Development of Specifications for ALDOT Mass Concrete Construction

by

Andrew D. Eiland

A thesis submitted to the Graduate Faculty of
Auburn University
in partial fulfillment of the
requirements for the Degree of
Master of Science

Auburn, Alabama
December 10, 2016

Keywords: aggregate type, size designation, ConcreteWorks, temperature sensor instrumentation, maximum temperature limit, maximum temperature difference limit

Approved by

Anton K. Schindler, Chair, Professor and Director of Highway Research Center
Robert W. Barnes, Associate Professor of Civil Engineering
James S. Davidson, Professor of Civil Engineering

ABSTRACT

In mass concrete construction, concrete temperature requirements must be established to prevent delayed ettringite formation (DEF) and thermal cracking. This involves limiting the maximum allowable concrete temperature and maximum allowable concrete temperature difference. It is also necessary for a mass concrete specification to define an appropriate size designation for mass concrete members. These size and temperature specifications are investigated for their potential inclusion in a future ALDOT mass concrete specification.

Six ALDOT mass concrete members were instrumented with temperature sensors and examined for signs of DEF and thermal cracking. These six members were also modeled in ConcreteWorks to assess the accuracy of this software's temperature predictions. Based on all of the recorded temperature data, predicted temperature data, and site observations, guidelines were developed for a future ALDOT mass concrete specification.

Because concrete materials, specifically SCMs and low-CTE coarse aggregates, play a major role in the occurrence of thermal cracking, tiered specifications for mass concrete size designation and temperature difference limit were developed. Depending on the condition, it is recommended to use a least dimension of 4 to 6 ft for an element to be designated as mass concrete. If the concrete CTE or aggregate type is known, then an age-dependent temperature difference limit is recommended for the first 7 days after placement. The lowest tier (worst case) specifies a mass concrete least dimension designation as 4 feet and a temperature difference limit of 35 °F. The concept of raising the maximum concrete temperature limit of 158 °F was investigated, but it is recommended that a limit of 158 °F be used for ALDOT.

ACKNOWLEDGEMENTS

I would like to thank my advisor, Dr. Anton Schindler. Your wisdom and guidance have been invaluable throughout the course of my graduate studies at Auburn. Aside from your passion and knowledge of the concrete industry, your leadership has encouraged me in my own pursuit of a successful career. I am grateful for the opportunity to work under a man of incredible work ethic, integrity, and intellect during my time here at Auburn. Certainly, this research would not be what it is without your constant guidance and direction. I would also like to thank Dr. Robert Barnes and Dr. James Davidson. Your willingness to serve on my thesis committee is much appreciated. Without your time and effort, the completion of thesis would be possible.

Over the course of my research, a number of other graduate students have assisted me, most notably Eric Gross. It has been a pleasure working with you and sharing an office with you during this past year. I wish you the best as you finish the work we started. I would also like to thank Mohammed Ali. Sharing an office with you was a great pleasure, and I am very grateful for your encouragement as a friend.

I would also like to thank my parents, Al and Sallie Eiland, for their constant support. Without you, I would certainly not be who I am today. Wherever I go, knowing that your love and support go with me is a gift I could never deserve. I would also like to thank the lovely Morgan Wesson, whose daily encouragement drives me to excellence.

I give all glory and honor to God. I acknowledge that all that I have, even down to the breath in my lungs, is a gift from you. I dedicate my work and my life to you in response to the goodness and love you have shown to me.

Table of Contents

Abstract ii

Acknowledgements iii

List of Tables xii

List of Figures xvi

Chapter 1: Introduction 1

1.1 Background 1

1.2 Project Objectives 3

1.3 Research Approach 4

1.4 Organization of Thesis 5

Chapter 2: Literature Review 6

2.1 Introduction 6

2.2 Thermal Cracking 6

2.2.1 Thermal Stress Development 6

2.2.2 Modeling Thermal Stress Development of Concrete Elements at Early Ages 12

2.2.2.1 Modeling Strength and Stiffness of Concrete at Early Ages 12

2.2.2.2 Modeling of Creep Effects 13

2.2.3 Causes and Effects of Thermal Cracking ... 15

2.2.4 Mitigation of Thermal Cracking ... 16

2.2.4.1 Temperature Difference Limits ... 16

2.2.4.2 Formwork Removal ... 17

2.2.4.3 Coarse Aggregate Selection ... 19

2.2.4.4 Pour Dimensions and Element Geometry ... 19

2.2.5 Examples of Thermal Cracking in Mass Concrete Elements ... 20

2.3 Delayed Ettringite Formation ... 21

2.3.1 Introduction ... 21

2.3.2 Causes and Effects of DEF ... 25

2.3.3 Preventative Measures for Controlling DEF in New Concrete Construction ... 26

2.3.3.1 Maximum Concrete Temperature Limits ... 26

2.4 Temperature Control Strategies ... 31

2.4.1 Introduction ... 31

2.4.2 Methods for Controlling Maximum Concrete Temperatures ... 31

2.4.2.1 Pre-Cooling Techniques ... 31

2.4.2.2 Post-Cooling Techniques ... 32

2.4.3 Concrete Temperature Monitoring Methods ... 34

2.5 Thermal Control Plan ... 35

2.6 ConcreteWorks ... 36

2.6.1 ConcreteWorks Background Information ... 36

Chapter 3: Review of Mass Concrete Specifications ... 41

3.1 Investigation of Current Mass Concrete Specifications ... 41

3.1.1 ACI 301 ... 41

3.1.2 AASHTO and FHWA ... 42

3.1.3 U.S. States ... 42

Chapter 4: Experimental Plan ... 53

4.1 Introduction ... 53

4.2 Field Instrumentation ... 53

4.3 ConcreteWorks Analysis ... 55

4.3.1 Overview of ConcreteWorks Analysis ... 55

4.3.2 Important Mass Concrete Variables ... 56

4.4 Numerical Modeling ... 56

4.4.1 Introduction ... 56

4.5 Temperature Sensors ... 59

4.5.1 Assembling Temperature Sensors ... 59

4.5.2 Programming and Installing Temperature Sensors ... 60

4.6 Laboratory Testing of Site Materials ... 62

4.6.1 Raw Materials ... 62

4.6.2 Mixing Procedure ... 63

4.6.2.1 Semi-Adiabatic Calorimetry 63
4.6.2.2 Compressive Strength and Modulus of Elasticity Test 64
4.6.2.3 CTE Test 64
Chapter 5: Presentation and Discussion of Instrumentation Data 66
5.1 Introduction 66
5.2 Field Instrumentation Data 66
5.2.1 Field Instrumentation Overview 66
5.2.2 Albertville Bent Cap 68
5.2.2.1 Site Details 68
5.2.2.2 Sensor Layout 69
5.2.2.3 14-Day Temperature Data 70
5.2.2.4 Long-Term Temperature Data 71
5.2.2.5 Post-Instrumentation Site Observations 71
5.2.2.6 Laboratory Testing Results 71
5.2.3 Harpersville Crashwall 72
5.2.3.1 Site Details 72
5.2.3.2 Sensor Layout 73
5.2.3.3 14-Day Temperature Data 74
5.2.3.4 Long-Term Temperature Data 75
5.2.3.5 Post-Instrumentation Site Observations 75

5.2.3.6 Laboratory Testing Results 75

5.2.4 Scottsboro Pedestal 76

5.2.4.1 Site Details 76

5.2.4.2 Sensor Layout 77

5.2.4.3 14-Day Temperature Data 77

5.2.4.4 Long-Term Temperature Data 79

5.2.4.5 Post-Instrumentation Site Observations 79

5.2.4.6 Laboratory Testing Results 79

5.2.5 Scottsboro Bent Cap 80

5.2.5.1 Site Details 80

5.2.5.2 Sensor Layout 81

5.2.5.3 14-Day Temperature Data 82

5.2.5.4 Long-Term Temperature Data 83

5.2.5.5 Post-Instrumentation Site Observations 83

5.2.5.6 Laboratory Testing Results 83

5.2.6 Elba Bent Cap 84

5.2.6.1 Site Details 84

5.2.6.2 Sensor Layout 85

5.2.6.3 14-Day Temperature Data 86

5.2.6.4 Long-Term Temperature Data 87

5.2.6.5 Post-Instrumentation Site Observations 87

5.2.6.6 Laboratory Testing Results.. 87

5.2.7 Birmingham Column 88

5.2.7.1 Site Details... 88

5.2.7.2 Sensor Layout 89

5.2.7.3 14-Day Temperature Data 90

5.2.7.4 Long-Term Temperature Data 91

5.2.7.5 Post-Instrumentation Site Observations 91

5.2.7.6 Laboratory Testing Results.. 92

5.3 Summary and Discussion of Temperature Data.. 92

Chapter 6: Results and Discussion of ConcreteWorks Analysis 94

6.1 Assessment of the Accuracy of ConcreteWorks Predictions 94

6.1.1 ConcreteWorks 7-Day Concrete Temperature Predictions.................................... 94

6.1.2 Accuracy of ConcreteWorks Predictions.. 102

6.2 Analysis with ConcreteWorks... 103

6.2.1 Statistical Analysis of Results.. 103

6.2.2 Summary of ConcreteWorks Analysis.. 106

6.2.2.1 Maximum Concrete Temperatures 107

6.2.2.2 Maximum Concrete Temperature Difference... 108

6.2.2.3 Maximum Concrete Cracking Risk 110

6.2.2.4 Temperature Difference versus Cracking Risk 111

6.2.2.5 Mass Concrete Size Designation 114

Chapter 7: Development of an ALDOT Mass Concrete Specification...................................... 116

7.1 Introduction 116

7.2 Mass Concrete Size Designation 116

7.3 Maximum Concrete Temperature Limit 118

7.4 Maximum Concrete Temperature Difference Limit 118

7.4.1 Thermal Cracking Analysis to Develop an Age-Dependent Maximum Temperature Difference Limit 118

7.4.2 Development of Maximum Concrete Temperature Difference Limit Using the Bamforth Creep Coefficient 126

7.4.3 Application of the Tiered Maximum Temperature Difference Limits on Instrumented ALDOT Mass Concrete Elements 130

7.5 Temperature Monitoring Recommendations 135

7.5.1 Duration of Temperature Monitoring 135

7.5.2 Placement Location of Temperature Monitoring Sensors 135

Chapter 8: Summary, Conclusions, and Recommendations 142

8.1 Summary of Project 142

8.2 Research Conclusions 143

8.3 Research Recommendations 147

References 149

Appendix A: Albertville Bent Cap 156

Appendix B: Harpersville Crashwall 159

Appendix C: Scottsboro Pedestal 162

Appendix D: Scottsboro Bent Cap 165

Appendix E: Elba Bent Cap 168

Appendix F: Birmingham Column 171

Appendix G: ConcreteWorks Analysis 174

LIST OF TABLES

Table 2-1: Limiting Temperature Differences Based on Assumed Values of CTE and Tensile Strain Capacity (adapted from Bamforth and Price, 1995) ($\Delta°F = 1.8 \times \Delta°C$) 11

Table 2-2: TxDOT Research Study Temperature Limit Recommendations (Folliard et al., 2006) 29

Table 2-3: Proposed Maximum in-Place Temperature Limit Specification in Mass and Precast Concrete Construction (Folliard et al. 2008) 30

Table 2-4: ConcreteWorks Input Categories 37

Table 2-5: ConcreteWorks Mass Concrete Outputs (adapted from Concrete Durability Center, 2005) 38

