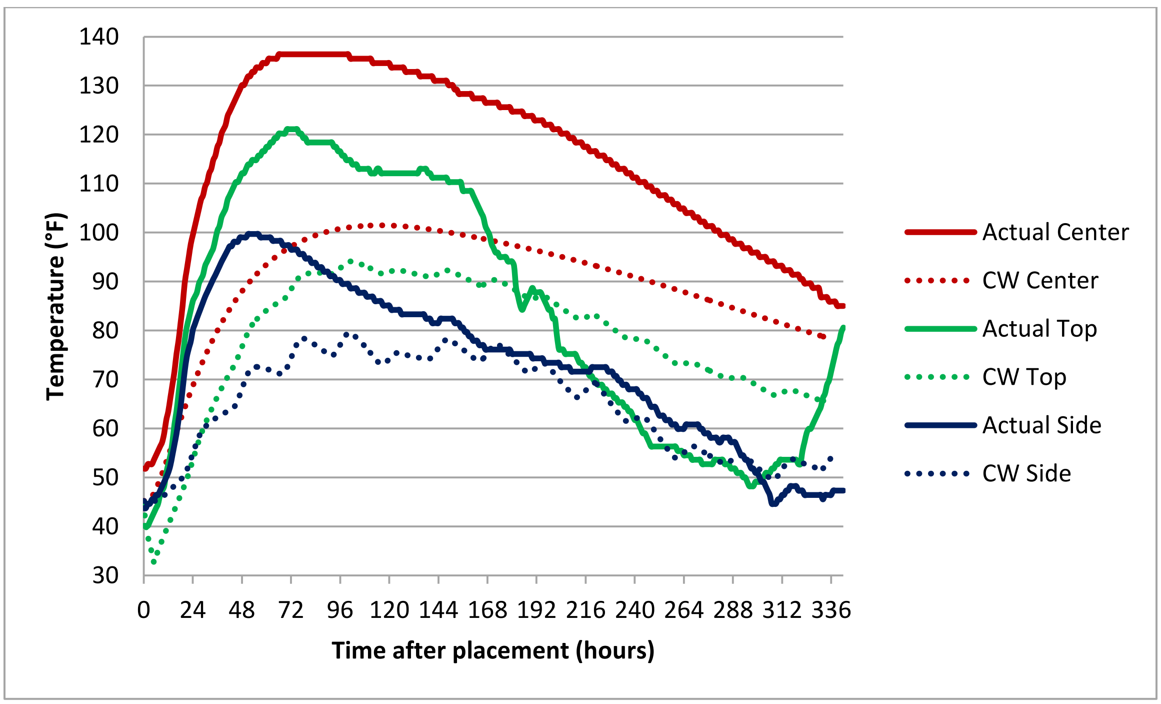


Figure B.16 WB I-80 case study thermal results – Pier 5 footing

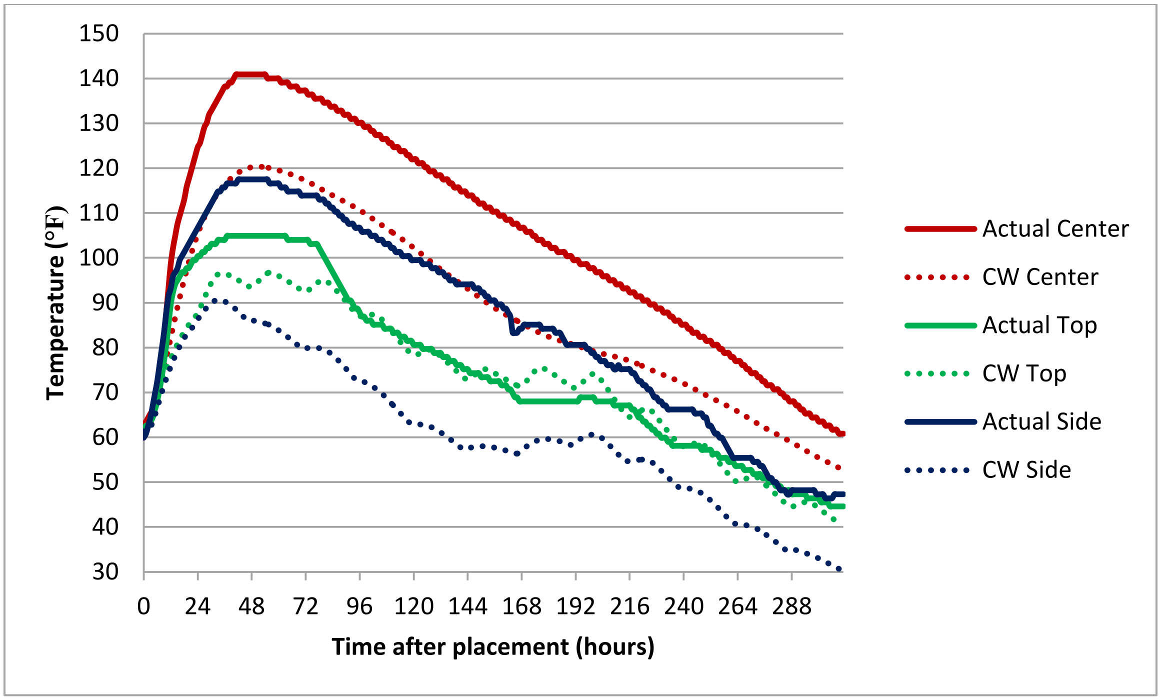


Figure B.17 WB I-80 case study thermal results – Pier 5 stem

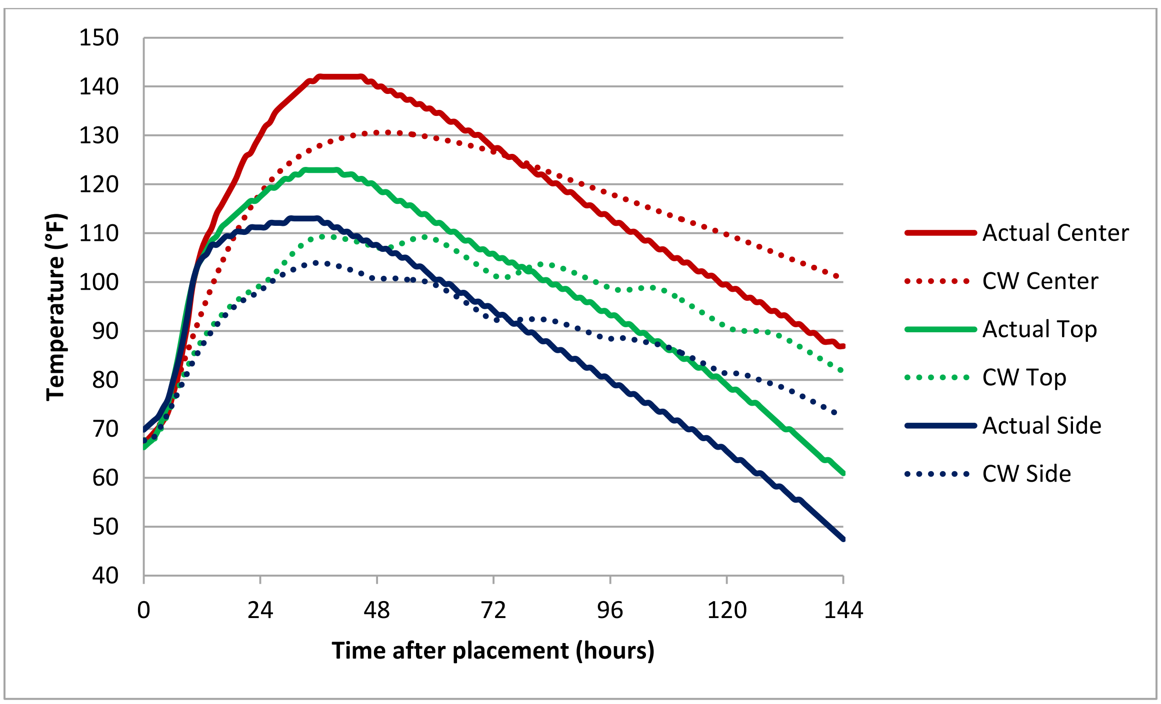


Figure B.18 WB I-80 case study thermal results – Pier 5 column

