Iowa Mass Concrete for Bridge Foundations Study – Phase II

Final Report
February 2014

IOWA STATE UNIVERSITY
Institute for Transportation

Sponsored by
Iowa Department of Transportation
Federal Highway Administration
(InTrans Project 10-384)

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The preparation of this report was financed in part through funds provided by the Iowa Department of Transportation through its "Second Revised Agreement for the Management of Research Conducted by Iowa State University for the Iowa Department of Transportation" and its amendments.

Technical Report Documentation Page

<table>
<tr><td>1. Report No.
InTrans Project 10-384</td><td>2. Government Accession No.</td><td colspan="2">3. Recipient's Catalog No.</td></tr>
<tr><td colspan="2" rowspan="2">4. Title and Subtitle
Iowa Mass Concrete for Bridge Foundations Study – Phase II</td><td colspan="2">5. Report Date
February 2014</td></tr>
<tr><td colspan="2">6. Performing Organization Code
InTrans Project 10-384</td></tr>
<tr><td colspan="2">7. Author(s)
Jacob J. Shaw, Charles T. Jahren, Kejin Wang, and Jinxin "Linda" Li</td><td colspan="2">8. Performing Organization Report No.</td></tr>
<tr><td colspan="2" rowspan="2">9. Performing Organization Name and Address
Institute for Transportation
Iowa State University
2711 South Loop Drive, Suite 4700
Ames, IA 50010-8664</td><td colspan="2">10. Work Unit No. (TRAIS)</td></tr>
<tr><td colspan="2">11. Contract or Grant No.</td></tr>
<tr><td colspan="2" rowspan="2">12. Sponsoring Organization Name and Address
Iowa Department of Transportation, 800 Lincoln Way, Ames, IA 50010
Federal Highway Administration, U.S. Department of Transportation, 1200 New Jersey Avenue SE, Washington, DC 20590</td><td colspan="2">13. Type of Report and Period Covered
Phase II Final Report</td></tr>
<tr><td colspan="2">14. Sponsoring Agency Code
SPR RB09-011</td></tr>
<tr><td colspan="4">15. Supplementary Notes
Visit www.intrans.iastate.edu for color pdfs of this and other research reports.</td></tr>
<tr><td colspan="4">16. Abstract
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In the Phase I study, the research team reviewed published literature and current specifications on mass concrete. In addition, the team observed construction and reviewed thermal data from the westbound (WB) I-80 Missouri River Bridge. Finally, the researchers conducted an initial investigation of the thermal analysis software programs ConcreteWorks and 4C-Temp&Stress.

The Phase II study is aimed at developing guidelines for the design and construction of mass concrete placements associated with large bridge foundations. This phase included an additional review of published literature and a more in-depth investigation of current mass concrete specifications. In addition, the mass concrete construction of two bridges, the WB I-80 Missouri River Bridge and the US 34 Missouri River Bridge, was documented.

An investigation was conducted of the theory and application of 4C-Temp&Stress. ConcreteWorks and 4C-Temp&Stress were calibrated with thermal data recorded for the WB I-80 Missouri River Bridge and the US 34 Missouri River Bridge. ConcreteWorks and 4C-Temp&Stress were further verified by means of a sensitivity study.

Finally, conclusions and recommendations were developed, as included in this report.</td></tr>
<tr><td colspan="2">17. Key Words
bridge foundations—concrete thermal damage—mass concrete construction—thermal stress analysis—specifications</td><td colspan="2">18. Distribution Statement
No restrictions.</td></tr>
<tr><td>19. Security Classification (of this report)
Unclassified.</td><td>20. Security Classification (of this page)
Unclassified.</td><td>21. No. of Pages
161</td><td>22. Price
NA</td></tr>
</table>

Form DOT F 1700.7 (8-72) **Reproduction of completed page authorized**

IOWA MASS CONCRETE FOR BRIDGE FOUNDATIONS STUDY – PHASE II

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February 2014

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Sponsored by
the Iowa Department of Transportation and
the Federal Highway Administration
State Planning and Research Funding
(SPR RB09-011)

Preparation of this report was financed in part
through funds provided by the Iowa Department of Transportation
through its Research Management Agreement with the
Institute for Transportation
(InTrans Project 10-384)

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ACKNOWLEDGMENTS

The research team would like to acknowledge the Iowa Department of Transportation (DOT) for sponsoring this research and the Federal Highway Administration for state planning and research (SPR) funds used for this project.

The research team would like to thank our technical advisory committee (TAC) including James Nelson, Wayne Sunday, Todd Hanson, Ahmad Abu-Hawash, Chris Cromwell, Mark Dunn, Curt Monk, and Linda Narigon. We would also like to thank Scott Nixon, Steve Maifield, Jeremy Purvis, and Jason Cole of the Iowa DOT.

The research team would also like John Gajda and Jon Feld for their cooperation with our research. Finally, the research team wishes to thank Landon Streit, Dan Timmons, Ryan Cheeseman, and Steve Hague for their assistance with our research.

EXECUTIVE SUMMARY

The early-age thermal development of structural mass concrete elements has a significant impact on the future durability and longevity of the elements. If the heat of hydration is not controlled, the elements may be susceptible to thermal cracking and damage from delayed ettringite formation.

In the Phase I study, the research team reviewed published literature and current specifications on mass concrete. The team also observed construction and reviewed thermal data from the westbound (WB) I-80 Missouri River Bridge. In addition, the researchers conducted an initial investigation of the thermal analysis software programs ConcreteWorks and 4-CTemp&Stress.

The present study is aimed at developing guidelines for the design and construction of mass concrete placements associated with large bridge foundations. This phase consisted of the following research activities:

- Update literature review and preliminary thermal stress analysis
- Observe mass concrete construction practices
- Review construction observations and data from the WB I-80 Missouri River Bridge and US 34 Missouri River Bridge
- Model thermal activity in ConcreteWorks and 4C-Temp&Stress
- Develop recommendations

This report describes the activities conducted and results obtained from the Phase II study.

The Phase II study included an additional review of published literature and a more in-depth investigation of current mass concrete specifications. In addition, the mass concrete construction of two bridges, the WB I-80 Missouri River Bridge and the US 34 Missouri River Bridge, was documented.

An investigation was conducted regarding the theory and application of 4C-Temp&Stress. ConcreteWorks and 4C-Temp&Stress were calibrated by using thermal data recorded for the WB I-80 Missouri River Bridge and the US 34 Missouri River Bridge. ConcreteWorks and 4C-Temp&Stress were further verified by means of a sensitivity study.

Finally, conclusions and recommendations were developed as included in this report.

CHAPTER 1. INTRODUCTION

Mass concrete is a structural element of concrete with dimensions large enough to require actions to prevent excessive heat development. Heat development in a concrete element is the result of hydration of the cement. If the heat development is not controlled, the element may experience thermal cracking or delayed ettringite formation.

Thermal cracking is the result of large thermal gradients in a massive placement. Thermal gradients induce stress in the placement, which results from the exterior portion of the placement dissipating heat more rapidly than the interior portion. If the induced stress exceeds the tensile strength of the recently placed concrete, the placement is likely to experience thermal cracking. Historically, keeping the maximum temperature differential below 35°F was found to reduce the likelihood of thermal cracking.

Delayed ettringite formation, also known as heat-induced delayed expansion (HIDE), results from excessively high temperatures in a concrete placement. High temperatures in a placement decompose the ettringite that had been previously formed in the concrete and suppresses further ettringite formation.

In the future, if moisture is present in the concrete, ettringite may begin to form in the now solid cement paste, causing expansive pressure in the concrete. If the expansive pressures become too extreme, the placement may experience cracking. It has been established that preventing the maximum temperature in the placement from reaching 160°F will reduce the probability of HIDE.

1.1 Objectives

The objectives of this research are to provide insight on the early-age thermal development of mass concrete, provide recommendations for the Iowa Department of Transportation (DOT) mass concrete specification, and present best practices for mass concrete construction. The research utilized the software packages ConcreteWorks and 4C-Temp&Stress to model the thermal development of mass concrete elements.

1.2 Iowa DOT Mass Concrete Specification

The Iowa DOT currently has a developmental specification for mass concrete (Control Heat of Hydration DS-09047, August 17, 2010). The specification was based on national industry practices and experiences on the westbound (WB) I-80 bridge over the Missouri River (between Council Bluffs, Iowa and Omaha, Nebraska). The goal of the specification is to provide concrete structures free of thermal damage resulting from heat of hydration during the curing of large concrete cross-sections.

To mitigate the effects of heat of hydration, the Iowa DOT specification has implemented thermal limits for mass concrete placements. To prevent delayed ettringite formation, the specification states that the maximum temperature in a placement may not exceed 160°F during the time of heat dissipation. To prevent thermal cracking, the specification has laid out maximum temperature differentials for placements as shown in Table 1.1.

Table 1.1. Iowa DOT maximum allowable temperature difference limits

Time after Placement (hrs)	Maximum Temperature Difference (°F)
0-24	20
24-48	30
48-72	40
>72	50

1.3 Literature Review

Historically, there have been many methods used to control the heat of hydration of mass concrete placements and reduce the thermal damage. Approaches that put limits on mix proportions and material properties include using a low-cement content, reduced heat cements and/or increased aggregate size; increasing coarse aggregate, fly ash, and/or ground granulated blast furnace slag (GGBFS) content; and requiring water-reducing admixtures.

Construction practices used to reduce thermal damage include reducing the fresh placement temperature, post-cooling the concrete with internal cooling pipes, pouring placements during cooler times (nighttime or cooler times of the year), water curing, reducing placement lift height, and using steel forms for rapid heat dissipation or wood forms and insulation for reduced heat dissipation (Kosmatka et al. 2002).

1.3.1 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.

1.3.1.1 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 1.1.

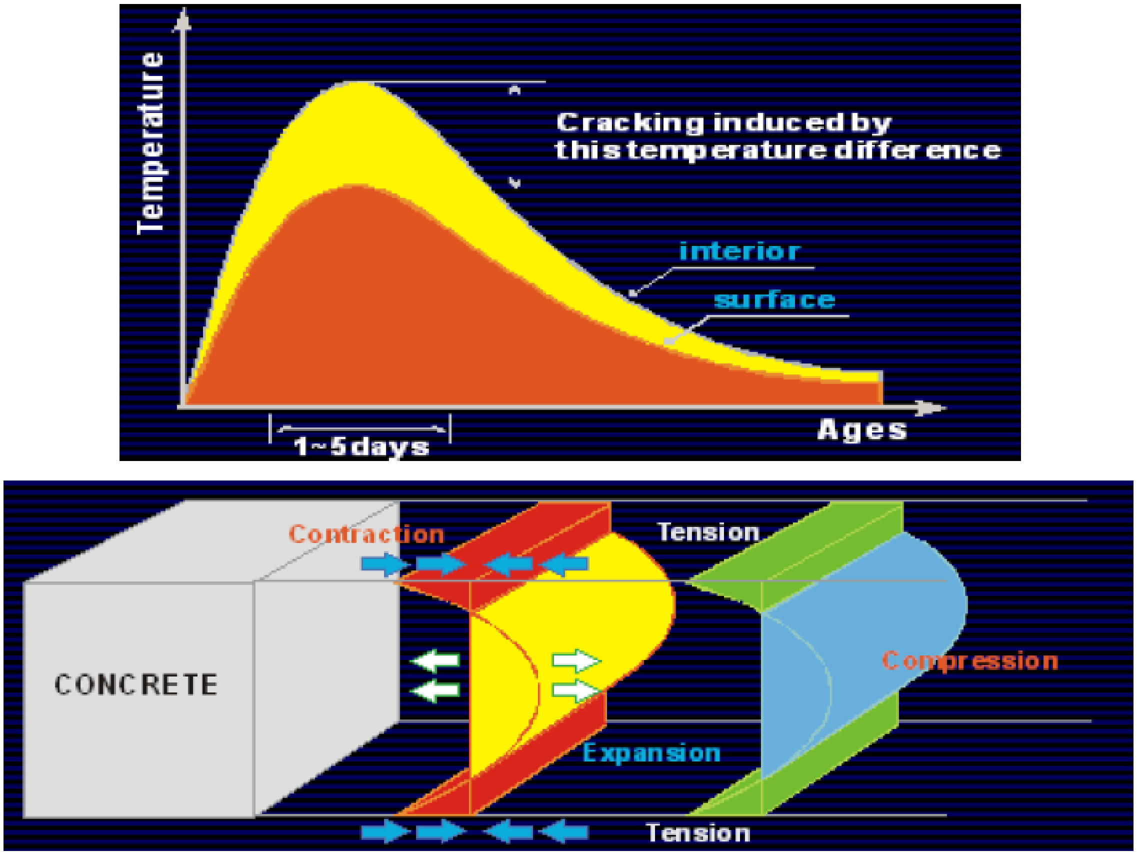


Figure 1.1. Internal restraint mechanism due to thermal gradients (Kim 2010)

If the tensile stress in the concrete exceeds the developed tensile strength of the concrete, the concrete will experience thermal cracking.

1.3.1.2 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 1.2 shows how the volumetric changes of mass concrete are resisted by external restraint.

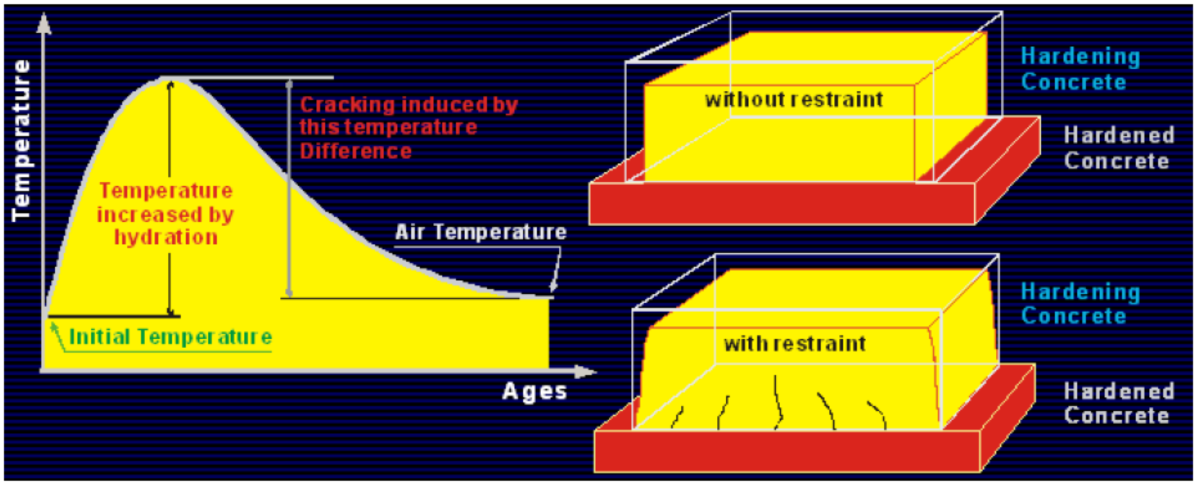


Figure 1.2. External restraint mechanism due to thermal gradients (Kim 2010)

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.

CHAPTER 2. SPECIFICATION SURVEY

2.1 Overview

The chapter describes the mass concrete specification survey for state and federal agencies in the US. The following three sections describe the methodology utilized to identify the specifications, the results of the identified specifications, and a discussion of the results.

2.2 Introduction

Mass concrete specification requirements throughout the US vary greatly between agencies. The goal of the specification survey was to identify current trends in mass concrete requirements in the US. Aspects of the mass concrete specification that were surveyed included the definition of mass concrete, concrete mix portion requirements, thermal control requirements, construction requirements, design requirements, and additional special requirements.

The next section of this chapter describes the methodology that was used to complete the specification survey. The section after that describes the results of the survey. The final section of provides a discussion of the sensitivity survey results.

2.3 Methodology

The specification survey was completed by investigating the mass concrete specification of the 51 state highway agencies, including the District of Columbia (DC) 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 identify specifications independently. 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 the agency either did not respond, was unable to identify the specification, or did not have a specification. Furthermore, agencies with minimal mass concrete specifications were excluded from the survey for lack of scope. As an example of lack of scope, the standard specification identifies only that mass concrete shall use type II cement.

2.4 Results

Thirteen different mass concrete specifications were identified including standard specifications, special provisions, special notes, developmental specifications, and structural design guidelines, as shown in Table 2.1. As listed on the right side of the table, the researchers were unable to identify a mass concrete specification for 40 agencies.

Table 2.1. 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 DOR
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

The type, reference, and year for the identified specifications are listed in Table 2.2.

Table 2.2. 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

2.4.1 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. In addition, mass concrete may be defined by the volume of placement, surface area of the placement, or 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 2.3 indicates the definition of mass concrete provided by the specifications identified in the survey. The definitions vary greatly between agencies, with the lesser dimensions varying from 3 to 5 ft. In addition, 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 2.3. Mass concrete definition by agency

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

NA- not available

2.4.2 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 2.4.

Table 2.4. 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 to 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 (CalTrans) 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 to 50°F. The gradient approach allows the contractor to take advantage of the increase in concrete strength over time.

2.4.3 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 2.5 and Table 2.6 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 2.5. 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 2.6. 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. In addition, there is little commonality between agencies in regard to mix proportion requirements.

2.4.4 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, only the fresh placement temperature of a placement is restricted typically for the construction of mass concrete elements.

Table 2.7 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. In addition, there is little commonality in fresh placement temperature restrictions between agencies. The range of maximum fresh placement temperature is 60 to 90°F for agencies with identified specifications.

Table 2.7. 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

2.4.5 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 with 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 gauge the thermal stresses in the placement accurately. If sensors are not installed properly, the temperature reading may have significant error, providing misleading results. In addition, the surface sensors may compromise the durability and cosmetic appearance of the concrete if installed too close to the surface. To capture accurate results, sensors must be installed in the proper location in the placement. Table 2.8 shows the sensor location requirements and the surface cover requirements for sensors placed near the surface.

Table 2.8. 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. In addition, the ambient air temperature and entrance/exit of cooling water	1-3 in.
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 in. 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

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

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 2.9 shows the thermal control completion time by agency.

Table 2.9. 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

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. In addition, this requirement will typically force the placement to reach a maximum temperature and to begin to cool.

2.5 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 still generally have large discrepancies in requirements.

CHAPTER 3. CASE STUDIES

3.1 Introduction

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

3.2 Westbound I-80 Missouri River Bridge Overview

The WB I-80 Missouri River Bridge is a 2,477 ft 10 in. by 84 ft continuous welded girder bridge. The bridge spans the Missouri River connecting Council Bluffs, Iowa to Omaha, Nebraska. 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 ft to 10.5 ft.

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

3.3 US 34 Missouri River Bridge Overview

The US 34 Missouri River Bridge is a 3,276 ft 1 in. by 86 ft 3 in. continuous welded girder bridge with pretensioned, prestressed concrete beam approaches. The bridge crosses the Missouri River south of Omaha, Nebraska and Council Bluffs, Iowa. 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 ft to 6.5 ft.

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

3.4 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 in Chapter 6.

3.4.1 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 retains the footings externally. To support the footings on the WB I-80 Bridge, two techniques were used: steel bearing piles and drilled shafts. Piers 1 through 5, 7, 10, and 11utilized HP 12 x 84 steel bearing piles to support the respective footings. The Pier 6 footing was supported by 48 in. diameter drilled shafts, and Piers 7 and 8 were supported by 72 in. diameter drilled shafts. Similarly, on the US 34 Bridge, Piers 1 through 4 and 7 through 17 were supported by HP 14 x 89 steel bearing piles. Piers 5 and 6 were supported by 30 individual 48 in. 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 water from seeping through the foundation soils into the area where the footing will be cast. 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 rivers in the Midwest. 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 3.1 and Figure 3.2, respectively.

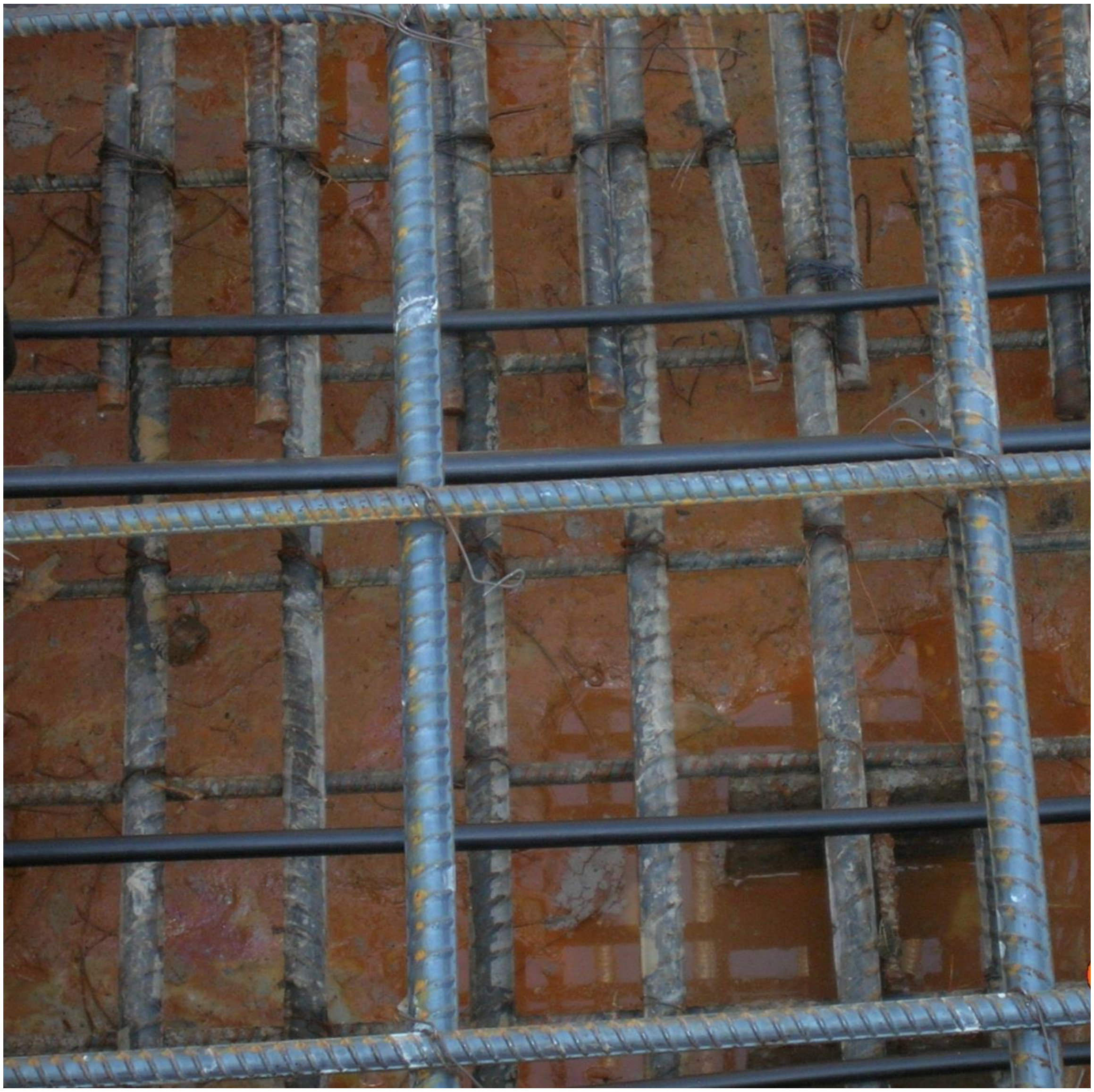


Figure 3.1. Clay subbase with steel bearing pile

Figure 3.2. Crushed rock subbase with steel bearing pile

3.4.2 Formwork Material

Two different formwork materials, wood and steel, were used to form the placements on both the WB I-80 and US 34 Bridge. 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 used typically 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 3.3. The steel formwork that was used on both projects consisted of yellow EFCO formwork. Figure 3.4 and Figure 3.5 show typical examples of the steel formwork used on both projects.

Figure 3.3. US 34 Missouri River Bridge Pier 3 footing

Figure 3.4. WB I-80 Missouri River Bridge column formwork

Figure 3.5. US 34 Missouri River Bridge column formwork

3.4.3 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, footing, stem, column, and cap, as shown by Figure 3.6. The allowable locations for the construction joints were designated by the bridge designer.

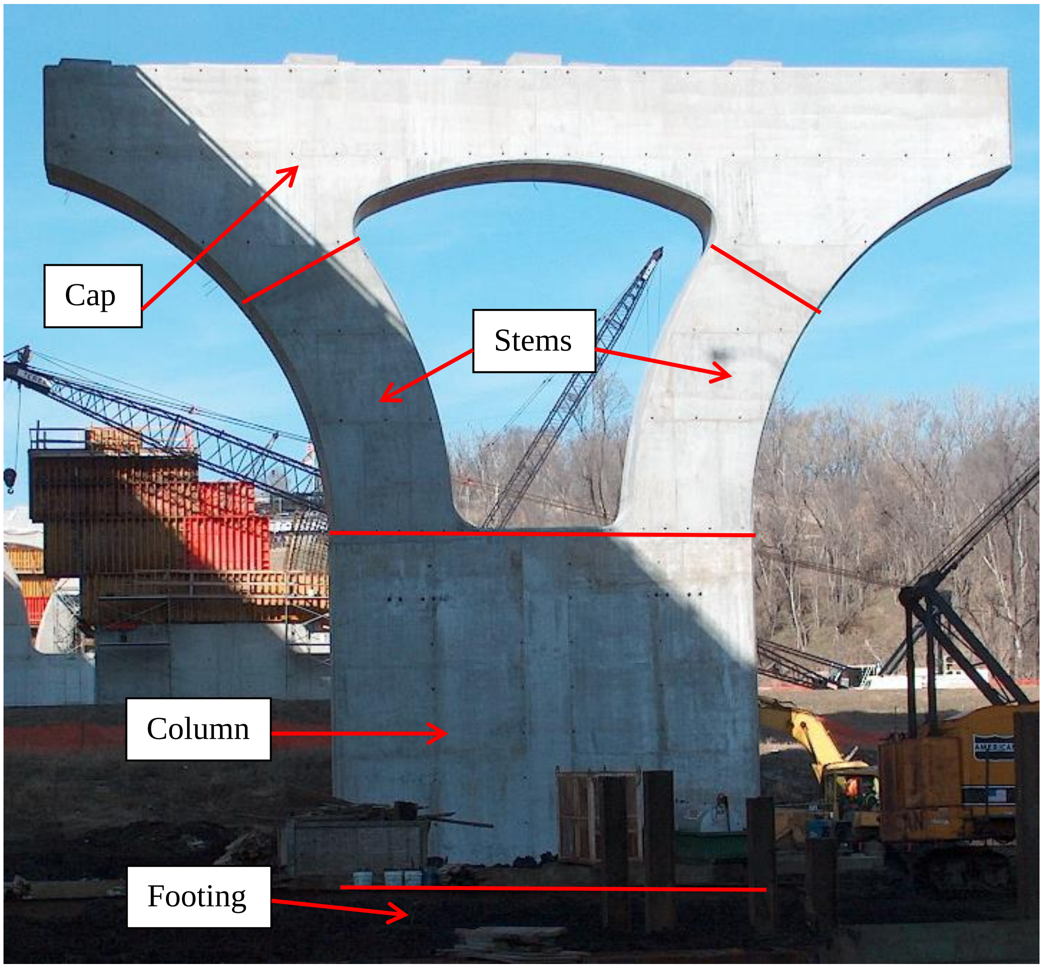


Figure 3.6. Typical bridge pier element sections

For small or simple elements, the number of pier elements was reduced for both bridges. The stem and column on Pier 1 from 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 through 3 and 8 through 17, which simplified the geometry and reduced the size of each element. As a result, the piers were poured in three sections: footing, column, and cap.

3.4.4 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 buckets in congested areas, as shown by Figure 3.7, especially when equipped with an extended tremie pipe.

Figure 3.7. US 34 Bridge Pier 4 footing concrete placement

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 reduces the risk of segregation for the concrete. 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 dropped generally 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 3.8 shows the use of a concrete hopper bucket to pour a 5.5 ft deep foundation.

Figure 3.8. US 34 Bridge Pier 2 footing concrete placement

3.4.5 Consolidation

Consolidation of concrete is an essential step in the placement of mass concrete. If concrete is not consolidated properly, the concrete element will have substantial voids reducing the overall strength and durability of the element. The need for concrete consolidation on both the WB I-80 and US 34 bridges required the utilization of concrete vibrators with flexible shafts to vibrate the concrete internally. To assure that the concrete was consolidated adequately, the concrete was vibrated at each individual concrete placement layer. A typical example of the vibratory compactor used on both bridges is shown in Figure 3.9.

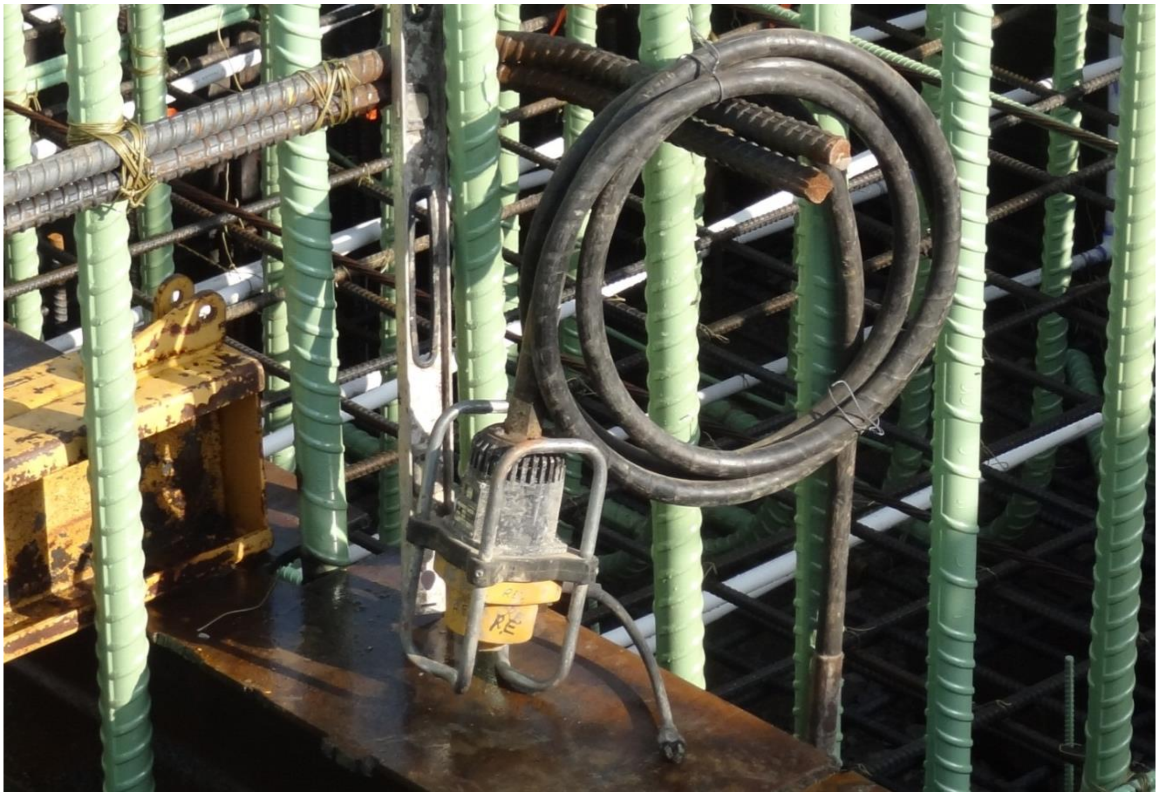


Figure 3.9. Jensen Construction Company flexible shaft vibratory compactor

3.4.6 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 (generally rebar) protruding from the placement. 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 control the thermal development of the placements efficiently, 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 adding insulating blankets to the placement to prevent exceeding the maximum temperature difference limits. In some instances, additional insulating blankets were added to all sides, but were limited typically 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, insulation blankets were removed occasionally to dissipate the heat generated in the placement more rapidly. 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 attached to previous formwork. In addition, many blankets had small rips and tears.

To attach the insulation to wooden formwork, the insulation was nailed typically 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 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 3.10 shows a typical situation with an insulating blanket attached to wood formwork.

Figure 3.10. Insulation attached to wood-formed footings

The insulation blankets were attached to the exterior of the formwork generally 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 and, therefore, extra care was taken to assure that the blankets were lapped properly on the top surfaces.

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 attached to the formwork directly. To provide additional rigidity to formwork, shores were installed to support the formwork walls by the cofferdam sheet pile walls as shown in Figure 3.11. As a result, the insulating blankets were unable to be attached directly to the formwork.

Figure 3.11. WB I-80 Bridge wood-formed footing shoring

In an effort to provide thermal insulation for 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 in Figure 3.12. The insulating blankets were intended to prevent airflow along the sides of the footing and to capture the heat of the placement in the void. The effectiveness of the insulating blanket installed on top of the shoring is unknown.

Figure 3.12. Shored formwork insulating blanket

Elevated placements on both bridges occasionally utilized catwalks 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 in Figure 3.13.

Figure 3.13. 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 the insulation blankets be tied to the formwork. The insulating blankets were tied with reinforcing tie-wire onto the formwork struts. A typical example of insulating blankets attached to steel formwork for both bridges is shown in Figure 3.14.

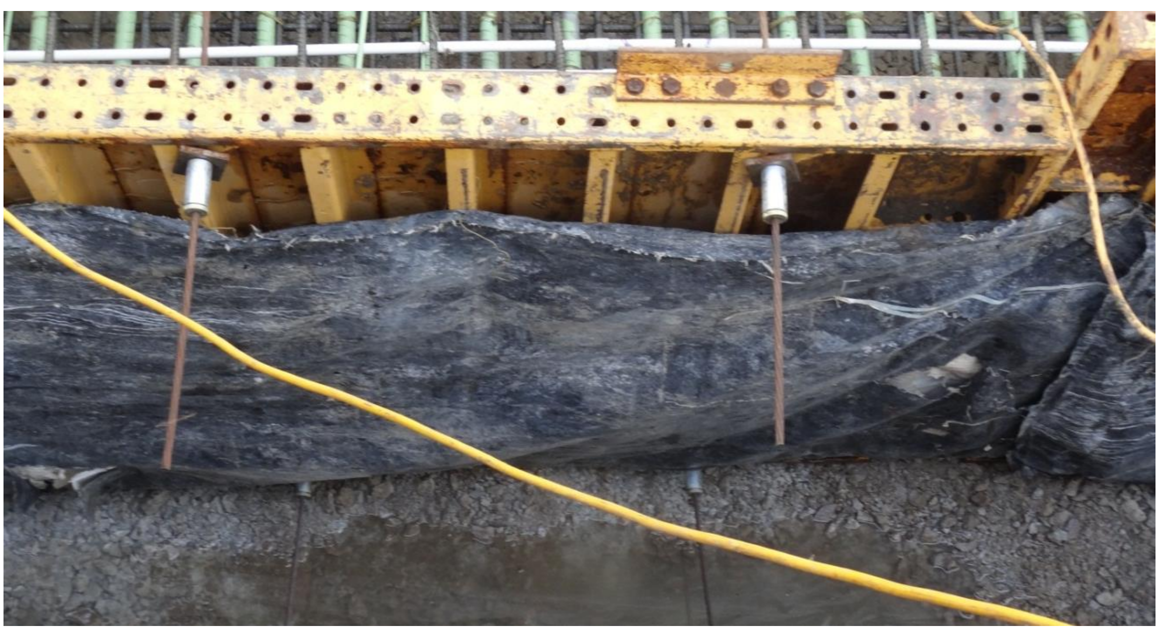


Figure 3.14. Steel formed footing with insulating blanket

3.4.7 Cooling Pipes

Cooling pipes were utilized on both bridges to control the thermal development of placements with relatively large dimensions. Cooling pipes were also used occasionally 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 3.15.