Table 3-1: ACI 301 (2016) Mass Concrete Requirements 41

Table 3-2: FWHA FP-16 Specifications... 42

Table 3-3: U.S DOTs with Mass Concrete Specifications 43

Table 3-4 – Mass Concrete Specification Reference Document 44

Table 3-5: Definition of Mass Concrete across U.S. States.. 45

Table 3-6: State DOT Cementitious Material and Chemical Admixture Requirements 46

Table 3-7: State DOT Temperature Restrictions 47

Table 3-8: State DOT Temperature Monitoring Requirements.. 50

Table 3-9: State DOT In-Place Concrete Temperature Control Requirements 51

Table 3-10: State DOT Thermal Control Plan.. 51

Table 4-1: Size, Type, and Time of Year of Each Instrumented Element................................... 54

Table 5-1: Albertville Bent Cap Site Information 68

Table 5-2: Albertville Bent Cap Laboratory Testing Results 71

Table 5-3: Harpersville Crashwall Site Information.. 72

Table 5-4: Harpersville Crashwall Laboratory Testing Results 75

Table 5-5: Scottsboro Pedestal Site Information 76

Table 5-6: Scottsboro Pedestal Laboratory Testing Results 79

Table 5-7: Scottsboro Bent Cap Site Information... 80

Table 5-8: Scottsboro Bent Cap Laboratory Testing Results 83

Table 5-9: Elba Bent Cap Site Information 84

Table 5-10: Elba Bent Cap Laboratory Testing Results 87

Table 5-11: Birmingham Column Site Information.. 88

Table 5-12: Birmingham Column Laboratory Testing Results 92

Table 5-13: Summary of Maximum Temperature Values from Instrumentation Data 92

Table 6-1: Concrete Age of Instrumented Elements at the Time of Formwork Removal............ 95

Table 6-2: Acceptable Amount of Error for ConcreteWorks Temperature Predictions............. 102

Table 6-3: Measured Maximum Temperature Values versus ConcreteWorks Predictions........ 103

Table 6-4: P-Values of Different Input Variables Based on ANOVA Test 105

Table 6-5: Rankings of Statistical Impact of Input Variables Based on ANOVA Test 106

Table 6-6: ConcreteWorks Analysis of an Appropriate Mass Concrete Size Designation 115

Table 7-1: Summary of Input Values for B3 Model Compliance Calculations 122

Table 7-2: Calculation of Actual Concrete Compressive Strength (ACI 214, 2011) 127

Table 7-3: Allowable Maximum Temperature Differences with River Gravel Concrete 128

Table 7-4: Default CTE Values for Concretes Made with Various Coarse Aggregate Types (Schindler et al., 2010) 130

Table 7-5: Tiered Maximum Temperature Difference Limit for a Mass Concrete Specification 130

Table 7-6: CTE Values for the Instrumented ALDOT Mass Concrete Elements 131

Table 8-1: Allowable Maximum Temperature Differences with River Gravel Concrete 145

Table 8-2: Default CTE Values for Concretes Made with Various Coarse Aggregate Types (Schindler et al., 2010) 146

Table 8-3: Tiered Maximum Temperature Difference Limit for a Mass Concrete Specification 147

Table A-1: Concrete Mixture Proportions – Albertville Bent Cap 156

Table A-2: Cement Composition – Albertville Bent Cap 156

Table A-3: Fly Ash Composition – Albertville Bent Cap 157

Table A-4: 7-Day Weather Data – Albertville Bent Cap 157

Table A-5: ALDOT Test Results of Project Concrete Specimens – Albertville Bent Cap 157

Table B-1: Concrete Mixture Proportions – Harpersville Crashwall 159

Table B-2: Cement Composition – Harpersville Crashwall 159

Table B-3: Fly Ash Composition – Harpersville Crashwall 160

Table B-4: 7-Day Weather Data – Harpersville Crashwall 160

Table B-5: ALDOT Test Results of Project Concrete Specimens – Harpersville Crashwall 160

Table C-1: Concrete Mixture Proportions – Scottsboro Pedestal 162

Table C-2: Cement Composition – Scottsboro Pedestal 162

Table C-3: Fly Ash Composition – Scottsboro Pedestal 163

Table C-4: 7-Day Weather Data – Scottsboro Pedestal 163

Table C-5: ALDOT Test Results of Project Concrete Specimens – Scottsboro Pedestal 163

Table D-1: Concrete Mixture Proportions – Scottsboro Bent Cap 165

Table D-2: Cement Composition – Scottsboro Bent Cap 165

Table D-3: Fly Ash Composition – Scottsboro Bent Cap 166

Table D-4: 7-Day Weather Data – Scottsboro Bent Cap 166

Table D-5: ALDOT Test Results of Project Concrete Specimens – Scottsboro Bent Cap 166

Table E-1: Concrete Mixture Proportions – Elba Bent Cap 168

Table E-2: Cement Composition – Elba Bent Cap 168

Table E-3: Fly Ash Composition – Elba Bent Cap 169

Table E-4: 7-Day Weather Data – Elba Bent Cap 169

Table E-5: ALDOT Test Results of Project Concrete Specimens – Elba Bent Cap 169

Table F-1: Concrete Mixture Proportions – Birmingham Column 171

Table F-2: Cement Composition – Birmingham Column 171

Table F-3: Fly Ash Composition – Birmingham Column 172

Table F-4: 7-Day Weather Data – Birmingham Column 172

Table F-5: ALDOT Test Results of Project Concrete Specimens – Birmingham Column 172

Table G-1: ConcreteWorks Analysis Output Data 174

LIST OF FIGURES

Figure 1-1: Thermal Cracking of a Bridge Column in Texas (Photo Courtesy of Dr. J.C. Liu) 2

Figure 2-1: Development of Cracks in a Massive Element due to Temperature Differences Assuming Only Internal Restraint (Bamforth, 2007) 7

Figure 2-2: Effect of Internal Restraint on the Development Location of Thermal Cracks (Tankasala et al., 2017) 8

Figure 2-3: Restraint of a Concrete Member by Adjacent Elements (Bamforth, 2007) 8

Figure 2-4: Effect of Aggregate Type on Concrete Coefficient of Thermal Expansion 10

Figure 2-5: Mechanism of Thermal Cracking (Schindler and McCullough, 2002) 16

Figure 2-6: Effect of Formwork Removal Time on Cracking Risk (Gajda and Alsamsam, 2006) ($°F = 1.8 \times °C + 32$) 18

Figure 2-7: Concrete Core from Footing Where Insulation Blew Off During Cold Weather (Gajda and Alsamsam, 2006) 18

Figure 2-8: Thermal Cracking of a Bridge Column in Texas (Photo Courtesy of Dr. J.C. Liu) .. 20

Figure 2-9: Suggested DEF Expansion Mechanism (Taylor et al., 2001) 21

Figure 2-10: Plan View Schematic of Cracks in a Seal Footing in Georgia (McCall, 2013) 23

Figure 2-11: Cracking Due to DEF at San Antonio Y Overpass (Thomas et al., 2008) 24

Figure 2-12: Ettringite Formation in Cracks around Coarse Aggregate in Cores Removed from San Antonio Y Overpass (adapted from Thomas et al., 2008) 25

Figure 2-13: Effect of Fly Ash (top) and Slag Cement (bottom) on Heat of Hydration (Schindler and Folliard, 2003) ($°F = 1.8 \times °C + 32$) 27

Figure 2-14: Expansion Test Results for Mortar Bars with Plain Portland Cement in Combination with Various SCMs Cured at 203 °F (95 °C) (adapted from Folliard et al., 2006) 28
Figure 2-15: Reduced Expansion in Class F Fly Ash versus Class C Fly Ash Concrete Specimens (Folliard et al., 2006) 29
Figure 2-16: Effect of Cooling Pipes (adapted from Kim et al., 2000) ($°F = 1.8 \times °C + 32$) 34
Figure 2-17: Output Summary (top) and Cracking Risk Profile (bottom) of Concrete Pedestal . 39
Figure 3-1: Mass Concrete Designation across U.S. DOTs.. 45
Figure 3-2: Allowable Temperature Differences across U.S. DOTs 48
Figure 3-3: Maximum Temperature Limits across U.S. DOTs 49
Figure 3-4: U.S. DOTs Requiring a TCP for Mass Concrete Construction 52
Figure 4-1: Locations of Instrumented Elements... 55
Figure 4-2: iButton Assembly... 60
Figure 4-3: Rebar and Edge Sensors in Harpersville Crashwall.. 62
Figure 4-4: Q-Drum Testing Apparatus.. 64
Figure 4-5: CTE Testing Apparatus.. 65
Figure 5-1: Continuous Restraint along Bottom Edge of Crashwall and Pedestal 67
Figure 5-2: Photograph of Albertville Bent Cap... 68
Figure 5-3: Albertville Bent Cap Elevation View (top) and Cross Section (bottom).................. 69
Figure 5-4: Albertville Bent Cap 14-Day Concrete Temperature Data 70
Figure 5-5: Albertville Bent Cap 14-Day Concrete Temperature Difference Data 70
Figure 5-6: Albertville Bent Cap Long-Term Concrete Temperature Data 71
Figure 5-7: Photograph of Harpersville Crashwall 72
Figure 5-8: Harpersville Crashwall Elevation View (top) and Cross Section (bottom) 73

Figure 5-9: Harpersville Crashwall 14-Day Concrete Temperature Data 74

Figure 5-10: Harpersville Crashwall 14-Day Concrete Temperature Difference Data 74

Figure 5-11: Harpersville Crashwall Long-Term Concrete Temperature Data 75

Figure 5-12: Photograph of Scottsboro Pedestal 76

Figure 5-13: Scottsboro Pedestal Elevation View (top) and Cross Section (bottom)................... 77

Figure 5-14: Scottsboro Pedestal 14-Day Concrete Temperature Data 78

Figure 5-15: Scottsboro Pedestal 14-Day Concrete Temperature Difference Data 78

Figure 5-16: Photograph of Scottsboro Bent Cap 80

Figure 5-17: Scottsboro Bent Cap Elevation View (top) and Cross Section (bottom) 81

Figure 5-18: Scottsboro Bent Cap 14-Day Concrete Temperature Data 82

Figure 5-19: Scottsboro Bent Cap 14-Day Concrete Temperature Difference Data 82

Figure 5-20: Photograph of Elba Bent Cap 84

Figure 5-21: Elba Bent Cap Elevation View (top) and Cross Section (bottom) 85

Figure 5-22: Elba Bent Cap 14-Day Concrete Temperature Data 86

Figure 5-23: Elba Bent Cap 14-Day Concrete Temperature Difference Data 86

Figure 5-24: Elba Bent Cap Long-Term Concrete Temperature Data 87

Figure 5-25: Photograph of Birmingham Column 88

Figure 5-26: Birmingham Column Elevation View (top) and Cross Section (bottom) 89

Figure 5-27: Birmingham Column 14-Day Concrete Temperature Data 90

Figure 5-28: Birmingham Column 14-Day Concrete Temperature Difference Data 90

Figure 5-29: Birmingham Column Long-Term Concrete Temperature Data 91

Figure 6-1: Albertville Bent Cap – ConcreteWorks Temperature Predictions versus Measured Concrete Temperatures 96