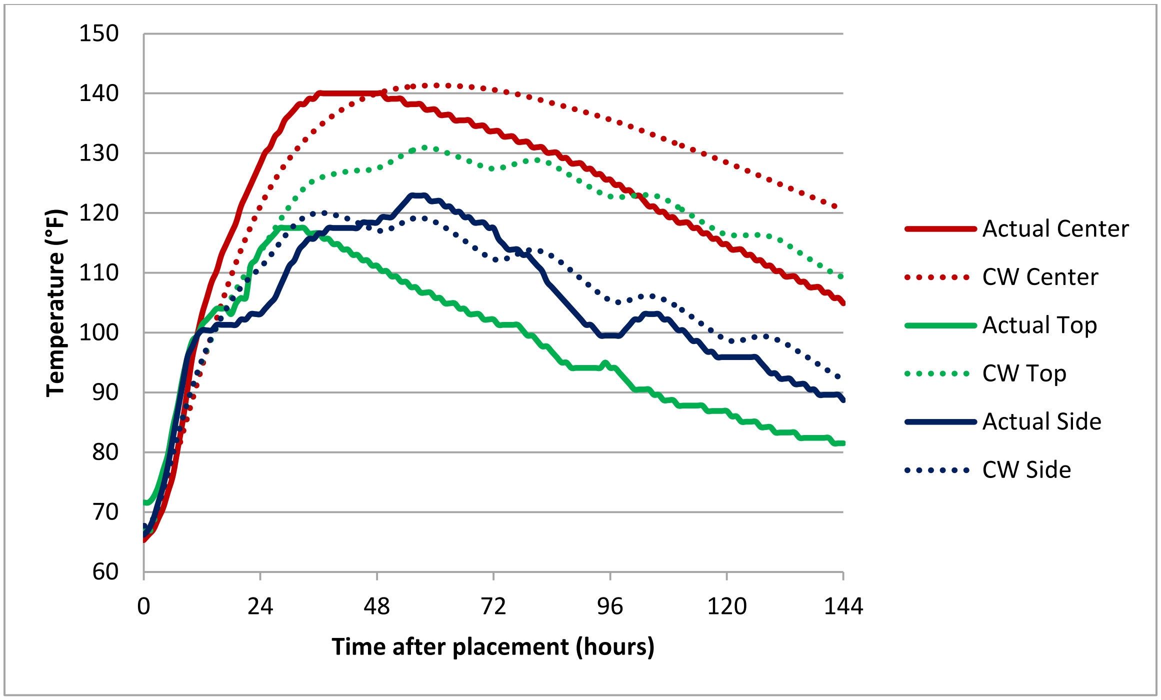


Figure B.19 WB I-80 case study thermal results – Pier 5 cap

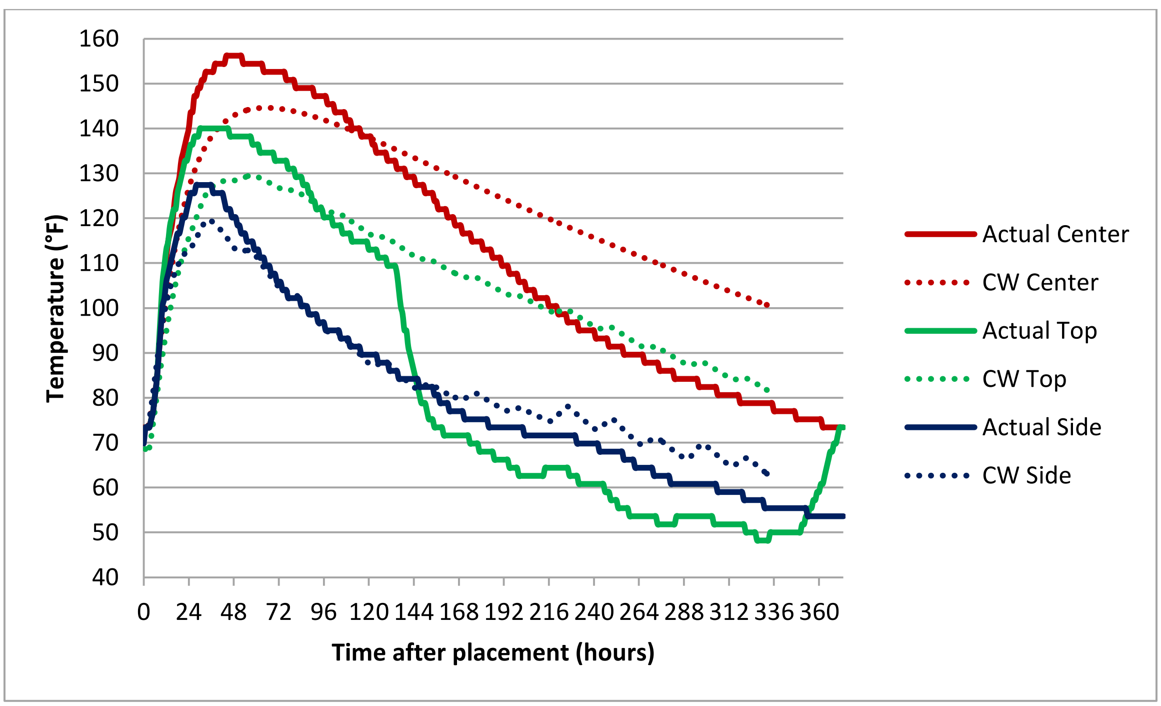


Figure B.20 WB I-80 case study thermal results – Pier 6 footing

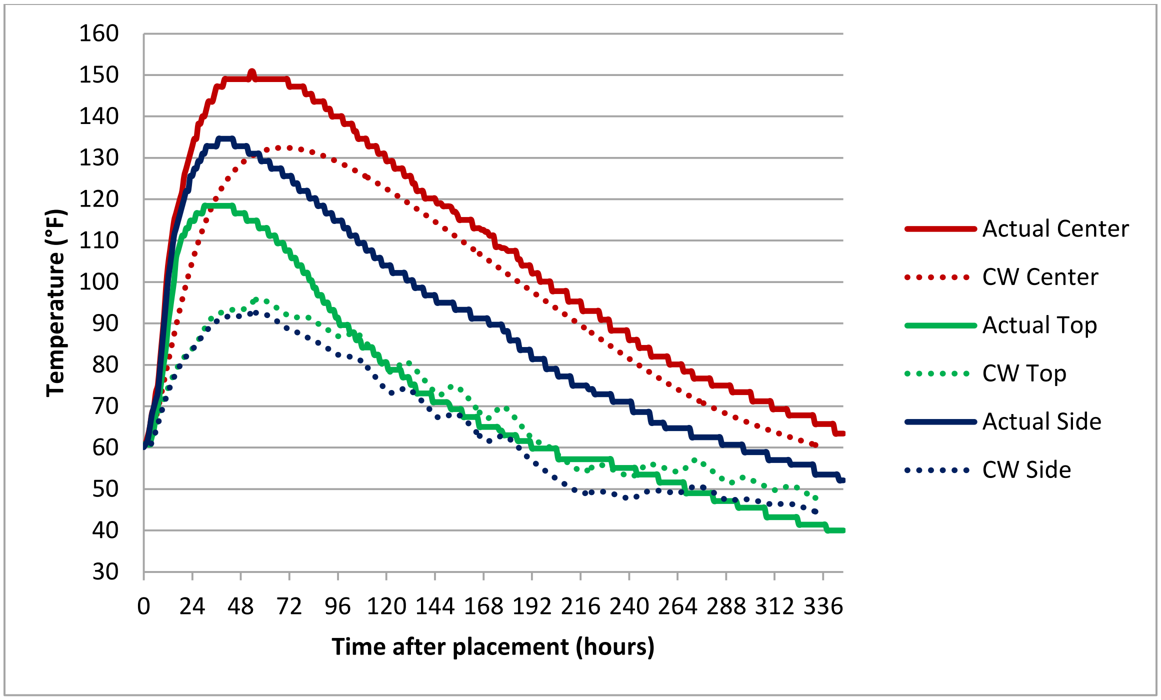