Figure 3.15. 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 more than 400 ft horizontally and more than 50 ft vertically to supply the cooling pipe system for the Pier 4 cap.

The water was pumped through piping, approximately 4 to 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 3.16.

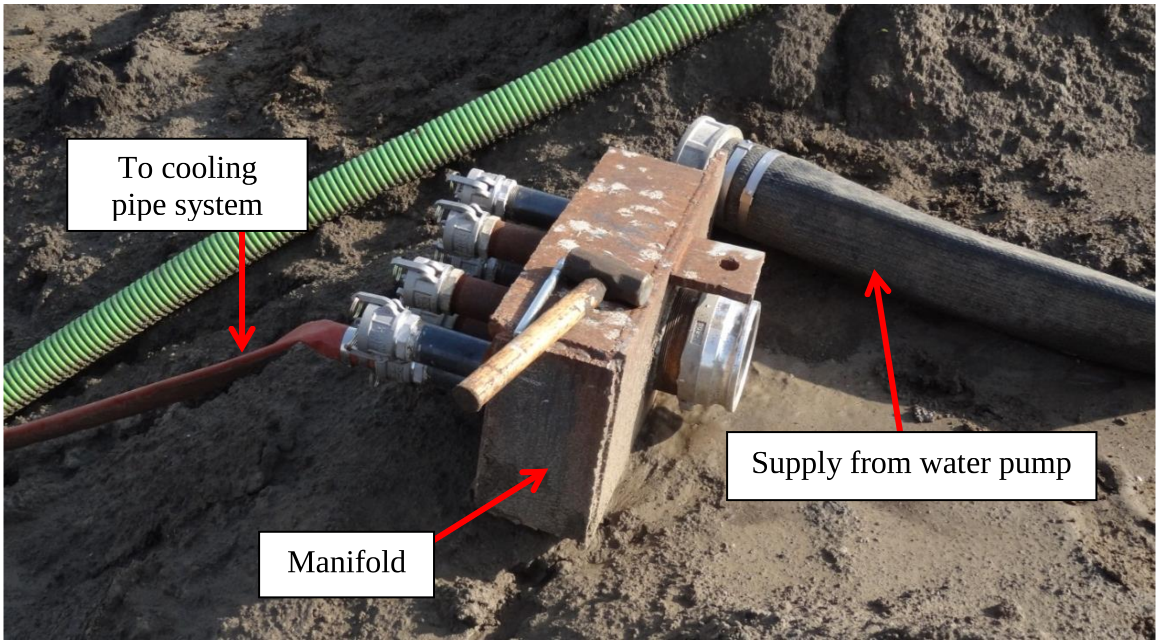


Figure 3.16. Cooling pipe system supply line manifold

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 in Figure 3.17 and Figure 3.18, 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 3.17. WB I-80 Bridge cooling pipe system manifold

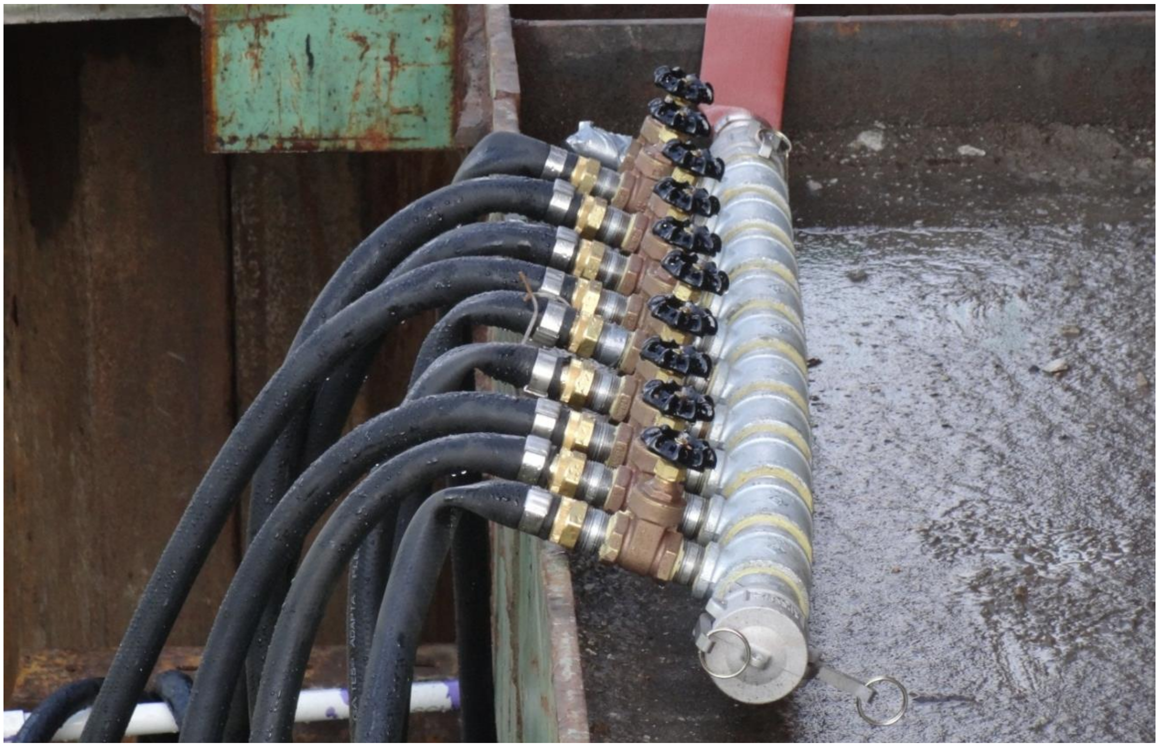


Figure 3.18. US 34 Bridge cooling pipe system manifold

Each cooling pipe system consisted of several loops that pumped the water through the placement. Each loop was spaced typically in both the vertical and horizontal directions by two to three feet. In addition, the material utilized to construct the loops inside the placement varied between the two projects. The WB I-80 Bridge utilized 3/4 inch PEX (cross-linked polyethylene) piping as shown in Figure 3.19 as well as 3/4 inch PVC (polyvinyl chloride) piping. The US 34 Bridge utilized 1 inch PVC piping as shown in Figure 3.20. 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 3.19. PEX cooling pipes being installed on a WB I-80 bridge footing (Iowa DOT)

Figure 3.20. 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 drained into the cofferdam. The water that was drained into the cofferdam would be pumped out subsequently by the cofferdam dewatering pumps.

Given that 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 shocking the placement thermally with the cooling pipes, the circulation of water began immediately after the completion of the pour. In addition, 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 continued generally 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 contractor-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 to 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 cut off at the surface of the placement and pumped full of high-strength grout.

3.4.8 Thermal Monitoring

In accordance with the Iowa DOT mass concrete developmental specification, each placement on both bridges defined as mass concrete was 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 1921 thermal 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, top surface, and 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, and 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. Given 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 3.21 and Figure 3.22. 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 3.23 shows the rebar concrete cover for a footing, with the red chalk line representing the finish pour height.

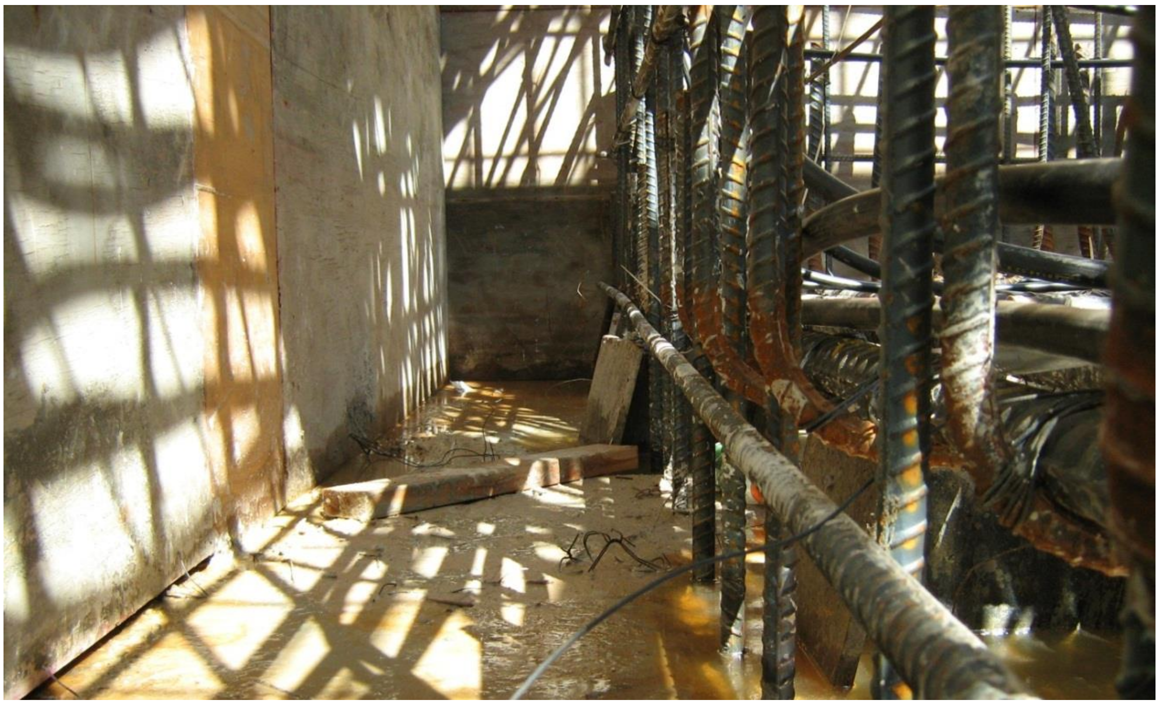


Figure 3.21. Distance between formwork and outermost rebar/thermal sensor location – large distance

Figure 3.22. Distance between formwork and outermost rebar/thermal sensor location

Figure 3.23. Typical rebar cover for mass concrete footing

To determine the location of each sensor, typically, no measuring devices were used. Generally, the sensors were placed approximately 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 show the installation of the thermal sensors and the 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 were recorded in one-hour intervals. In addition, the data were monitored remotely, by checking the thermal readings visually, 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 were submitted to the Iowa DOT as part of the required field reports.

3.4.9 Formwork Removal

To prevent thermal damage to the placement, formwork was retained on the placement typically 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 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 contractors, ranged from 91 to 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.

3.5 Concrete Mix Proportion

Both bridges utilized the same mix proportion. The concrete mix proportion along with the material and mechanical properties are described in detail in Chapter 5.

3.6 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 are described in detail in Chapter 7.

CHAPTER 4. 4C-TEMP&STRESS

4.1 Overview

4C-Temp&Stress (4C) is a computer program developed by the Danish Technological Institute that provides the abilities to outline concrete geometry, carry out full or approximated calculations, and view the results in a graphical interface. The program can perform thermal analysis and stress analysis of mass concrete development.

Assumptions are used in 4C to simplify the FE analyses. For example, the ambient temperature is an assumed sinusoidal curve varying from a single maximum and minimum temperature value for the entire duration of the analysis, while actual weather conditions differ from day to day. The maximum temperature sensor is assumed to be located at the center of concrete, and the minimum temperature sensor is assumed to be placed at 3 in. from the top surface of concrete.

Furthermore, specific heat, thermal conductivity, and coefficient of thermal expansion can be assumed using obtained literature values to simplify the analysis. Other assumptions, such as mesh size, analysis period, and calculation parameters, can be found in the 4C user manual (1998).

Table 5.1 describes the required inputs of 4C.

Table 5.2 further indicates prediction models that can be used to obtain those inputs. The models were established based on literature findings (Ge 2005).

Table 5.1. Summary of 4C inputs and how to obtain them

Item	Detailed description	Source
Structural Geometry	Width, depth, and length Cooling pipe layout Foundation types (concrete or soil)	Structure Design
Construction	concrete placement temperature cast rate insulation methods formwork material form removal time cooling process	Designed, Measured, or Assumed
Environmental and boundary conditions	ambient temperature, wind speed	Measured or Collected
Fresh concrete properties	slump, w/c ratio, air content, density specific heat, thermal conductivity, coefficient of thermal expansion	Collected or Predicted
Hardened concrete properties	maturity and its relationship with heat development	Measured
	E-modulus	Measured or Predicted
	Poison ratio	4C default
	Compressive strength	Measured or Predicted
	Tensile strength	Measured or Predicted
	Creep	Measured or Predicted

Table 5.2. Models used for prediction of concrete properties in 4C program

	Source
E-modulus	$\mathbf{E_c = 80811(f'_c)^{0.4659}}$ (Ge 2005)
Tensile strength	$f_t = 1.019 \times fc^{0.7068}$ (Ge 2005)
Creep	$J(\Delta t_{load,} t_0) = \frac{1}{(E(t))} + \Delta J(\Delta t_{load,} t_0)$ For $\Delta t_0 \leq \Delta t_{load} < \Delta t_1$ $\Delta J(\Delta t_{load,} t_0) = a_1 * \log(\frac{\Delta t_{load}}{\Delta t_0})$ For $\Delta t_{load} \geq \Delta t_{t1}$ $\Delta J(\Delta t_{load,} t_0) = a_1 * \log\left(\frac{\Delta t_{load}}{\Delta t_0}\right) + a_2 * \log\left(\frac{\Delta t_{load}}{\Delta t_0}\right)$ $a_i(t_0) = a_i^{min} + (a_i^{max}\text{-}a_i^{min}) * \exp(-(\frac{t_0 - t_s}{t_{ai}})^{n^{ai}})$ (Westman 1999)

Table 5.3 shows the output item comparison between 4C and ConcreteWorks, another computer software package used by the research team in the first phase of these projects. Compared to ConcreteWorks, 4C-Temp&Stress has capacity to build databases for structures, concrete, formwork/insulation, and materials, etc., but users can only choose established concrete database and geometry of concrete members in ConcreteWorks.

Table 5.3. Comparison of 4C outputs with ConcreteWorks

Output Items	4C	ConcreteWorks
Max. temperature of the volume	x	x
Min. temperature of the volume	x	x
Max. temperature of specific point	x	
Min. temperature of specific point	x	
Max. temperature difference of the volume		x
Ambient temperature	x	x
Average temperature of the volume	x	
Average temperature of specific point	x	

The 4C program has the other advantage of presenting more detailed temperature data. In the 4C program, the analysis at specific points could also be applied on maturity, strength, and stress results demonstration. Not only are multiple choices to exhibit analysis results along the time available, but showing iso-curves at a given time is also an advantage of 4C-Temp&Stress. (An iso-curve is a curve along which the function has a constant value in the cross section of the concrete structure.)

Furthermore, 4C also considers the effects of cooling pipes or heating wires, which are not considered in ConcreteWorks. Users can define the cooling pipe/heating wires used in the project and simulate thermal development more closely to the real construction. In addition, the 4C-Temp&Stress software package is more effective in terms of calculation time than ConcreteWorks, and a longer analysis period could be designed, while ConcreteWorks could only consider a 14 day temperature prediction and a 7 day cracking potential prediction.

The following points could be considered as challenges for users of 4C-Temp&Stress:

- 4C software normally works in the Windows XP environment, and might be compatible with Windows 7 with 32 bit, but not 64 bit
- Comparing both programs, ConcreteWorks is free software and uses English units, which makes the program more applicable in the US, while 4C is a commercial program with SI units only
- 4C-Temp&Stress has more inputs, which require users to be more knowledgeable in order to collect the information and make reasonable assumptions, while ConcreteWorks has many defaults and does not require so much information to input
- The other potential shortfall for 4C occurs when the volume of concrete becomes extremely large, generated meshes are relatively fine, or cement content is increased to relatively high, and the calculation for the analysis could not be extended for a relatively long period
- 4C ambient temperature inputs are not flexible and could only be assumed as sine-curve or constant, while actual ambient temperature varies day by day, so that the prediction results might be different from actual measurements
- The 4C cross-section results viewer can be displayed only at the mid span along the longest edge of the concrete and no diagonal or other perpendicular cross section results can be analyzed and presented

CHAPTER 5. SOFTWARE CALIBRATION

5.1 Overview

This chapter details the calibration of ConcreteWorks and 4C-Temp&Sress through the use of two case studies, the WB I-80 Missouri River Bridge and the US 34 Missouri River Bridge. The following sections include a description of the case studies, the calibration of ConcreteWorks and 4C-Temp&Stress for the WB I-80 Missouri River Bridge, the calibration of ConcreteWorks and 4C-Temp&Stress for the US 34 Missouri River Bridge, and a discussion of the results.

5.2 Westbound I-80 Missouri River Bridge

The WB I-80 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 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 thermal data that were provided were not sufficient to provide accurate results or a valid comparison.

5.2.1 ConcreteWorks

5.2.1.1 Inputs Overview

The construction of the WB I-80 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. In addition, the thermal data provided 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, photos of the construction, thermal control plans, and mix designs. In addition, to identify how the placements were constructed, interviews were conducted with personnel who worked on the project, 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.

5.2.1.2 Concrete Mix Proportion Inputs

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

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, as shown in Table 5.4.

Table 5.4. Ready Mixed Concrete Co. mix design for WB I-80 Missouri River Bridge

Iowa Mass Concrete (5,000 psi) with Slag		
Component	**Amount**	**Price**
Cement, IPF	420 lb	2.28
Slab GGBFS	207 lb	1.13
Water (263#)	0.42 lb/lb	4.21
Class V Sand-Gravel	1,586 lb	9.70
#557 Limestone	1,322 lb	7.93
Air Content	6.5%	1.75
Water Reducer	3 oz/100 lb	.00
High-Range Water Reducer	4-8 oz/100 lb	.00
		27.00

Cement type IPF is a blended cement that contains approximately 75 percent type I cement and 25 percent 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 10 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 used commonly in general concrete construction. ConcreteWorks assumes a slag grade of 120, which is a reasonable assumption for the project. In addition, the water-reducing agents are assumed to be type F naphthalene, a high-range water reducer. The concrete mix proportion inputs used for all of the mass concrete elements on the WB I-80 Missouri River Bridge as used in ConcreteWorks are listed in Table 5.5.

Table 5.5. Mixture proportion inputs for WB I-80 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*

*Naphthalene high-range water reducer (type F)

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 Louisville, Nebraska plant for type I/II cement. The values were calculated by ASTM test method C114 and represent the average values for cement produced between May 1 and May 31 of 2010. The values as input to ConcreteWorks are listed in Table 5.6.

Table 5.6. 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 fine aggregate type used most commonly in the area. Typical Iowa concrete has a coefficient of thermal expansion in the range of 4.1 to 7.3 (10^{-6}/°F) (Wang et al. 2008). The analysis utilized the value of a 4.1.

Table 5.7 shows the material properties used to model all of the elements from the WB I-80 Missouri River Bridge. The values that are denoted as ConcreteWorks default values are believed to represent the actual material properties accurately. In addition, the cement hydration properties were not altered from the ConcreteWorks default values.

Table 5.7. Material property inputs for the WB I-80 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]
Coarse Aggregate 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 5.8

Mechanical Property Inputs

The mechanical properties were assumed to be the same for all elements on the WB I-80 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.

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

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 Missouri River Bridge were 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 5.8. 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 5.8. Calculated Nurse-Saul constants for each placement for the WB I-80 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

5.2.1.3 Construction Parameter Inputs

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

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 US is English units. The location of all the placements is Omaha, Nebraska, 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 5.9. Placement start times that do not fall on the hour are rounded up to the nearest hour, as required by ConcreteWorks.

Table 5.9. Placement date and time for each element of the WB I-80 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

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 input as rectangular footings, columns and stems were input as rectangular columns, and caps were input as rectangular bent caps.

Dimension Inputs

The dimensional size of each element as provided by the final design plans of the bridge are listed in Table 5.10.

Table 5.10. Dimensions of elements for the WB I-80 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	-

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 Missouri River Bridge did not have elements that were soil formed or submerged in water. Footings may be analyzed as two- or three-dimensional (2D or 3D) to account for the length of the elements; our models utilized the 2D analysis, and assumed the footings were infinitely long.

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 placement, 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 documentation could not be found that indicated what kind of formwork was use for each placement. Photos found in construction records indicated that both wood and steel forms were used. For the purposes of this analysis; 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. The average ambient air temperature was provided by the National Weather Service historical data.

From interviews with the contractors and 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 discussions with contractors and Iowa DOT inspectors, it was determined that the footings were constructed on two different subbase conditions: clay and concrete. For footings that were constructed above the water table, no seal coat was needed and the footing was poured directly onto the clay-like material found in the riverbed. 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 to be infinitely long.

Table 5.11 shows the construction inputs for the WB I-80 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 5.11. Construction inputs for the WB I-80 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	-	-

5.2.1.4 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 input as provided by the National Weather Service historical data archive for Omaha, Nebraska. All other weather data were set as the default.

5.2.1.5 Sensor Location Corrections

For each placement constructed, there were sensors installed at three locations: center of the top surface, center of the side surface closest to the center, and 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 that the center sensor was installed at the exact center of the placement. These assumptions were developed from 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 to 12 inches, depending on which placement was being addressed. 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 correspond exactly 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 5.1.

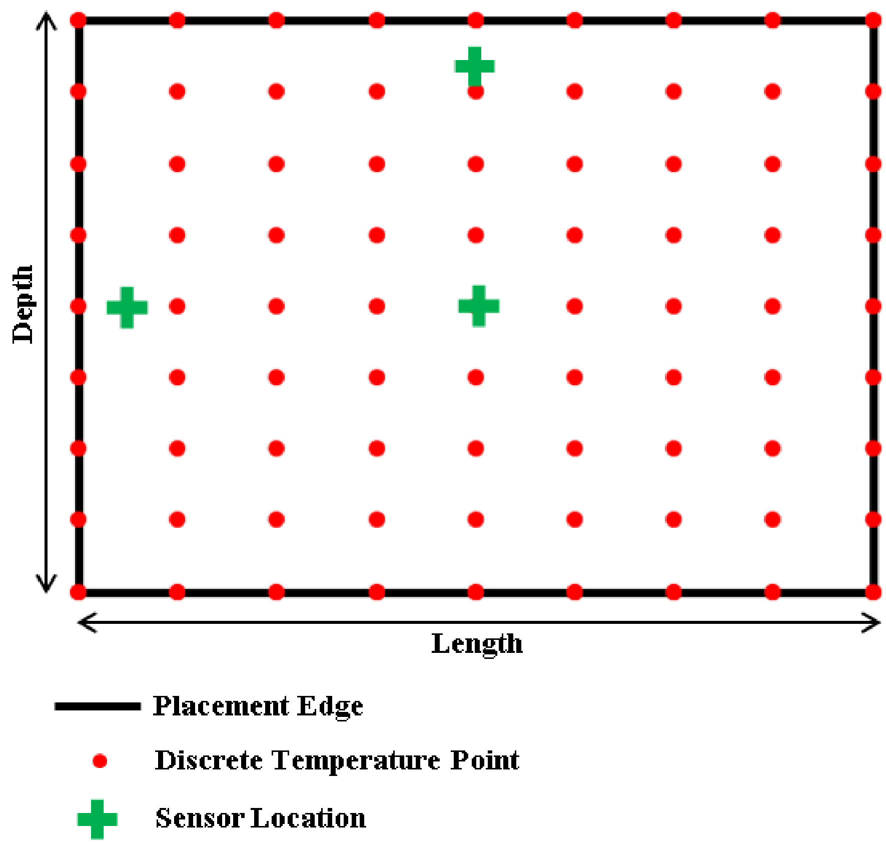


Figure 5.1. ConcreteWorks thermal analysis discrete temperature point layout

5.2.1.6 Results

The results of the WB I-80 Missouri River Bridge case study are listed in Table 5.12, Table 5.13, Table 5.14, and Table 5.15. 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. Each 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, each table also indicates negative errors, representing an underestimation by ConcreteWorks, and the positive errors, representing an overestimation by ConcreteWorks.

Table 5.12. 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 5.13. 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 5.14. 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 5.15. WB I-80 case study thermal results - caps

Pier	Maximum Temperature (°F)			Maximum Temperature Difference (°F)		
	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 C shows the results of each individual placement. The graphs in Appendix C show the comparison of the analysis results to the actual recorded data for the three discrete sensor locations with time.

5.2.1.7 Discussion

A statistical analysis of the maximum temperature error and maximum temperature difference error is provided in Table 5.16 and Table 5.17. 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 5.16. Maximum temperature error statistical analysis of the WB I-80 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 5.17. Maximum temperature difference error statistical analysis of the WB I-80 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 Missouri River Bridge case study show that ConcreteWorks is capable of predicting the general trends of the maximum temperature and maximum temperature difference of mass concrete placements for Midwest border bridges to a reasonable degree. The WB I-80 case study was also able to confirm the ability of ConcreteWorks to predict the temperature development of distinct points accurately in a mass concrete element, as shown by the individual placement thermal results. However, it appears that it would be prudent to make adjustments to the results because ConcreteWorks usually underestimates the maximum temperature and the maximum temperature difference, which is not conservative.

The error in the ConcreteWorks analysis might be attributed to differences between assumed and actual construction parameters. The lack of knowledge of the formwork type, insulation properties, and sensor locations is likely to be responsible largely for the analysis errors. Additional errors for the top surface sensors for the footings and columns arises from the ConcreteWorks assumption that the top surfaces are wet-cured. Although white pigmented curing compound or wet curing of top surfaces is required by specification, the researchers could not verify whether or not that this was done in all cases.

5.2.2 4C-Temp&Stress

5.2.2.1 Inputs

A total of 26 concrete members in the WB I-80 Bridge were selected for this case study. None of these units utilized cooling pipes. There are 7 footings, 8 columns, 7 stems, and 5 caps. This was done to keep the properties consistent to enhance comparison. These units had different dimension size, environmental temperature, and formwork removal time. However, the mix design and insulation material are presumed to be the same. The general inputs are described in Table 5.18.

Table 5.18. Concrete properties and material properties inputs used in I-80 Bridge case study

Description Concrete Volume	Input Value	Notes
Concrete Properties		
Slump	101mm	Obtained from data collection
W/c ratio	0.42	Obtained from data collection
Air content	6.50%	Obtained from data collection
Measured density	2320Kg/m^3	Obtained from data collection
Specific heat	0.84Kj/kg/°c	Obtained from data collection
Thermal conductivity	13	Obtained from data collection
Act. Energy factor 1	33500 J/mol	Default in 4c program
Act. Energy factor 2	1470 J/mol/°c	Default in 4c program
Material Properties		
Maturity vs. Heat development data	W/o cpipes: total:650 KJj/Kg, time :28h, curvature: 0.7 W/ cpipes: total:490 KJ/Kg, time :28h, curvature: 0.7	Based on temp. development data from collected data
Maturity vs. E-modulus	Total:40000 Mpa, time :15h, curvature: 0.8 cementitious material	Based on model prediction
Maturity vs. Poison ratio	Total: 0.17, time: 22.4 hr, curvature :1, fresh: 0.34	Default value in computer program
CW'S default value	0.00000736 /c	Obtained from data collection
Maturity vs. Compressive strength	Total:50Mpa, time :70h, curvature: 0.7	Based on estimated compressive strength data from collected data
Maturity vs. Tensile strength	Total:4.5Mpa, time :70h, curvature: 0.41	Based on model prediction
Creep		Based on model prediction

5.2.2.2 Results and Discussion

Verification for the 4C program was conducted through the analysis of 26 mass concrete elements (without cooling pipes) selected from the WB I-80 Bridge. The results are presented in Figures 5.2 and 5.3. Figure 5.2 provides a typical comparison between the measured and predicted temperature development for the Pier 1 footing concrete. When the age is less than 3 days (72 hours), the prediction values are generally consistent with those measured. However, the predicted values were lower than those measured for ages greater than 3 days. The discrepancy is 25% at 200 hours.

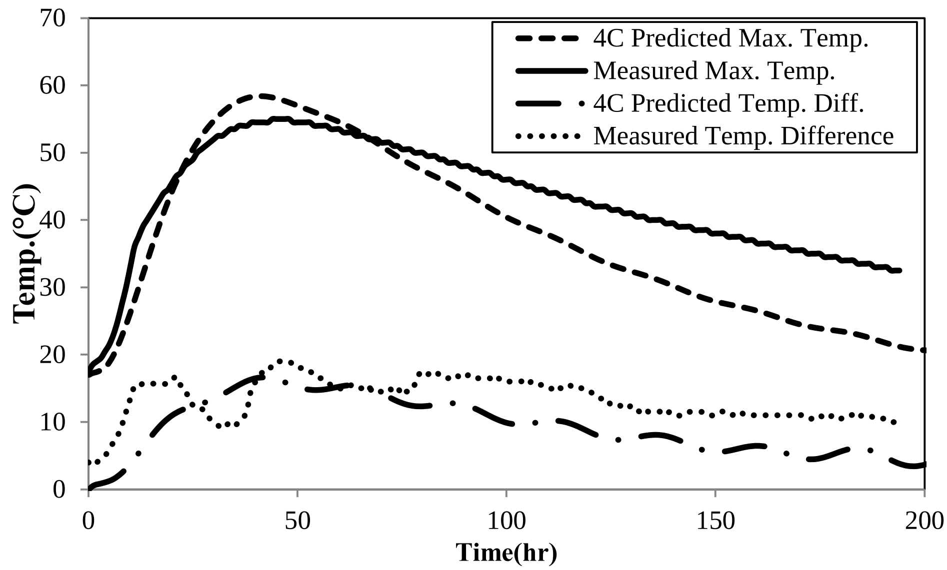


Figure 5.2. Comparison between the measured and 4C predicted temperatures (Pier 1 footing of WB I-80 Bridge)

Figure 5.3 shows the relationship between the measured and predicted maximum temperatures for the 26 concrete members analyzed. There are 780 temperature data points in Figure 5.3, 30 of which were selected from each structure member (footing, column, stem, or cap). As observed in Figure 5.3, most predictions are acceptable since their data points are close to the line of equality, suggesting that the 4C prediction differs little from the measured data. However, there are apparent outliers that are shown as light gray data points in Figure 5.3.

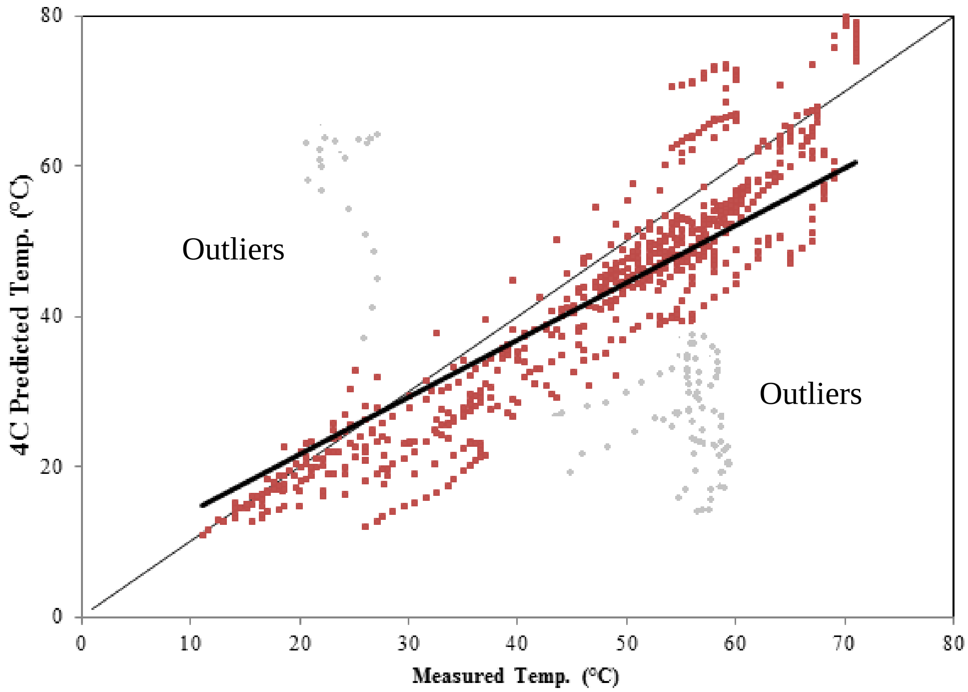


Figure 5.3. Line of equality plot for 780 data points of maximum temperature results from measured temperatures and 4C predicted temperatures

To identify these outliers, Figure 5.4 provides plots of the discrepancies between the 4C predicted and measured maximum temperatures. The data are categorized with the types of structural elements at selected time analysis points (0, 3, 6, …87 hours) to ensure there are enough data points to provide a reasonable analysis, but not so many as to create a heavy calculation load. The figure shows that stem elements have larger discrepancies between the measured and predicted temperatures compared to other elements, and that the discrepancies increase with time. The outliers (light gray data points) for the stem elements are shown in Figure 5.3. The outliers indicate that stems may not be modeled with complete accuracy by 4C, due to the simplification of the shape and size of the stem elements in the 4C model.