Figure 6-2: Harpersville Crashwall – ConcreteWorks Temperature Predictions versus Measured Concrete Temperatures 97

Figure 6-3: Scottsboro Pedestal - ConcreteWorks Temperature Predictions versus Measured Concrete Temperatures 98

Figure 6-4: Scottsboro Bent Cap - ConcreteWorks Temperature Predictions versus Measured Concrete Temperatures 99

Figure 6-5: Elba Bent Cap - ConcreteWorks Temperature Predictions versus Measured Concrete Temperatures 100

Figure 6-6: Birmingham Column - ConcreteWorks Temperature Predictions versus Measured Concrete Temperatures 101

Figure 6-7: Effect of SCM Type and Placement Date on Maximum Concrete Temperature (Mobile, AL with limestone aggregate) 107

Figure 6-8: Effect of Element Least Dimension on Maximum Concrete Temperature (100% PC in July) 108

Figure 6-9: Effect of SCM Type and Element Least Dimension on Maximum Concrete Temperature Difference (Mobile, AL on July 15) 109

Figure 6-10: Effect of Placement Date, Element Size, and Aggregate Type on Maximum Concrete Cracking Risk (100% PC in Mobile, AL) 110

Figure 6-11: Effect of Placement Date, Element Size, and SCM Type on Maximum Concrete Cracking Risk (River Gravel in Mobile, AL) 111

Figure 6-12: Cracking Risk versus Maximum Temperature Difference (7-ft least cross sectional dimension) 113

Figure 7-1: Mass Concrete Size Designation Flow Chart 117

Figure 7-2: Compliance Values – B3 versus Modified B3 123

Figure 7-3: Creep Factors – Bamforth versus B3 versus Modified B3 124

Figure 7-4: 7-day Maximum Allowable Temperature Difference Limits – Limestone 125

Figure 7-5: 7-day Maximum Allowable Temperature Difference Limits – River Gravel 126

Figure 7-6: Allowable Temperature Difference Limits – Limestone and River Gravel............. 127

Figure 7-7: Albertville Bent Cap Temperature Difference Data versus Potential Limits 131

Figure 7-8: Harpersville Crashwall Temperature Difference Data versus Potential Limits....... 132

Figure 7-9: Scottsboro Pedestal Temperature Difference Data versus Potential Limits 132

Figure 7-10: Scottsboro Bent Cap Temperature Difference Data versus Potential Limits......... 133

Figure 7-11: Elba Bent Cap Temperature Difference Data versus Potential Limits 133

Figure 7-12: Birmingham Column Temperature Difference Data versus Potential Limits........ 134

Figure 7-13: Temperature Difference between Edge and Rebar Sensors Measured for the Albertville Bent Cap, Scottsboro Bent Cap, and Elba Bent Cap 137

Figure 7-14: Temperature Difference between Edge and Rebar Sensors Measured for the Harpersville Crashwall, Scottsboro Pedestal, and Birmingham Column 138

Figure 7-15: Recommended Location of Sensor Cross Sections in a Rectangular Bent Cap 139

Figure 7-16: Recommended Sensor Locations in a Horizontal Rectangular Concrete Element (e.g. Bent Cap) 140

Figure 7-17: Recommended Sensor Locations in a Vertical Rectangular Concrete Element (e.g. Square Column) 140

Figure 7-18: Recommended Sensor Locations in a Vertical Circular Concrete Element (e.g. Circular Pier)... 141

Figure A-1: Cross-Section #1 Temperature Data – Albertville Bent Cap 158

Figure A-2: Cross-Section #2 Temperature Data – Albertville Bent Cap 158

Figure B-1: Cross-Section #1 Temperature Data – Harpersville Crashwall.............................. 161

Figure B-2: Cross-Section #2 Temperature Data – Harpersville Crashwall.............................. 161

Figure C-1: Cross-Section #1 Temperature Data – Scottsboro Pedestal 164

Figure C-2: Cross-Section #2 Temperature Data – Scottsboro Pedestal 164

Figure D-1: Cross-Section #1 Temperature Data – Scotttsboro Bent Cap 167

Figure D-2: Cross-Section #2 Temperature Data – Scottsboro Bent Cap 167

Figure E-1: Cross-Section #1 Temperature Data – Elba Bent Cap 170

Figure E-2: Cross-Section #2 Temperature Data – Elba Bent Cap 170

Figure F-1: Cross-Section #1 Temperature Data – Birmingham Column................................. 173

Figure F-2: Cross-Section #2 Temperature Data – Birmingham Column................................. 173

ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
CFA	Class C fly ash
CIP	Cast-in-place
CTE	Coefficient of thermal expansion
DEF	Delayed ettringite formation
DOT	Department of Transportation
FFA	Class F fly ash
FHWA	Federal Highway Administration
LS	Limestone
MH	Moderate heat
NOAA	National Oceanic and Atmospheric Administration
PC	Portland cement
PCC	Portland cement concrete
pcy	Pounds per cubic yard
RG	River gravel
SL	Slag
TCP	Thermal control plan
U.S.	United States

CHAPTER 1: INTRODUCTION

1.1 Background

Mass concrete is defined by ACI 207 as "any volume of concrete with dimensions large enough to require that measures be taken to cope with the generation of heat from hydration of the cement and attendant volume change to minimize cracking" (ACI 207, 2005). Based on this definition, there are three distinct, but closely related, issues associated with mass concrete: size, heat of hydration, and cracking.

Mass concrete construction began in the United States with the construction of concrete dams. The frequency of such projects increased significantly during the early 1900s with improving concrete placement capabilities. As more and more dams were constructed, engineers began to observe significant cracking in the large concrete elements. It was not until 1930 that the ACI Committee 207 was formed to examine and solve problems associated with mass concrete elements. At that time, the Hoover Dam in Nevada was in the early stages of planning, and the committee began investigating the causes of cracking in mass concrete elements (ACI 207, 2005). Over the years, as concrete technology improved and structures grew larger, mass concrete elements became a common occurrence in construction sites. Today, mass concrete construction includes much more than dam construction, as large buildings and bridges often have multiple mass concrete elements. As the demand for mass concrete has increased, so has the volume of associated research.

With regard to size, the primary concern is the least dimension of the concrete element. Suppose a structure with large concrete elements is to be placed. The first is a 1 ft thick concrete slab. Its dimensions are 1 ft × 40 ft × 40 ft, with a volume of 1600 ft^3. The second element is a rectangular concrete footing with dimensions 20 ft × 10 ft × 6 ft, giving it a volume of 1200 ft^3. While, the slab has a greater volume, only the footing would be classified as mass concrete. This is due to a large volume of interior concrete, such that the heat from hydration in the center of the concrete element cannot escape as quickly. While many states and agencies vary as to what least dimension constitutes mass concrete, most state mass concrete specifications designate mass concrete as concrete with a least dimension that exceeds either 4 or 5 feet.

Another primary concern in mass concrete construction is temperature. During hydration, entrapped heat causes the element's core temperature to rise significantly. Two common types of distress can occur due to a major temperature increase. The first of these—known as thermal cracking—is primarily attributed to a large difference between the core concrete temperature and external concrete temperature. Thermal cracking can be severe and cause premature deterioration of a concrete element. Figure 1-1 is an example of thermal cracking in a bridge column in Texas.

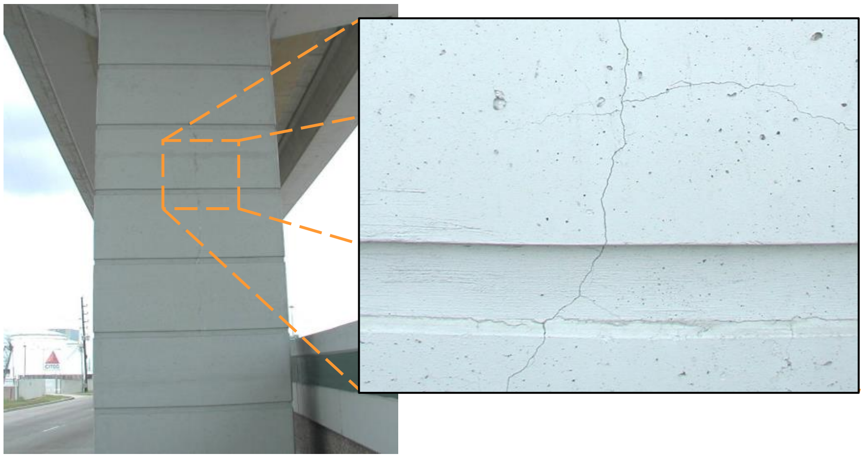

Figure 1-1: Thermal Cracking of a Bridge Column in Texas (Photo Courtesy of Dr. J.C. Liu)

The second of type of distress is known as delayed ettringite formation (DEF). DEF is an internal sulfate attack that causes expansion that can lead to cracking. This phenomenon occurs as a result of the high concrete core temperatures during hydration (Taylor et al., 2001).

In order to mitigate these distresses, mass concrete construction specifications can be developed. These specifications can provide guidance in the way of temperature limits, temperature control strategies, and other construction recommendations and requirements. Such a specification should contain information and guidelines that minimize the risk of mass concrete issues and are easy to understand and implement. Currently, the Alabama Department of Transportation (ALDOT) Standard Specifications for Highway Construction do not contain a mass concrete specification. For this reason, researchers at Auburn University were tasked with developing a specification for ALDOT mass concrete construction.

1.2 Project Objectives

This primary purpose of this project is to develop an ALDOT specification for mass concrete construction. The work done in this project sets the stage for the development of a comprehensive and practical specification. The primary objectives of this specification include recommendations for temperature prediction, size designation, materials requirements, temperature limits, and on-site procedures for mass concrete elements. In order to accomplish these objectives, the following tasks were performed:

- Review current mass concrete practices across the United States,
- Evaluate the effect of SCMs on the probability of DEF occurring in mass concrete members,
- Evaluate the effect of coarse aggregate type on the probability of thermal cracking in mass concrete placements,
- Measure in-place temperatures of mass concrete members during construction and observe those members for signs of thermal cracking and DEF,
- Assess the accuracy of ConcreteWorks' temperature and cracking risk predictions,

- Develop an improved method to determine the temperature difference to minimize the risk of thermal cracking,
- Investigate methods for predicting tensile strength development and creep effects in order to develop an age-dependent temperature difference limit,
- Use ConcreteWorks and field-instrumented temperature data to develop temperature control requirements for ALDOT mass concrete construction, and
- Provide recommendations for the assembly and placement of temperature sensors in mass concrete members.

1.3 Research Approach

The following five tasks were performed to accomplish the research objectives outlined in Section 1.2. Task 1 consisted of a literature review on the current state of practice. Task 2 involved the modeling of stress development in mass concrete members in order to develop a method for the prevention of thermal cracking. Task 3 consisted of the measurement of in-place concrete temperatures of six ALDOT mass concrete elements. The instrumentation of these elements allowed for the assessment of the accuracy of ConcreteWorks' temperature predictions in ALDOT mass concrete applications. Task 4 involved the development of implementation guidelines for a future ALDOT mass concrete specification.

This thesis presents the full procedures of Tasks 1 through 3, as well as parts of Task 4. The literature review (Task 1) was completed in the summer of 2014 and updated during the summer of 2016. ConcreteWorks modeling (Task 2) was performed during the fall of 2014. Task 3 began in the summer of 2015 with the assembly of temperature sensors in the Auburn University Structural Laboratory. Long-term temperature data were gathered in May 2016. All of the temperature data to that date are presented in this report, completing Task 3. Future research will be performed to complete the remaining tasks outlined by the project proposal.