Figure B.21 WB I-80 case study thermal results – Pier 6 column

APPENDIX C US 34 CASE STUDY THERMAL RESULTS

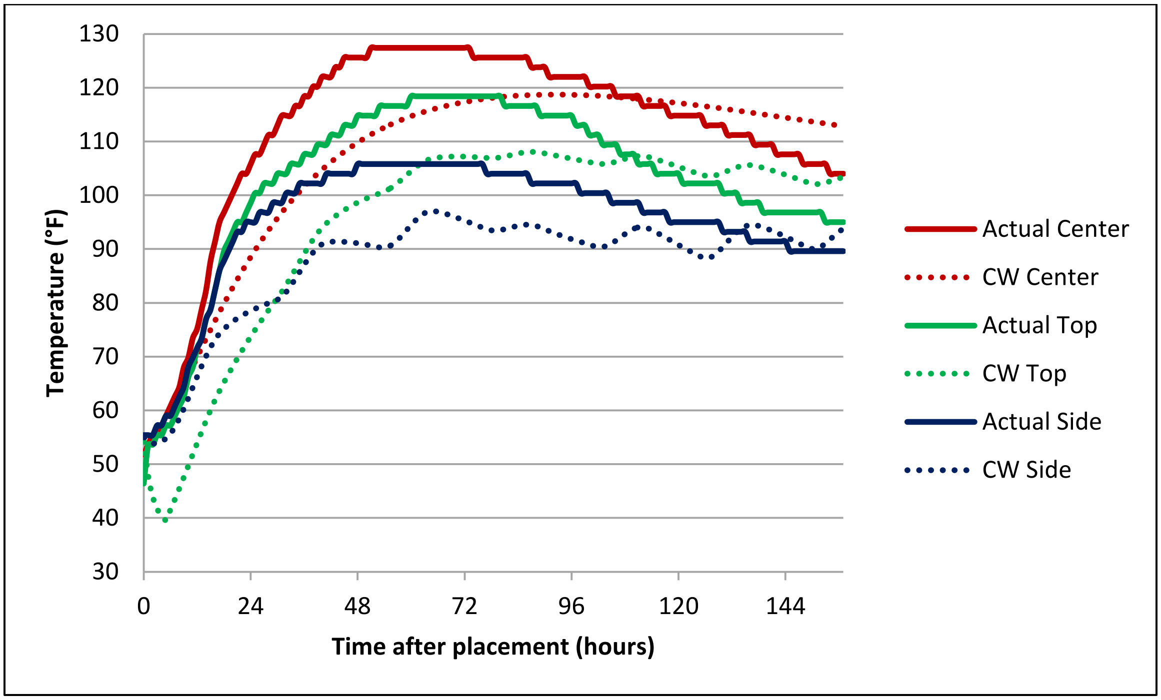


Figure C.1 US 34 case study thermal results – Pier 2 footing – A

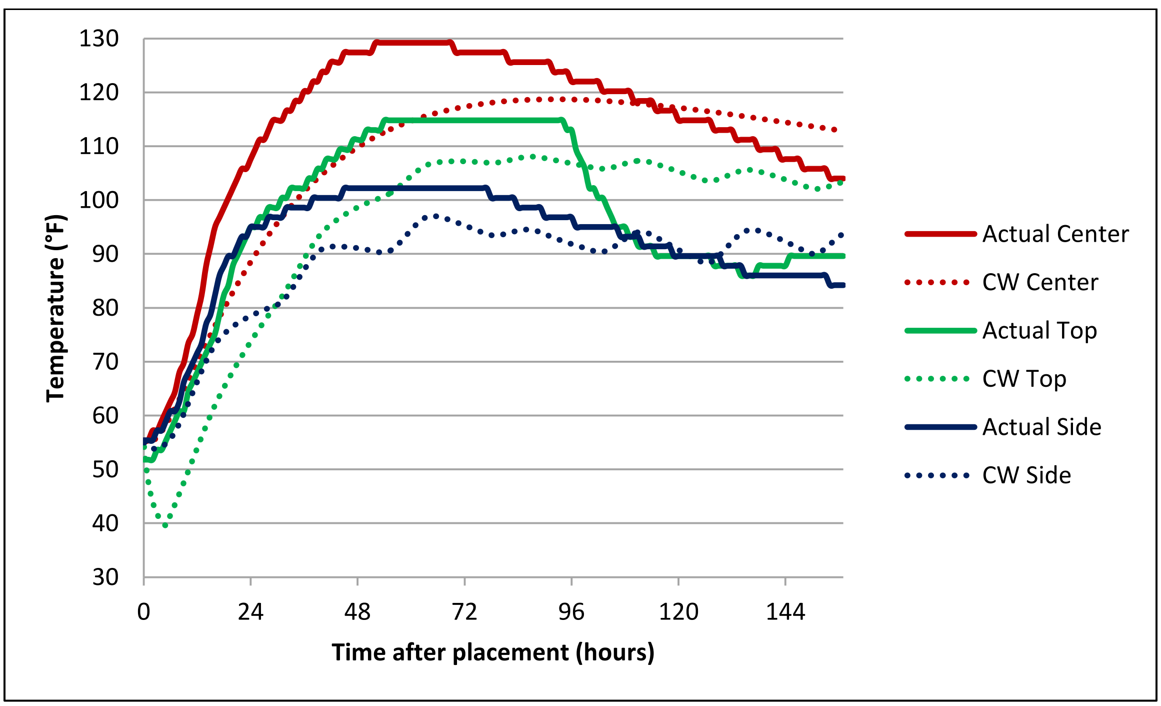