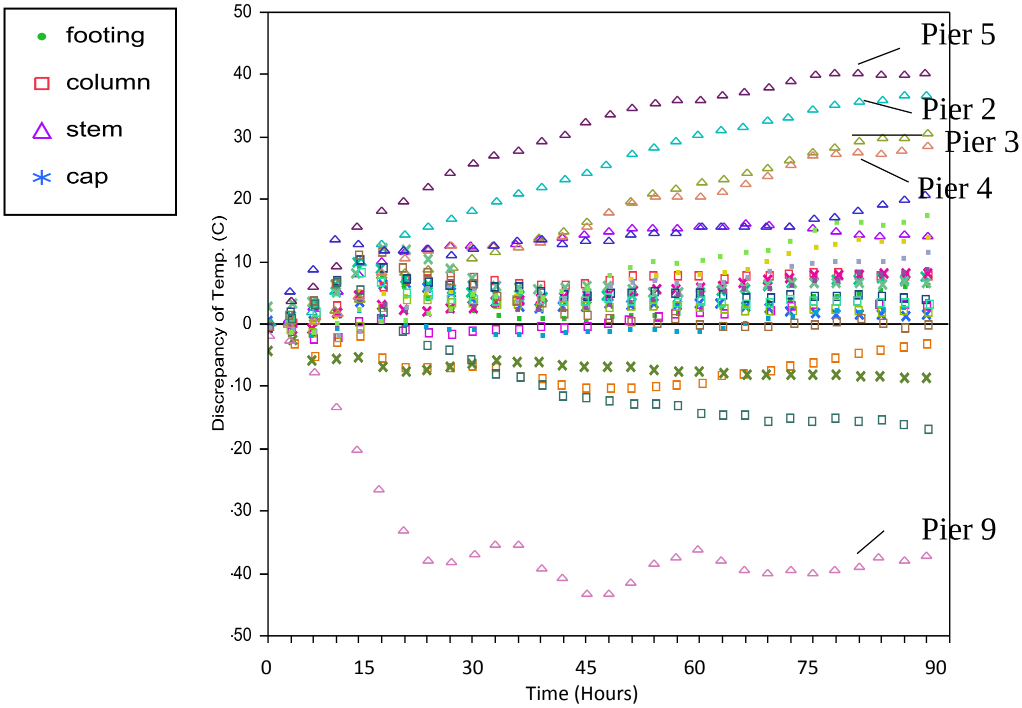


Figure 5.4. Temperature discrepancy plot for various structural elements

A statistical analysis of the temperature discrepancies was conducted to further evaluate the agreement between the 4C predictions and the field measurements for concrete at early age (0 to 72 hours). This period was selected because the maximum concrete temperature and the maximum temperature difference generally develop during this time. The outliers were eliminated before this analysis. The null hypothesis in this case is that the mean of the discrepancies is equal to 0, that is that the prediction will be accurate.

$$H_{0:}\ \overline{d_1} = \overline{d_2} = \overline{d_3} \cdots = \overline{d_n} = 0$$

$$H_{a:}\ \overline{d_1} \neq \overline{d_2} \neq \overline{d_3} \cdots \neq \overline{d_n} \neq 0$$

The results indicate that the p-values of the H_0 are larger than 0.05, which indicates that we do not reject the null hypothesis. That is, the temperature predictions from the 4C program are not significantly different from the measurements. During this period, the confidence level of the prediction is within 95%.

4C can provide iso-curve development during the analysis period. Iso-curve results for the case study Pier 3 footing with cooling pipes are shown below at 48 hours. Figure 5.5 shows the iso-curve of temperature development for the right third of the cross-section (or cut view) of the concrete member at 48 hours, when the concrete reached peak temperature during the analysis period.

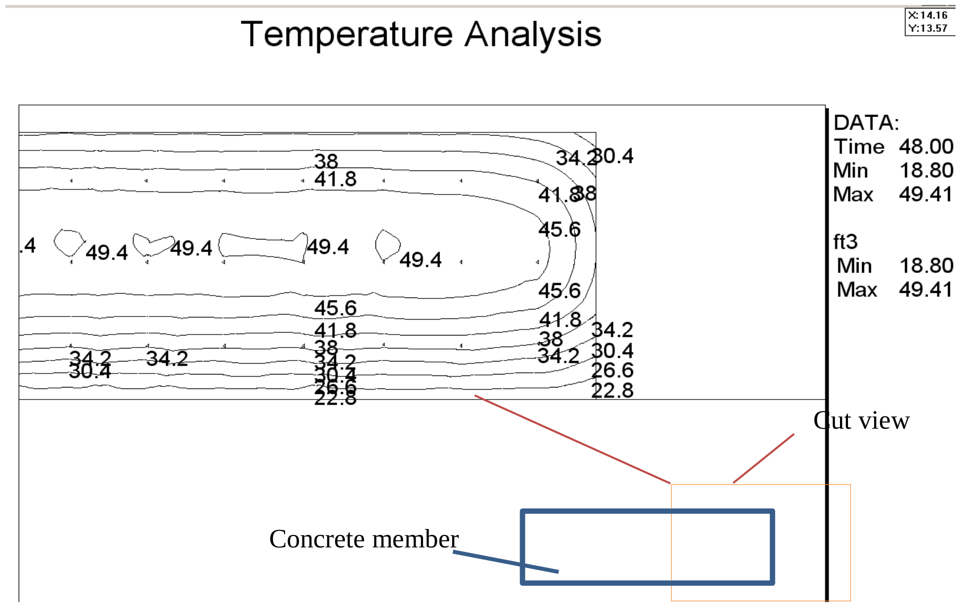


Figure 5.5. Sample temperature development iso-curve results for right-third cross section of Pier 3 footing with cooling pipe applied at 48 hours (not to scale)

The highest stress/strength ratio occurs during the first 24 hours at edges and corners of the concrete member. With passing time, the higher stress/strength ratio is likely to appear at the center of the structure. Examples of iso-curve results on stress/strength ratios are shown in Figure 5.6 and Figure 5.7. The iso-curve graphic results are generated by the 4C program. The user could zoom in and out on the cross section to get readable iso-curve results.

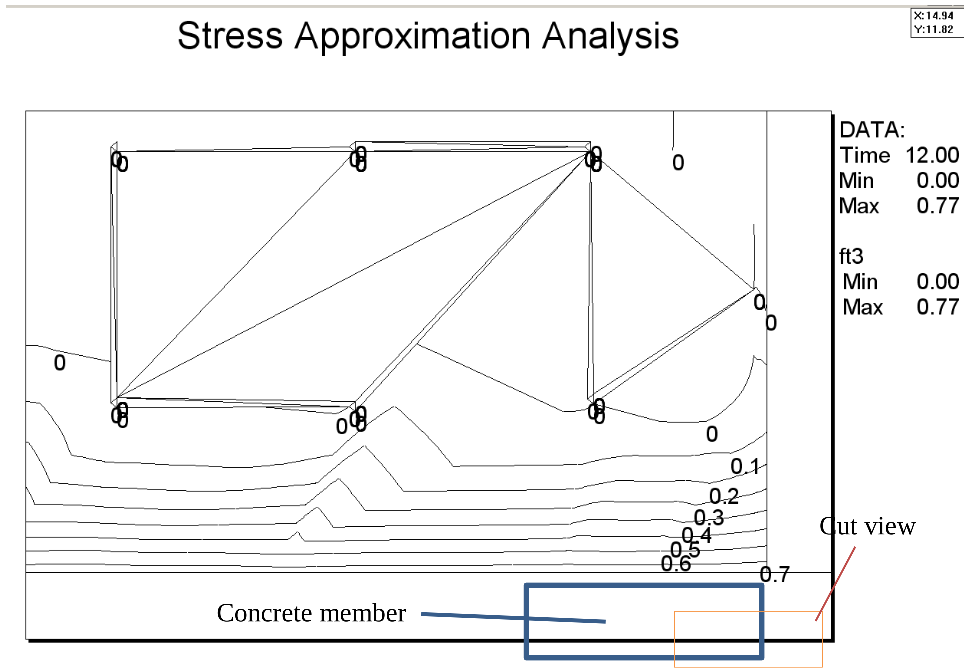


Figure 5.6. Sample tensile stress/strength iso-curve results for lower right corner of cross section of Pier 3 footing with cooling pipe applied at 12 hours (not to scale)

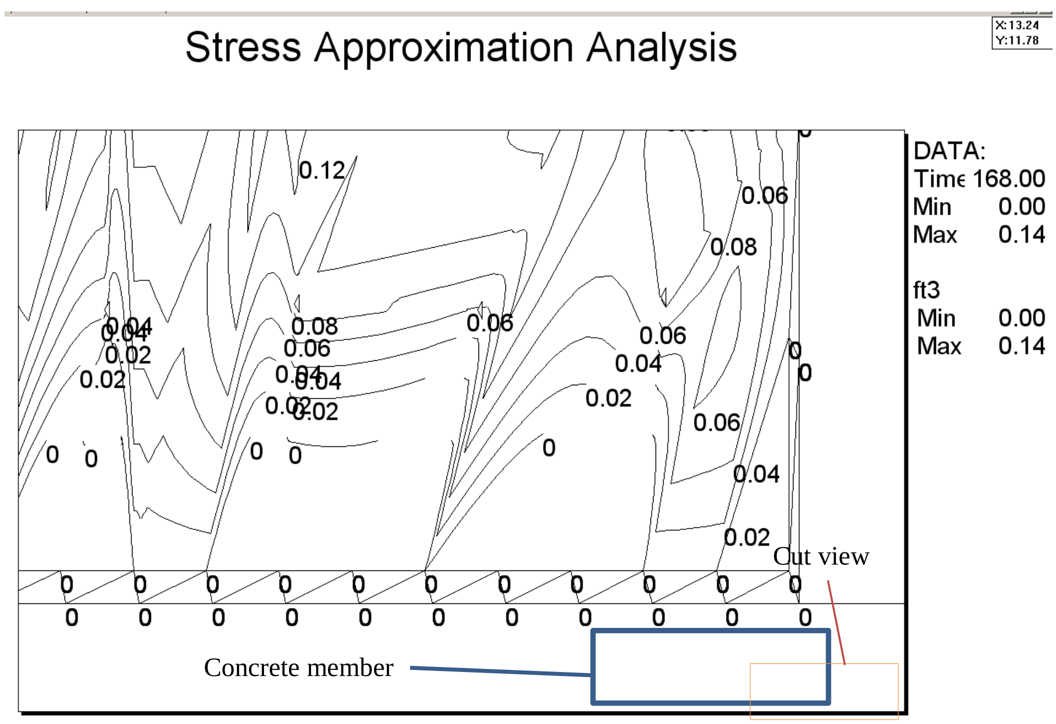


Figure 5.7. Sample tensile stress/strength iso-curve results for lower right corner of cross section of Pier 3 footing with cooling pipe applied at 168 hours (not to scale)

5.3 US 34 Missouri River Bridge

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 given 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 feet. 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.

5.3.1 ConcreteWorks

5.3.1.1 Inputs Overview

While the WB I-80 Missouri River Bridge was not constructed during the time that this research was conducted, the US 34 Missouri River Bridge was constructed partially during the time this research was conducted. The inputs for the case study were developed largely from firsthand 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.

5.3.1.2 Concrete Mix Proportion Inputs

The US 34 Missouri River Bridge utilized the same concrete mix proportion that was utilized on the WB I 80 Missouri River Bridge. Given that the mix proportions were 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 were used to model the US 34 Missouri River Bridge case study.

5.3.1.3 Construction Parameter Inputs

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

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, Nebraska, which is approximately 15 miles north of the actual bridge location. The placement date and start time was supplied by the contractor in the thermal data field report and is included in Table 5.19. Placement start times that do not fall on the hour are rounded up to the nearest hour, as required by ConcreteWorks.

Table 5.19. Placement date and time for each element of the US 34 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

Shape Inputs

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

Dimension Inputs

The dimensional size of each element was developed from the final bridge plans that were provided by the Iowa DOT. The dimensions of each placement required to run the ConcreteWorks analysis is provided in Table 5.20, with the column diameter defined as the width. ConcreteWorks assumes that columns and caps are infinitely long in comparison to the width and depth and therefore do not require a length input. None of the elements analyzed were submerged in water or soil formed. Similar to the WB I-80 case study, the footings are analyzed as two-dimensional elements.

Table 5.20. Dimensions of elements for the US 34 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	-

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.

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. Given the subbase material options that are available 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 analysis, 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 input into ConcreteWorks are listed in Table 5.21.

Table 5.21. Construction inputs for the US 34 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

* Wood was used to form the bottom of the cap

The fresh placement temperature of the concrete utilized in the analysis was measured by the contractor at the time when the concrete arrived at 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.

5.3.1.4 Environmental Conditions Inputs

Similar 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. In addition, all other environmental conditions were left as the default values. The ConcreteWorks default values are calculated from the start date and time of placement that were input and are based off the previous 30 years of historical weather data.

5.3.1.5 Sensor Location Corrections

While firsthand observations of the construction of the mass concrete placements from the US 34 bridge were conducted, the exact sensor locations could not be measured because of safety concerns. It was observed that, in general, the sensors were placed in locations similar to those 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 locations for the US 34 Bridge circular columns varied compared to those for the WB I 80 Bridge. Two sensor locations were utilized for the circular columns: one at the center of the column and one at the side surface. It was assumed that the center sensor was placed at the exact center of the placement, and the side sensor was located at an arbitrary location around the perimeter of the column with three inches of concrete cover.

For the footings and caps, it is assumed that one sensor was located at the exact geometric center of the placement, one in the center of the top surface with three inches of concrete cover, and one in the center of the side surface closest to the center with three inches of concrete cover.

5.3.1.6 Results

The results of the US 34 Missouri River Bridge case study are shown in Table 5.22, Table 5.23, and Table 5.24. 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 5.22. 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 5.23. 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 5.24. 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 D. The graphs in the appendix show the comparison of the analysis results to the actual recorded data for the three discrete sensor locations with time.

5.3.1.7 Discussion

A statistical analysis of the temperature prediction error was developed for the maximum temperature and maximum temperature difference as shown in Table 5.25 and Table 5.26, 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 5.25. Maximum temperature error statistical analysis of US 34 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 5.26. Maximum temperature difference error statistical analysis of US 34 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 confirmed that ConcreteWorks generally underestimated 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 Missouri River Bridge case study show that ConcreteWorks is capable of predicting the general trends of the maximum temperature and maximum temperature difference of placements to a reasonable degree. In addition, the results show that ConcreteWorks is capable of predicting the thermal development of placements at discrete locations with time to a reasonable degree. Adjustments should be considered to address recurring discrepancies between the predicted and actual temperatures. The results of the US 34 Missouri River Bridge case study confirm the results of the WB I-80 Missouri River Bridge case study.

5.3.2 4C-Temp&Stress

5.3.2.1 Inputs

In the US 34 Bridge case study, 6 footings, 4 columns, and 1cap were analyzed. Sample results of the Pier 4 footing using cooling pipes are presented below. The mix design and insulation material are the same as those for the I-80 Bridge. The dimensional size, environmental temperature, and construction procedures are shown in Table 5.27.

Table 5.27. 4C-Temp&Stress Inputs

	Pier 4 footing
Dimensional Size(ft×ft×ft)	51×20×6 ft
Ambient Temp(F)	max:2°C(-37.8°F)
Fresh Place. Temp.	15.6°C(60°F)
Insulation material	Same as I-80 Bridge
Form removal time	200 hours

Insulation removal	168 hours

5.3.2.2 Results

The analysis of the concrete units resulted in findings that were similar to those for the I-80 Bridge. The discrepancies between 4C predicted and measured values are acceptable. Sample results for the Pier 4 footing are shown in Figure 5.8.

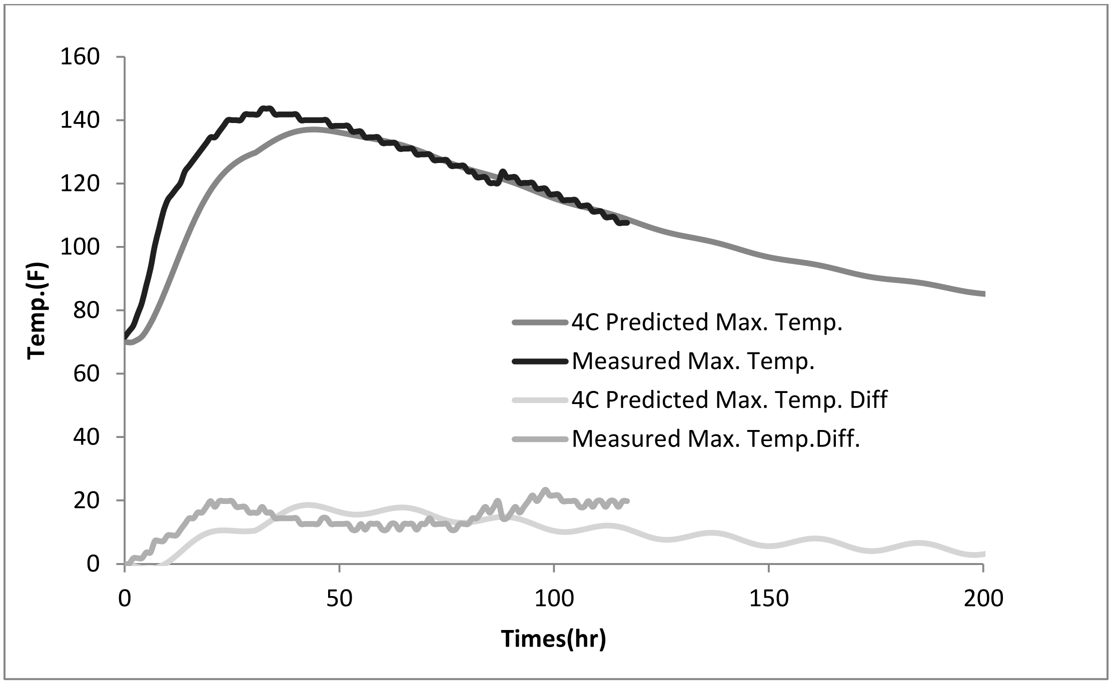


Figure 5.8. Pier 4 footing temperature results for US 34 Bridge

The stress/strength ratios (σt/ft) for the Pier 4 footing are shown in Figure 5.9. High cracking potential occurs when the σt/ft approaches unity. The final set of fresh concrete with fly ash occurs generally 10 to 13 hours after placing. During the first 12 hours, the concrete is still hardening and is relatively weak; the concrete is still restrained by substructure and formwork. Even the tensile stress/strength ratio is large within the first 10 to 13 hours; the concrete is likely to be too plastic or elastic to propagate cracks (Ge and Wang 2003).

Other actions that may delay the setting time include decreasing concrete temperature, using slag, excessive plasticizer, and water-to-cement ratio. The peak stress/strength ratio occurs at approximately 20 hours after casting, and this might occur after the concrete final set.

After final set, the concrete temperature keeps rising and the concrete will experience peak temperature at around 48 hours. A large temperature difference may occur at this time, which results in a large stress/strength ratio. Thus, the stress/strength ratio during 24 to 48 hours should be considered as important criteria on evaluation of mass concrete thermal cracking when a structure is placed on the soil substructure.

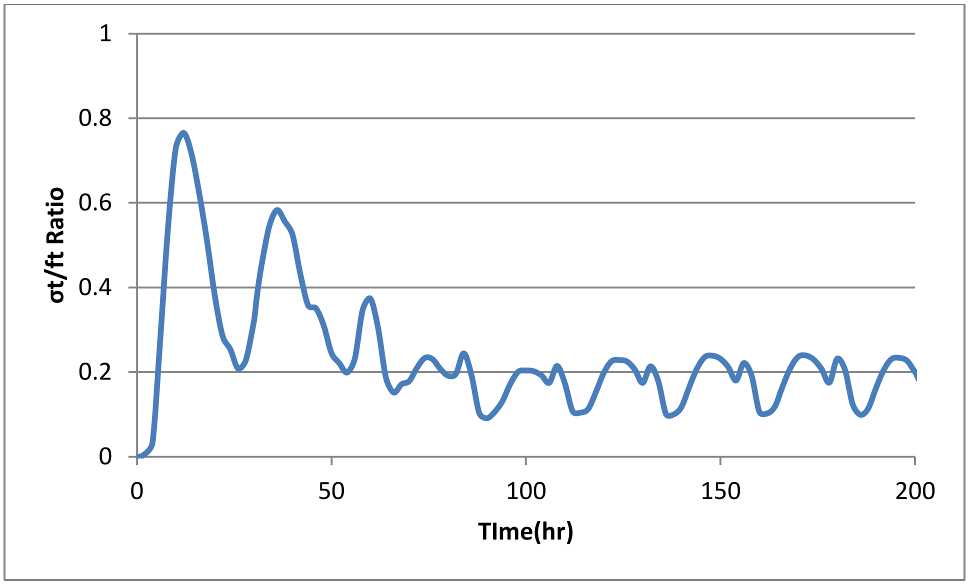


Figure 5.9. Pier 4 footing σt/ft ratio results for US 34 Bridge

CHAPTER 6. SENSITIVITY STUDY

6.1 Overview

This chapter provides a sensitivity study of parameters having the largest effect on the thermal development of mass concrete. Two separate case studies are documented utilizing ConcreteWorks and 4C-Temp&Stress. The following sections provide the ConcreteWorks sensitivity study results, the 4C-Temp&Stress sensitivity study results, and a discussion of the results.

6.2 ConcreteWorks Sensitivity Study

6.2.1 Overview

The early age development of mass concrete is affected by numerous mix proportion, construction, and environmental factors. To design and construct a mass concrete element properly, 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 that are believed to have the largest effect on the development of mass concrete placements typical for Midwest border bridges. The parameters were selected through a literature review of common practices used in the US 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 6.1.

Table 6.1. 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.

6.2.2 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 border bridge. To assure realistic inputs, an element was selected from the WB I-80 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 6.2 with additional values supplied in Table 6.3 and Table 6.4. 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.

In addition, 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 6.2. 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, Nebraska
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	Naphthalene 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 6.3
b – denotes values listed in Table 6.4

Table 6.3. Ash Grove type I/II Bogue calculated values

Bogue Value	Percent (%)
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 6.4. 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

6.2.3 Results

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

6.2.3.1 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 6.5.

Table 6.5. 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 6.6. The results show that an increase in the dimension of the placement typically 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. In addition, the length of the placement had no effect on the temperature development of the placement.

Table 6.6. Dimensional size sensitivity study results

Parameter Changed	Dimensional Size Depth (ft)	Width (ft)	Length (ft)	Maximum Temperature (°F)	Maximum Temperature Difference (°F)
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

Given 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 effect from increasing the 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.

6.2.3.2 Fresh Placement Temperature

The fresh placement temperature sensitivity study analyzed fresh placement temperatures that are seen commonly in mass concrete construction. A temperature of 40°F was selected as a minimum, which is the minimum temperature typically allowed by state agencies for general construction. A maximum temperature of 90°F was selected to represent the maximum fresh placement temperature, which is the maximum typically seen in general concrete construction. The sensitivity study examined the effect of fresh placement temperature in ten-degree increments from 40 to 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 shown in Table 6.7.

Table 6.7. 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

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 to 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.

Fresh placement temperature directly affects the thermal development of a placement by providing initial heat to the placement. In addition, 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. In addition, 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.

6.2.3.3 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 6.8, providing the maximum temperature and the maximum temperature difference, as provided by ConcreteWorks analysis.

Table 6.8. 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 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.

The 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 and, 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.

6.2.3.4 Forming Method

The forming method sensitivity study considered 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 were 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 6.9.

Table 6.9. 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 results show that the formwork material and color had no effect on the maximum temperature of a placement. In addition, 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.

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.

6.2.3.5 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 for concrete elements subject to flexure (i.e., some surfaces could be in tension) 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, or 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 6.10.

Table 6.10. 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 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 and 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 to 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.

The results show that the formwork removal time has no effect on the maximum temperature of the placement except for formwork removal times of 48 hours. Figure 6.1 shows that the maximum temperature occurs around 85 hours after the element was placed for these conditions.

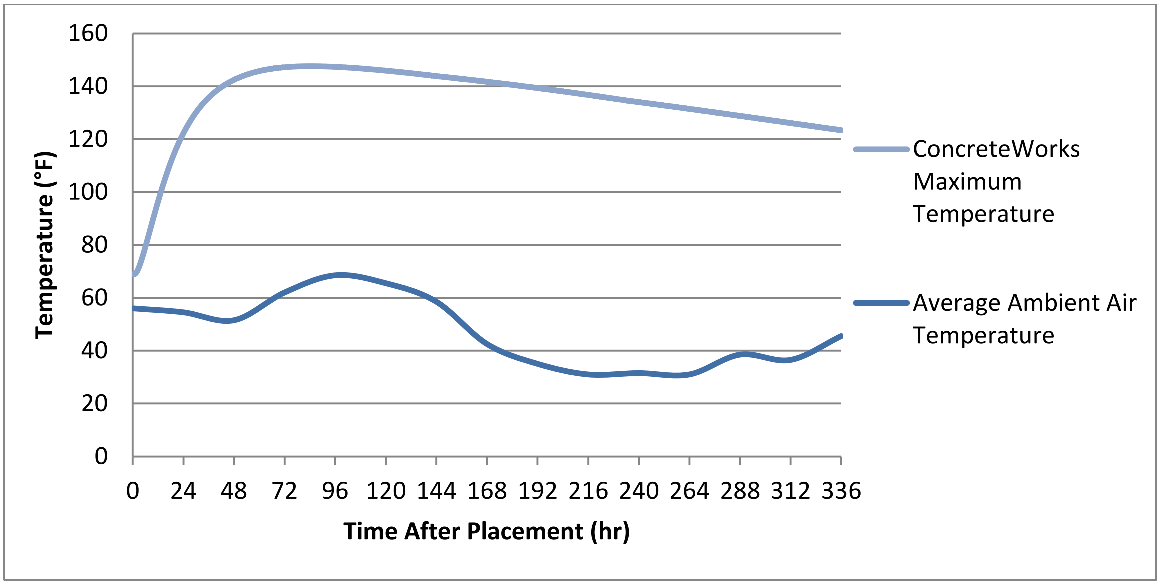


Figure 6.1. ConcreteWorks maximum temperature development and average ambient air temperature with time

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 6.1 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 cool gradually, shown by the decrease in the maximum temperature.

6.2.3.6 Subbase Material

The subbase sensitivity study considered the effect of various 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 maximum temperature and maximum temperature difference as calculated by ConcreteWorks are listed in Table 6.11.

Table 6.11. 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 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 to 151°F, and the maximum temperature difference ranged from 72 to 105°F.

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 6.12. ConcreteWorks does not use a standard set of thermal properties for concrete subbase, assuming the same thermal properties as the concrete being analyzed.

Table 6.12. 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 6.2 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.

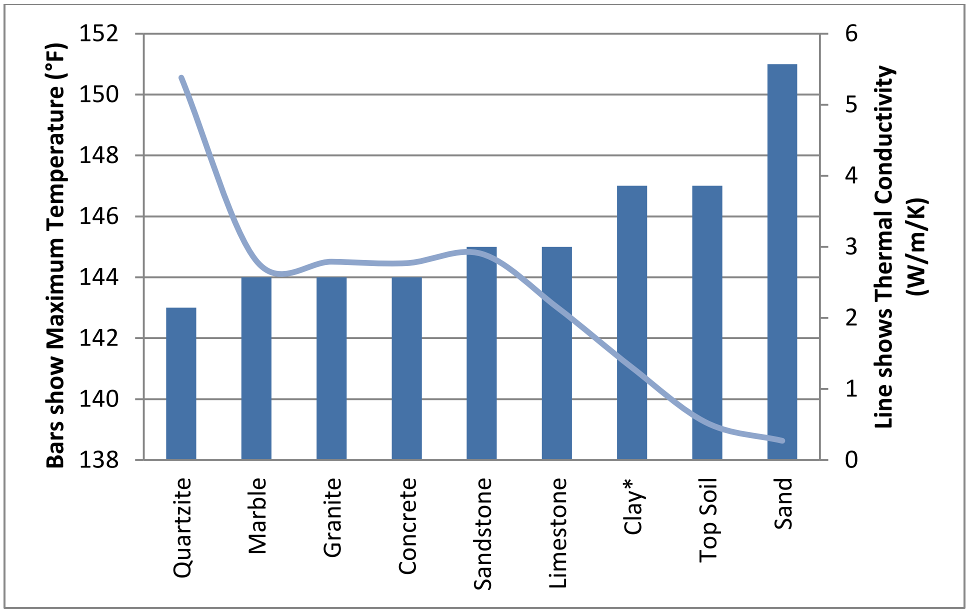


Figure 6.2. Placement temperature versus subbase material thermal conductivity

6.2.3.7 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 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 6.3 to identify the general thermal gradient pattern of the placement.

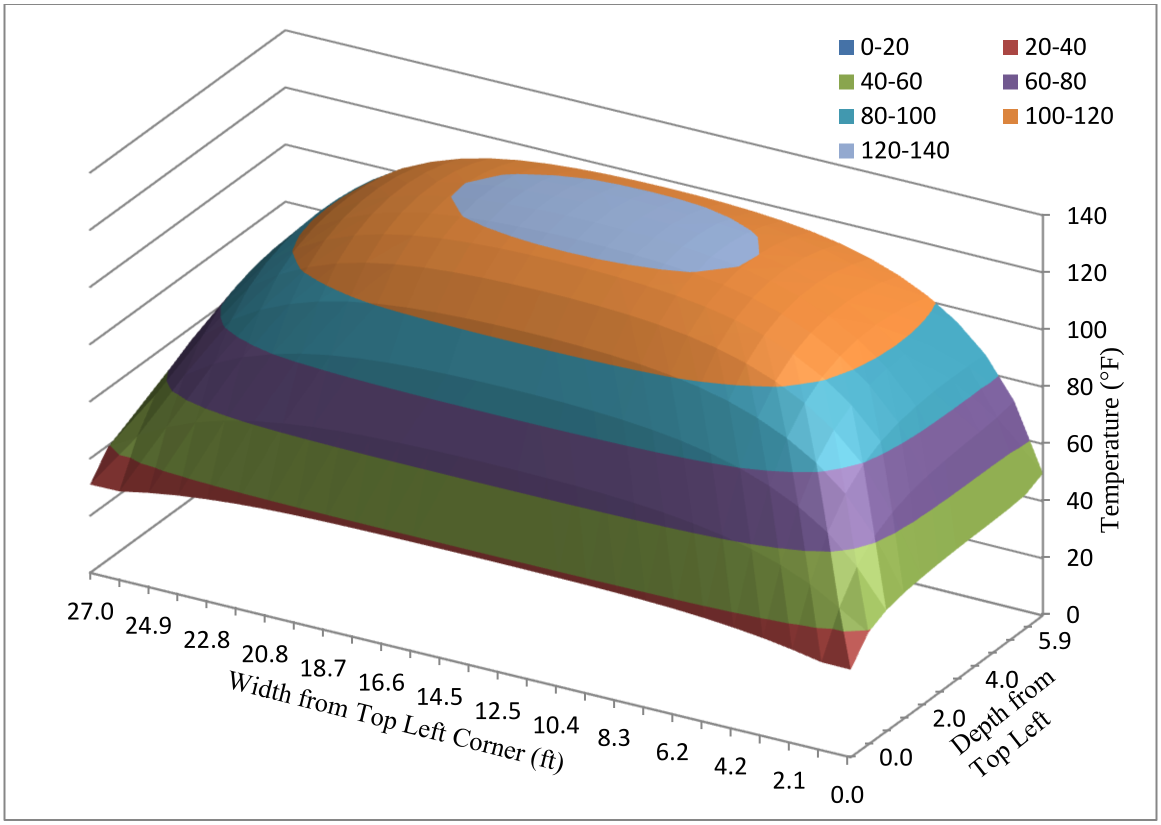


Figure 6.3. Pier 3 footing contour plot at time of maximum temperature difference

To examine the effects of incorrect sensor location, the cross-sectional thermal data were analyzed in the width direction at the center line of the depth for the side surface sensor, center line of the width in the depth direction for the top surface sensor, and 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 shown with the solid bold (red) lines in Figure 6.4.

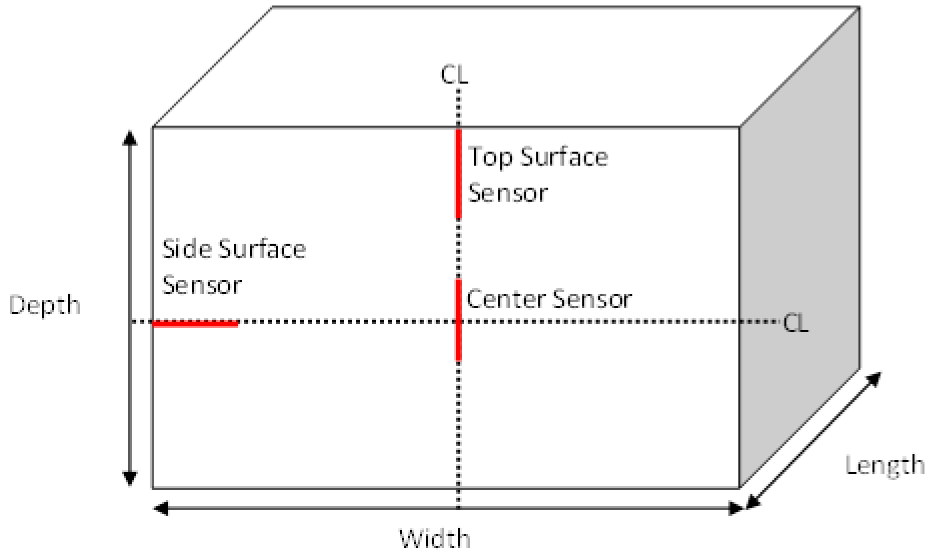


Figure 6.4. Top, side, and center sensor error locations

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. In addition, 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 changes from the baseline conditions, the thermal data from the surface to 15 inches below the surface were 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 respective 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 6.5, 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. 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.

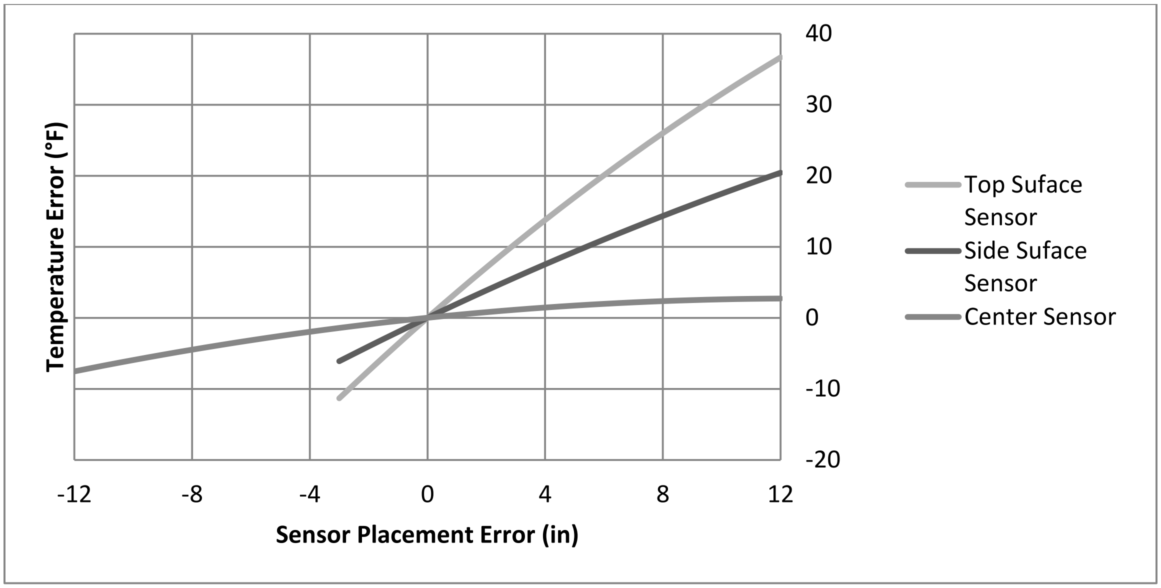


Figure 6.5. Temperature errors for sensor placement errors

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 toward 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 compared 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. In addition, the top surface sensor temperature error is the most affected by a given error in sensor placement. Because 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 6.13 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 6.13. 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 locate the sensors in the placement accurately. It is important to note that the maximum temperature of the placement is not located generally at the exact geometric center of the placement because of 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 attributed largely 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 concrete 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 reduced greatly compared to that of the raw ConcreteWorks maximum temperature difference of 92°F.