1.4 Organization of Thesis

Chapter 2 explores literature discussing the issues pertinent to mass concrete members and their construction. Chapter 3 details the review of mass concrete specifications of other agencies and DOTs across the United States. Chapter 4 lays out the experimental plan implemented for this project. Chapter 5 summarizes the results from the field instrumentation phase of this research project. Chapter 6 discusses the results from a comprehensive ConcreteWorks analysis. Chapter 7 presents guidelines for the development of an ALDOT mass concrete specification. Chapter 8 summarizes all of the information presented in this report. Appendices A through F contain the additional information and data from each of the six ALDOT mass concrete elements instrumented during the field instrumentation phase of this project. Appendix G contains all output data from the ConcreteWorks analysis discussed in Chapter 6.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

This chapter reviews published literature pertaining to mass concrete construction. The information presented in this chapter should provide a thorough overview of the mass concrete issues addressed in the remainder of this report.

As discussed in Section 1.1, due to the size of mass concrete elements, there are two primary distresses in mass concrete construction. The first of these distresses is thermal cracking; the second is DEF. The mechanisms, causes, mitigation practices, and examples of thermal cracking and DEF are covered in Sections 2.2 and 2.3, respectively.

2.2 Thermal Cracking

2.2.1 Thermal Stress Development

The hydration of cementitious materials is an exothermic reaction. As heat is generated and high temperatures begin to develop in the core of a mass concrete element, the volume change associated with the temperature change induces a strain on the concrete. A concrete element's resistance to these strains is defined as "restraint." Restrained thermal strains result in thermal stresses. There are two forms of restraint: internal and external.

Internal restraint exists as a result of varied temperatures across a cross section (Bamforth, 2007). Due to exposure to the air, the edge concrete releases heat and cools much more quickly than the core. This uneven cooling mechanism produces uneven amounts of expansion and contraction between the core and the edge concrete. The core concrete restrains the contraction of the cooling edge concrete. This concept is demonstrated in Figure 2-1.

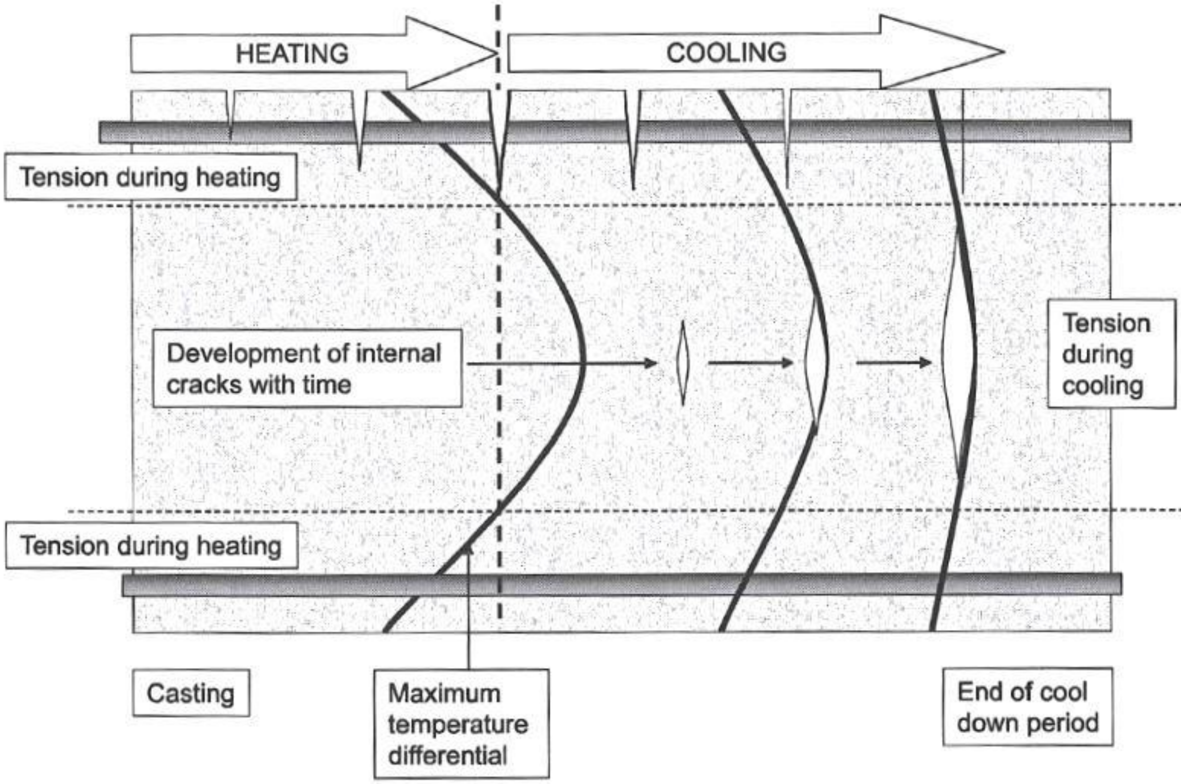

Figure 2-1: Development of Cracks in a Massive Element due to Temperature Differences Assuming Only Internal Restraint (Bamforth, 2007)

Large temperature differences are especially prominent in mass concrete elements due to the differences in temperature that develop between the face and core. A larger cross section typically results in a large temperature difference. Although the largest temperature difference typically occurs at the corner of a rectangular concrete element, thermal cracks are most likely to form along the face of the member, rather than the corner. This is due to the fact that restraint, namely internal restraint, is largest at the element edge, as indicated by Figure 2-2.

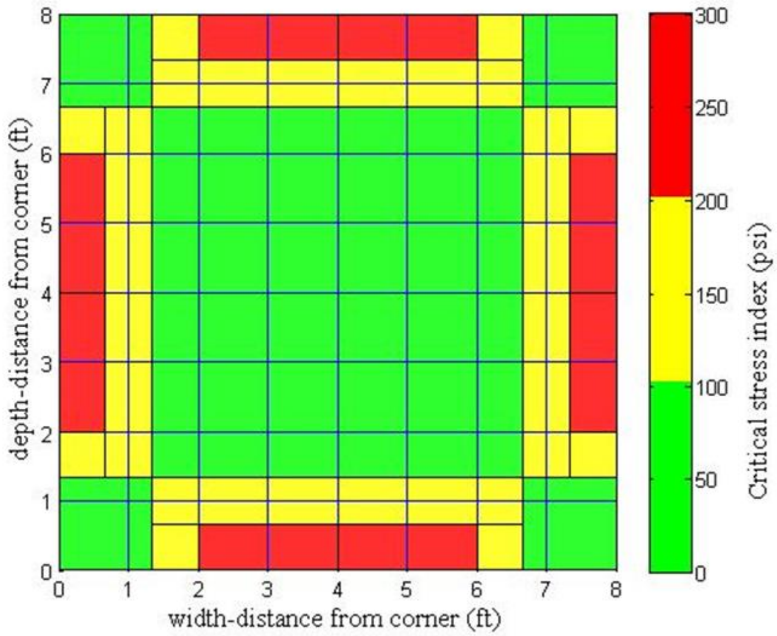

Figure 2-2: Effect of Internal Restraint on the Development Location of Thermal Cracks (Tankasala et al., 2017)

External restraint occurs when the concrete member's volume change is externally resisted, often by a previously placed adjacent concrete member or other restraining boundary condition (Rostasy et al., 1998). As depicted in Figure 2-3, an adjacent member restrains the contraction of cooling concrete member, resulting in large tensile stresses across the entire concrete cross section.

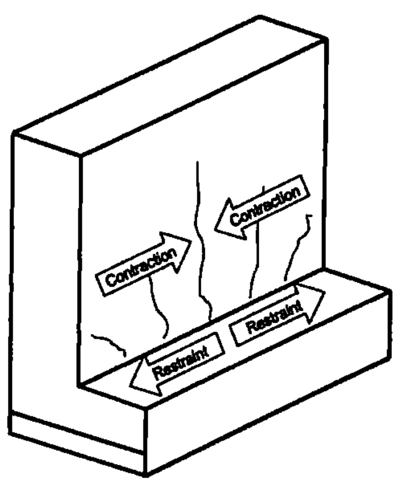

Figure 2-3: Restraint of a Concrete Member by Adjacent Elements (Bamforth, 2007)

In order to quantify thermal cracking in a concrete member, Bamforth and Price (1995) developed an equation for thermal strain, measured in microstrains ($\mu\varepsilon$), in concrete. Equation 2-1 shows this relationship between tensile strain, temperature difference, and restraint.

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

Where,

e_{tsc} = tensile strain capacity (in./in.),

K = creep modification factor (unitless),

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

ΔT_{max} = maximum allowable temperature difference (°F), and

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

As shown Equation 2-1, there are four primary factors in determining the thermal strain in a concrete element. First, the creep modification factor (K) is recommended by Bamforth (1995) at a constant value of 0.8 for concrete at early ages. This value accounts for the creep and relaxation in the concrete at early ages. In a later publication, Bamforth (2007) modified the default creep modification factor to 0.65 to account for the effects of creep and sustained loading due specifically to temperature differences across a cross section on concrete at early ages.

Next, there is the coefficient of thermal expansion (CTE) which is primarily governed by the type of coarse aggregate selected (Browne, 1972). A larger CTE indicates that a concrete specimen will undergo a larger volume change when exposed to the same temperature change as a specimen with a smaller CTE. Figure 2-4 shows typical CTE values for concretes of different coarse aggregate types. Alabama concretes typical contain either limestone or siliceous river gravel coarse aggregates.

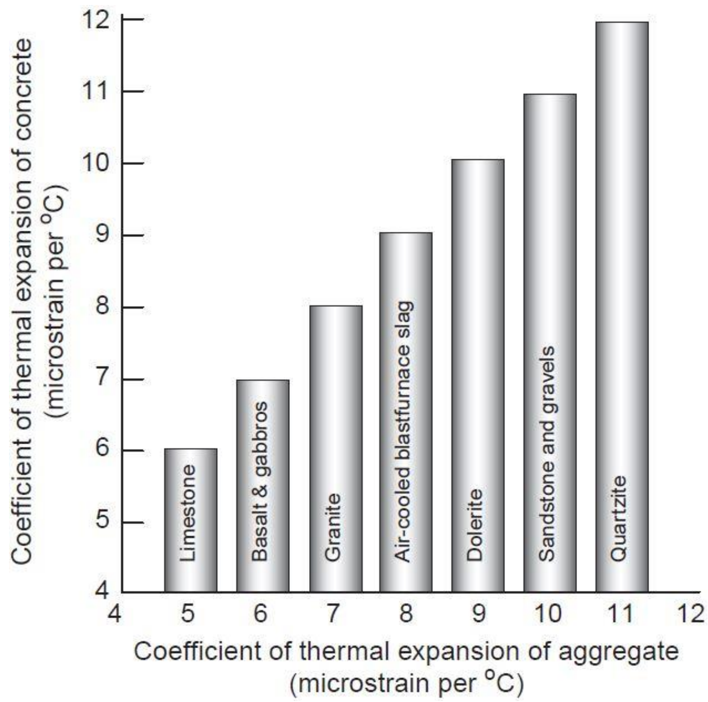

Figure 2-4: Effect of Aggregate Type on Concrete Coefficient of Thermal Expansion

($\Delta°\mathrm{F} = 1.8 \times \Delta°\mathrm{C}$) (Mehta and Monteiro, 2006)

The third variable is the restraint factor (R). This variable is used to estimate the magnitude of the effects of both internal and external restraint in a concrete member. Bamforth and Price (1995) assume for mass concrete a value of 0.36 for the restraint factor, as restraint can be a difficult factor to quantify. This assumption corresponds to restraint quantities from some typical mass concrete elements (ACI 207, 2007). In reality, restraint can vary greatly due to the age of the concrete, member geometry, and external restraint condition. Bamforth (2007) has guidelines for determining the appropriate restraint factor for concrete elements with different sizes, shapes, and boundary conditions. The final factor is the temperature change, ΔT, which is the difference in temperature at different locations within the concrete element. As previously stated, mass concrete members tend to experience large temperature differences due to large dimensions of the cross section.