Figure C.2 US 34 case study thermal results – Pier 2 footing – B

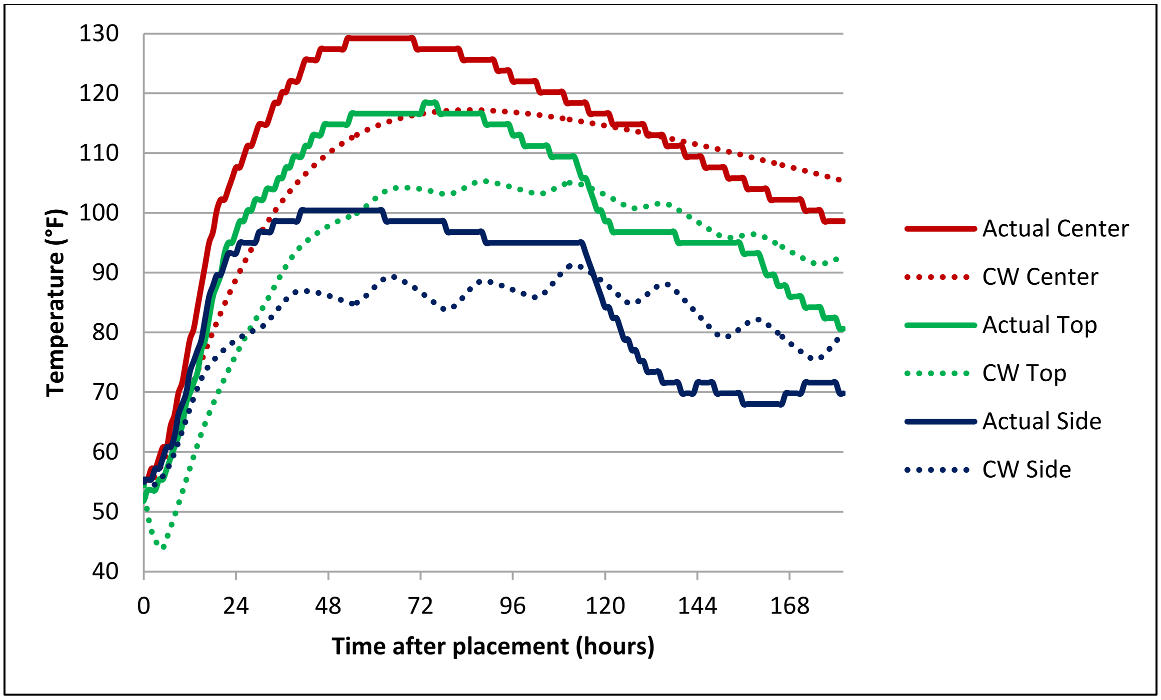


Figure C.3 US 34 case study thermal results – Pier 2 footing – C

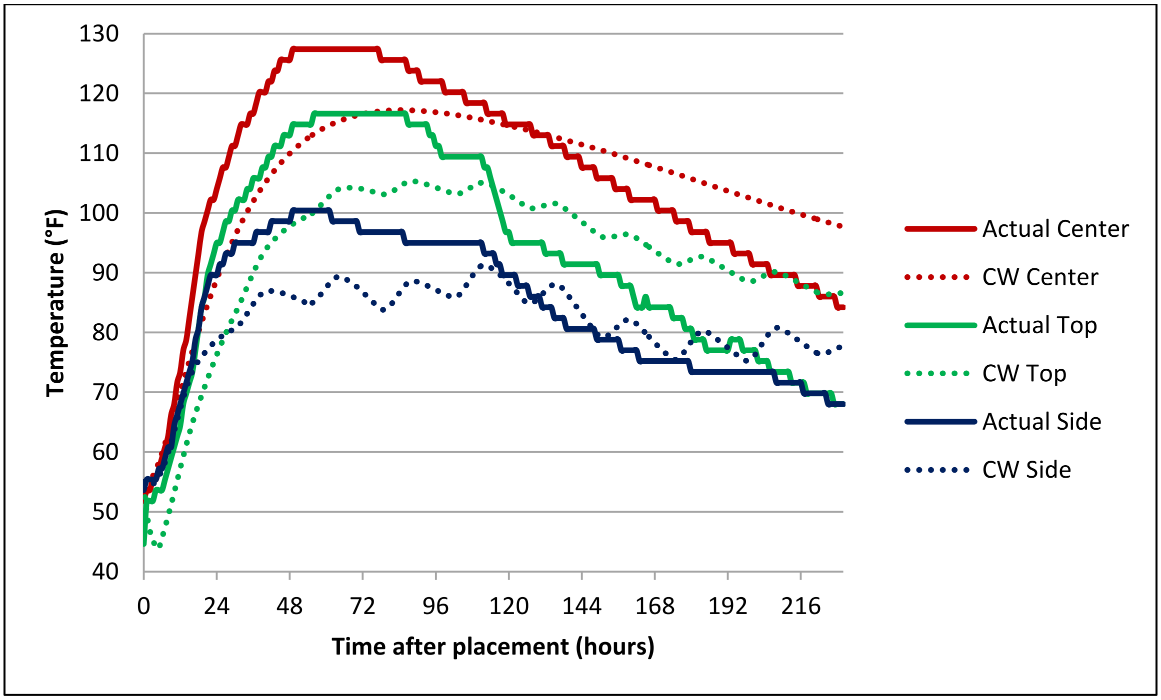


Figure C.4 US 34 case study thermal results – Pier 2 footing – D

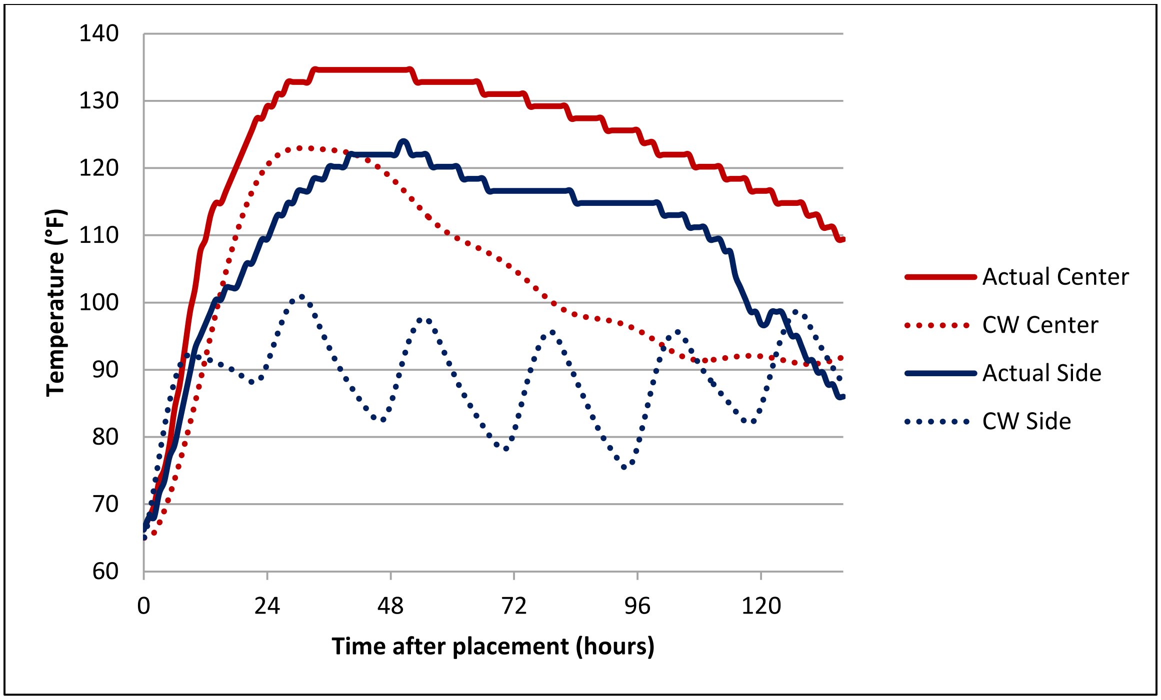