6.2.3.8 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 30 represents a cool ambient air temperature, where freezing of the concrete is of little concern. July 30 is one of the warmest times of the year typically 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 input. 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 6.14 as inputted into ConcreteWorks.

Table 6.14. 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 shown in Table 6.15.

Table 6.15. 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 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 for July 30, 2008 generated a higher maximum temperature and a reduced maximum temperature difference compared to that of the cooler ambient air temperature for October 30, 2008.

The ambient temperature and maximum temperature development with time, as calculated by ConcreteWorks, is shown in Figure 6.6.

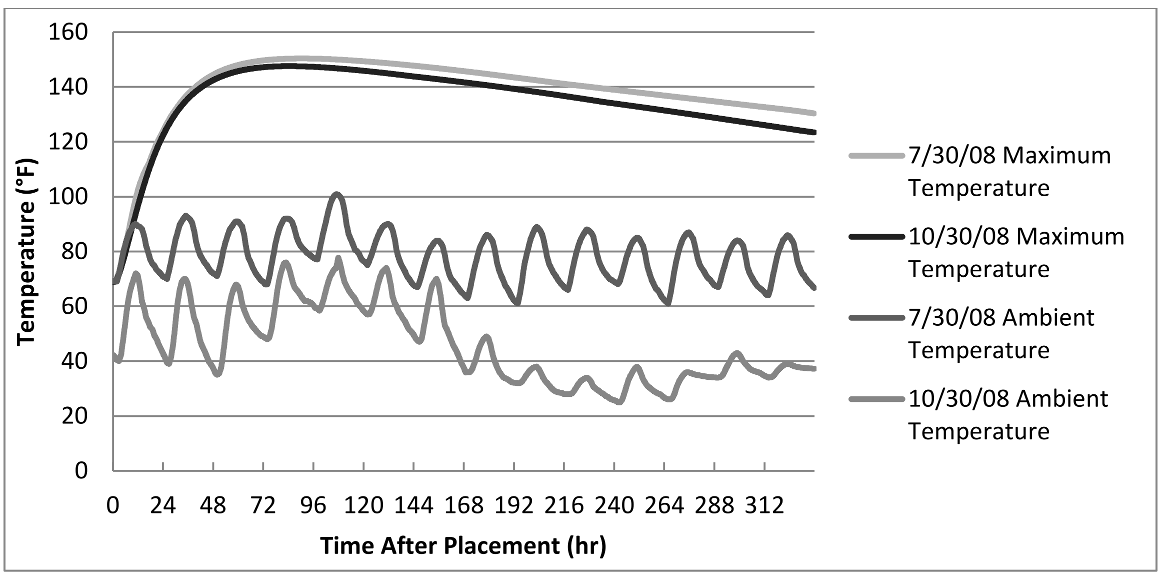


Figure 6.6. ConcreteWorks ambient air temperature and maximum temperature with time after placement

The figure shows how ConcreteWorks approximates the ambient air temperature surrounding the placement from the daily maximum and minimum temperautres. In addition, the graph shows that the maximum temperature is reduced for the lower ambeint air temperature conditions for October 30, 2008.

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 October 30, 2008 had a slightly reduced maximum temperature, the ambeient air temperature is greatly reduced compared to that of the placement poured on July 30, 2008. The greatly reduced ambient air temperature causes an increase in the maximum temperature difference compared to the placement poured on July 30, 2008.

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.

6.2.3.9 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 to 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 6.16.

Table 6.16. 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 6.17. 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 fact that additional material is undergoing hydration. The additional heat generated in the placement results in an increased maximum temperature and, subsequently, an increased maximum temperature difference.

Table 6.17. 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

6.2.3.10 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 percent increments from 0 to 50 percent of the total cementitious content. The upper limit of 50 percent was set to represent typical mass concrete specifications. The total cementitious content of 627 lb/cy was selected to following the previous baselines. Table 6.18 and Table 6.19 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 simplify the study.

Table 6.18. 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 6.19. 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 6.20 and Table 6.21 show the results of the sensitivity study for 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. In addition, the substitution of class C reduced the maximum temperature of the placement, and the maximum temperature difference slightly.

Table 6.20. 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 6.21. 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 reduced the maximum temperature in the placement substantially as a result of 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 lowers the maximum temperature in the placement slightly, which correlates to the minimal reduction in the maximum temperature difference.

6.2.3.11 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 to 50 percent in 10 percent increments. Table 6.22 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 6.22. 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 6.23 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 increases the maximum temperature difference of the placement slightly.

Table 6.23. 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 6.7 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.

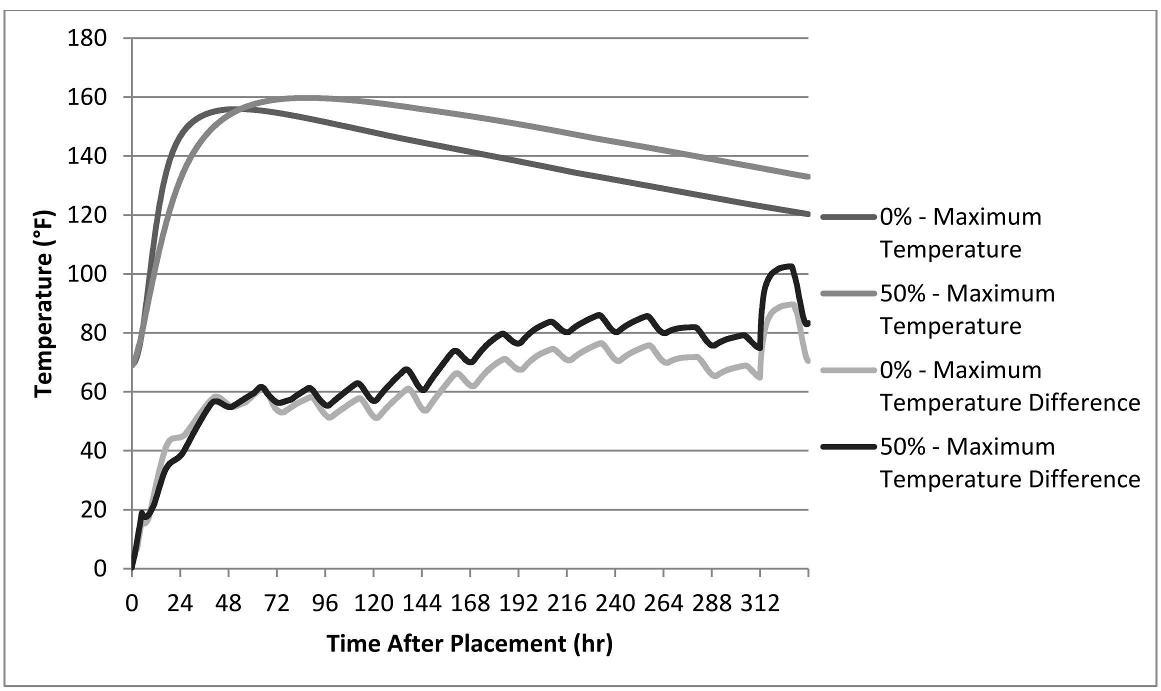


Figure 6.7. Maximum temperature and maximum temperature difference sensitivity study results for 0% and 50% GGBFS substitution

6.2.3.12 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 6.24, Table 6.25, and Table 6.26, respectively. The tables are organized with each column representing a different total cement substitution percentage. In addition, each row identifies a class F fly ash to GGBFS percentage, with the percentage of the cement substitution being fly ash in the left-most column and GGBFS in the right-most column.

Table 6.24. 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 6.25. 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 6.26. 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 in Table 6.27 and Table 6.28. 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. Similar to the class F fly ash and GGBFS substitution sensitivity study, class F fly ash reduces the maximum temperature and maximum temperature difference, while GGBFS substitution increases both.

Table 6.27. 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 6.28. 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 graphic representation of the maximum temperature results is shown in Figure 6.8. In accordance, the maximum temperature difference follows the same trend as that shown for the maximum temperature.

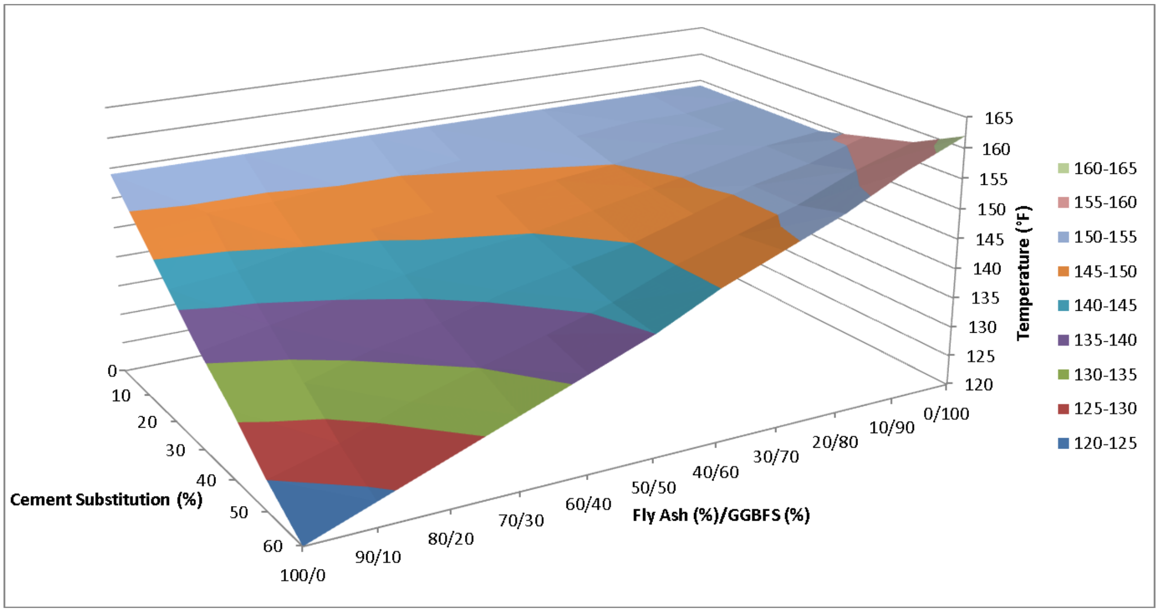


Figure 6.8. Combined class F fly ash and GGBFS substitution maximum temperature results

6.2.4 Discussion

The results of the sensitivity study show that all 12 of the parameters examined effect the thermal development of typical Midwest border 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.

In addition, 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.

6.3 4C-Temp&Stress Sensitivity Study Results

The sensitivity study was conducted using the software program 4C-Temp&Stress and varying construction, environmental, mix proportion, and thermal properties parameters as follows:

Construction Parameters

- Temperature sensor location
- Dimensional size
- Insulation method
- Form removal time
- Substructure material
- Cooling pipes

Environmental Parameters

- Fresh placement temperature
- Ambient temperature

Mix Proportion, Thermal Properties, and Others

- Cement content, fly ash, GGFBS
- Thermal conductivity
- Coefficient of thermal expansion
- Creep
- Coarse degree of meshes

The range for each set of parameters were selected in a manner that was similar to the one used for ConcreteWorks. The summary of sensitivity analysis results are provided in Table 6.29.

The major findings can be summarized as follows:

- As sensor depth beneath the surface increases, the temperature increases.
- It is recommended that surface sensors be installed 3 inches below the concrete surface, where the sensor could be easily attached to the steel rebar.
- Ethafoam, plast foam (10mm), foil with 5 mm air space, and plastic foil were calculated to provide the lowest cracking risk in this analysis and are recommended to use as top insulation.
- Formbord (25 mm), plywood, plywood formwork, and timber formwork, which were calculated to provide the lowest cracking risk in this analysis are recommended for use as side formwork.
- Maximum temperature and maximum temperature difference were found to increase with the following:
    - Increase of the least dimensional size
    - Increase of fresh placement temperature during fall season weather (October)
    - Decrease of form removal time
    - Increase of cement content
- The use of supplementary cementitious materials, high thermal conductivity aggregate, and low coefficient of expansion aggregate were effective in reducing the cracking potential.
- Cooling pipes were effective in reducing the maximum temperature and the thermal cracking potential. The layout and numbers of cooling pipes were important in terms of reducing cracking potential and construction cost.
- 4C inputs could be adjusted to reflect changes in temperature sensor locations. Temperature sensors near the surface are usually buried about 3 inches below the surface. The temperature values for subsurface sensors are higher than what they would be for a sensor located on the surface. This means that calculations directly using the temperature sensor data would underestimate maximum temperature differences.
- To provide the best predictions using 4C, input methods that involve measured concrete properties should be selected.
- It is recommended that users input changes of measured heat development and compressive strength in 4C when mix design of concrete is changed.

Table 6.29. Parameters, ranges, and results considered in sensitivity study

	Description	Detailed Items and Range	Tmax	ΔTmax	Max. σt/ft Ratio		Baseline
					Soil	Concrete	Footing #
Material	cement content	427, 527, 627, 727 pcy	175-284°F	60-90°F	1.1-3.3		Pier 3
	F fly ash replacement	0%, 10%, 20%, 30%, 40% ,50%	175-120°F	60-40°F	1.1-0.6		Pier 3
	C fly ash replacement	0%, 10%, 20%, 30%, 40% ,50%	175-150°F	60-45°F	1.1-0.75		Pier 3
	Slag replacement	0%, 10%, 20%, 30%, 40% ,50%	175-160°F	60-100°F	1.1-1.1		Pier 3
Structure Size	Depth	1.4, 1.5, 1.8 ,2.1, 2.7 m	122-165°F	42-80°F	0.52-1.30	0.5-2.2	Pier 1
	Width	3, 4.5, 6, 7.5, 9, 10.5 m	122°F	42-90°F	0.52-1.35	0.6	Pier 1
	Length	9, 12, 15, 18, 21 m	122°F	42°F	0.52	0.6	Pier 1
Concrete properties	Thermal conductivity	5.39, 8, 10,13,18 KJ/kg/°C	130-110°F	40-18°F	0.75-0.5	0.55-0.48	Pier 1
	Thermal expansion coefficient	7.36, 9, 11, 13 $*10^{-6}$/°C,	122°F	42°F	0.52-1.1	0.5-1.3	Pier 1
	Creep	w/ or w/o creep influence	156°F	45°F	0.75/3.5		Pier 3
Construction	Form removal time	48,72, 96, 120, 144, 168, 192, 216, 240, 264, 288, 312 hours	156°F	72-42°F	1.5-0.75		Pier 3
	Formwork materials	steel, plywood, plywood formwork, timber, formbord (0.75, 1.0 in)	153-158°F	40-50°F	0.75-1.55		Pier 3
	Curing method	etha foam, foil with 5mm air space, plastic foil, foam plastic,(0.4, 0.8, 1.2in), winter blanket (2, 4in.)	153-158°F	35-80°F	0.75-1.1		Pier 3
	Cooling pipes	w/ or w/o cpipes	156/130°F	40/20°F	0.81/0.75		Pier 3
Environmental	Fresh placement temperature & placement date & time	summer: 40, 50, 60, 70, 80, 90°C	122-185°F	20-35°F	1.4-1.2		Pier 3
		winter: 40,50,60,70,80,90 °C	118-176°F	32-53°F	0.75-1.5		Pier 3
Others	Sensor locations	from surface to center of concrete with 3in increment	80-122°F				Pier 1
	Mesh sizes of finite element analysis	2% (fine) or10% (coarse)	156/158°F	45-47°F	0.82-0.75		Pier 3
	Substructure	soil or concrete	122°F	42°F	0.75	0.57	Pier 1

6.4 Discussion on Sensitivity Studies

Although both 4C and ConcreteWorks provide reasonable predictions of concrete thermal behavior, there are some differences in the predictions. This was especially true for the maximum temperature difference. ConcreteWorks predicts a higher temperature difference consistently, because it compares the temperature at the surface with the temperature in the middle of the placement. However, the temperature is usually not measured at the surface but rather at the location of a temperature sensor, which was usually buried three or more inches in the concrete. The surface temperature is influenced directly by the ambient temperature conditions and thus a larger temperature difference is predicted.

Several forming and insulation alternatives can be selected in ConcreteWorks and the analysis in 4C using the same selections provided similar predictions. However, 4C provided more options for forming and insulation materials. Furthermore, 4C provided results that were similar to those of ConcreteWorks regarding the effect of changes in placement date. Generally, smaller maximum temperature difference and less cracking potential were predicted for colder weather placements in comparison to warmer weather placements. Issues with warmer weather placement were mitigated when the fresh placement temperature was held to less than 70°F.

ConcreteWorks was developed to allow considering the influences of changes in mix design. The results appear to be reasonable. Even though the 4C output confirmed the general trends provided by ConcreteWorks, the maximum concrete temperatures were noticeably different. The research team was not able to find a satisfactory method to input mix design parameters into 4C to conduct a sensitivity analysis on mix design.

CHAPTER 7. TEMPERATURE DIFFERENCE CASE STUDIES

An objective of the present case study is to find the relationship between the maximum temperature differences and the cracking potential of mass concrete. Field measurements are often monitored by embedded temperature sensors and are often used to identify the maximum temperature and maximum temperature differences of mass concrete elements.

7.1 I-80 Bridge

A total of 13 concrete structural elements, including the footings for Piers 1 through 6, the columns for Piers 1 through 5, and the column for Pier 10 of the I-80 Bridge were analyzed using the 4C program. Some of these elements had a soil subbase and some had a concrete subbase. The maximum temperature difference and the $б_t/f_t$ ratio of these concrete elements were obtained from the analyses. Four different time intervals were considered: 0-24, 24-48, 48-72, and after 72 hours. A computer software application that performs statistical analyses, JMP 9 (JMP 9, 2012), was used to analyze the data and to identify a relationship between the predicted maximum temperature difference and the $б_t/f_t$ ratio. The results are presented in Figure 7.1. Illustrated is a linear relationship between the predicted maximum temperature difference and $\ln(б_t/f_t)$ for the time interval investigated.

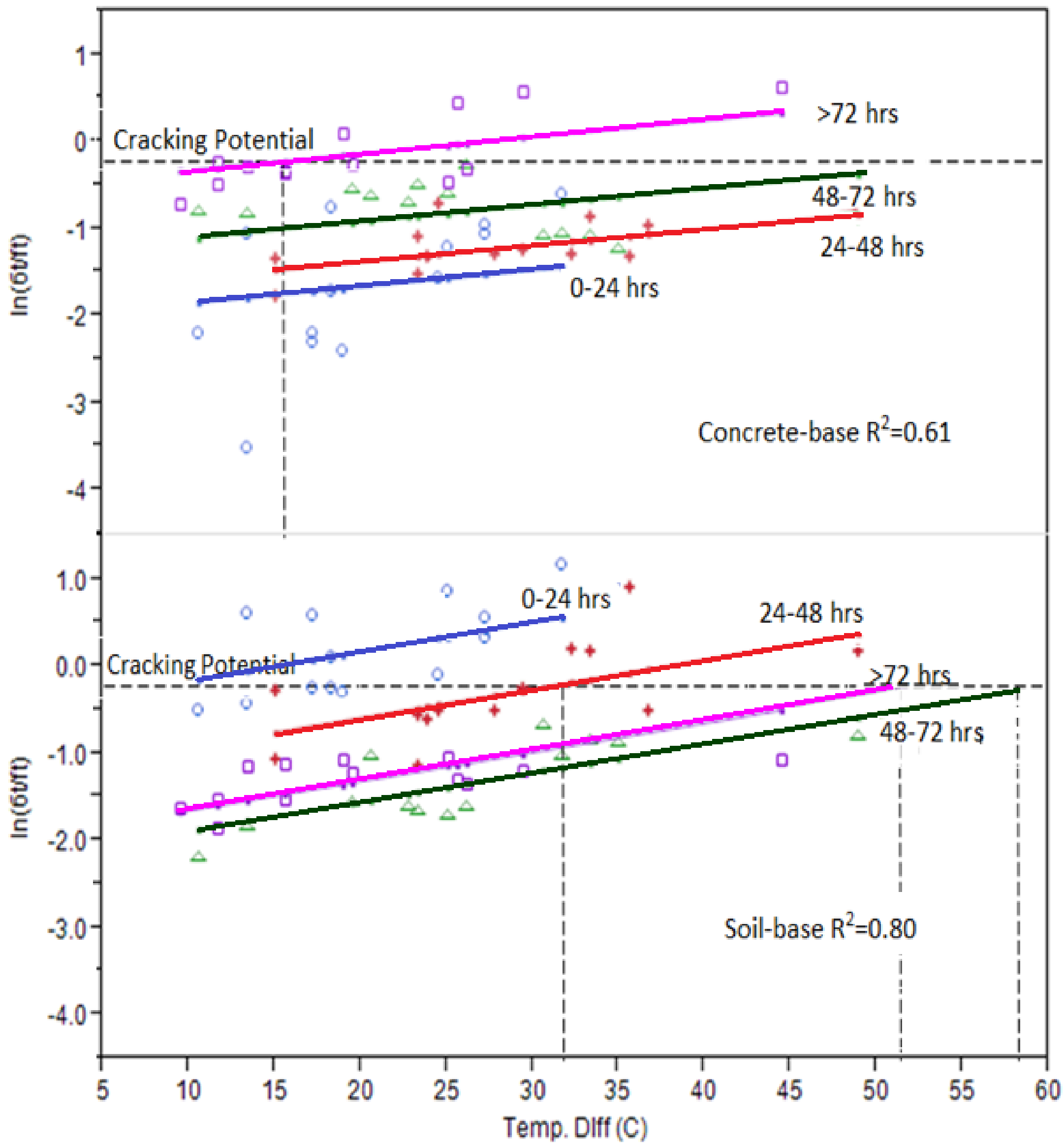


Figure 7.1. Relationship between $б_t/f_t$ ratio and the maximum temperature difference

When the subbase is soil, the $б_t/f_t$ ratios of concrete in footings or columns are very high during first 24 hours (lower part of Figure 7.1). This may not be problematic because the concrete is relatively soft and can deform without significant cracking before it fully sets and hardens. The simulation or stress/strength prediction is often less accurate at such an early age because the concrete properties are difficult to measure or assess accurately at an early age. Special attention should be given to the high $б_t/f_t$ ratios during the age of 24-48 hours. Figure 7.1 shows that during the time from 24 to 48 hours, the $б_t/f_t$ ratio reaches the critical value of 0.75 when the concrete maximum temperature difference increases to approximately 32°C. After 48 hours, the concrete maximum temperature and temperature difference are reduced, obviously due to the reduced heat of cement hydration. Therefore, the $б_t/f_t$ ratio generally remains less than 0.75 in the present analysis; this stress ratio is considered to represent a low cracking potential.

When a concrete member is placed on a concrete subbase, the upper part of Figure 7.1 shows that the concrete elements generally have a low cracking potential ($б_t/f_t$ <0.75) for the first72 hours. However, after 72 hours, the $б_t/f_t$ ratio reaches the critical value of 0.75 when the concrete maximum temperature difference increases to approximately 16 °C. Furthermore, the $б_t/f_t$ ratio further increases as the maximum temperature difference increases in the concrete. Coincidentally, formwork removal for the case study mass concrete construction projects generally occurred after 72 hours or 3 days of placement. The highest temperature difference often occurs shortly after the formwork removal. Therefore, the $б_t/f_t$ ratios after formwork removal are important for mass concrete placed on a concrete subbase, while the $б_t/f_t$ ratios before formwork removal are important for mass concrete placed on a soil subbase.

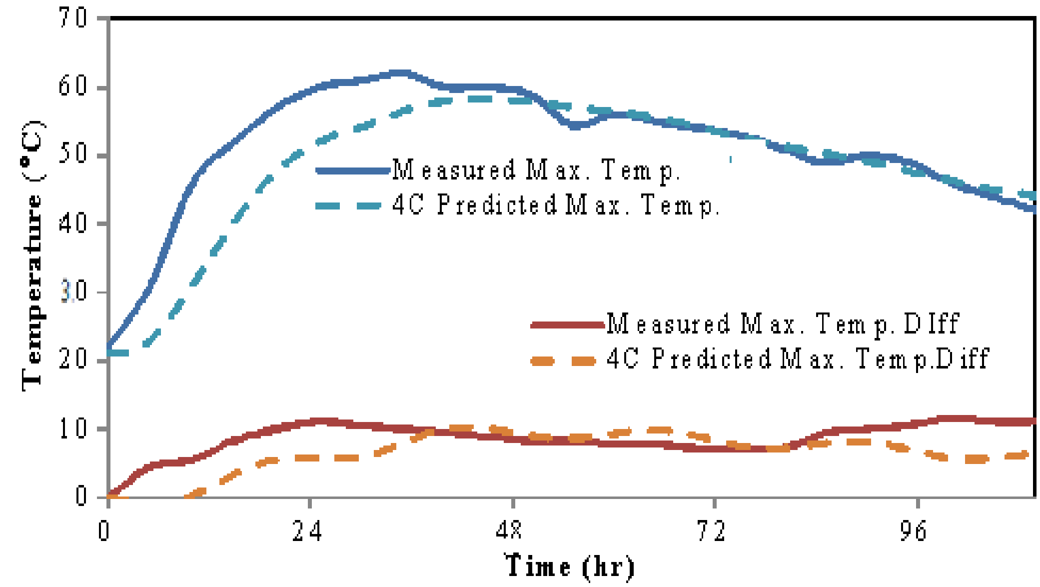
Based on the discussion above, in order to ensure that $б_t/f_t$ <0.75, it is recommended that the critical maximum temperature difference limits should be set at 30 °C for 24 to 48 hours after concrete is placed when the subbase is soil and at 15 °C for after 72 hours when the subbase is concrete. When the subbase material is soil, the allowable maximum temperature difference is increasing during 0 to 72 hours. When the subbase material is concrete, the allowable maximum temperature difference is decreasing. This may be due to the following:

- A concrete subbase may provide more restraint to footings and columns in comparison to soil, thus increasing the stresses in footings and columns with a concrete subbase with increasing age
- The data from 4C analyses are fitted using the covariance model for all of the various time intervals that were analyzed, so that the R^2 value of each model is the same

Further study is needed to fully explain the trends that were observed regarding the cracking potential for mass concrete, especially for elements that have concrete subbases.

7.2 US 34 Bridge

In an effort to further investigate the applicability of the 4C program to river bridges in the US Midwest, the Pier 4 footing of the US 34 Bridge project was analyzed using the 4C program. The Pier 4 footing was constructed on a soil subbase on March, 30, 2012. The ambient temperature and fresh placement temperature (15.6 °C) were monitored on-site. Cooling pipes were used for this footing, and the temperature prediction results were compared with the collected field measurements as shown in the top graph of Figure 7.2.



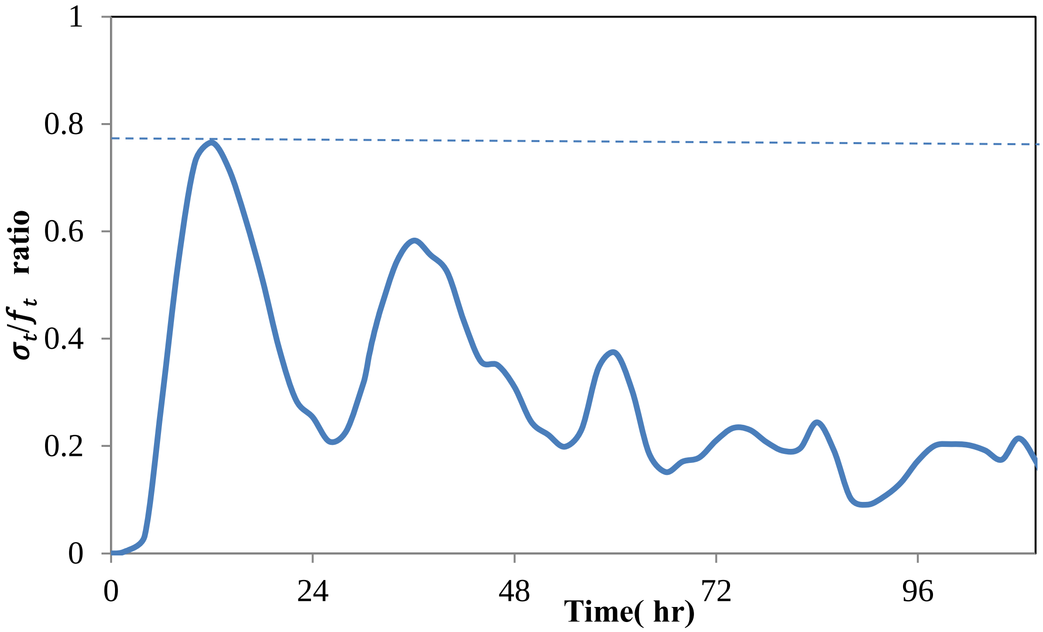


Figure 7.2. Case study results for Pier 4 footing of US 34 Bridge

The maximum temperature of concrete had a 3% discrepancy between the predicted and actual values. The prediction discrepancies for the temperature during the analysis period may be due to the assumptions made in 4C analysis, such as input ambient temperature not being exactly the same as actual monitored environmental temperature. The critical $б_t/f_t$ ratio is below 0.75. This matches with field observations that found the concrete element showed no evidence of thermal cracking upon field investigation. These observations corroborate the finding from the study of the I-80 Bridge that indicated the 4C program is useful in predicting the thermal behavior of mass concrete for larger US Midwest river crossing bridges.

CHAPTER 8. CONCLUSIONS AND RECOMMENDATIONS

The research yielded the following conclusions and recommendations:

1. ConcreteWorks is capable of predicting the general trend of thermal development of mass concrete elements. In a comparison between actual and predicted maximum temperatures for 22 different concrete elements on theI-80 WB Bridge, the errors ranged from underestimates of 35°F to overestimates of about 1°F; the average was an underestimate of 12.3°F. In a comparison between actual and predicted maximum temperature differences, errors ranged from underestimates of 21°F to overestimates of 14°F with an average of 1.9°F. Some adjustment to the inputs and outputs could be made to ensure that the results are conservative. Input values would be easily available to Iowa DOT personnel. Output regarding cracking potential is only available for the first seven days of the placement and cracking potential is described qualitatively as low, medium, and high. Because of a programming limitation, the entire analysis ends in 14 days, while thermal development continues on some typical concrete placements in Iowa for a longer period.

2. 4C-Temp&Stress is also capable of predicting the general trend of thermal development of mass concrete elements. A comparison between actual and predicted maximum concrete temperatures for 26 concrete elements were within 25 degrees, except for stem elements, which had predictions of lesser quality. Many input values would be easily available to Iowa DOT personnel; however, some effort to correlate or calculate some input values is required. The length of time for the output covers the entire thermal development period for the type of construction in the case studies of I-80 WB and US 34. Output is provided as temperatures and the stress ratio (tensile stress: tensile strength) at various locations. Iso-curves are also available for temperature and stress ratio.

3. Sensitivity analysis using both Concrete Works and 4C both confirm actions that are documented in the literature that are effective in controlling the thermal performance of mass concrete elements. For example, reducing the fresh placement temperature, limiting cement content, and substituting fly ash for concrete all tend to improve the thermal performance of mass concrete. The sensitivity studies provide further verification regarding the operation of ConcreteWorks and 4C-Temp&Stress.

4. The Iowa DOT maximum allowable temperature difference gradient limits specified in Control Heat of Hydration DS-09047, August 17, 2010 are confirmed to be applicable for bridges similar to that of the WB I-80 Missouri River Bridge and the US 34 Missouri River Bridge, where bridge elements are founded on concrete. By having lower limits on the maximum allowable temperature difference at earlier ages, the specification recognizes that concrete is relatively weak shortly after placement and becomes stronger and more able to resist thermal cracking as it matures.

5. Further investigation regarding the influence of subbase material on cracking and how to model cooling pipes in mass concrete elements would be useful.

6. Enhancing ConcreteWorks to have longer analysis periods would increase its usefulness for modeling mass concrete placements that are similar to those for the I-80 WB and US 34 bridges over the Missouri River.

7. The Iowa DOT could consider allowing contractors to have greater latitude in developing plans for mass concrete placements if the potential success of such plans can be verified by 4C-Temp&Stress or ConcreteWorks.