Bamforth's thermal strain equation can be utilized to determine allowable temperature differences in mass concrete elements. Table 2-1 was adapted from Bamforth and Price (1995) to provide guidance on limiting temperature difference values with varying aggregate types and restraint factors.

Table 2-1: Limiting Temperature Differences Based on Assumed Values of CTE and Tensile Strain Capacity (adapted from Bamforth and Price, 1995) ($\Delta°\mathrm{F} = 1.8 \times \Delta°\mathrm{C}$)

Aggregate Type	**Gravel**	**Granite**	**Limestone**	**Lightweight**
Coefficient of Thermal Expansion ($\mu\varepsilon$/°C)	12.0	10.0	8.0	7.0
Tensile Strain Capacity ($\mu\varepsilon$)	70	80	90	110
Limiting temperature change for different restraint factors (°C):				
R = 1.0	7	10	16	20
R = 0.75	10	13	19	26
R = 0.50	15	20	32	39
R = 0.36	**20**	28	39	55
R = 0.25	29	40	64	78

The limiting temperature difference of 36 °F (20 °C), as calculated with a restraint factor of 0.36, is very important. This value is adopted as the recommended allowable temperature difference limit for mass concrete construction in ACI 301 (2016). Evidence of the widespread adoption of this temperature limit by U.S. Departments of Transportation (DOT) is in Chapter 3 of this report. Based on this table, however, this limit can be increased when a concrete with a lower CTE is used. In 2007, Bamforth republished this table of values to reflect the change of early-age creep modification factor from 0.8 to 0.65. Bamforth (2007) also revisited the values of CTE and tensile strain capacity applied to each aggregate type. In order to maintain the limiting temperature difference of 36 °F (20 °C) for gravel, the restraint factor modified proportionally from 0.36 to 0.42. For this research project, the default values from Bamforth's 2007 report are used.

2.2.2 Modeling Thermal Stress Development of Concrete Elements at Early Ages

In Bamforth's (2007) thermal cracking equation, the creep modification factor is assumed as a constant value of 0.65. Because the effects of creep at early ages are highly dependent on time (Bazant and Baweja, 2000), it would be beneficial to develop a time-dependent creep modification factor when calculating thermal stress. In order to quantify the effect of creep on a concrete element, the concrete strength and stiffness must first be determined. Bazant and Baweja (2000) developed a model for the prediction of creep, shrinkage, and temperature effects in concrete structures. This model is known as the B3 Model, and it is covered in more detail in the remainder of this section.

2.2.2.1 Modeling Strength and Stiffness of Concrete at Early Ages

The equation used in the B3 model to calculate compressive strength is shown in Equation 2-2.

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

Where,

f_{cmt} = concrete mean compressive strength at any time t (psi),

t = age of the concrete (days),

a = 4.0 for moist-cured concrete with Type I cement (days),

b = 0.85 for moist-cured concrete with Type I cement (unitless), and

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

In 1984, Raphael developed a method for calculating concrete tensile strength as a function of the concrete compressive strength. This particular function was selected because it was obtained through splitting tension applications, which is the primary mechanism of thermal cracking in mass concrete elements. This function is demonstrated in Equation 2-3 (Raphael, 1984).

$$f_t = 1.7 f_c^{2/3}$$ **Equation 2-3**

Where,

f_t = concrete tensile strength (psi), and

f_c = concrete compressive strength (psi).

The concrete stiffness can also be calculated as a function of the concrete compressive strength. ACI 318 (2014) provides guidance on how to calculate the concrete modulus of elasticity, as shown in Equation 2-4.

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

Where,

E_c = concrete modulus of elasticity (psi),

w_c = concrete unit weight (pcf), and

f_c = concrete compressive strength (psi).

When using normalweight concrete, Equation 2-4 can be simplified into Equation 2-5 (ACI 318, 2014).

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

Once the strength and stiffness have been calculated, the final time-dependent variable to the quantified is the creep modification factor.

2.2.2.2 Modeling of Creep Effects

The Bazant-Baweja B3 model was designed to predict the effect of creep and shrinkage in concrete elements (Bazant and Baweja, 2000). The parameters of this model, as well as the ACI 209R-92 model, are summarized by ACI 209 (2008). In accordance with ACI 214 (2011), the B3 model uses the second equation from Table 2-2 to calculate actual concrete strength when only the design strength is known (Bazant and Baweja, 2000).

The B3 model uses a creep coefficient to represent the effects of creep in a thermal cracking analysis. The equation for the B3 model's creep coefficient is shown in Equation 2-6.

$$\varphi(t, t_o) = E(t_o) J(t, t_o) - 1 \qquad \textbf{Equation 2-6}$$

Where,

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

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

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

t = age of concrete, and

t_o = age of concrete loading.

In this model, the creep coefficient is directly calculated from the compliance function. The compliance function found in the B3 model is shown in Equation 2-7.

$$J(t, t_o) = q_1 + C_o(t, t_o) + C_d(t, t_o, t_c) \qquad \textbf{Equation 2-7}$$

Where,

q_1 = instantaneous strain due to unit stress,

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

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

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

Lastly, the B3 creep coefficient can be converted into a creep modification factor using Equation 2-8. The creep modification factor adjusts the concrete stiffness to account for the effects of creep (ACI 209, 1982). By calculating a creep modification factor, the creep effects in Bamforth's thermal cracking equation (Equation 2-1) can be modeled as a time-dependent quantity.

$$K(t) = \frac{1}{1 + \varphi(t, t_o)} \qquad \textbf{Equation 2-8}$$

Where,

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

$K(t)$ = creep modification factor

Unfortunately, the compliance function in the Bazant-Baweja B3 model underestimates the effects of creep and relaxation at concrete early ages and does not account for the development of the modulus of elasticity with age. Because of this, Byard and Schindler (2015) developed a Modified B3 model. The Modified B3 model is simple and incorporates the existing B3 model's compliance function. This Modified B3 model uses two modifications to improve the B3 compliance function at early ages (Byard and Schindler, 2015).

2.2.3 Causes and Effects of Thermal Cracking

In order to better understand the concept of thermal cracking, it is helpful to look at Figure 2-5. The vertical dotted line marked "A" signifies the time of final set. At this condition, no thermal stresses have developed because the concrete has remained in a fluid state to this point, incapable of developing stress. As time passes from condition "A" to condition "B", the concrete temperature increases due to initial cement hydration. This results in a compressive stress developing in the restrained concrete. Condition "B" signifies the time of zero-stress, when sufficient cooling has occurred and the concrete experiences tensile stress for the first time. As concrete temperatures continue to decrease, thermal stresses continue to develop. Tensile strength begins developing at the time of final set. Once the tensile stresses due to thermal effects exceed the tensile strength of the concrete, thermal cracks will develop.

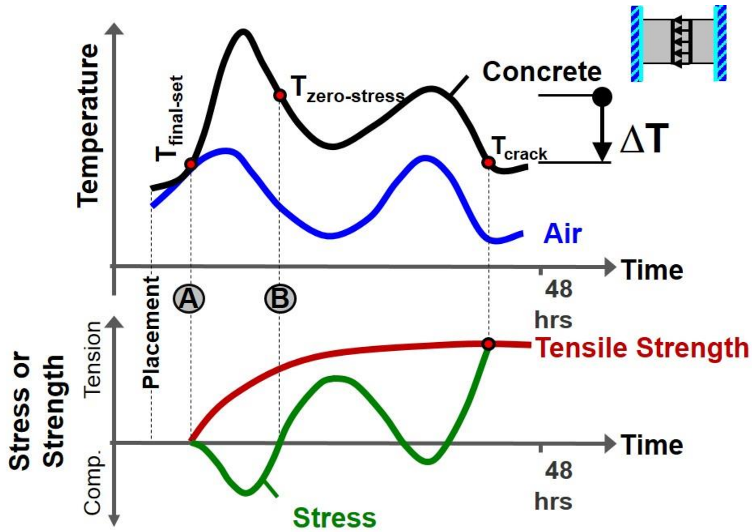

Figure 2-5: Mechanism of Thermal Cracking (Schindler and McCullough, 2002)

Aside from being aesthetically displeasing, thermal cracking can impact the durability of the concrete member. If thermal cracks grow wide enough, they can allow for the ingress of water, salts, and other harmful chemicals. This can expose reinforcement leading to corrosion and concrete to other long-term deterioration mechanisms. To mitigate this, there are a few methods that can be implemented to control thermal cracking.

2.2.4 Mitigation of Thermal Cracking

2.2.4.1 Temperature Difference Limits

The primary method for mitigating thermal cracking is to limit the temperature difference that the concrete may experience during construction. In mass concrete elements, the temperature difference is defined by ACI 207 (2007) as "the cooling of the surface concrete relative to the more stable internal temperature". The cooling concrete surface contracts more rapidly than the concrete core. These temperature differences produce tensile stresses and can produce thermal cracks. Passive techniques, such as proper formwork removal and material selection, and active cooling techniques, such as internal cooling pipes, can be used to reduce temperature differences in mass concrete elements.

2.2.4.2 Formwork Removal

Proper formwork selection and removal times can reduce temperature difference in a mass concrete element. If a concrete element is poorly insulated and the ambient temperature is significantly cooler than the concrete temperature, rapid heat loss will occur at the edges of the element, resulting in a large temperature difference (ACI 207, 2007). The use of wooden forms or blanket-insulated forms provides better insulation and allows the concrete temperature to decrease at a more uniform rate across the entire element (ACI 207, 2007). Wooden forms often have better insulating properties than steel forms. Steel forms tend to allow heat to escape a concrete element more quickly, resulting in a rapid decline of the edge temperature with respect to the core temperature.

A phenomenon known as “thermal shock” can also occur in mass concrete elements as a result of early formwork removal. If formwork is removed too soon or in extremely cold conditions, the edge concrete’s immediate exposure to cold air results in the rapid decrease of edge temperatures and can result in thermal cracking (ACI 207, 2007). Figure 2-6 shows this phenomenon in detail.