Figure C.5 US 34 case study thermal results – Pier 2 column – A

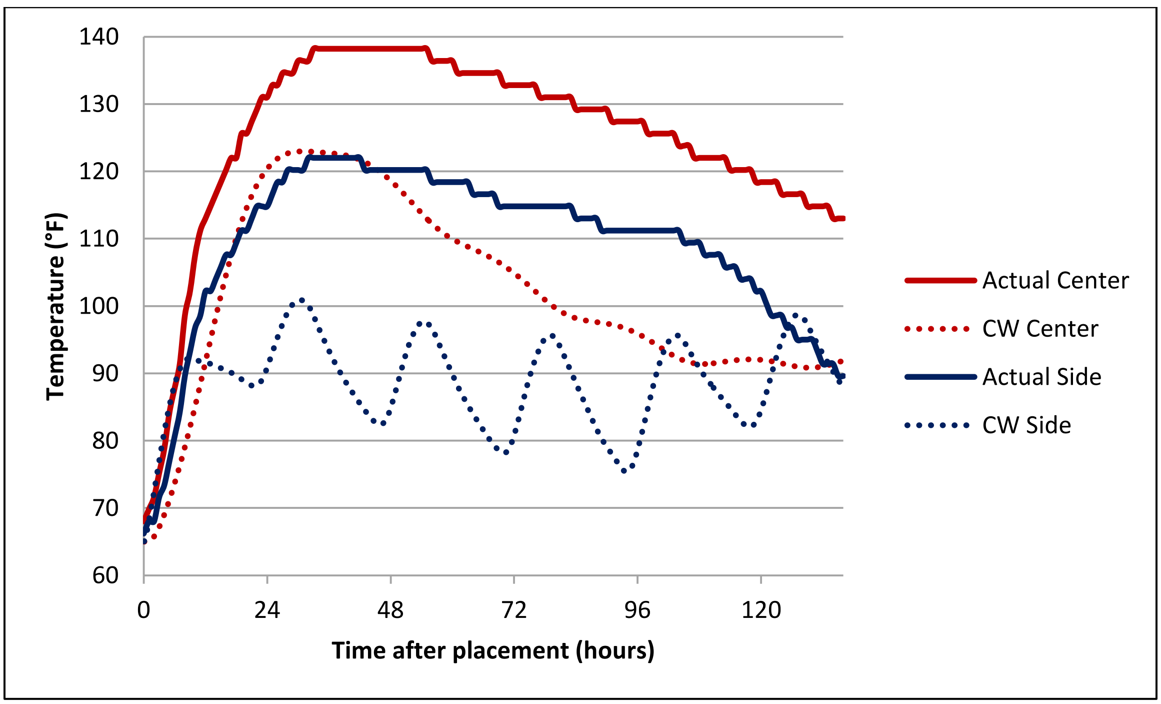


Figure C.6 US 34 case study thermal results – Pier 2 column – B

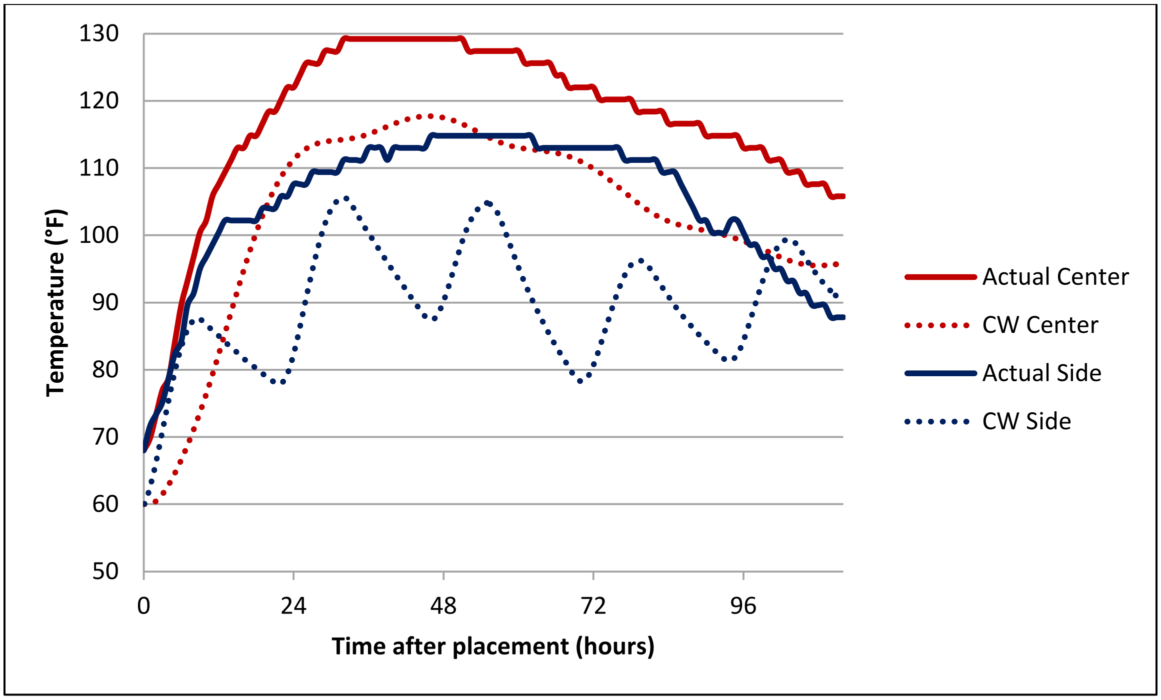


Figure C.7 US 34 case study thermal results – Pier 2 column – C