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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. COMPARISON BETWEEN 4C (PREDICTION) AND CTL (ACTUAL)

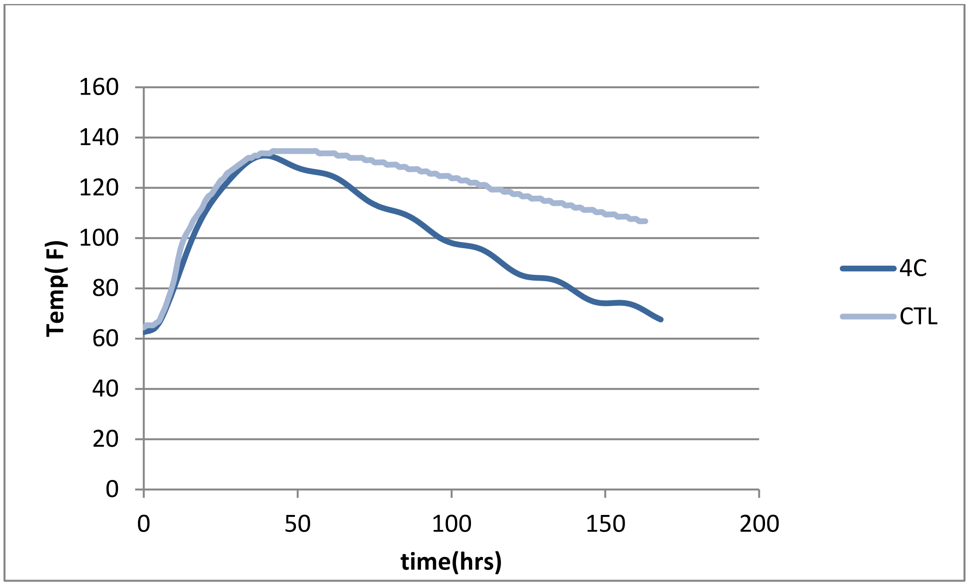


Figure B.1. Maximum temperature development for Pier 2 footing comparison between measured (CTL) and predicted (4C)

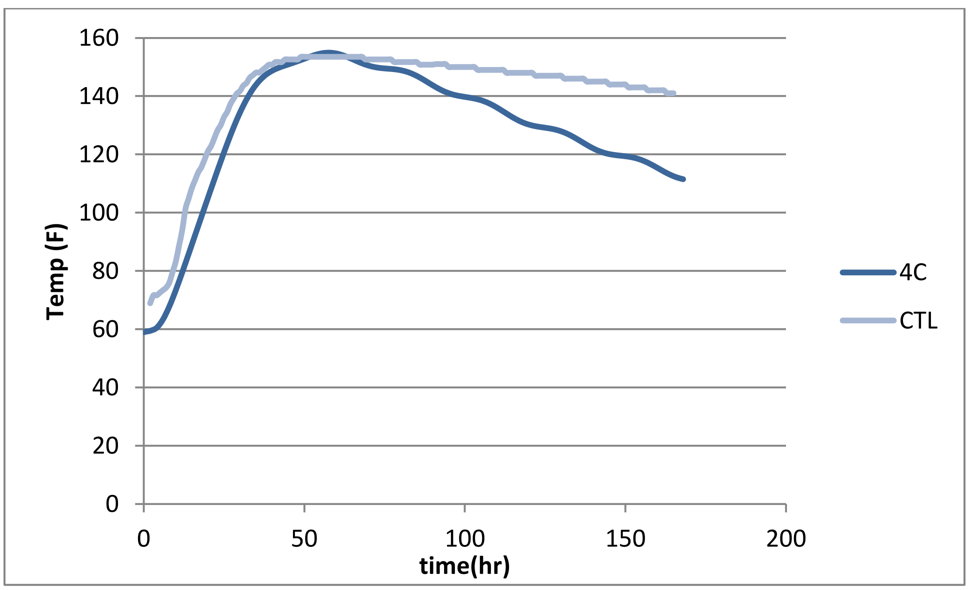


Figure B.2. Maximum temperature development for Pier 3 footing comparison between measured (CTL) and predicted (4C)

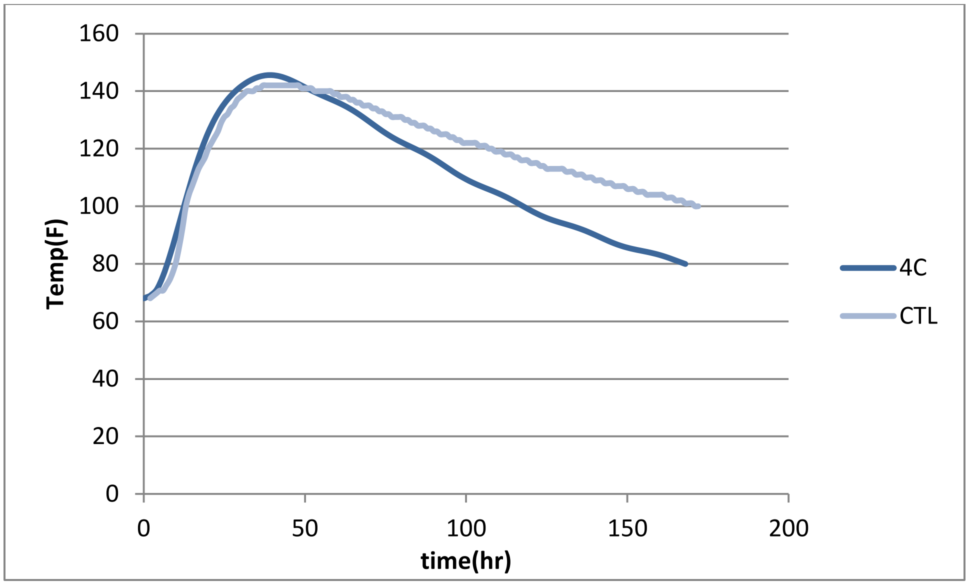


Figure B.3. Maximum temperature development for Pier 4 footing comparison between measured (CTL) and predicted (4C)

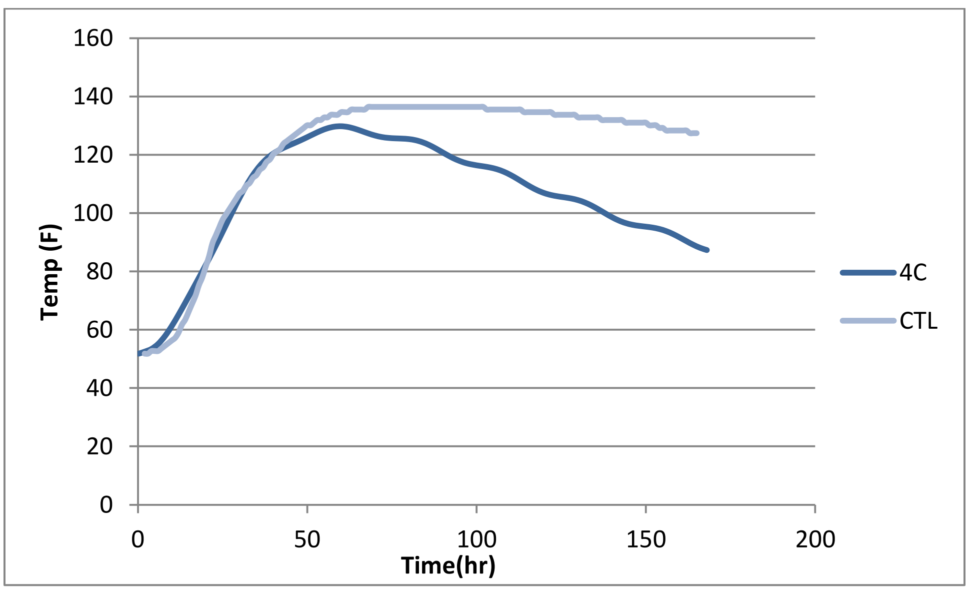


Figure B.4. Maximum temperature development for Pier 5 footing comparison between measured (CTL) and predicted (4C)

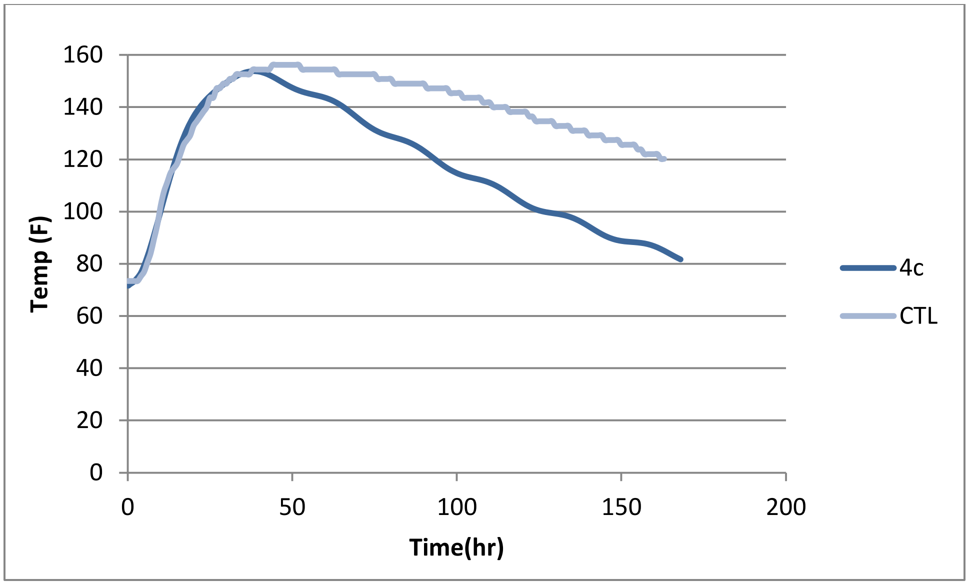


Figure B.5. Maximum temperature development for Pier 6 footing comparison between measured (CTL) and predicted (4C)

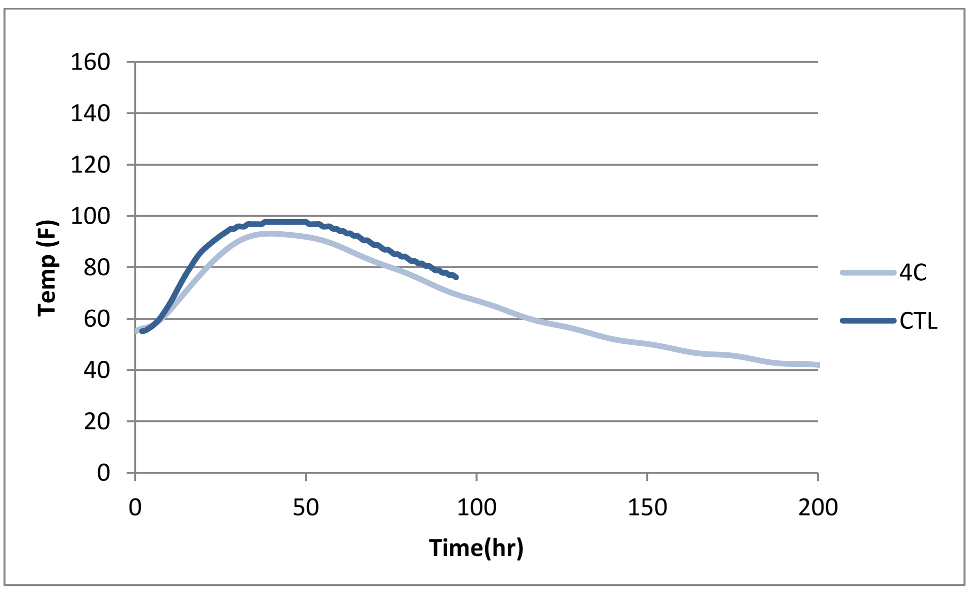


Figure B.6. Maximum temperature development for Pier 1 stem comparison between measured (CTL) and predicted (4C)

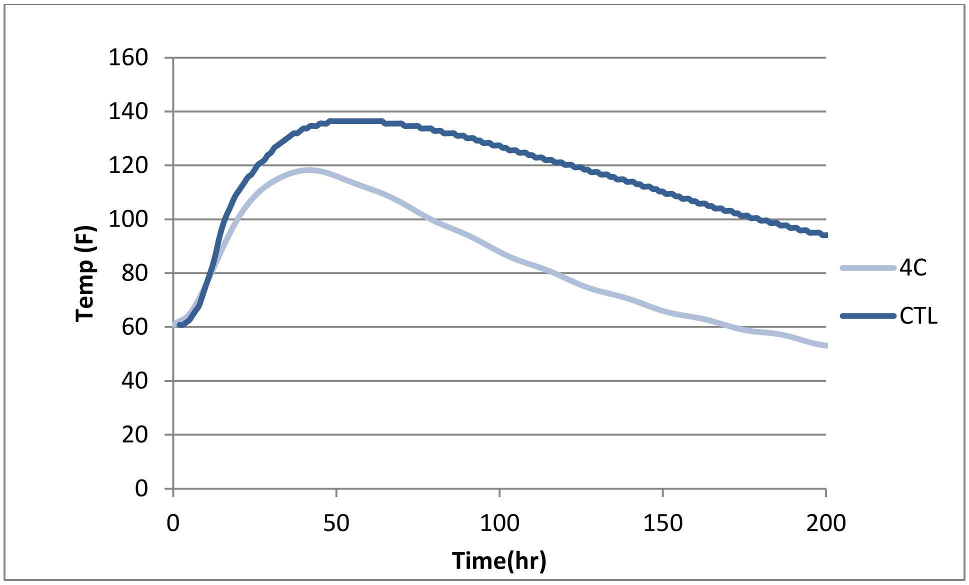


Figure B.7. Maximum temperature development for Pier 2 stem comparison between measured (CTL) and predicted (4C)

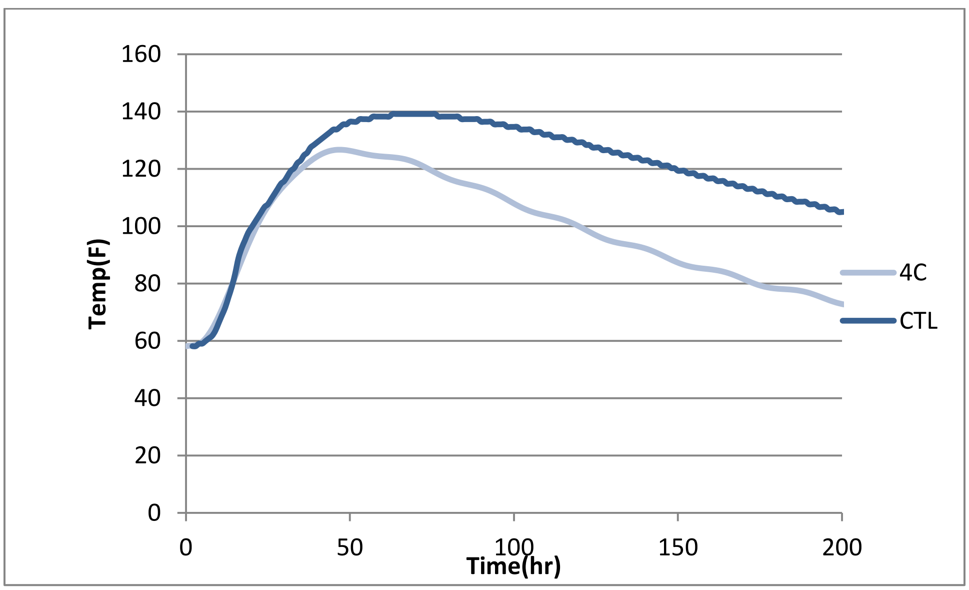


Figure B.8. Maximum temperature development for Pier 3 stem comparison between measured (CTL) and predicted (4C)

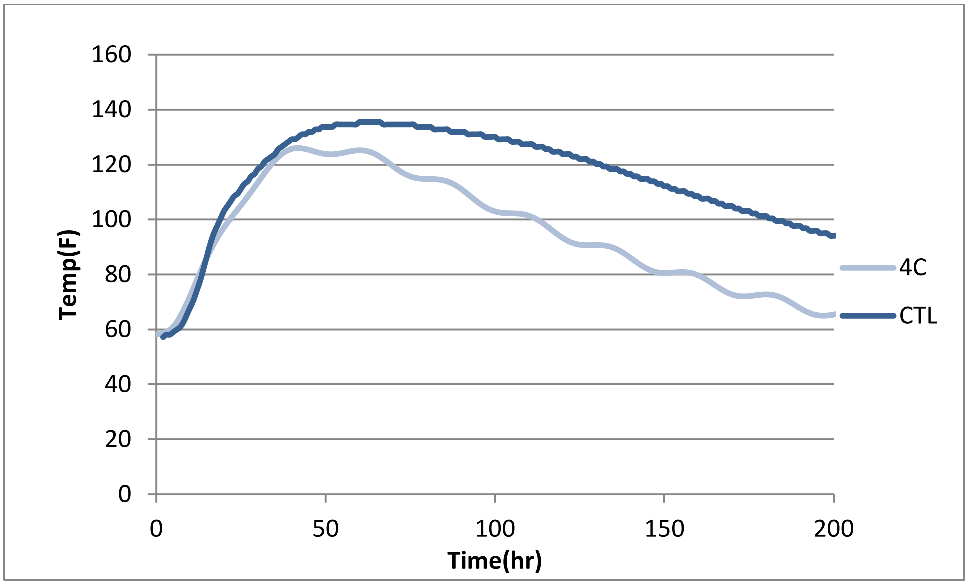


Figure B.9. Maximum temperature development for Pier 4 stem comparison between measured (CTL) and predicted (4C)

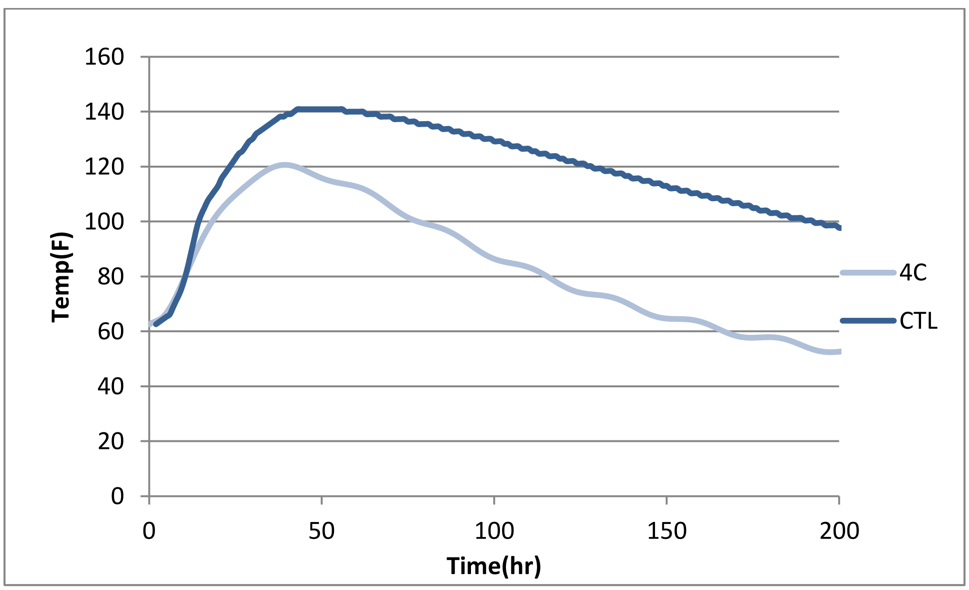


Figure B.10. Maximum temperature development for Pier 5 stem comparison between measured (CTL) and predicted (4C)

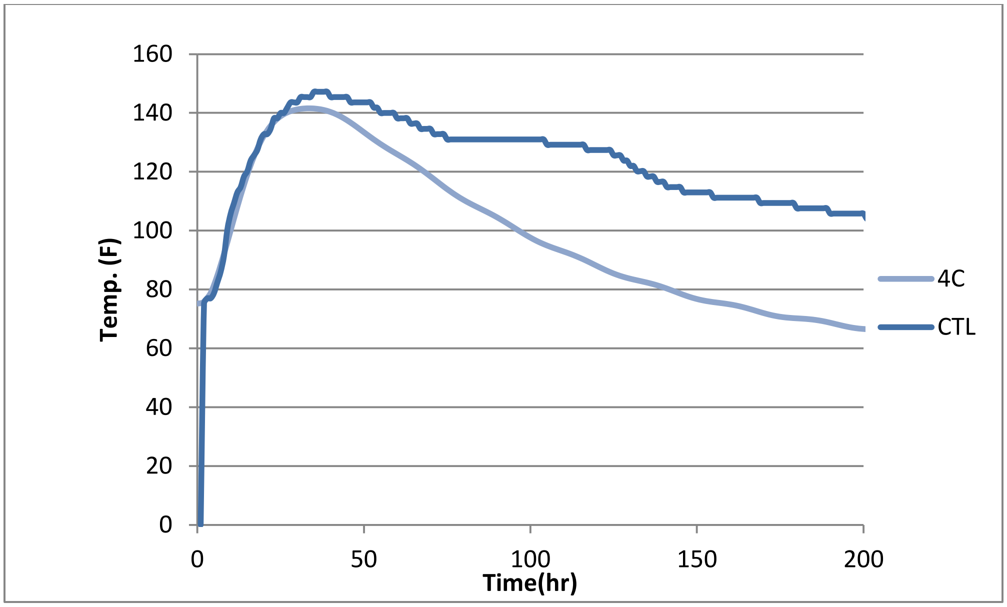


Figure B.11. Maximum temperature development for Pier 7 stem comparison between measured (CTL) and predicted (4C)

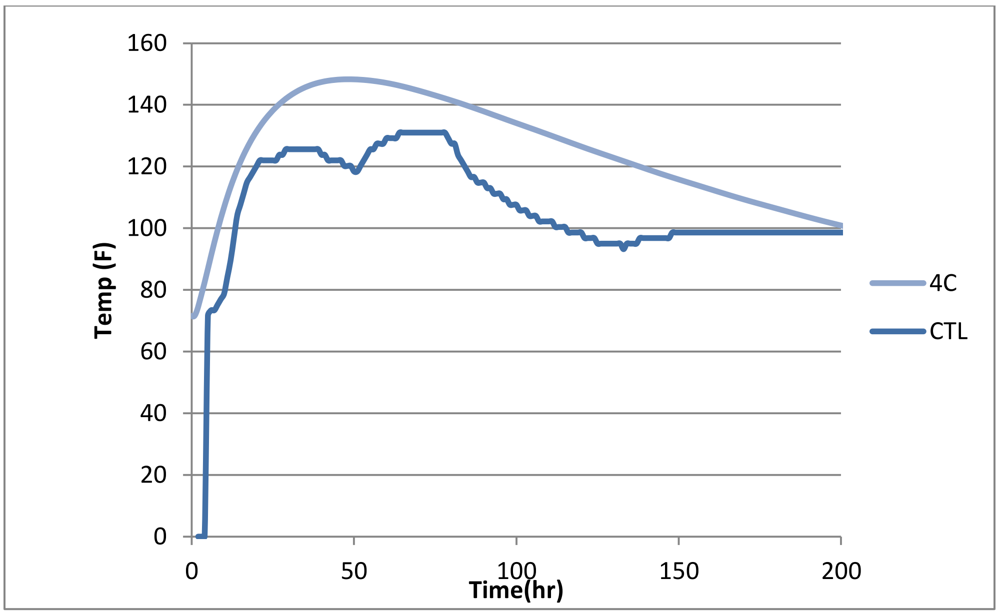


Figure B.12. Maximum temperature development for Pier 9 stem comparison between measured (CTL) and predicted (4C)

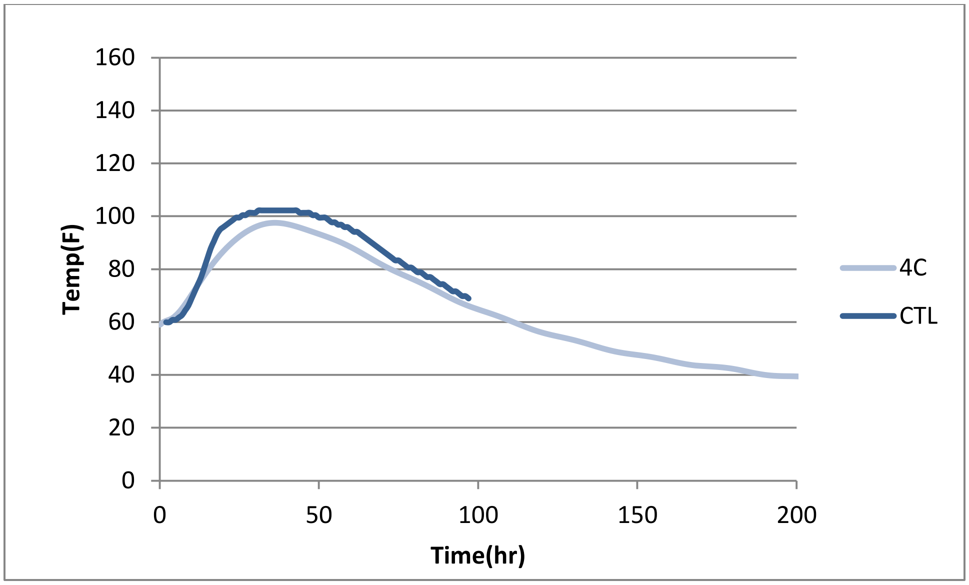


Figure B.13. Maximum temperature development for Pier 1 cap comparison between measured (CTL) and predicted (4C)

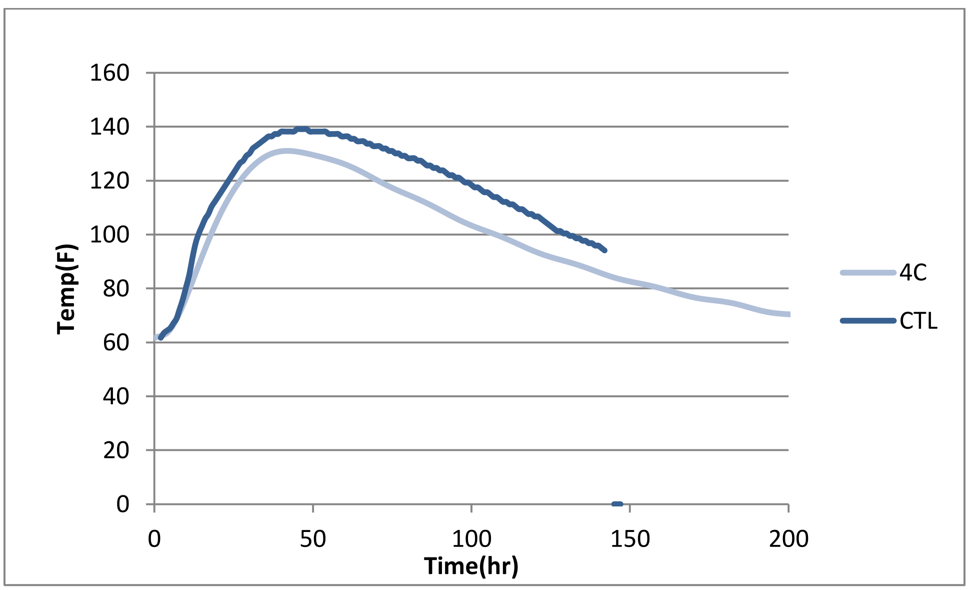


Figure B.14. Maximum temperature development for Pier 2 cap comparison between measured (CTL) and predicted (4C)

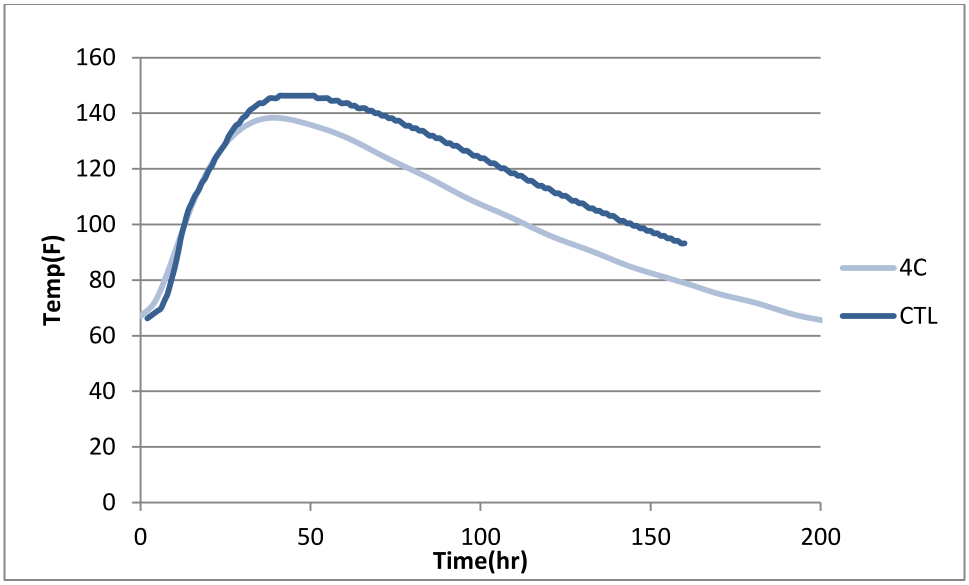


Figure B.15. Maximum temperature development for Pier 3 cap comparison between measured (CTL) and predicted (4C)

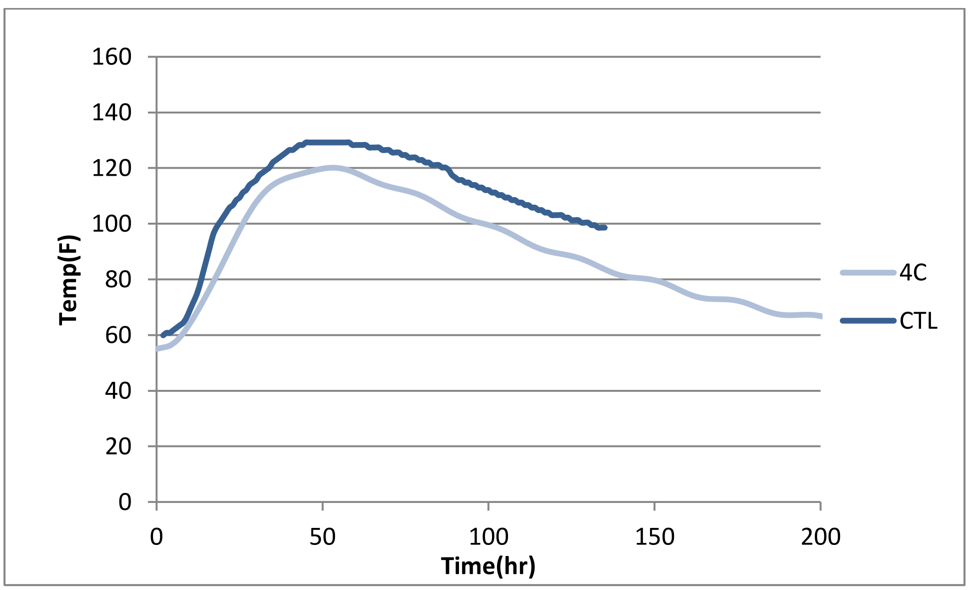


Figure B.16. Maximum temperature development for Pier 4 cap comparison between measured (CTL) and predicted (4C)

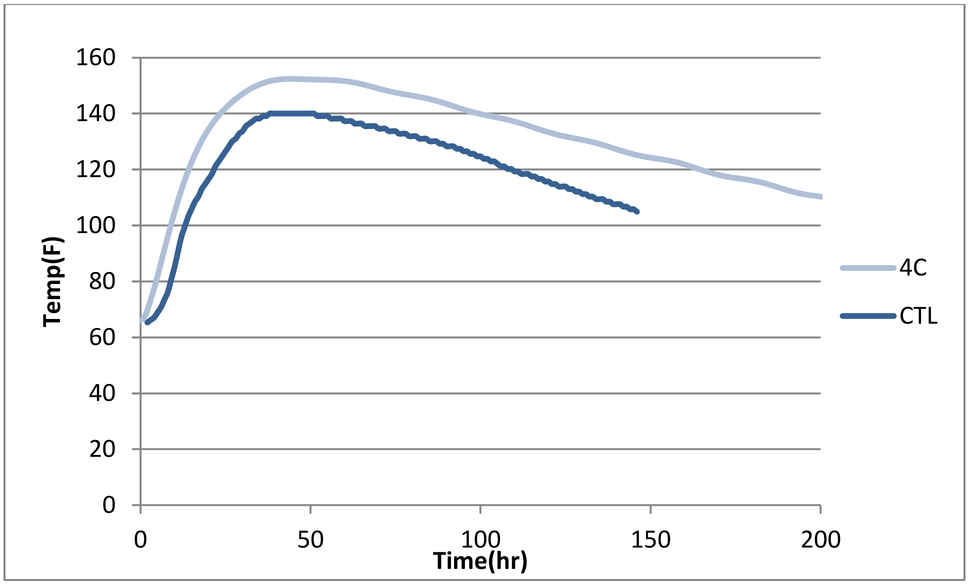


Figure B.17. Maximum temperature development for Pier 5 cap comparison between measured (CTL) and predicted (4C)

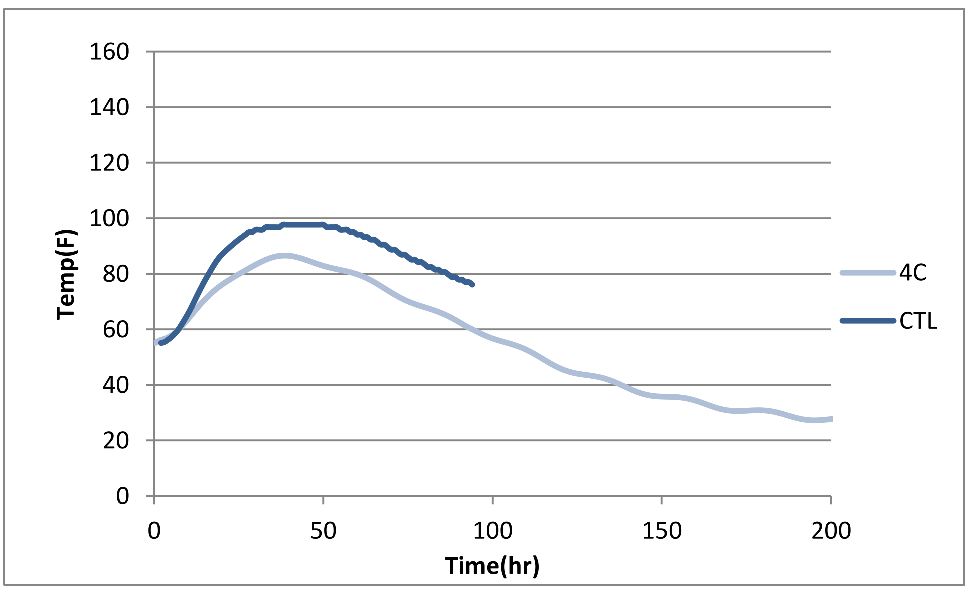


Figure B.18. Maximum temperature development for Pier 1 column comparison between measured (CTL) and predicted (4C)

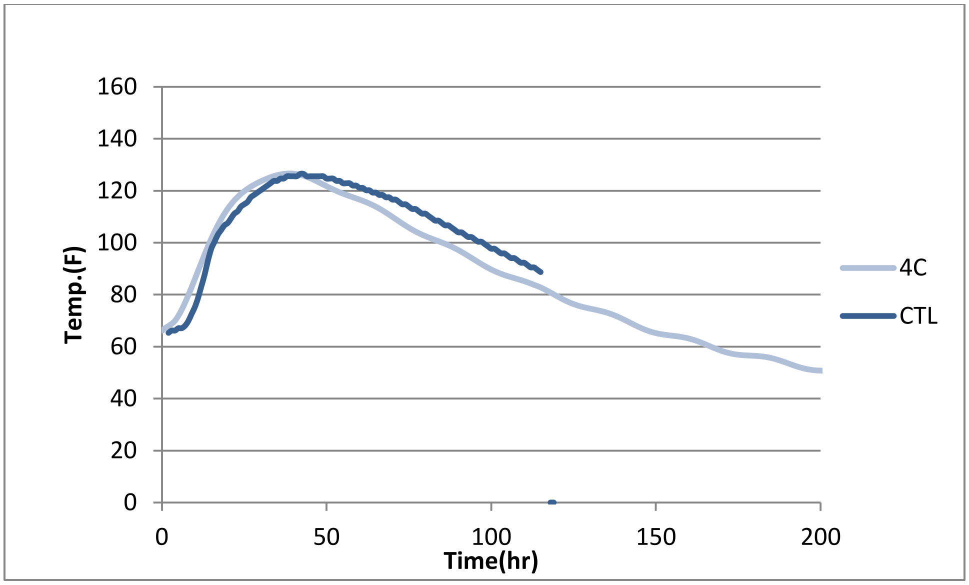


Figure B.19. Maximum temperature development for Pier 2 column comparison between measured (CTL) and predicted (4C)