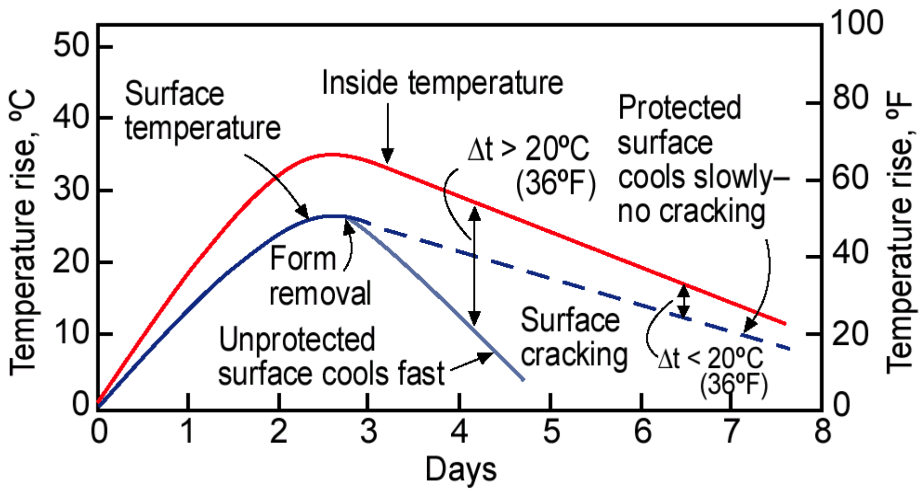

Figure 2-6: Effect of Formwork Removal Time on Cracking Risk (Gajda and Alsamsam, 2006)

$(°\mathrm{F} = 1.8 \times ℃ + 32)$

In Figure 2-6, the dashed line shows the benefit of waiting to remove forms until after the core temperature has reached its peak and cooled significantly. By removing formwork when there is a low temperature difference, the risk of thermal cracking is significantly reduced. An example of cracks developed from early removal of insulation in a footing is shown in Figure 2-7. The thermal cracks shown are extremely wide and propagate deep into the element.

Figure 2-7: Concrete Core from Footing Where Insulation Blew Off During Cold Weather (Gajda and Alsamsam, 2006)

2.2.4.3 Coarse Aggregate Selection

Selecting the proper coarse aggregate can improve a concrete element's resistance to thermal cracking in two ways. First, selecting a coarse aggregate with a lower CTE can directly reduce the risk of thermal cracking because the coarse aggregate has the greatest effect on the concrete's overall CTE (Browne, 1972). In Alabama, the two primary coarse aggregate types are siliceous river gravel and limestone. Based upon research performed at Auburn University, typical CTE values for Alabama concretes are 6.95×10^{-6} in./in./°F for siliceous river gravel and 5.52×10^{-6} in./in./°F for limestone (Schindler et al., 2010). Because concrete with limestone aggregate has a lower CTE, it can tolerate higher temperature differences without inducing a higher tensile stress. This means limestone concrete has a lower susceptibility to cracking.

Second, coarse aggregate selection determines a concrete's tensile strain capacity (ε_{tsc}). This value represents the amount of strain a concrete member can withstand without developing a crack. Bamforth (2007) assigned river gravel and limestone concrete ε_{tsc} values of 65 and 85 $\mu\varepsilon$, respectively. These values on differ slightly from the values of 70 and 90 $\mu\varepsilon$ published by Bamforth and Price (1995). Based on these values, highly restrained concrete made with limestone can withstand 30% greater temperature change than concrete made with river gravel before cracking.

2.2.4.4 Pour Dimensions and Element Geometry

The final methods for preventing thermal cracking involve the construction schedule and element design (ACI 207, 2005). As described in Section 1.1, the least dimension of a mass concrete element is what drives the majority of the temperature concerns associated with mass concrete. A concrete element with a smaller least dimension will typically experience a lower maximum temperature and lower temperature differences. Sometimes in very large concrete pours, such as footings, pedestals, or very large columns, the concrete can be cast in lifts. For instance, a mat footing with plan dimensions of 100 ft × 100 ft and a depth of 30 ft has a least dimension of 30 ft. With an element this size, extremely high core temperatures and temperature differences are likely to occur, creating a high risk for DEF and extensive thermal cracking. However, if the footing were cast in four equal-depth lifts, the least dimension could be reduced to 7.5 ft. This would allow heat trapped in the placement core to be released more quickly, which in turn lowers the maximum temperature reached in the section.

2.2.5 Examples of Thermal Cracking in Mass Concrete Elements

Figure 2-8 is an example of thermal cracking in a mass concrete member in Texas. In this figure, it is easy to identify thermal cracks running primarily vertically up the center of the column face. These cracks likely occurred at this location because this location is likely to experience both a high degree of internal restraint and a large temperature difference. Based on Equation 2-1, an element with a high degree of restraint and a high temperature difference will likely experience high tensile strains due to thermal effects, which leads to thermal cracking.

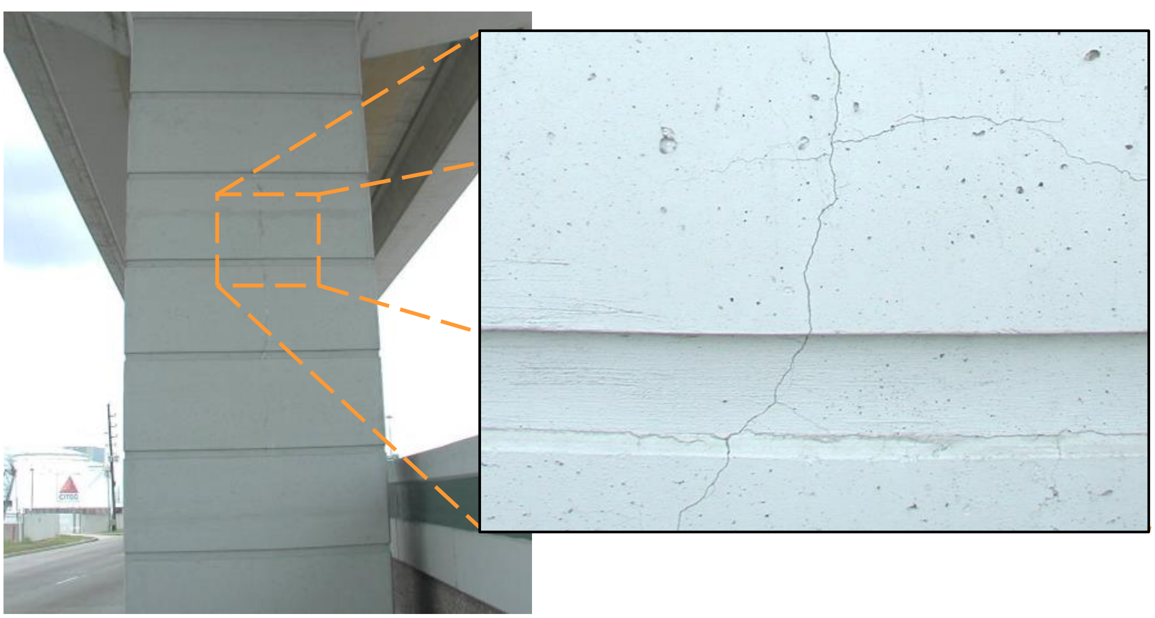

Figure 2-8: Thermal Cracking of a Bridge Column in Texas (Photo Courtesy of Dr. J.C. Liu)

2.3 Delayed Ettringite Formation

2.3.1 Introduction

DEF is an internal sulfate attack that causes expansion of the hydrated cement paste. This phenomenon occurs in concrete as a result of elevated concrete temperatures and exposure of concrete to moisture (Pavoine et al., 2006). During the mid-1990s, some concrete pavements and precast members began showing signs of premature deterioration. These instances gained notoriety and significance because the causes of the distress was unknown. At this point, many researchers and consultants to take a deeper look into the cause of these crack formations (PCA, 2001). The effect of high temperatures and moisture presence were examined, as well as the chemical composition of the mixture components. Unfortunately, there remained much uncertainty when it came to DEF for quite some time. In 2001, a paper published by Taylor, Famy, and Scrivener (2001) answered many questions associated with DEF and provided a now widely accepted theory for the formation of DEF in plain portland cement concretes. The primary hypothesis of this paper suggests that DEF-induced expansion is dependent upon three factors: chemistry, paste microstructure, and concrete microstructure (Taylor et al., 2001). The suggested expansion mechanism detailed in this paper is shown in Figure 2-9 (Taylor et al., 2001).

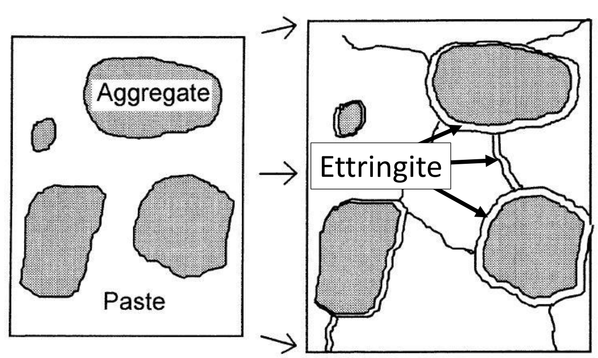

Figure 2-9: Suggested DEF Expansion Mechanism (Taylor et al., 2001)

The cracking mechanism portrayed in Figure 2-9 suggests that uneven paste expansion causes cracking in the paste and at the paste-aggregate interfaces. Ettringite then forms in these cracks. Initially, expansive pressures are likely low in these cracks. However, as the DEF-damaged concrete is reheated, further expansion occurs in these ettringite-filled cracks, further damaging the concrete element (Taylor et al., 2001).

Mass concrete elements are particularly susceptible to DEF due to the large amount of heat from hydration that becomes entrapped in the concrete after placement. In recent years, there have been limited reports of structural damage in mass concrete elements due to DEF in the United States. One such occurrence was discovered in Georgia in 2008, as large ettringite-filled cracks up to 6 in. wide were found in a seal foundation (McCall, 2013). A schematic of the cracking in this seal footing is shown in Figure 2-10 (McCall, 2013). Microscopic analysis of core samples taken from the seal was performed, and it was concluded that DEF was the primary contributor to the cracking (McCall, 2013). According to a report submitted by the Georgia Department of Transportation, these seal foundations reached temperatures of approximately 200 °F during hydration, as well as a temperature difference of approximately 111 °F (Kurtis et al., 2012). Those high temperatures resulted in thermal cracking as well as expansion and cracking from DEF.