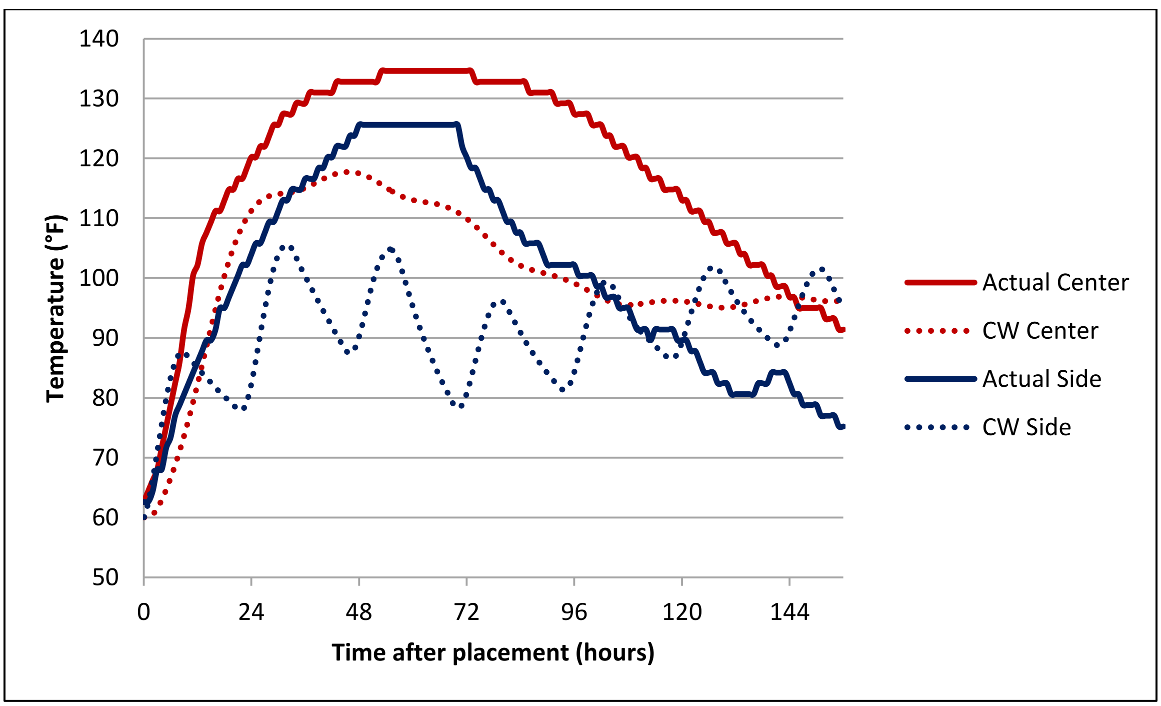


Figure C.8 US 34 case study thermal results – Pier 2 column – D

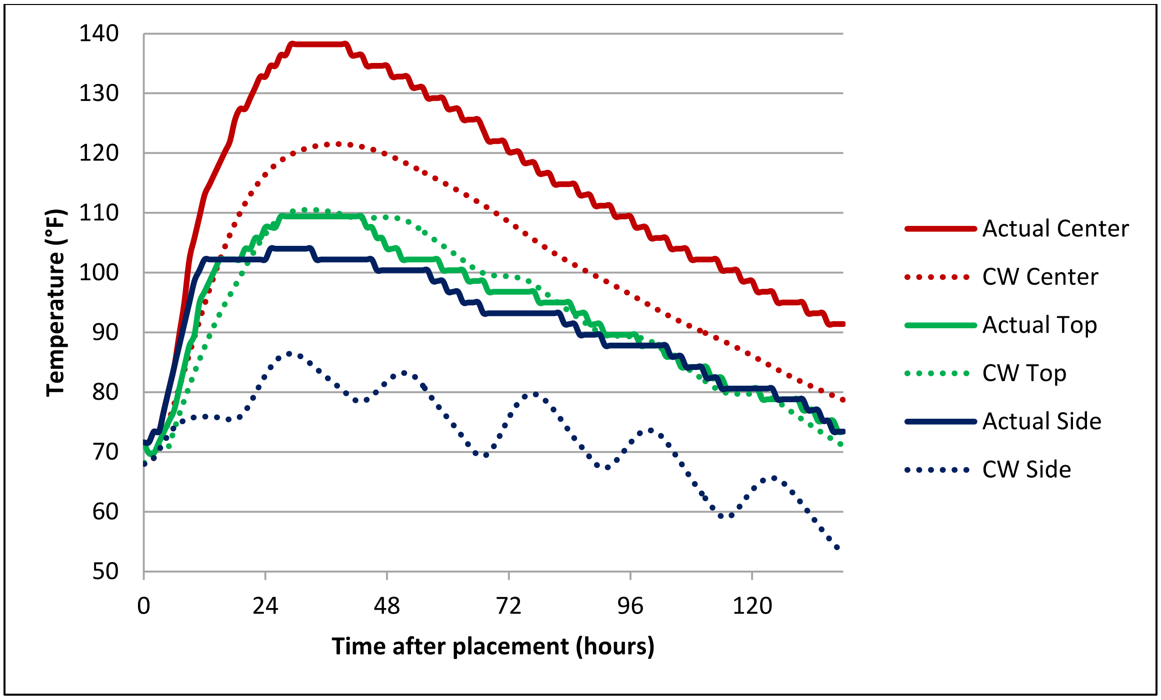


Figure C.9 US 34 case study thermal results – Pier 2 cap

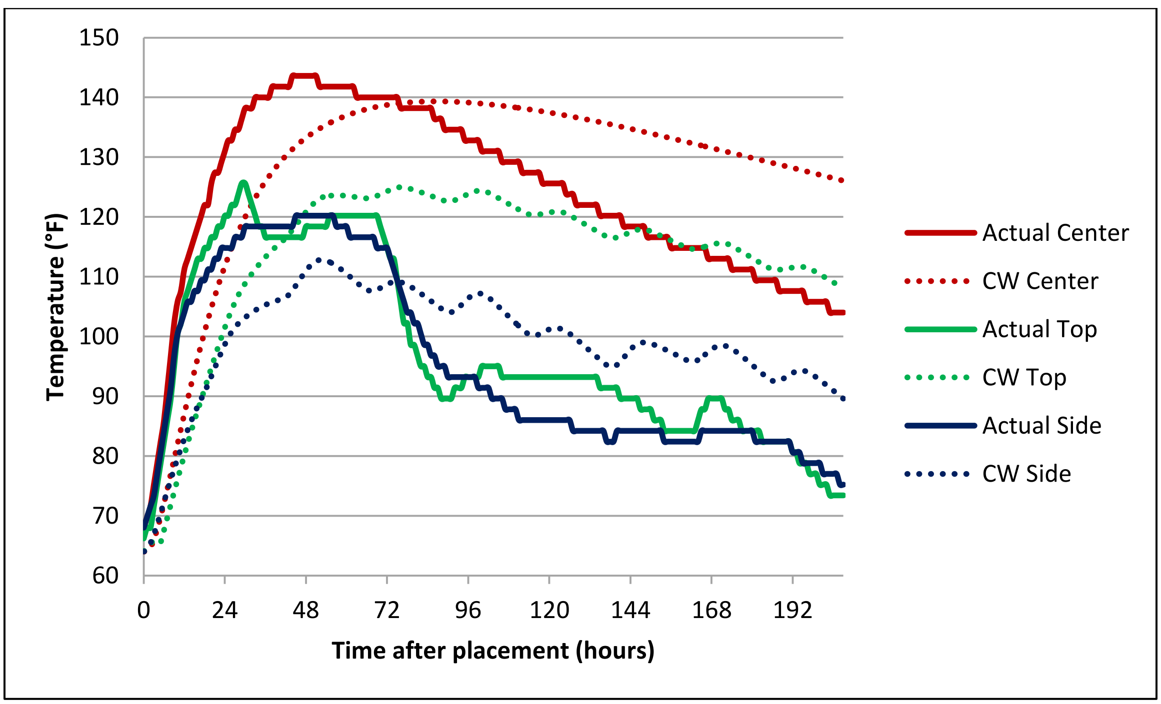


Figure C.10 US 34 case study thermal results – Pier 3 footing – C