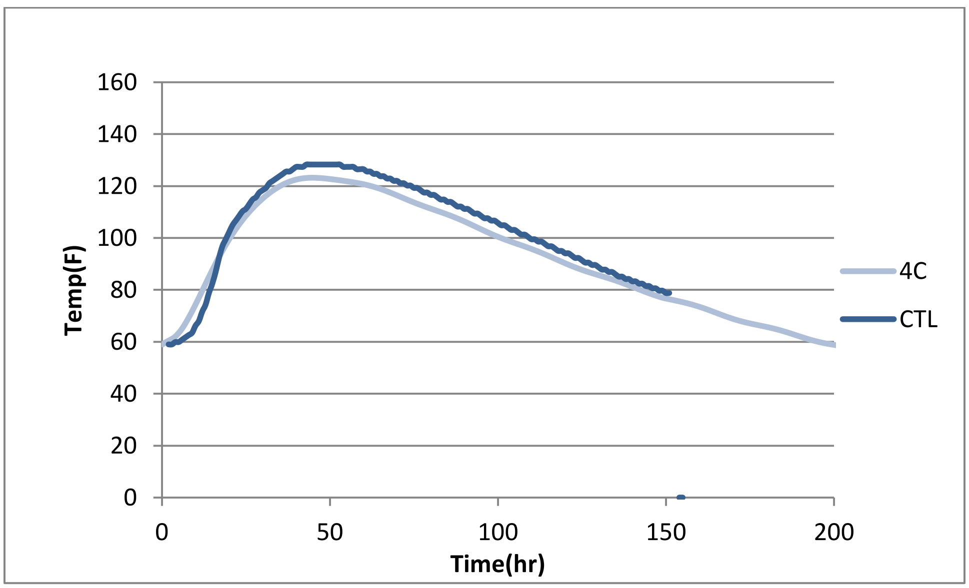


Figure B.20. Maximum temperature development for Pier 3 column comparison between measured (CTL) and predicted (4C)

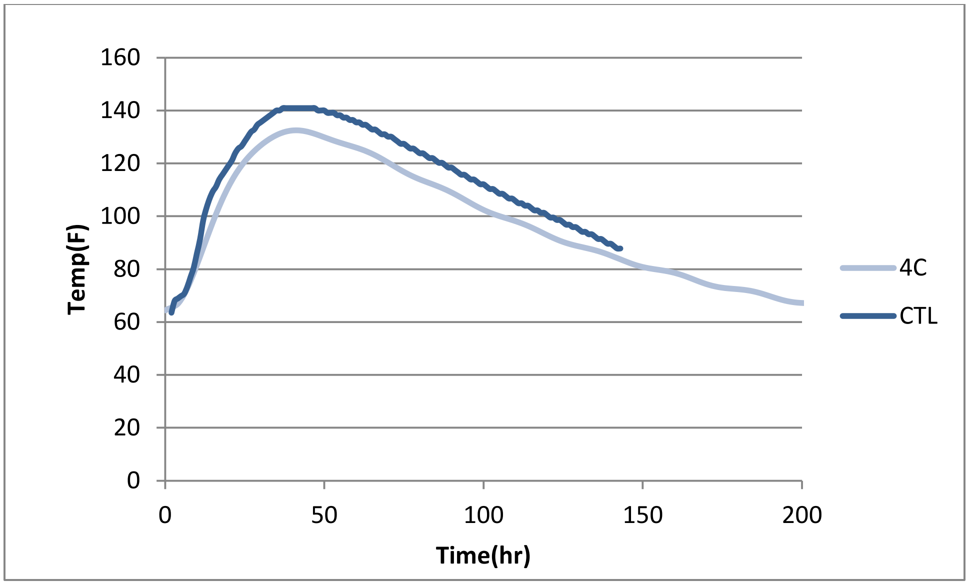


Figure B.21. Maximum temperature development for Pier 4 column comparison between measured (CTL) and predicted (4C)

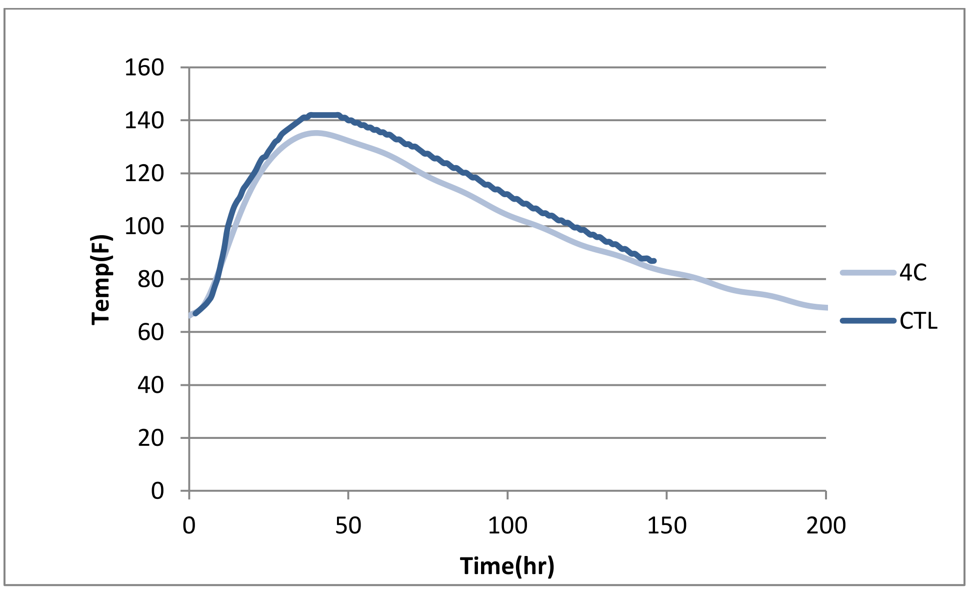


Figure B.22. Maximum temperature development for Pier 2 column comparison between measured (CTL) and predicted (4C)

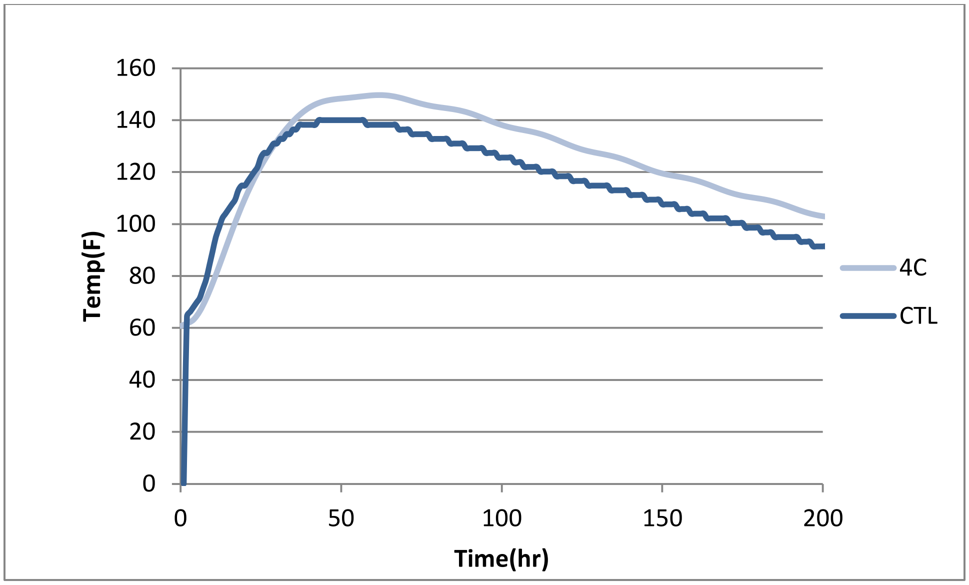


Figure B.23. Maximum temperature development for Pier 7 column comparison between measured (CTL) and predicted (4C)

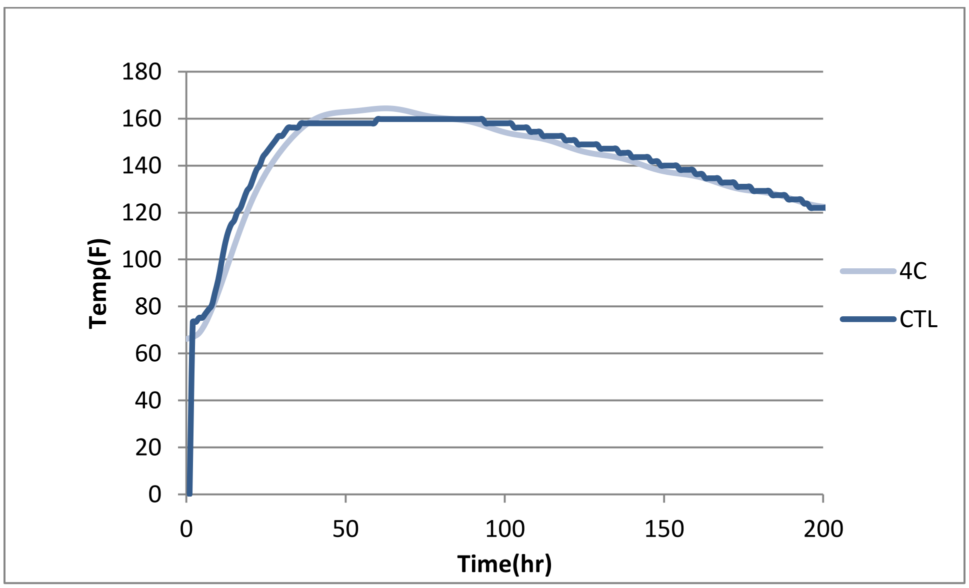


Figure B.24. Maximum temperature development for Pier 10 column comparison between measured (CTL) and predicted (4C)