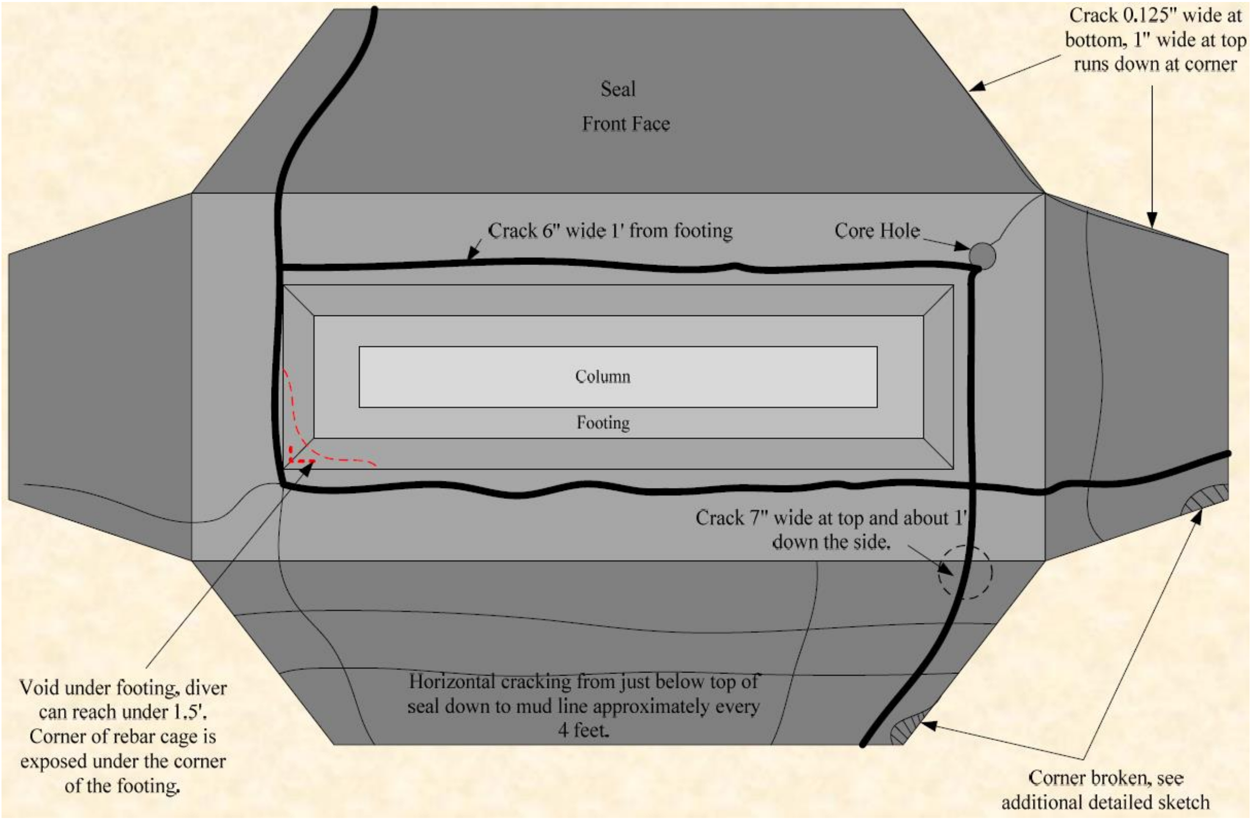

Figure 2-10: Plan View Schematic of Cracks in a Seal Footing in Georgia (McCall, 2013)

Cast-in-place concrete columns on the San Antonia Y Overpass in San Antonio, Texas were investigated by UT-Austin for signs of DEF-induced expansion (Folliard et al., 2006). Researchers were able to identify DEF as the primary cause of premature deterioration in at least one of these columns. Photographs of the column with the most extensive cracking are shown in Figure 2-11.

Figure 2-11: Cracking Due to DEF at San Antonio Y Overpass (Thomas et al., 2008)

An internal memorandum and batch tickets from TxDOT indicated that Type III cement was likely included in the mixture proportions for this column (Folliard et al., 2006). Researchers observed that of three adjacent columns with presumably similar mixture proportions, the column with the most exposure to weather experienced the most extensive cracking due to DEF (Folliard et al., 2006).

Another group of researchers examined this same bridge element in order to develop methods for diagnosing DEF in concrete structures. Back-scattered electron images were produced on concrete samples taken from the column. These microscopic images show significant cracking around coarse aggregate particles, as well as the formation of ettringite in the gaps caused by the cracking. An example of these images is shown in Figure 2-12.

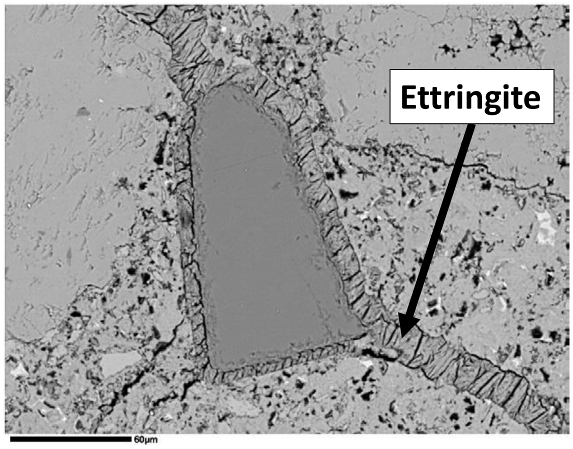

Figure 2-12: Ettringite Formation in Cracks around Coarse Aggregate in Cores Removed from San Antonio Y Overpass (adapted from Thomas et al., 2008)

2.3.2 Causes and Effects of DEF

In mass concrete, it is paramount for engineers to monitor the core temperature of a concrete element, ensuring that it does not exceed the maximum allowable temperature. The widely accepted limit for concrete temperature is 158 °F (70 °C) (Taylor et al., 2001). However, this maximum temperature limit is not absolute and can be decreased or increased based on certain conditions which can accelerate or decelerate DEF in mass concrete. For example, expansion is not only influenced by temperature, but also by the amount of sulfate in the concrete (Pavoine et al., 2006).

The second necessary condition for DEF to occur is that "the material must be kept wet or moist" (Taylor et al., 2001). In research performed in 1999, it was concluded that "the driving force for growth (of ettringite) is provided by the supersaturation, and the hydrostatic pressure that is needed to stop growth increases with the degree of supersaturation" (Scherer, 1999).

Though elevated temperatures and exposure to water are the two undisputed necessary criteria for DEF, there are other parameters that can lead to the development of DEF in mass concrete elements. Pavoine, Divet, and Fenouillet (2006) reported the following parameters also have an effect on occurrence of DEF:

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

2.3.3 Preventative Measures for Controlling DEF in New Concrete Construction

2.3.3.1 Maximum Concrete Temperature Limits

Maximum core temperature limits are often set in place to prevent DEF. As previously stated, the most common maximum temperature limit is 158 °F (70 °C), but the use of SCMs could potentially increase this limit (Folliard et al., 2006). The use of SCMs, such as fly ash or slag cement, reduces the amount of sulfate in the concrete (Thomas et al., 2008). For this reason, the use of SCMs at appropriate replacement levels can provide a chemical composition less susceptible to DEF (Myuran et al., 2015). In addition to a better chemical composition, concrete containing fly ash also produces less heat during hydration than a typical portland cement concrete. This is demonstrated clearly in Figure 2-13.

DEF expansion tests were performed for a TxDOT research project to assess the validity of the 158 °F limit and potentially develop a higher temperature limit for concrete containing SCMs. This study determined that the following SCMs at sufficient replacement levels are effective in mitigating DEF-induced expansion at temperatures above the 158 °F limit: Class F fly ash, Class C fly ash, slag cement, metakaolin, and ultra-fine fly ash. These claims are supported by the expansion test results shown in Figure 2-14.

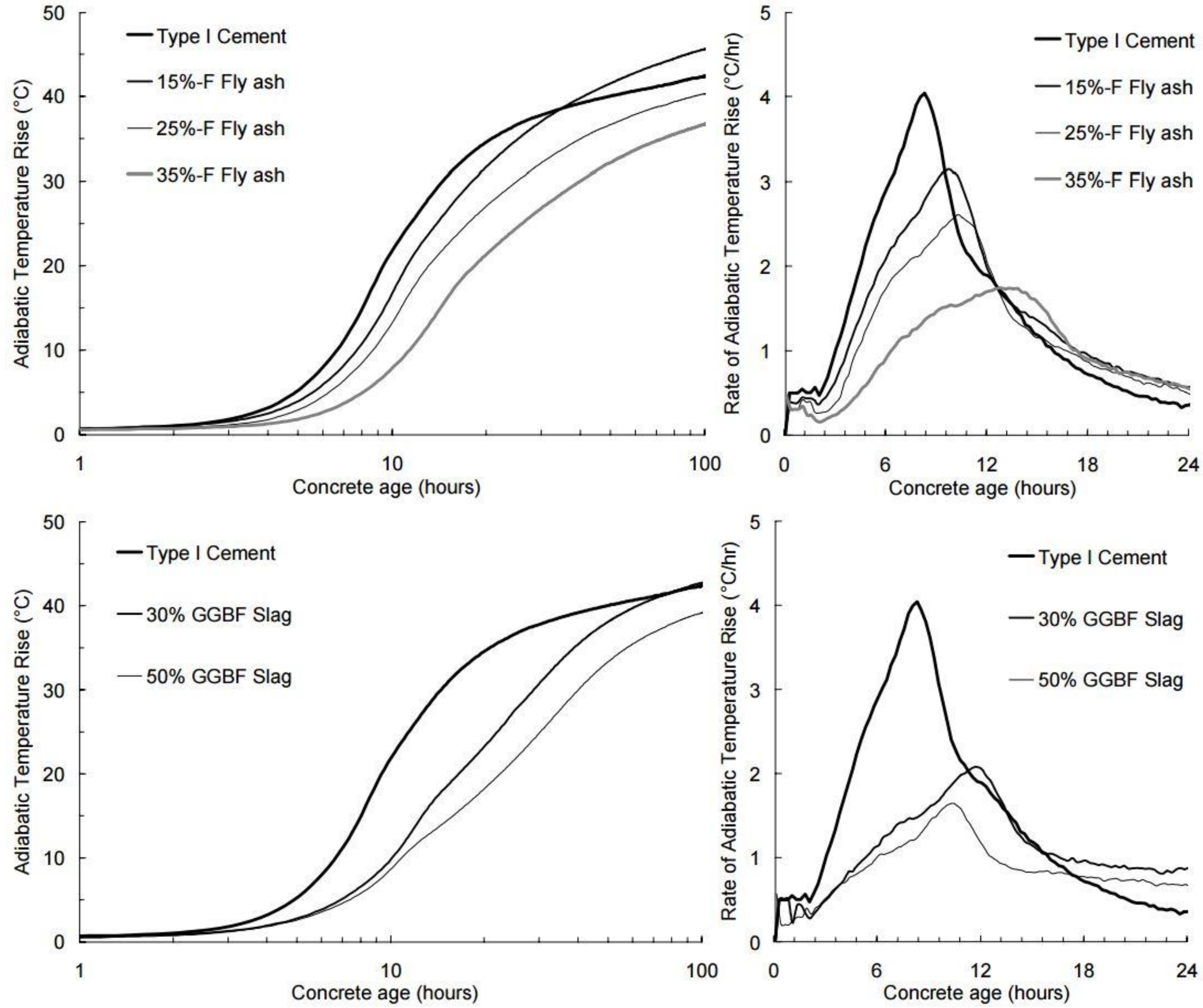

Figure 2-13: Effect of Fly Ash (top) and Slag Cement (bottom) on Heat of Hydration (Schindler and Folliard, 2003) ($°\mathrm{F} = 1.8 \times °\mathrm{C} + 32$)