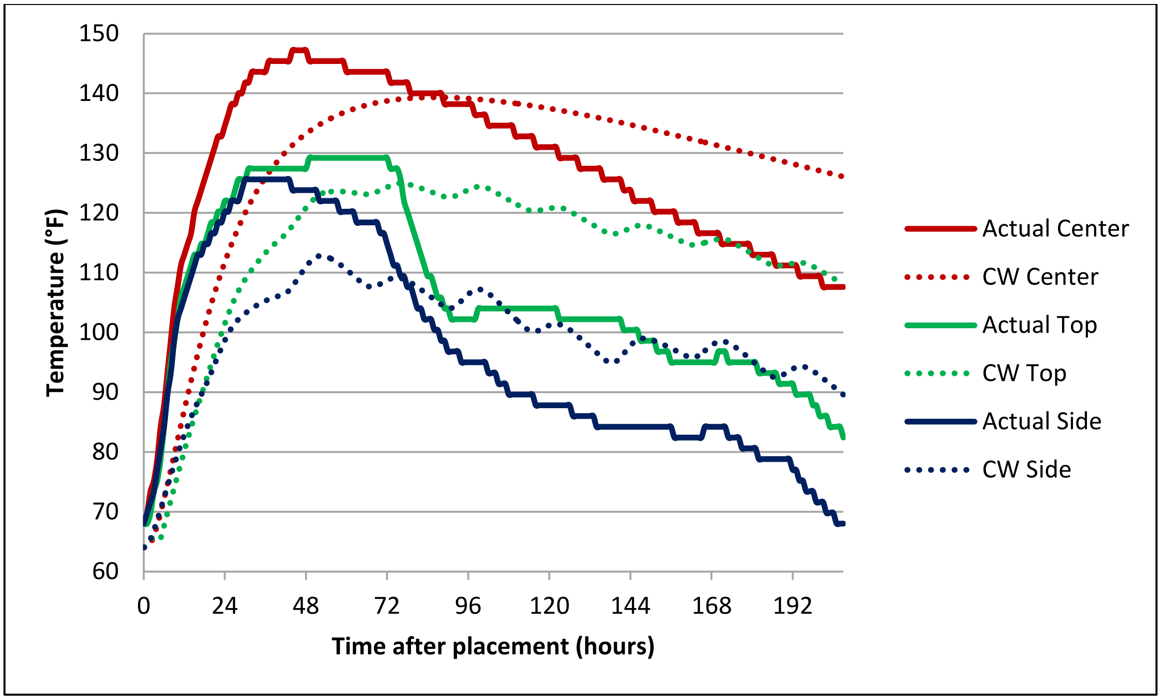


Figure C.11 US 34 case study thermal results – Pier 3 footing – D

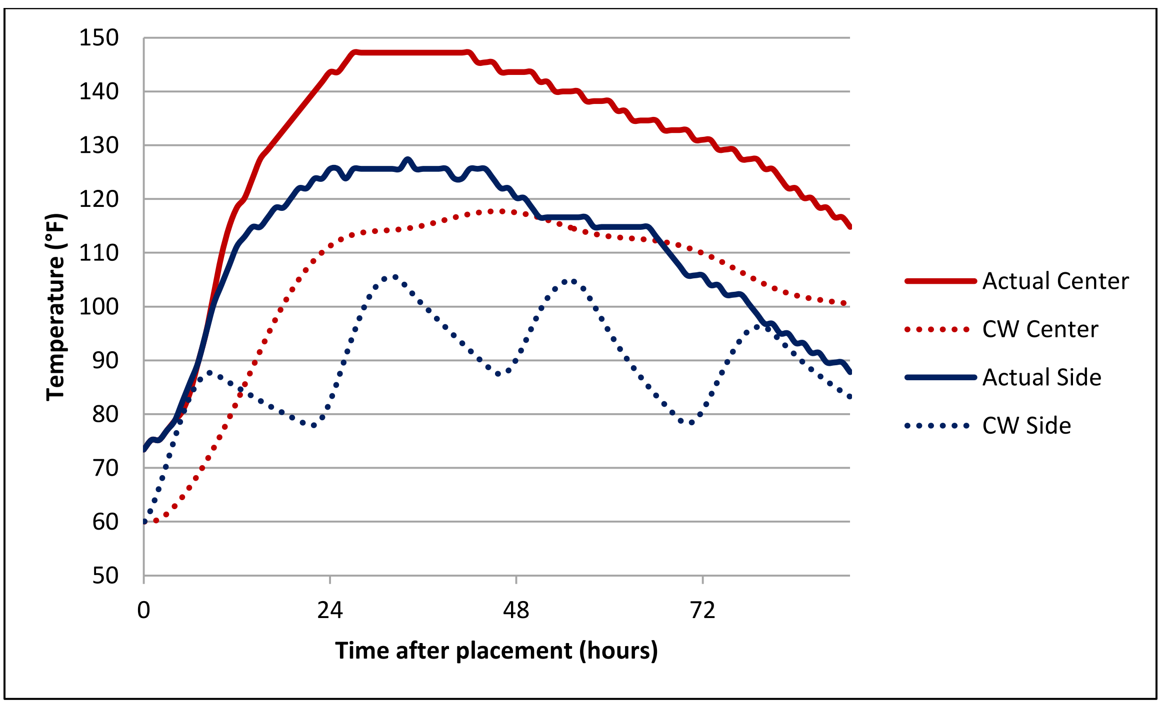


Figure C.12 US 34 case study thermal results – Pier 3 column – A

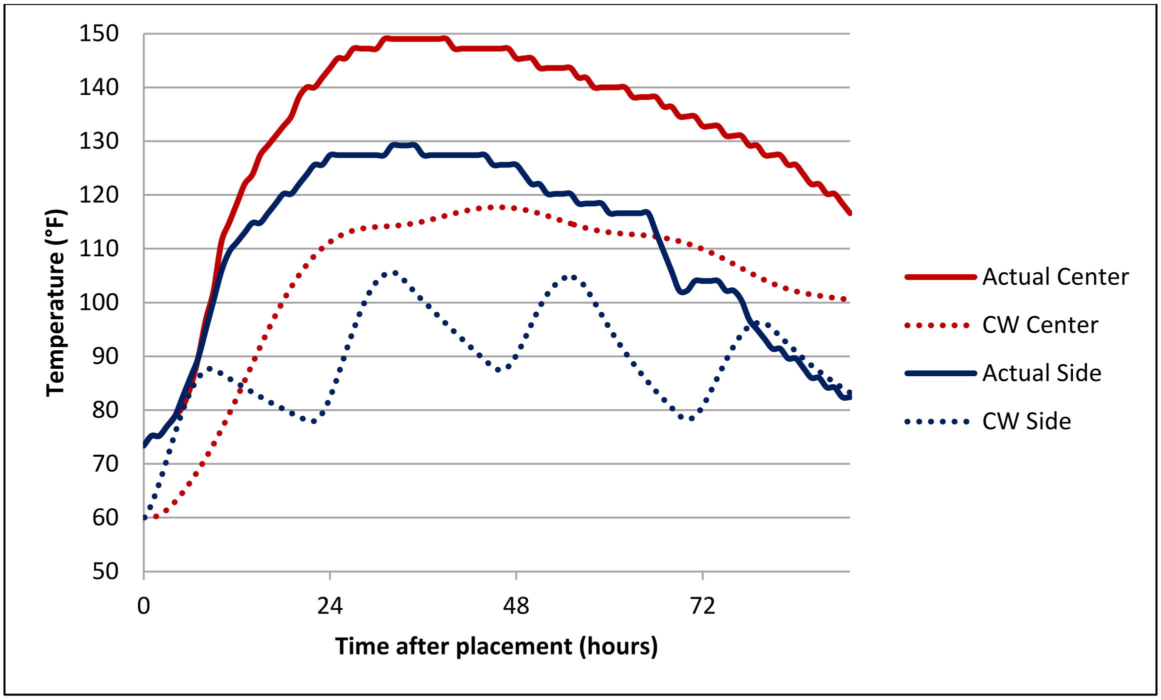