APPENDIX C. CONCRETEWORKS WESTBOUND I-80 CASE STUDY THERMAL RESULTS

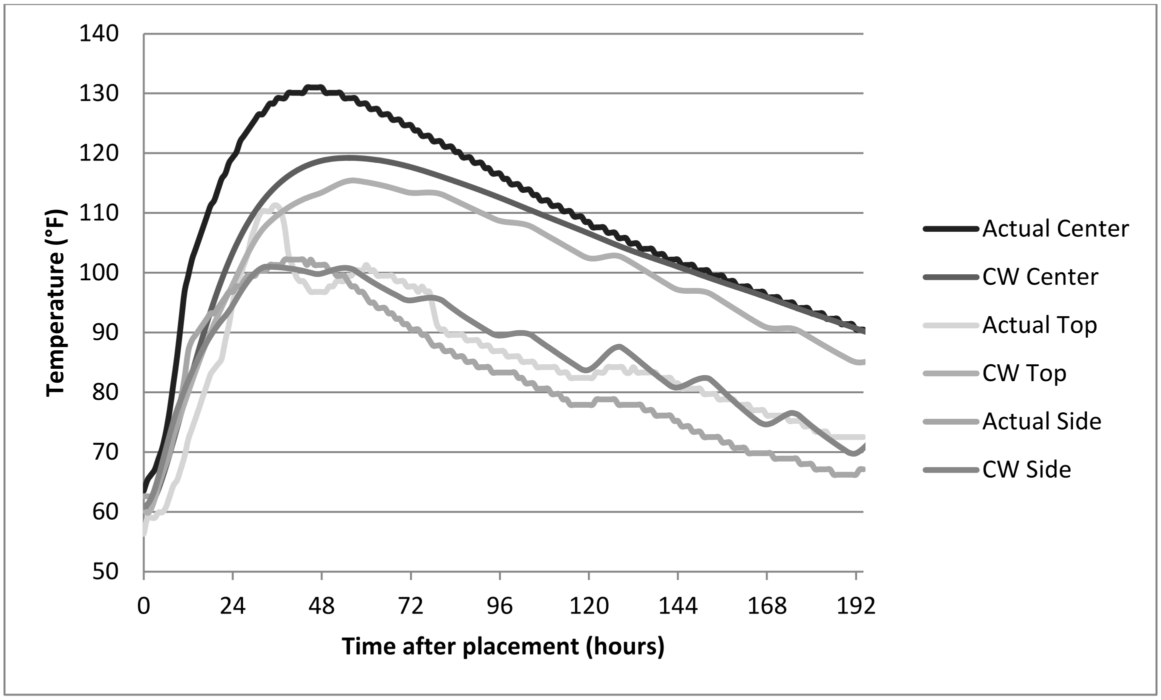


Figure C.1. WB I-80 case study thermal results – Pier 1 footing

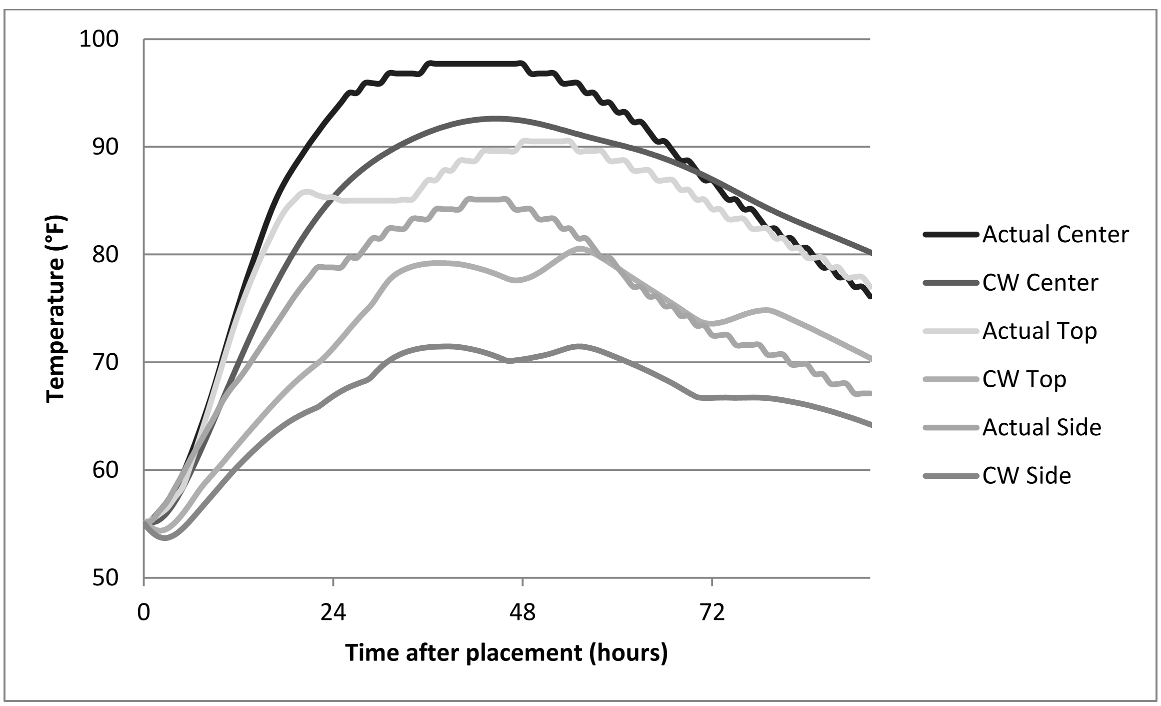


Figure C.2. WB I-80 case study thermal results – Pier 1 stem/column

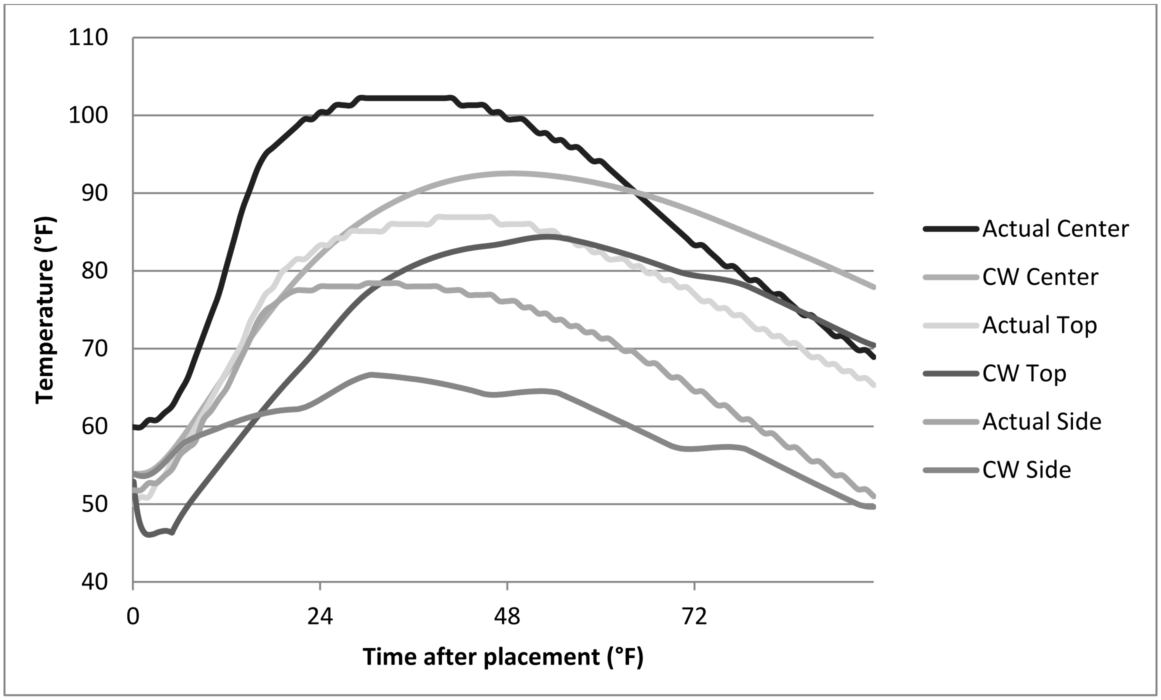


Figure C.3. WB I-80 case study thermal results – Pier 1 cap

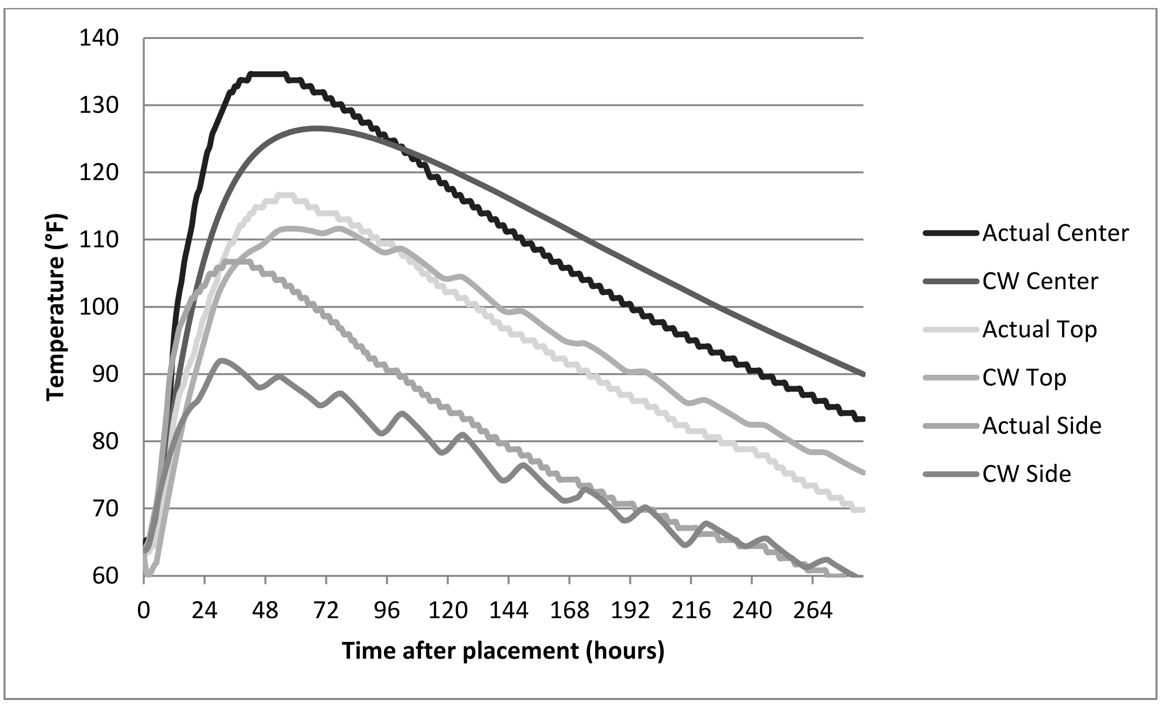


Figure C.4. WB I-80 case study thermal results – Pier 2 footing

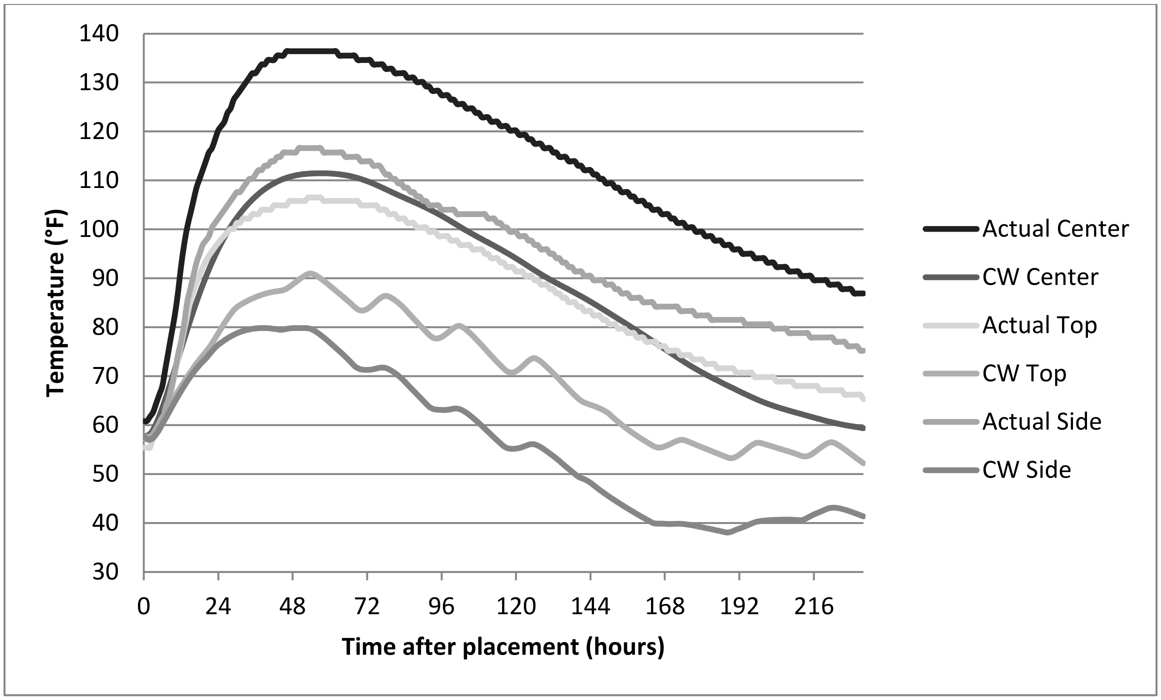


Figure C.5. WB I-80 case study thermal results – Pier 2 stem

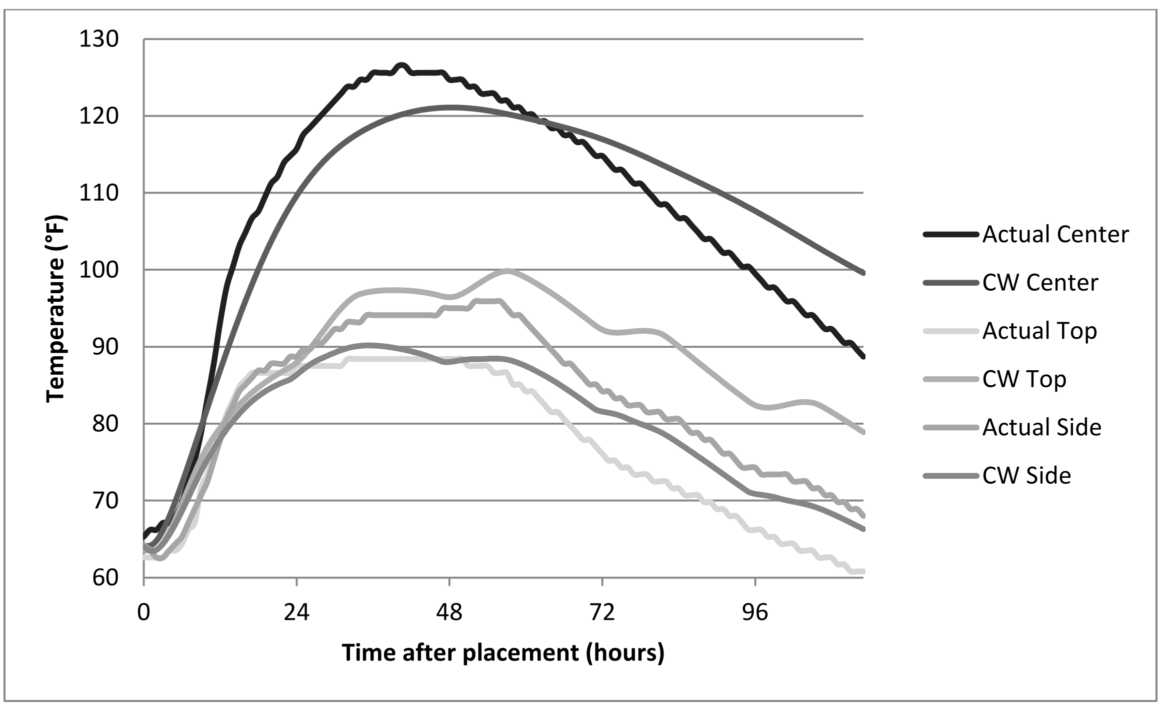


Figure C.6. WB I-80 case study thermal results – Pier 2 column

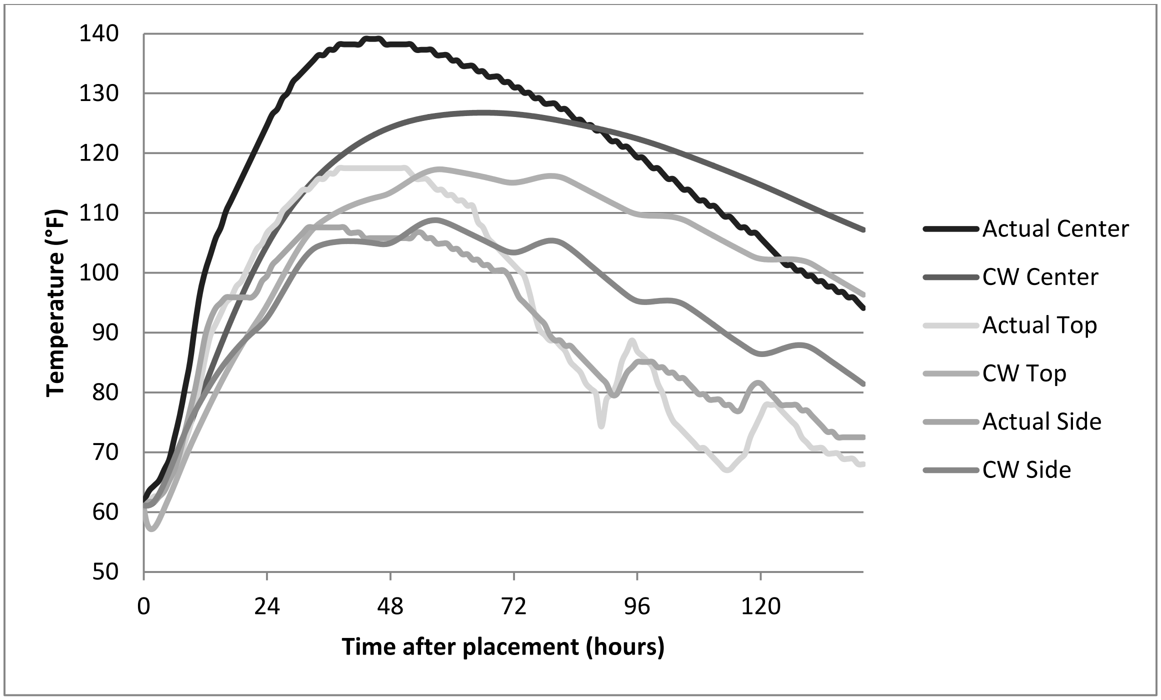


Figure C.7. WB I-80 case study thermal results – Pier 2 cap

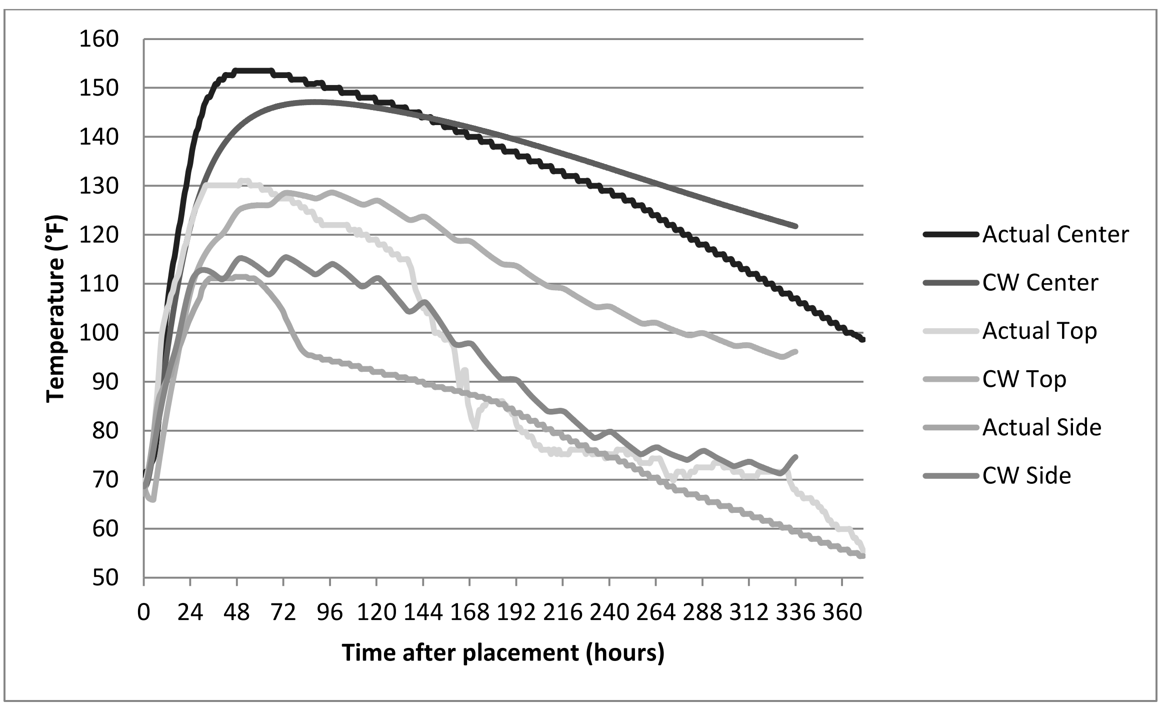


Figure C.8. WB I-80 case study thermal results – Pier 3 footing

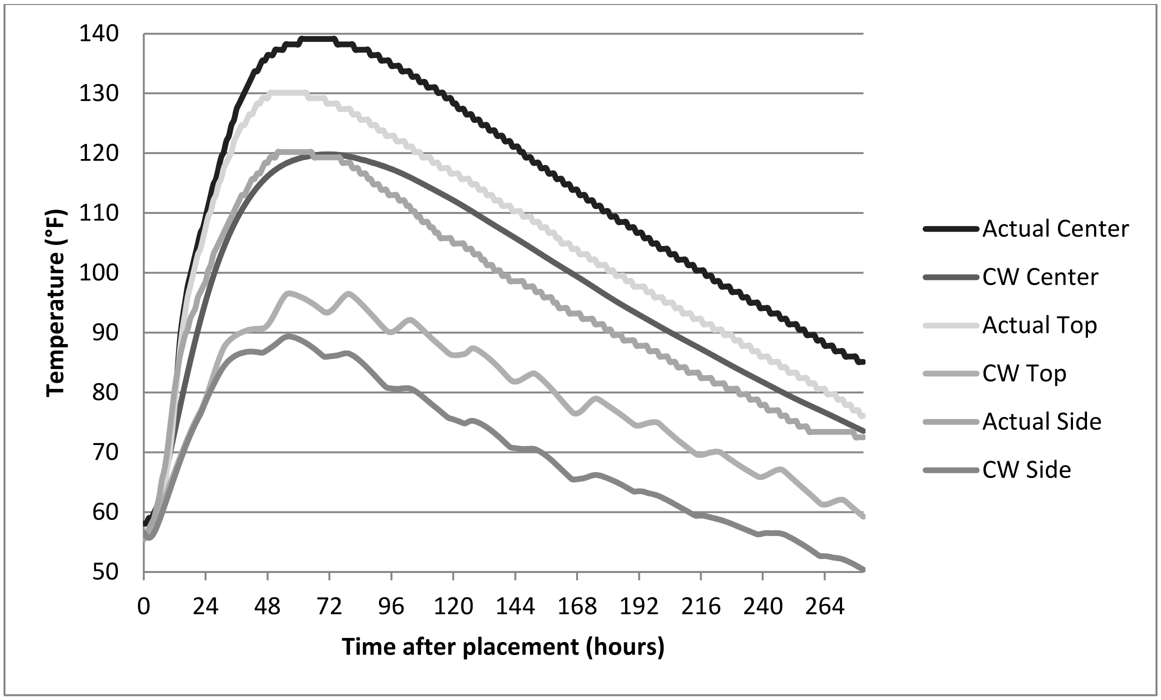


Figure C.9. WB I-80 case study thermal results – Pier 3 stem

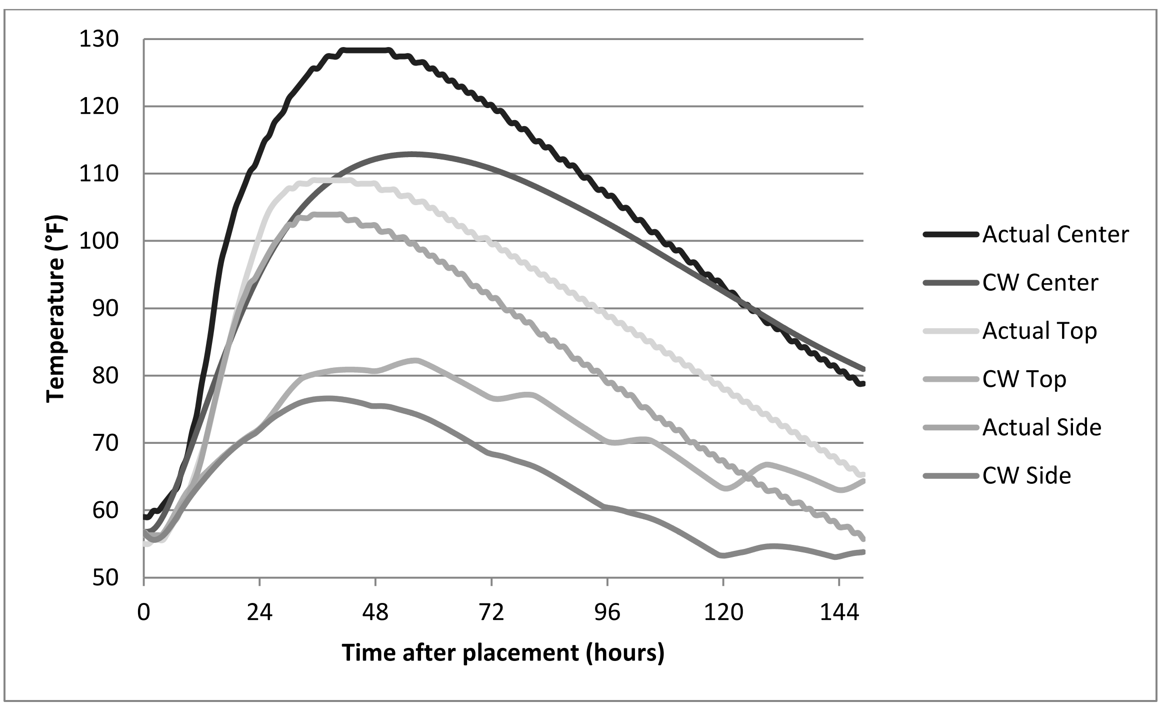


Figure C.10. WB I-80 case study thermal results – Pier 3 column

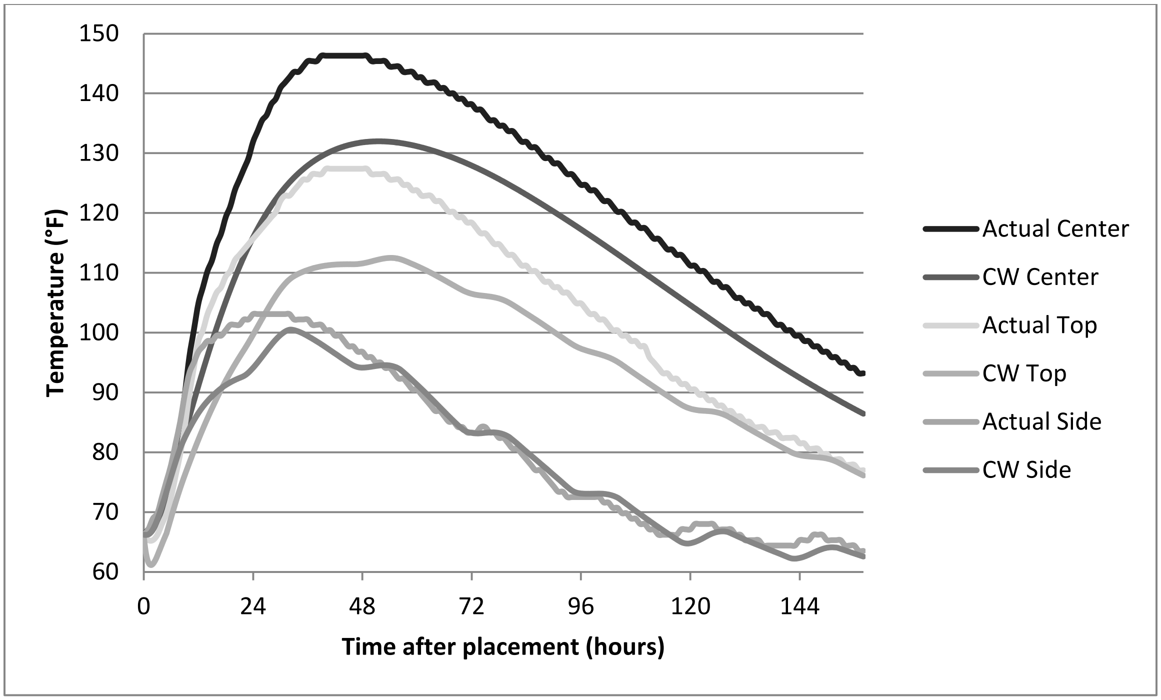
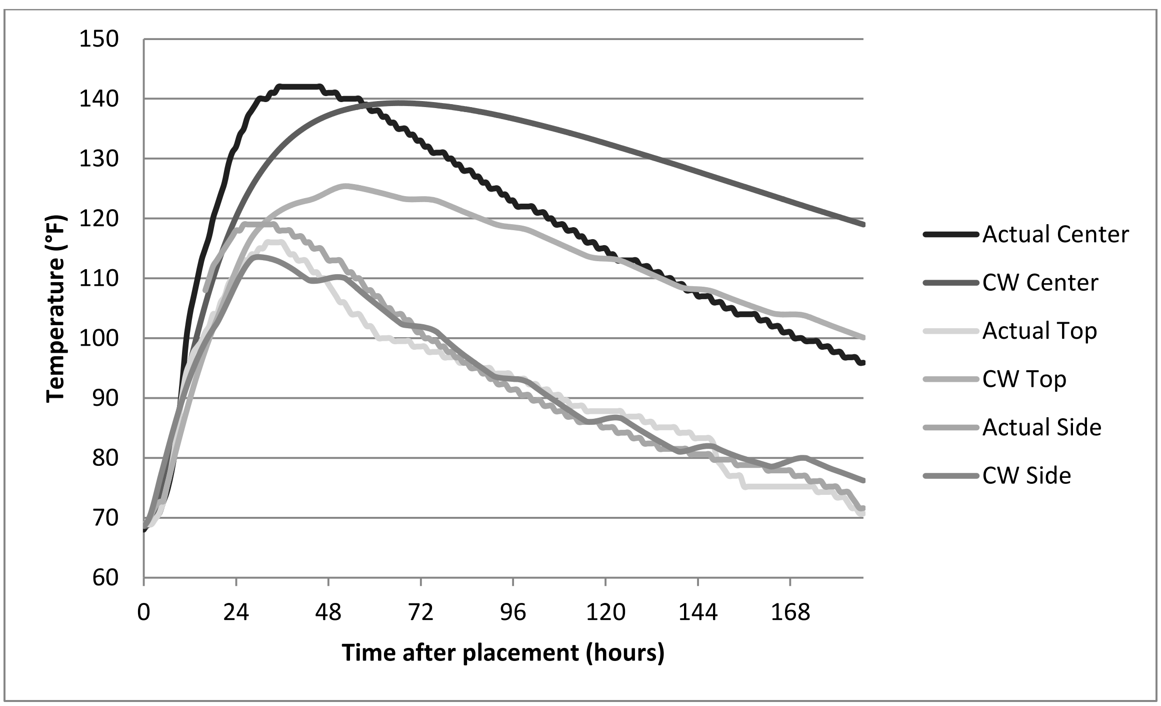