Figure C.13 US 34 case study thermal results – Pier 3 column – B

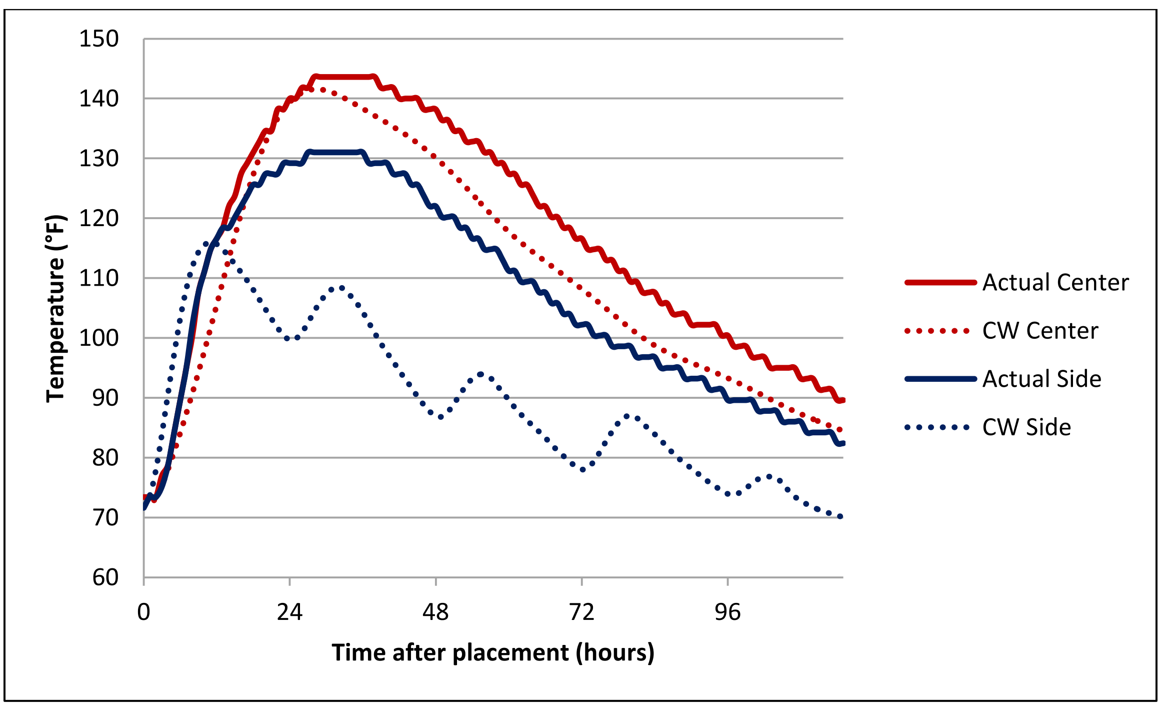


Figure C.14 US 34 case study thermal results – Pier 3 column – C

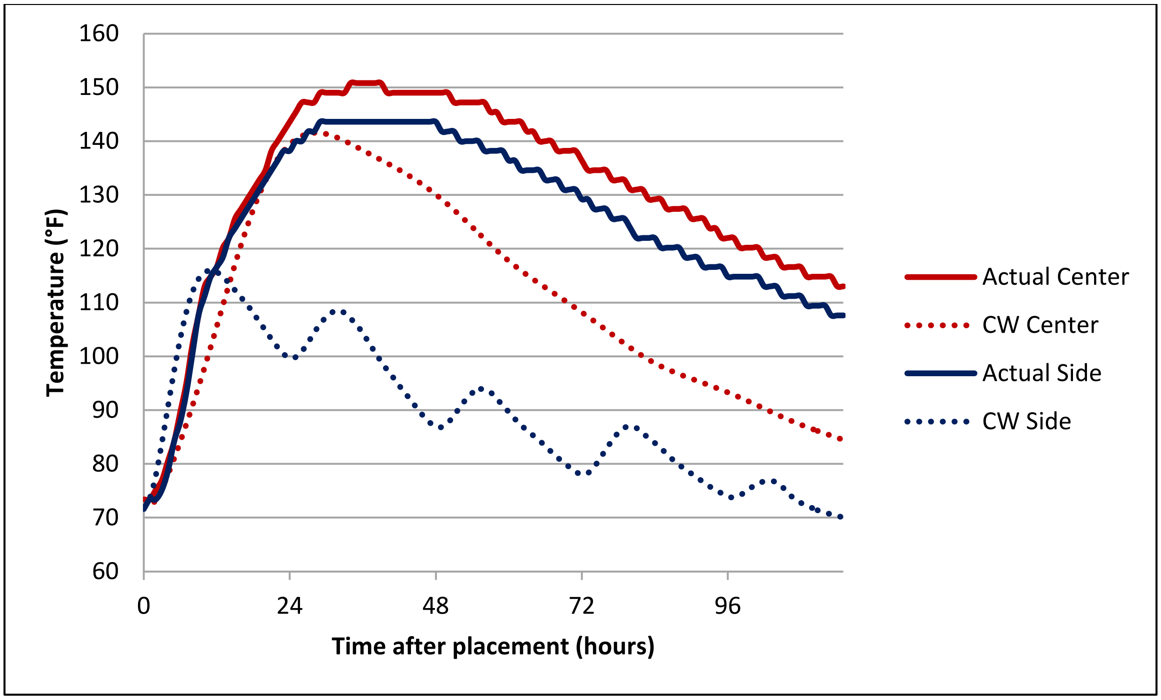


Figure C.15 US 34 case study thermal results – Pier 3 column – D