Figure C.11. WB I-80 case study thermal results – Pier 3 cap



*side sensor was not turned on until hours 16

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

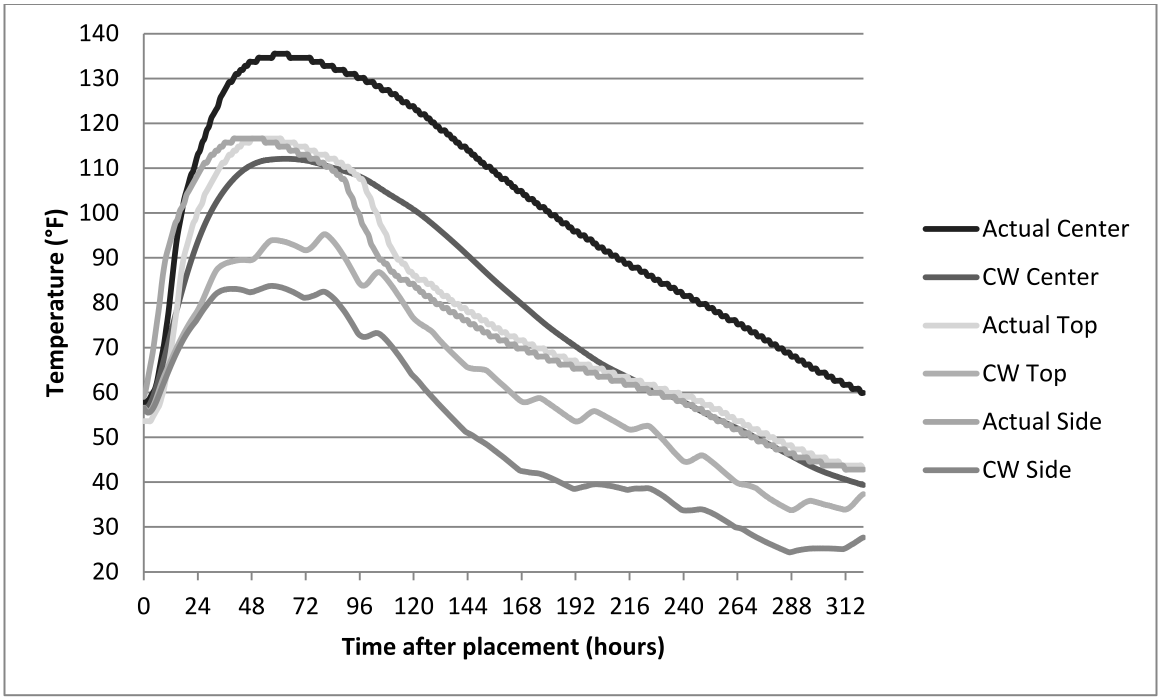


Figure C.13. WB I-80 case study thermal results – Pier 4 stem

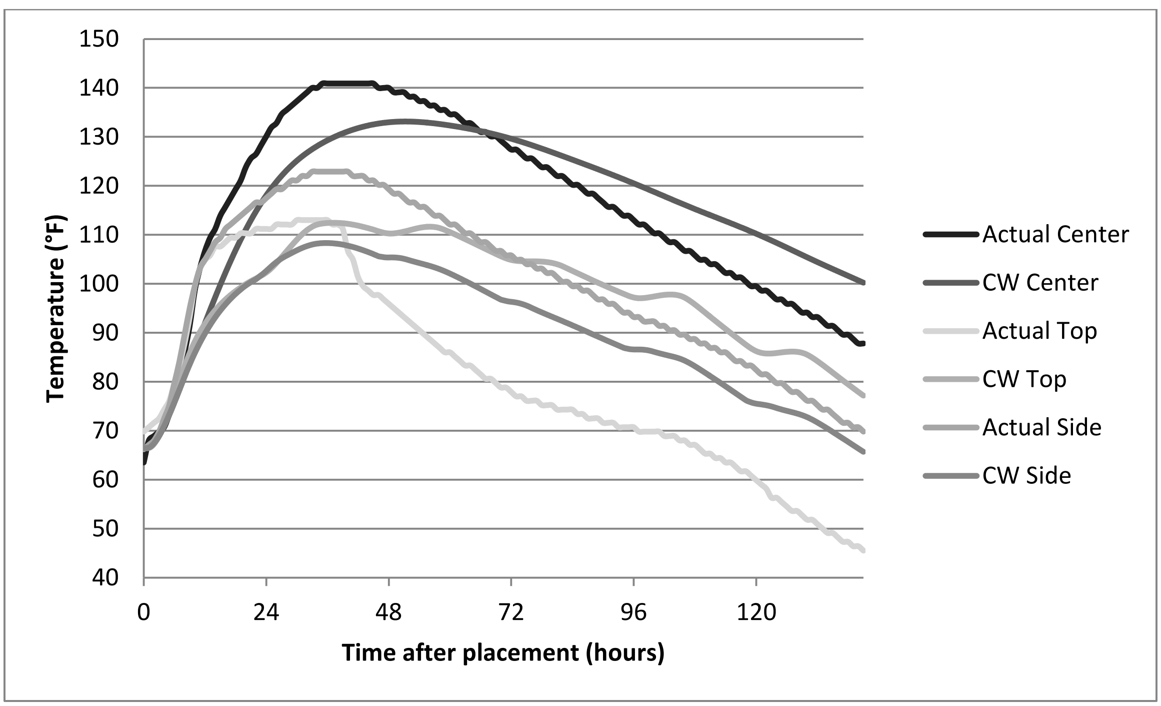


Figure C.14. WB I-80 case study thermal results – Pier 4 column

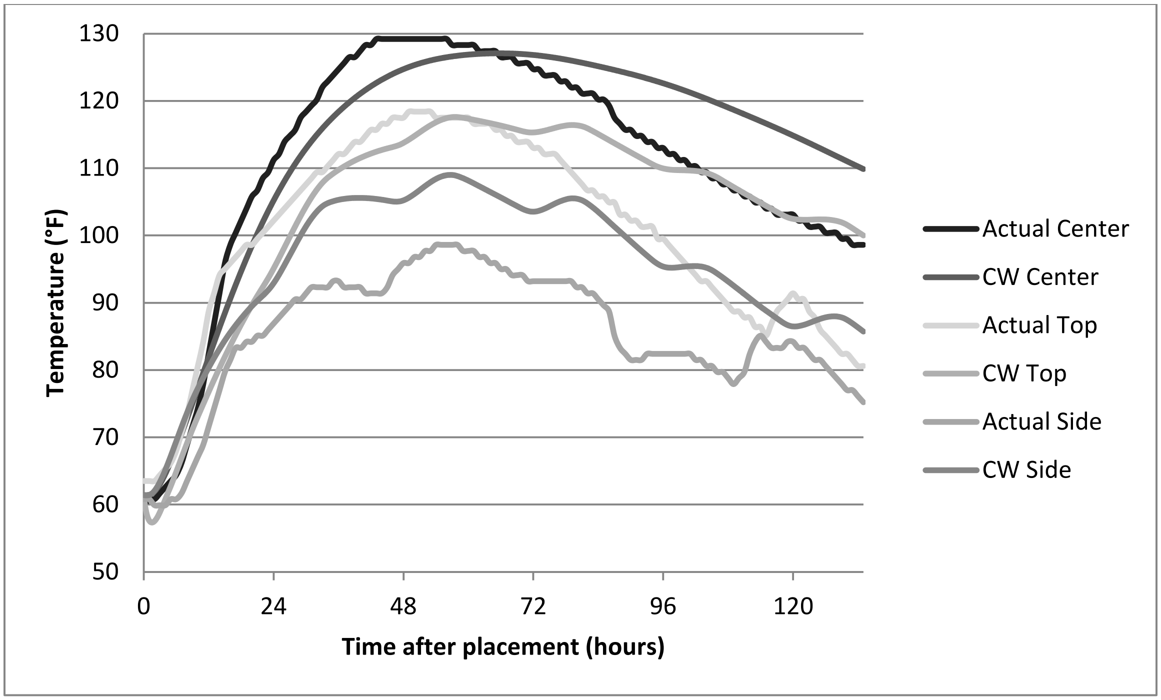


Figure C.15. WB I-80 case study thermal results – Pier 4 cap

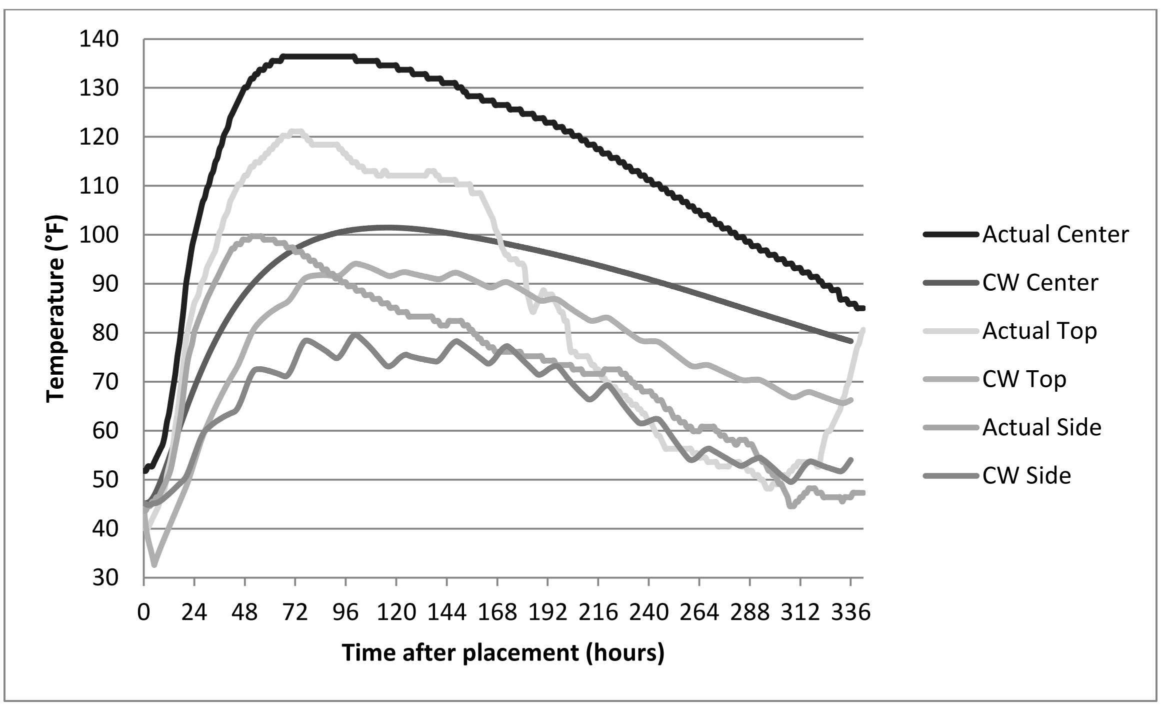


Figure C.16. WB I-80 case study thermal results – Pier 5 footing

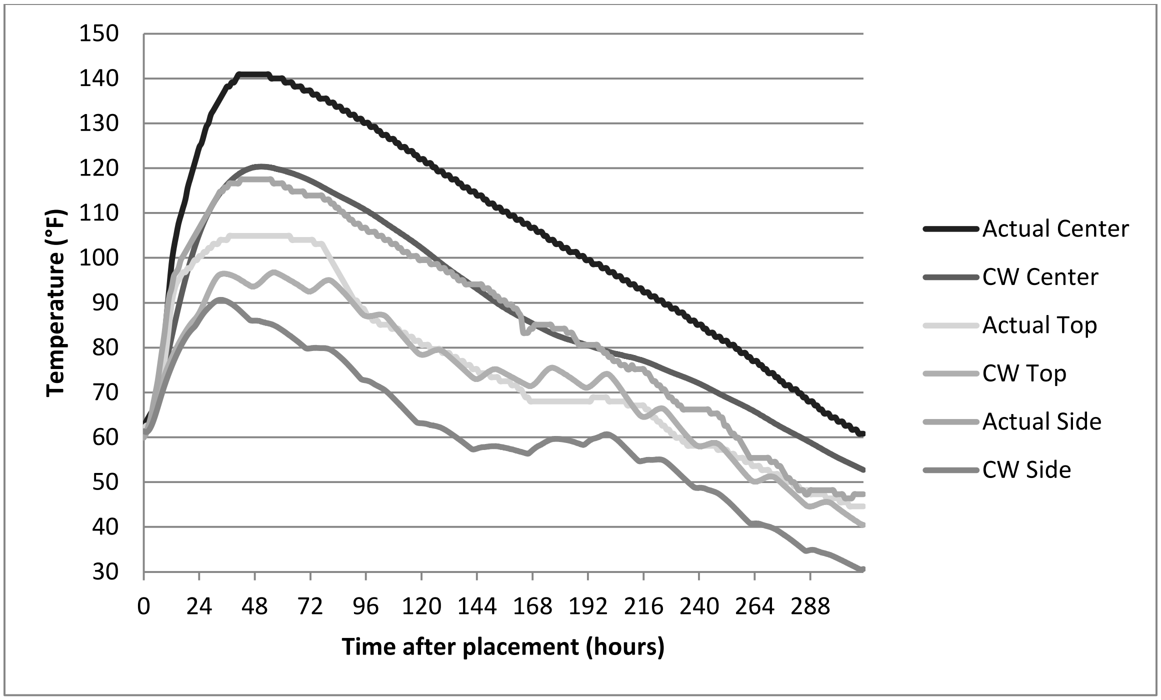


Figure C.17. WB I-80 case study thermal results – Pier 5 stem

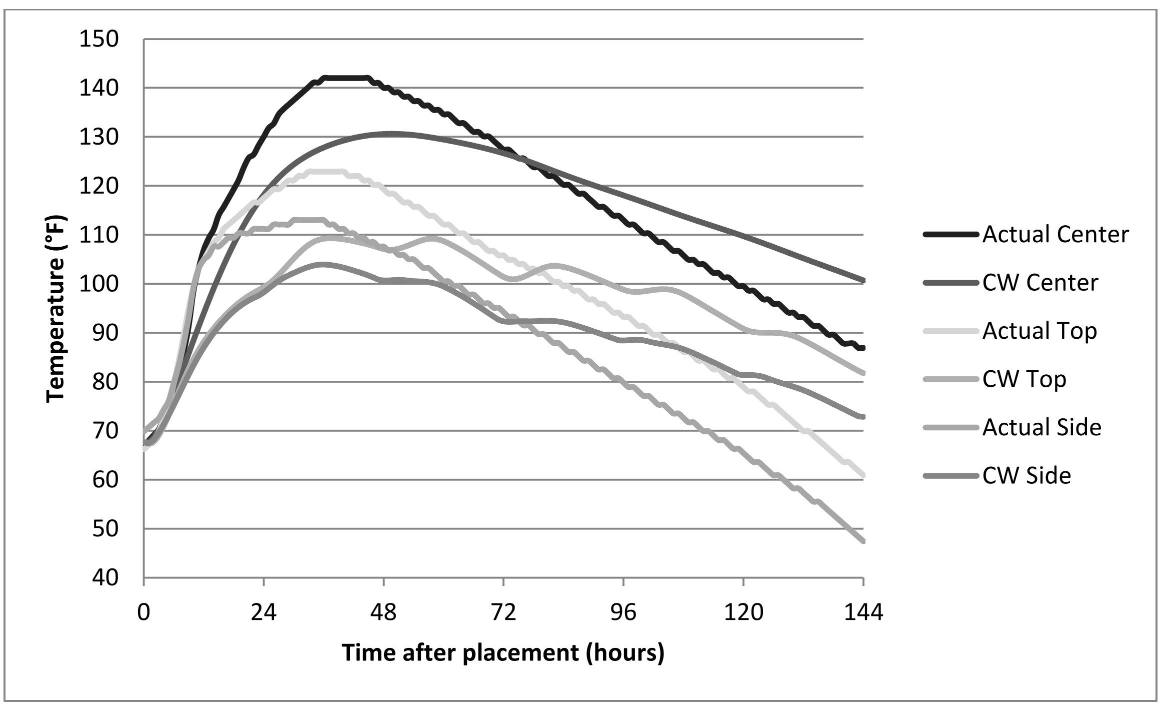


Figure C.18. WB I-80 case study thermal results – Pier 5 column

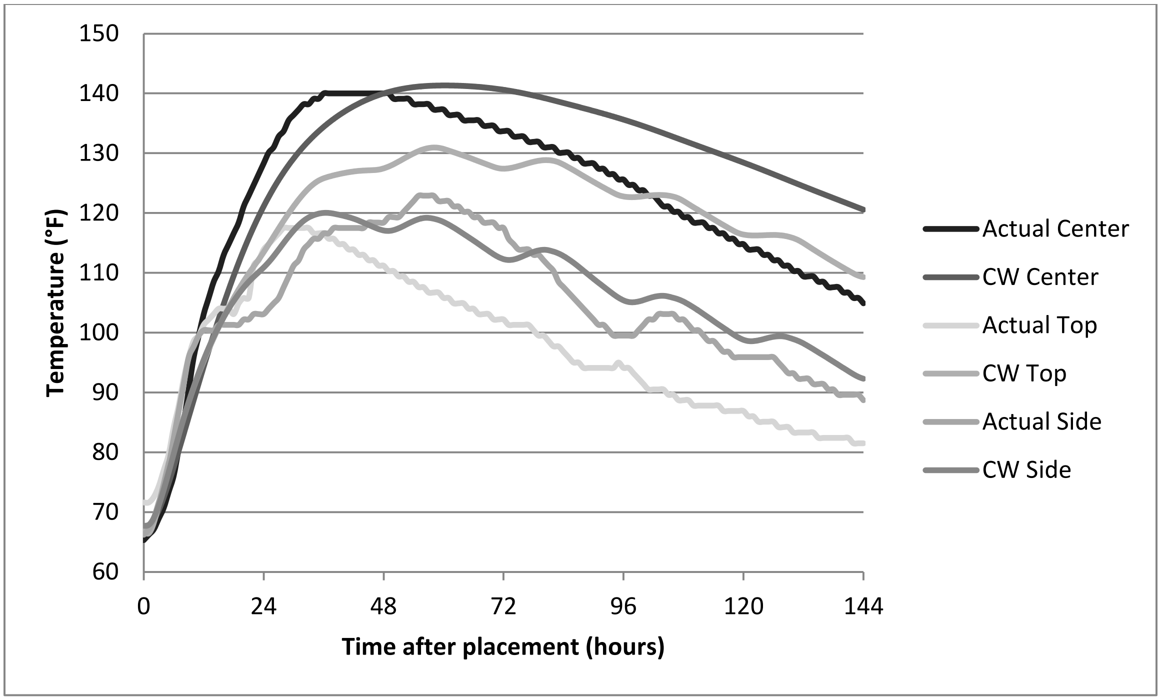


Figure C.19. WB I-80 case study thermal results – Pier 5 cap

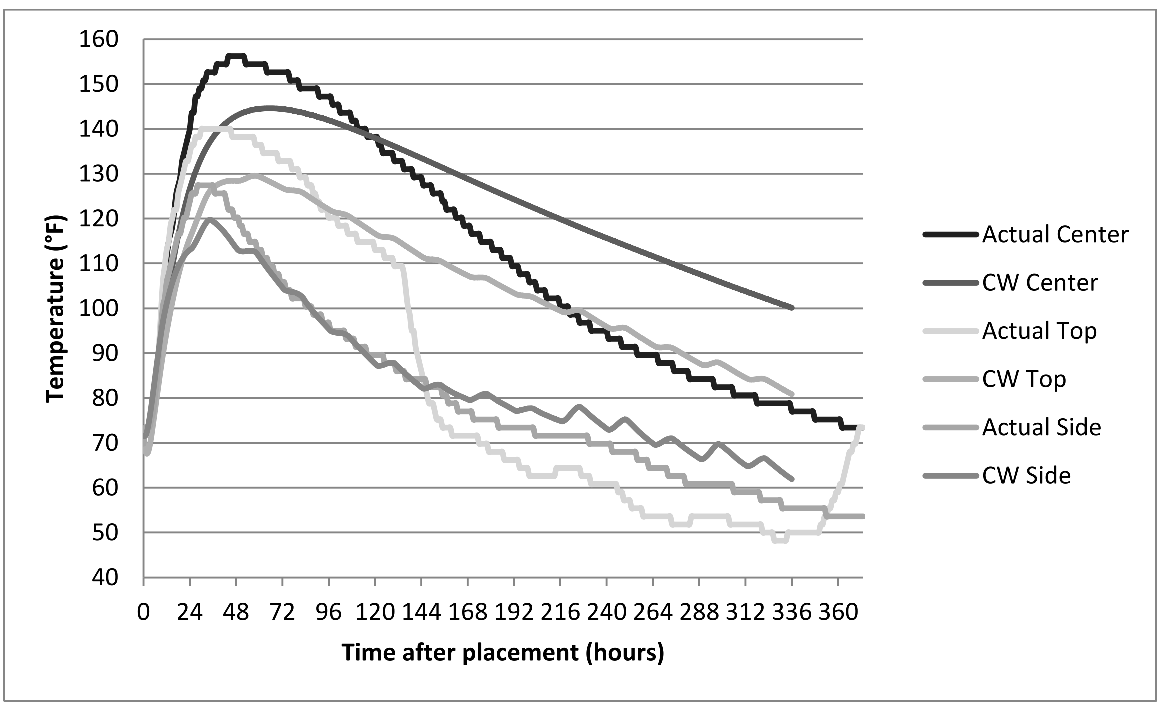


Figure C.20. WB I-80 case study thermal results – Pier 6 footing

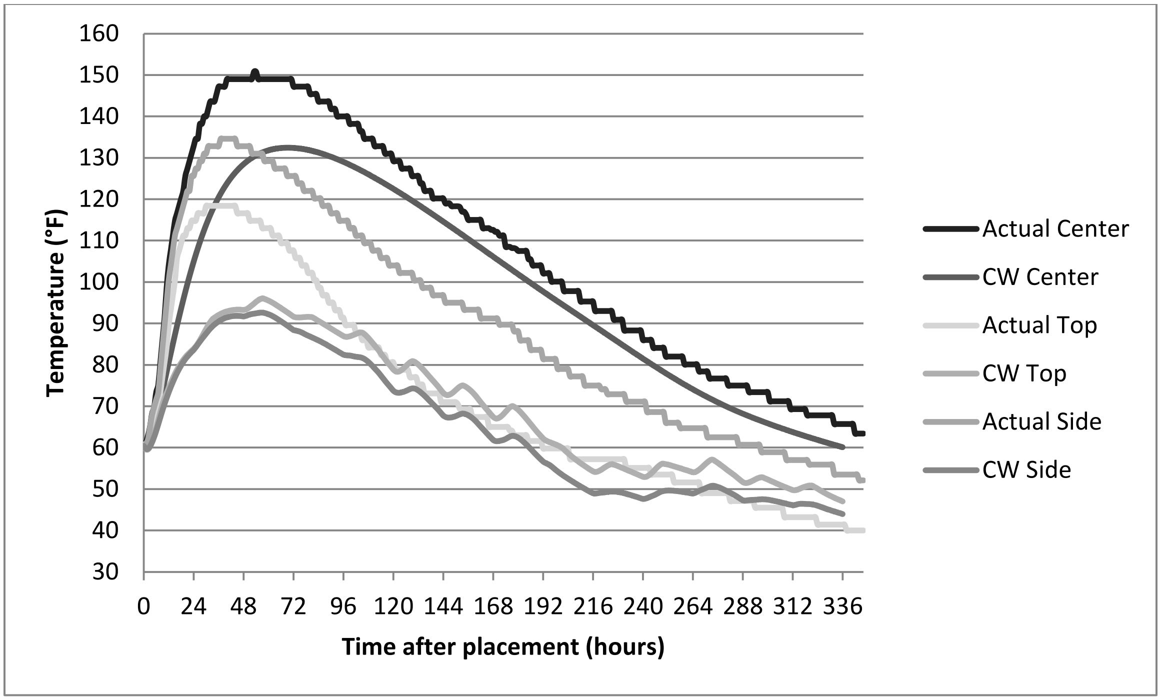


Figure C.21. WB I-80 case study thermal results – Pier 6 column

APPENDIX D. US 34 CASE STUDY THERMAL RESULTS

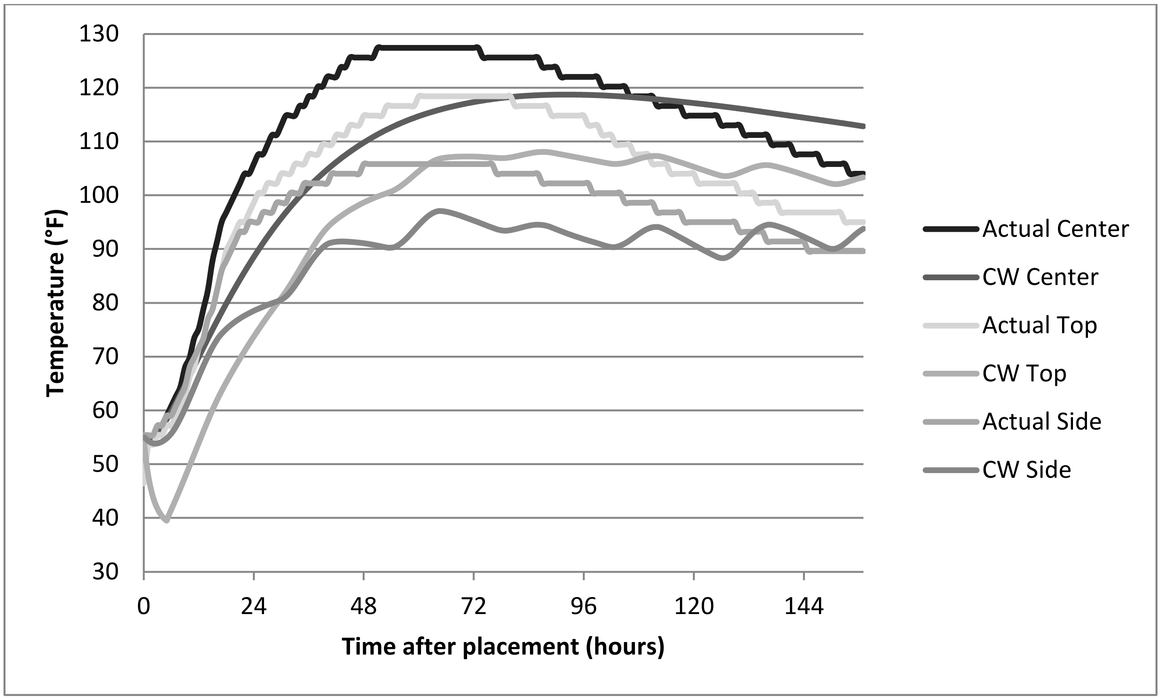


Figure D.1. US 34 case study thermal results – Pier 2 footing – A

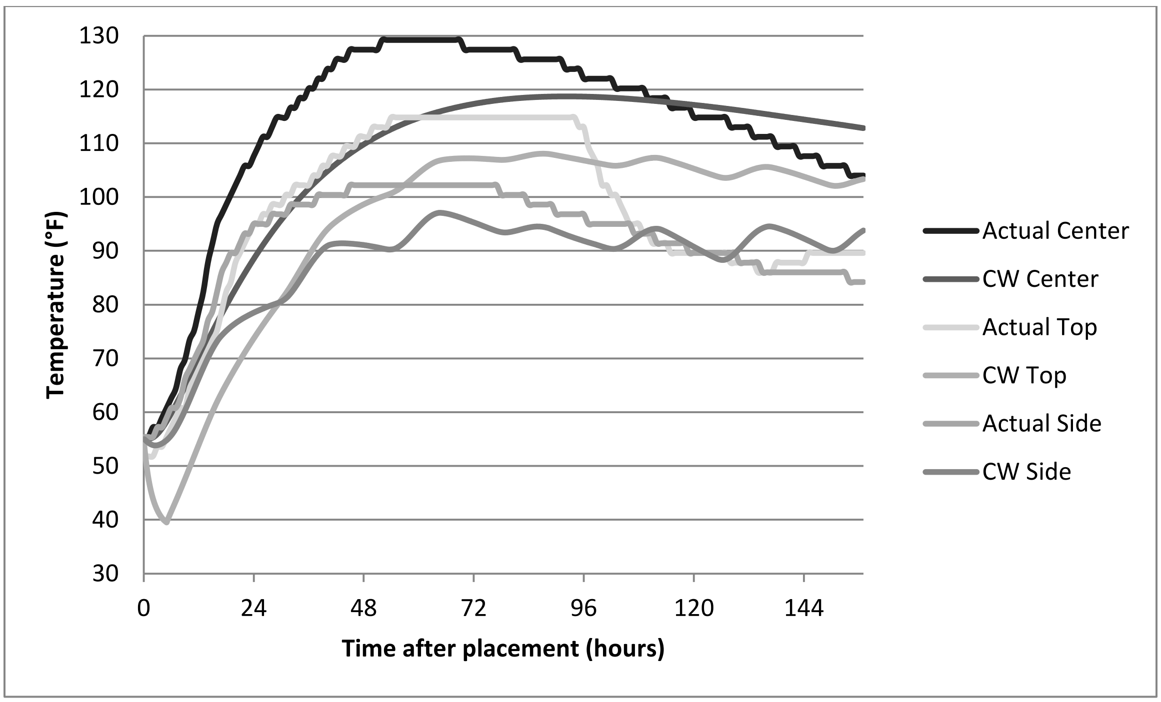


Figure D.2. US 34 case study thermal results – Pier 2 footing – B

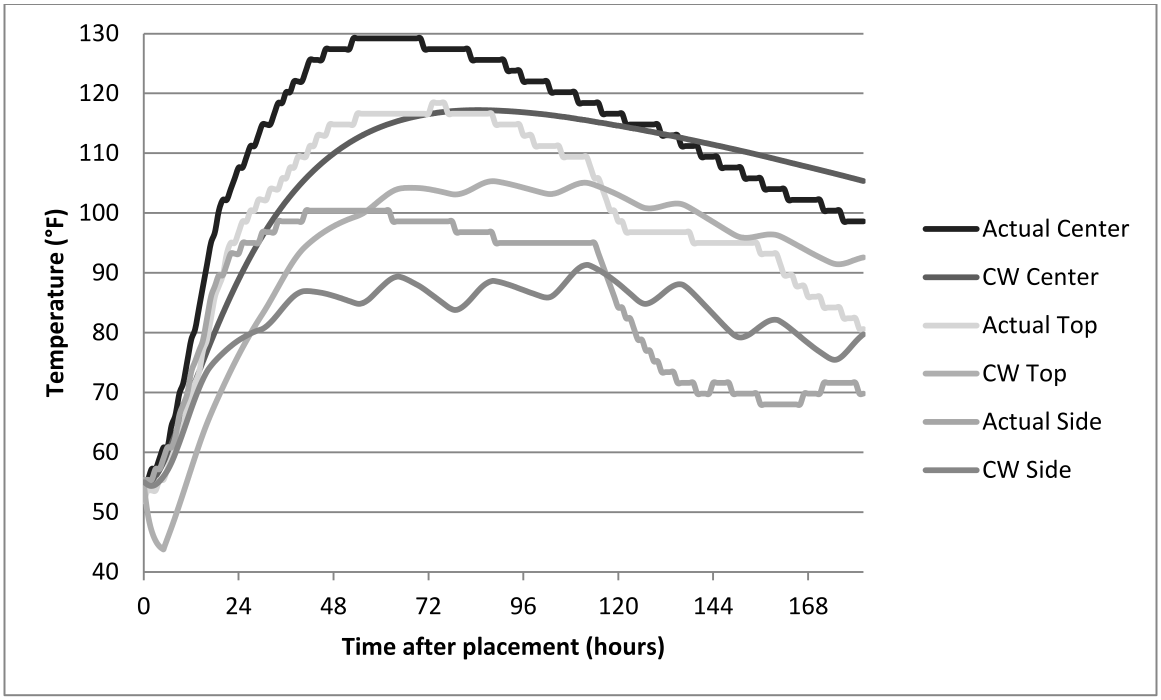


Figure D.3. US 34 case study thermal results – Pier 2 footing – C

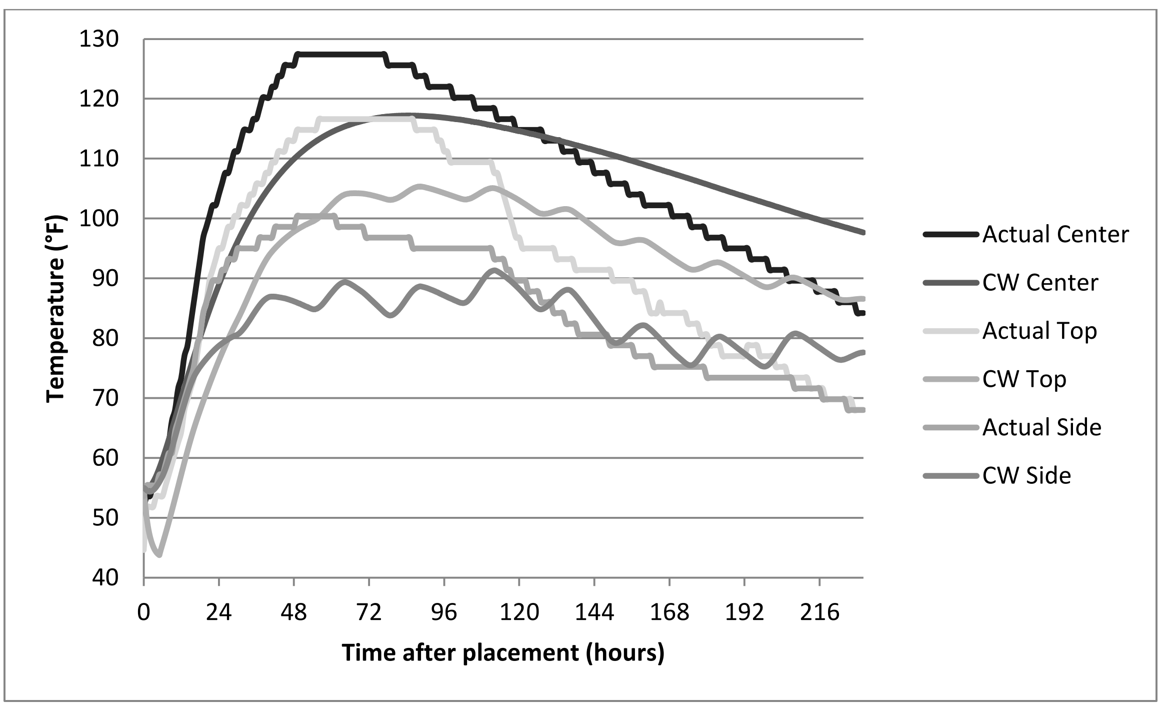


Figure D.4. US 34 case study thermal results – Pier 2 footing – D

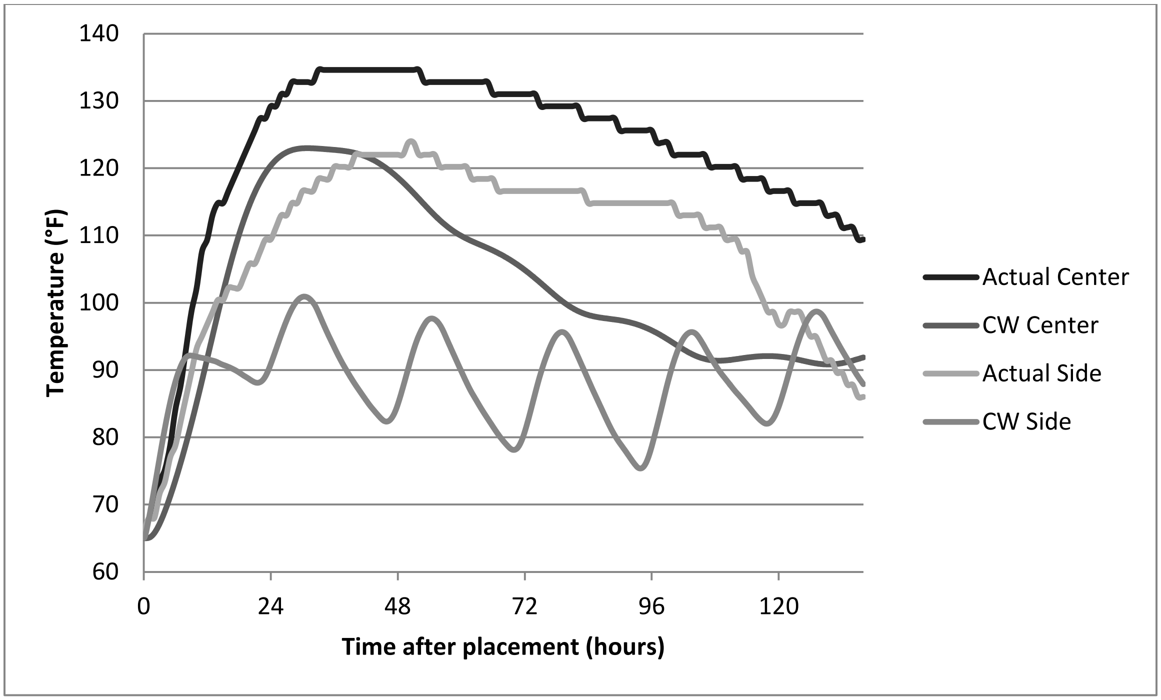


Figure D.5. US 34 case study thermal results – Pier 2 column – A

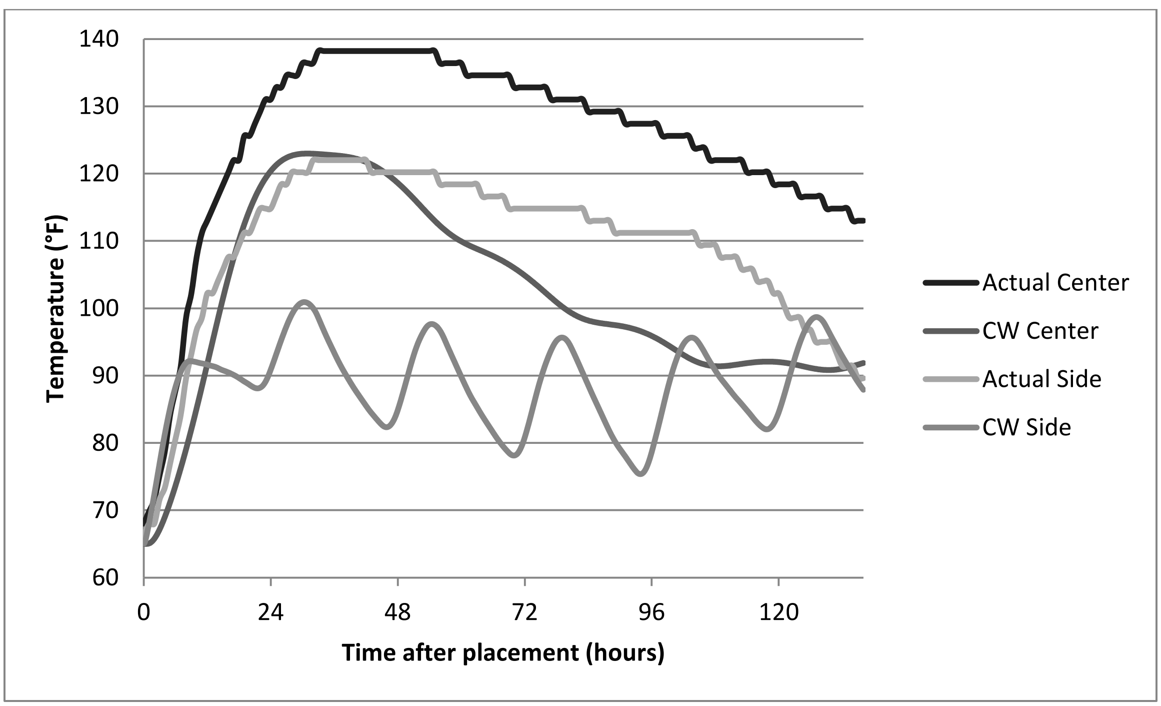


Figure D.6. US 34 case study thermal results – Pier 2 column – B

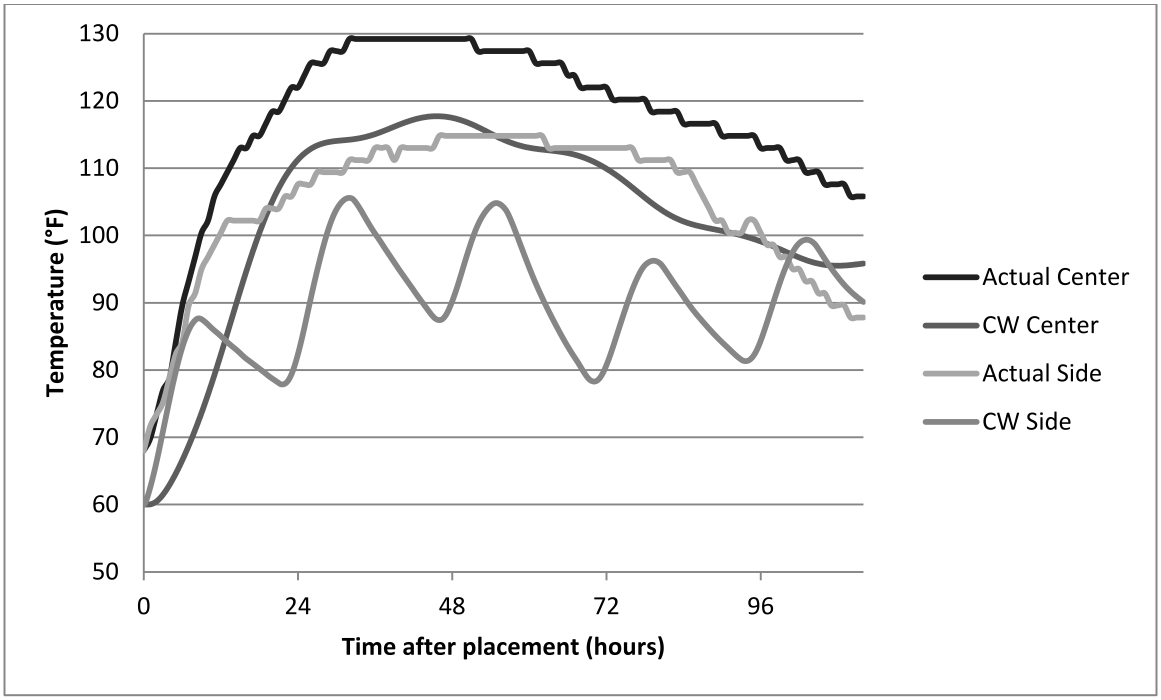


Figure D.7. US 34 case study thermal results – Pier 2 column – C

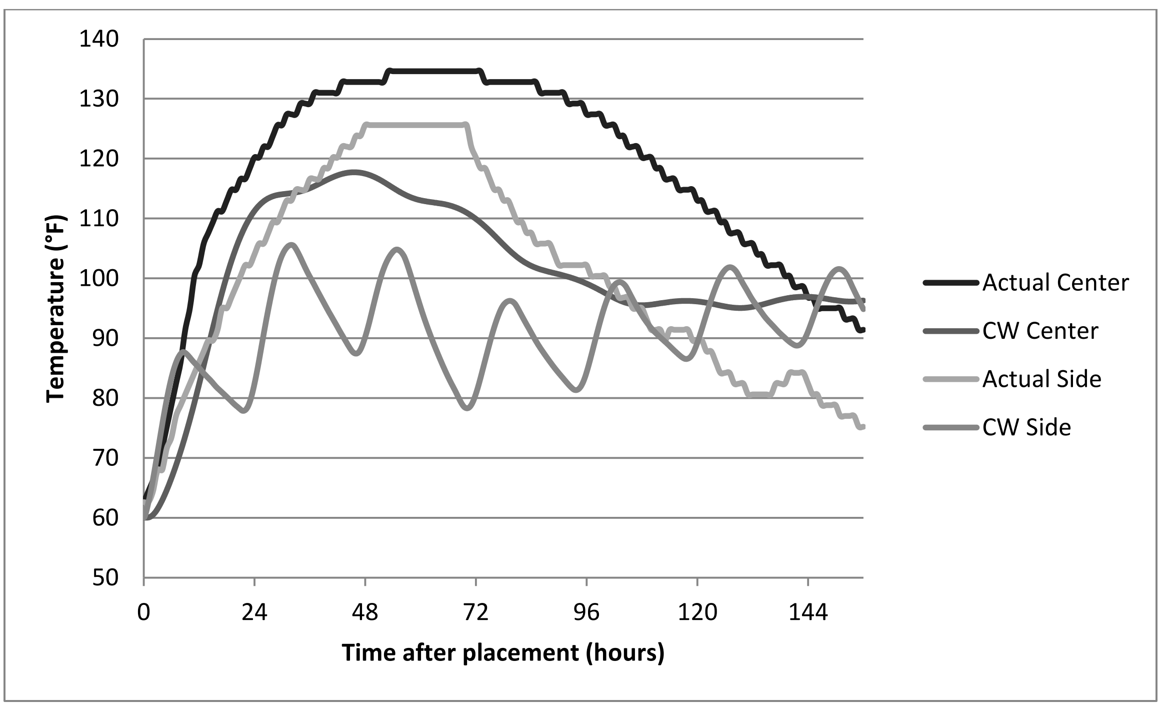


Figure D.8. US 34 case study thermal results – Pier 2 column – D

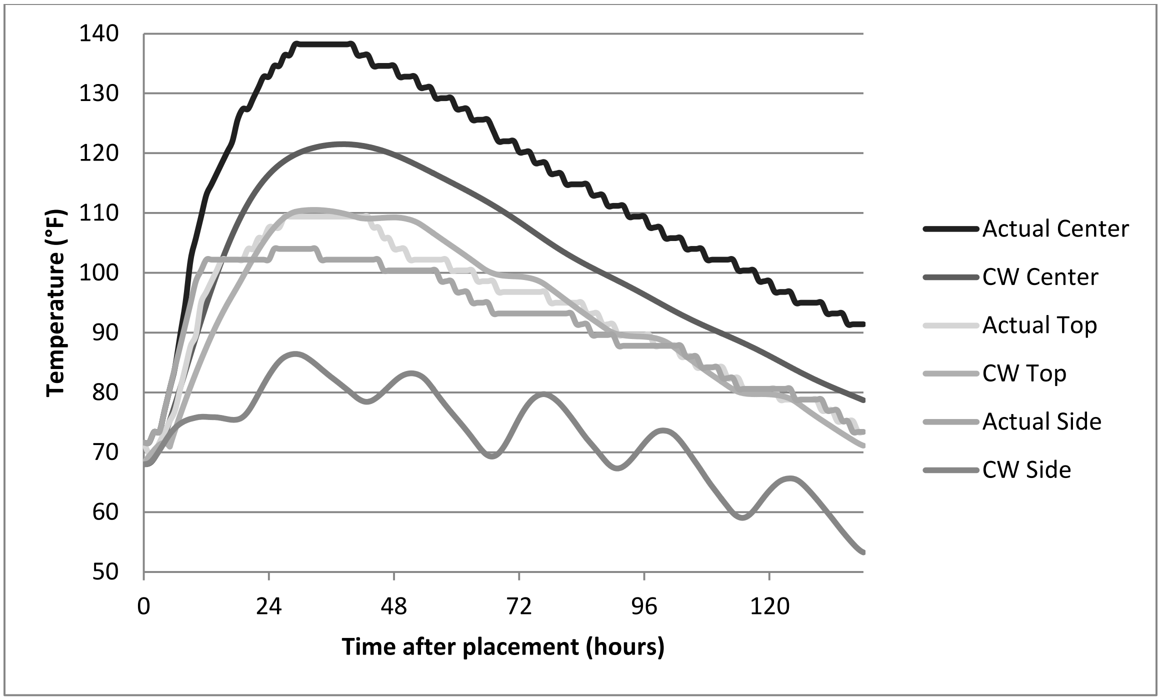


Figure D.9. US 34 case study thermal results – Pier 2 cap

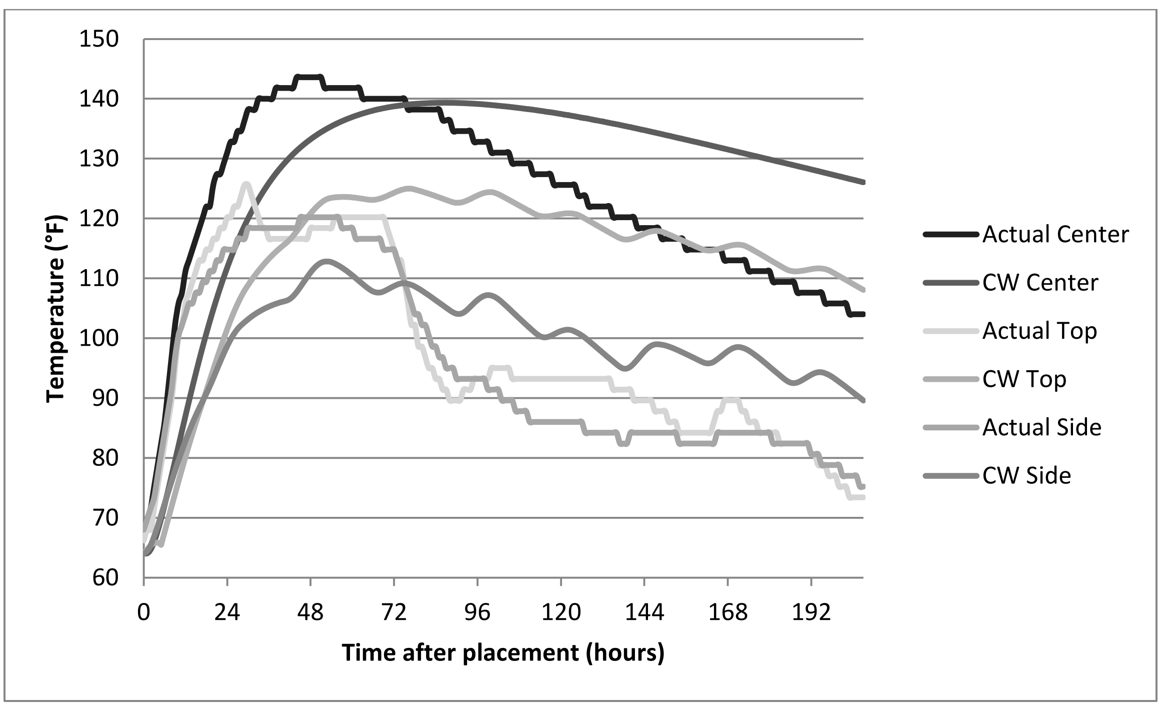


Figure D.10. US 34 case study thermal results – Pier 3 footing – C

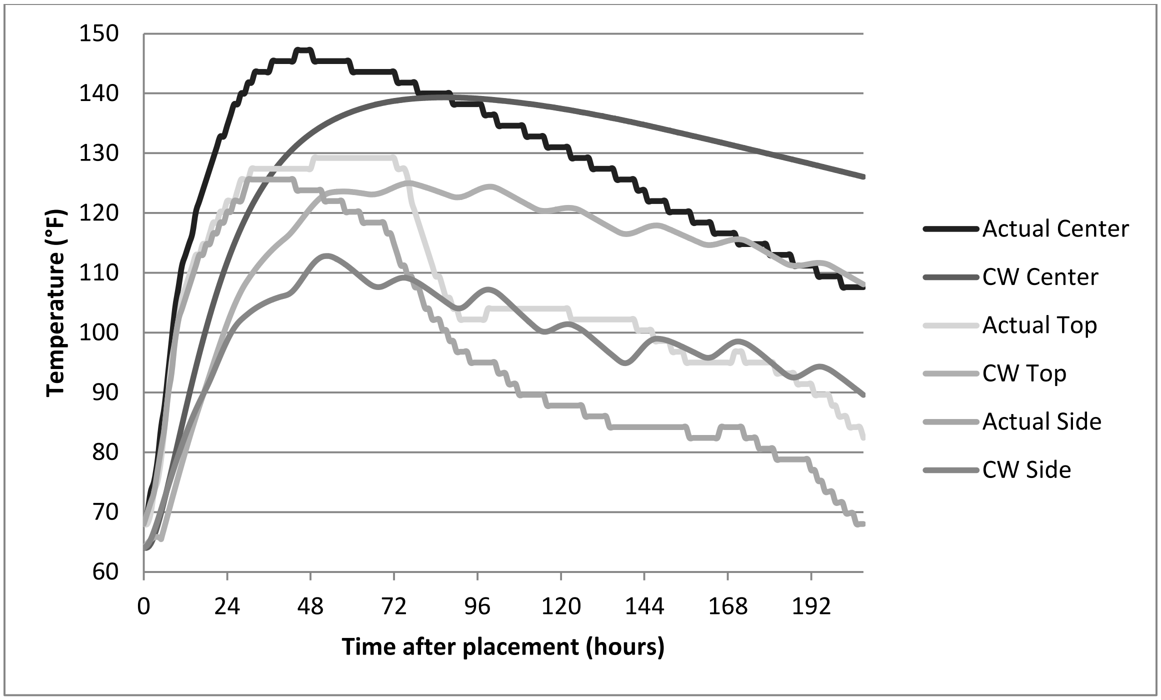


Figure D.11. US 34 case study thermal results – Pier 3 footing – D

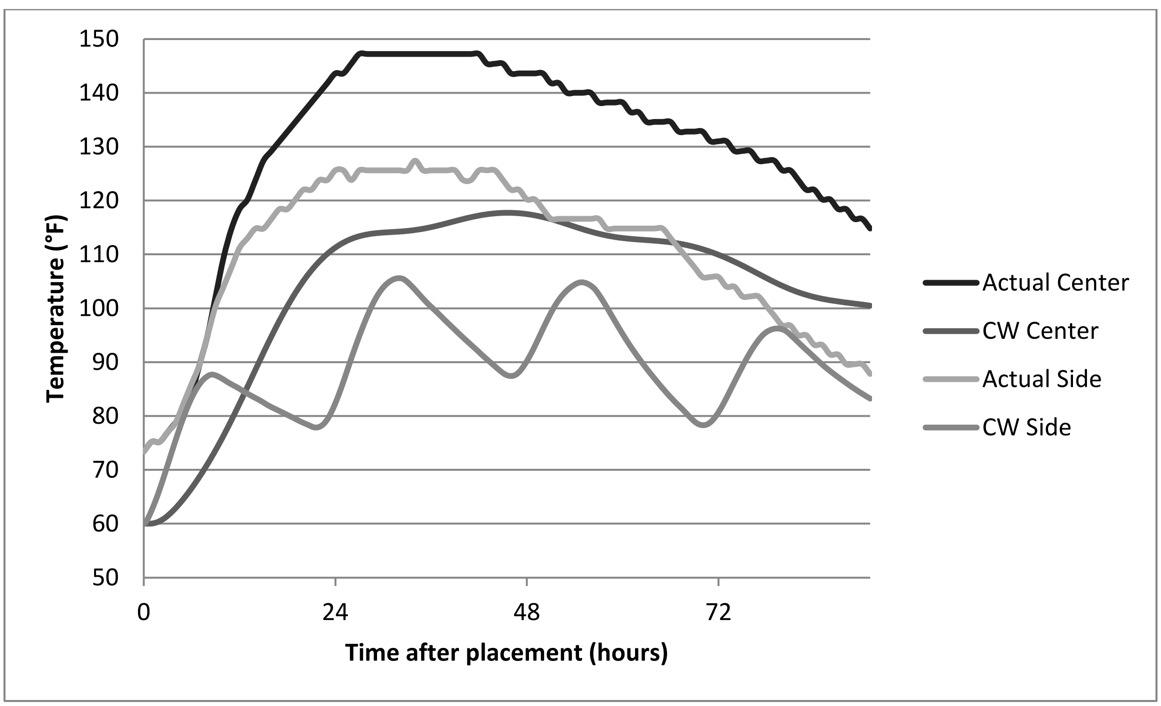


Figure D.12. US 34 case study thermal results – Pier 3 column – A

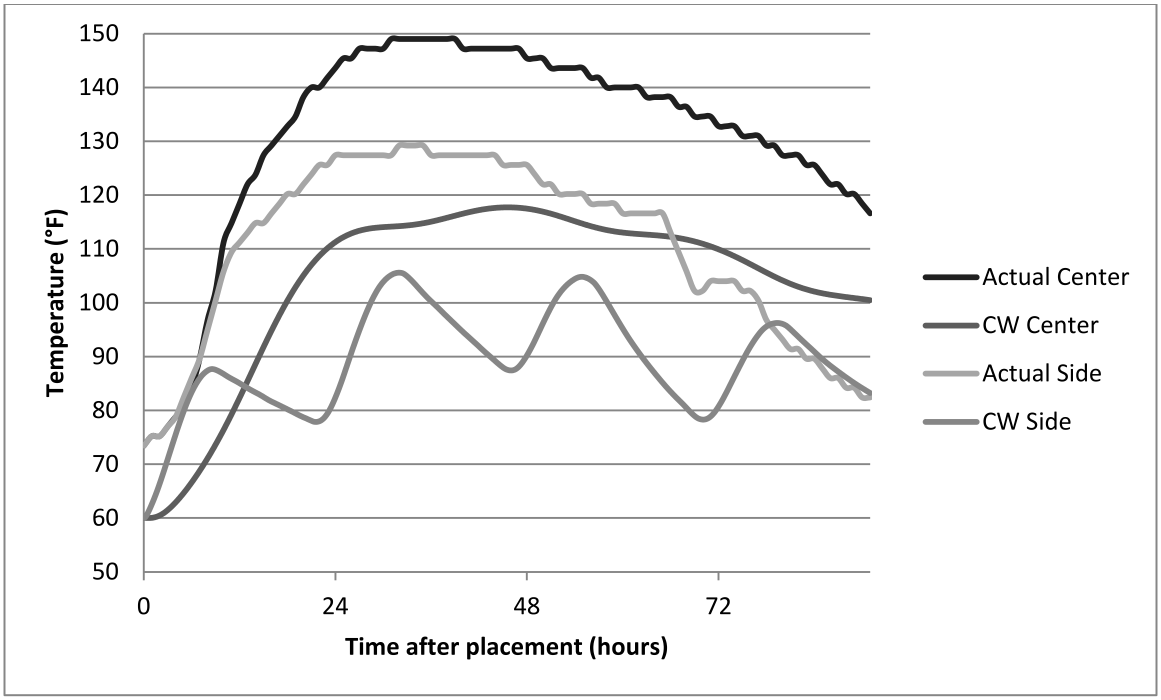


Figure D.13. US 34 case study thermal results – Pier 3 column – B

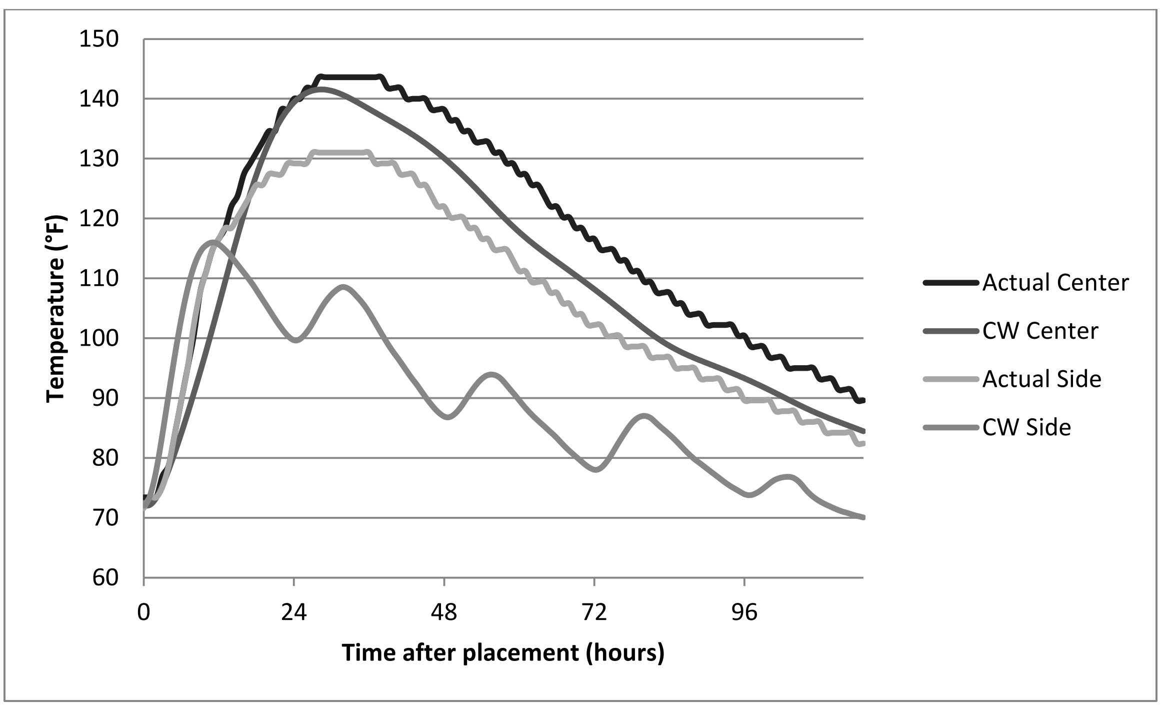


Figure D.14. US 34 case study thermal results – Pier 3 column – C

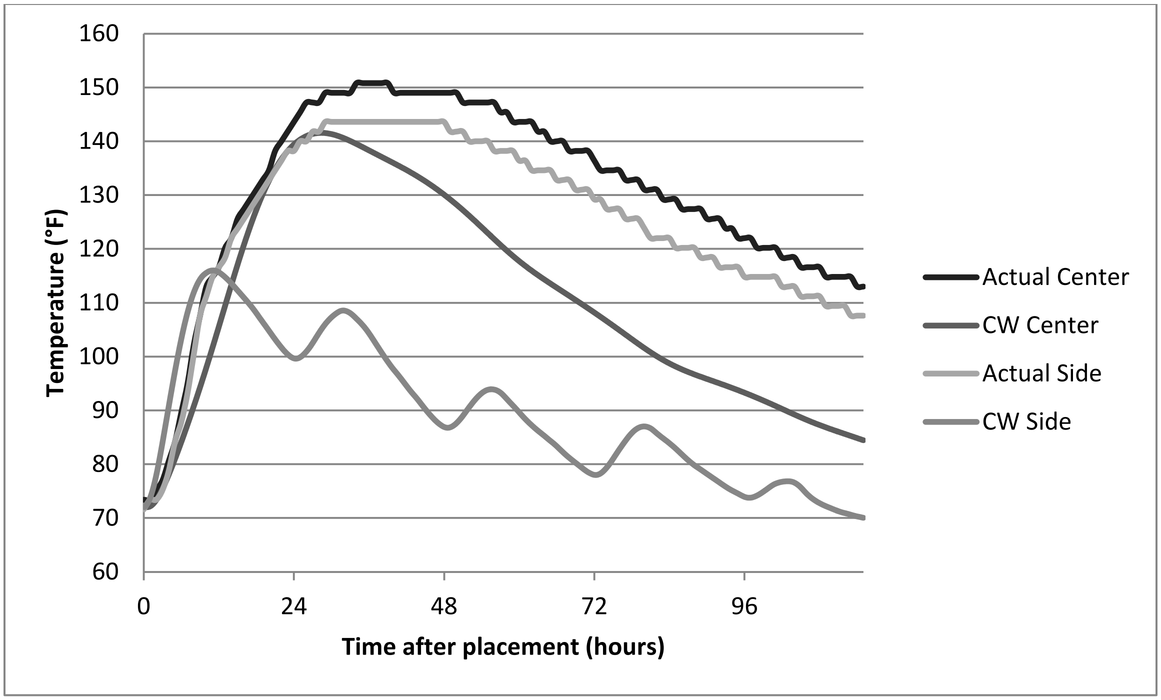


Figure D.15. US 34 case study thermal results – Pier 3 column – D