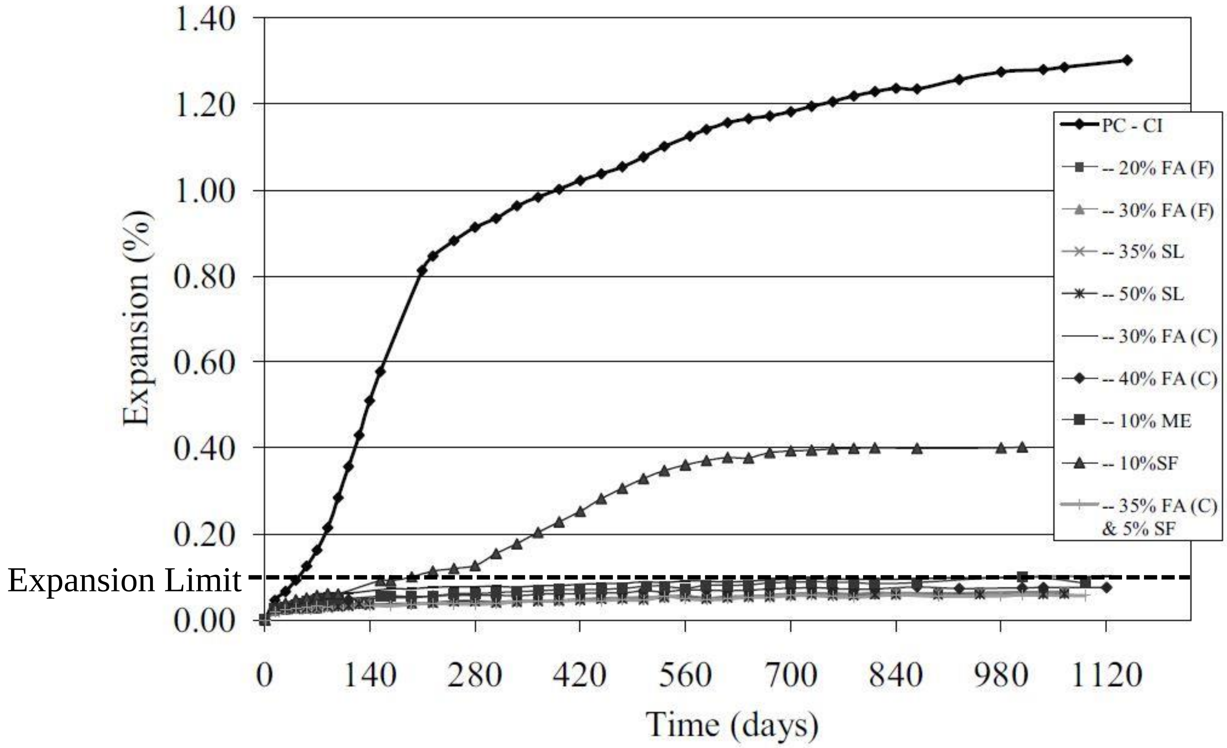

Figure 2-14: Expansion Test Results for Mortar Bars with Plain Portland Cement in Combination with Various SCMs Cured at 203 °F (95 °C) (adapted from Folliard et al., 2006)

The only mortar bars to show signs of expansion above the expansion limit of 0.10% were those with plain portland cement and 10 percent silica fume replacement. It should be noted that silica fume not used in combination with another SCM is not effective in mitigating DEF (Folliard et al., 2008).

While both Class F fly ash and Class C fly ash can be used to reduce DEF-induced expansion, Class F fly ash tends produce less expansion even at lower replacement levels, making it a more favorable option than Class C fly ash (Folliard et al., 2006). This concept is demonstrated graphically by 14-day expansion test results in Figure 2-15.

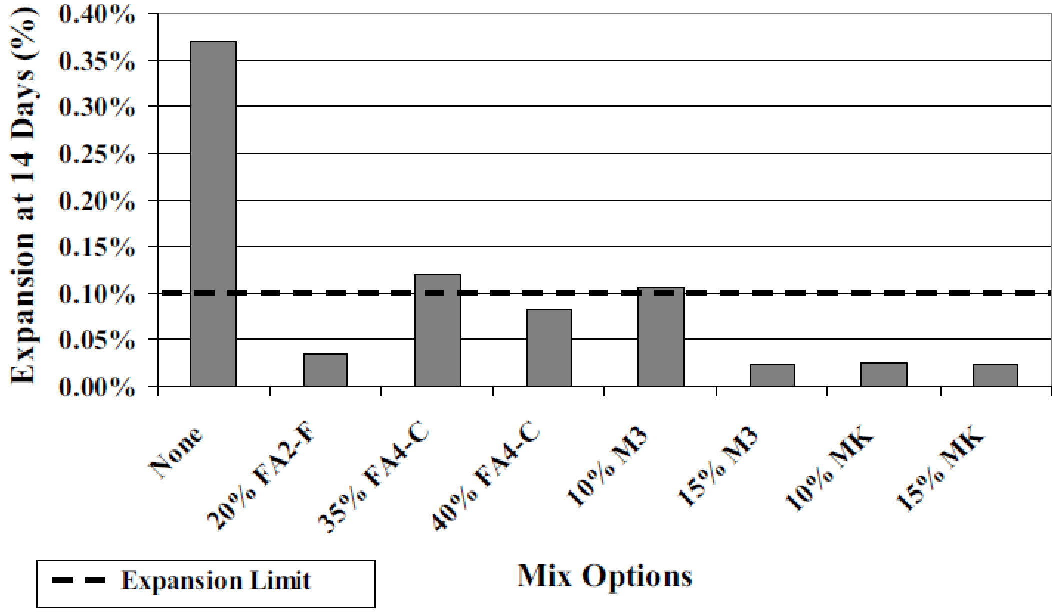

Figure 2-15: Reduced Expansion in Class F Fly Ash versus Class C Fly Ash Concrete Specimens (Folliard et al., 2006)

Based on the experimental results from all of these expansion tests, new temperature limits were recommended by Folliard et al. (2006) for various types of concrete construction. These limits are summarized in Table 2-2.

Table 2-2: TxDOT Research Study Temperature Limit Recommendations (Folliard et al., 2006)

Concrete Element Type	Maximum Temperature Limit
Precast girders with plain concrete	150 °F
Precast girders with concrete containing SCMs	170 °F
Mass concrete	160 °F

The type and dosage for SCMs in "concrete containing SCMs" is not directly specified. However, the types and dosages from the expansion tests shown in Figure 2-14 can be referenced, as they were part of the research as the information in Table 2-2. It is unknown why the 170 °F limit does not apply to mass concrete placements. Many mass concrete specifications require low-heat cements and SCMs which might also reduce the risk of DEF. This claim is supported by the study of U.S. mass concrete specifications in Chapter 3 of this report. An ensuing UT-Austin report sponsored by TxDOT proposed mass concrete maximum temperature limits in order to be more consistent with precast concrete temperature limits (Folliard et al., 2008). Table 2-3 summarizes the mass concrete specifications changes recommended by this report.

Table 2-3: Proposed Maximum in-Place Temperature Limit Specification in Mass and Precast Concrete Construction (Folliard et al. 2008)

Maximum In-Place Concrete Temperature (*T_{max}*)	**Prevention Required**
T_{max} < 158°F (70 °C)	None.
158°F (70 °C) ≤ T_{max} ≤ 185°F (85 °C)	Use one of the following approaches to minimize the risk of DEF: 1. Use portland cement that meets the requirements of ASTM C 150 for Type II, IV, or V cement and has a Blaine fineness ≤ .58 psi (400 m2/kg) 2. Use portland cement with a 1-day mortar strength (ASTM C 109) ≤ 20 MPa 3. Use any of the following suitable combinations of pozzolan or slag cement with portland cement: —at least 25% fly ash meeting the requirements of ASTM C 618 for Class F fly ash —at least 35% fly ash meeting the requirements of ASTM C 618 for Class C fly ash —at least 35% slag cement meeting the requirements of ASTM C 989 —5% silica fume (meeting ASTM C 1240) in combination with at least 25% slag cement.
T_{max} > 185°F	This condition is not allowed.

Although these mass concrete limits are supported by the research results and recommended by research performed for TxDOT, TxDOT has not yet adopted these limits in its mass concrete specification.

2.4 Temperature Control Strategies

2.4.1 Introduction

Temperature control is essential in the construction of mass concrete elements in order to prevent excessive concrete core temperatures and limit temperature differences in the concrete element. This section highlights the methods for monitoring and controlling these high temperatures in mass concrete elements. Lowering concrete temperatures can be performed by a number of methods.

2.4.2 Methods for Controlling Maximum Concrete Temperatures

The best and most efficient method for preventing DEF is simply minimizing the maximum in-place concrete temperature. This can be accomplished by cooling the concrete before or after placement. Cooling the concrete prior to placement is referred to as pre-cooling; cooling the concrete after placement is referred to as post-cooling.

2.4.2.1 Pre-Cooling Techniques

The implementation of pre-cooling techniques is very common in mass concrete construction (ACI 207, 2005). The most simple and cost effective method for reducing maximum in-place concrete temperature is a well-engineered low-heat concrete (Gajda et al., 2005). Placing concrete during cooler ambient conditions can lower the placement temperature of the concrete. Typically, a 1 °F decrease in concrete placement temperature results in a 1 °F reduction in maximum in-place concrete temperature (Gajda and Vangeem, 2002). If more pre-cooling is needed to avoid high temperatures, other cooling techniques can be implemented. Four of these techniques are described in the following numbered list (Gajda et al., 2005).

1. *Evaporative cooling*: this is the most economical cooling method (Gajda et al., 2005). Evaporative cooling is done by sprinkling water on the aggregate stockpile and using

evaporation to remove heat from the aggregate stockpile. The amount of cooling depends on the cooling effect of multiple ambient factors, such as temperature, wind, and relative humidity (ACI 207, 2005).

2. *Ice*: This is the most common method of pre-cooling. This method simply involves replacing batch water with ice. This cools the mixture first by lowering the batch water temperature and second by removing heat during the melting process. Ice is typically allowed to replace up to 80 percent of the batch water (Gajda et al., 2005) and can lower the concrete temperature up to approximately 21 °F (ACI 207, 2005).
3. *Chilled water*: this method is very similar to the "ice" method. By simply lowering the temperature of the batch water, the maximum temperature can be lowered up to about 8 °F.
4. *Liquid Nitrogen*: this is the most effective method for lowering the fresh concrete temperature more than 20 °F (Gajda et al., 2005). Liquid nitrogen can be added to the concrete mixture in multiple ways. It can be injected directly into the ready-mixed concrete truck drum and mixed in with the plastic concrete (ACI 207, 2005). Liquid nitrogen is sometimes selected because it allows contractors to make on-site adjustments to the placement temperature. Local availability should always be considered when considering liquid nitrogen as a pre-cooling option (ACI 207, 2005).

2.4.2.2 Post-Cooling Techniques

Where pre-cooling is effective in lowering the placement temperature, post-cooling techniques are used to remove heat from the concrete during the hydration process. Post-cooling is primarily conducted by use of internal cooling pipes. Prior to concrete placement, cooling pipes are threaded through the concrete element's interior and embedded in the concrete. After the element is cast, cool water is pumped through the pipes, removing heat from the concrete with post-cooling. There are multiple variables that determine how much heat is removed from the concrete. These variables include the size and type of pipe used, temperature of the water, length of pipe, and velocity of the water flowing through the pipes (Kim et al., 2000). The primary disadvantage of internal cooling pipes is the difficulty in placing and maintaining the pipes before and during the concrete placement. In order to protect the tubing from damage, only durable materials should be used. Options for tubing materials include aluminum tubing, thin-walled steel tubing, and heavy duty PVC piping (ACI 207, 2005). Pipe spacing and diameters can vary from one element to another, tubing with a diameter greater than 1 in. tend to be most effective in post-cooling practice (ACI 207, 2005). Figure 2-16 demonstrates the benefit of installing cooling pipes in a mass concrete element. In this figure, it is important to observe the decrease in maximum concrete temperature from approximately 163 °F (73 °C) to 144 °F (62 °C). If the element in Figure 2-9 specified maximum in-place concrete temperature limit of 158 °F (70 °C), this post-cooling would be effective in maintaining concrete temperatures below that limit.

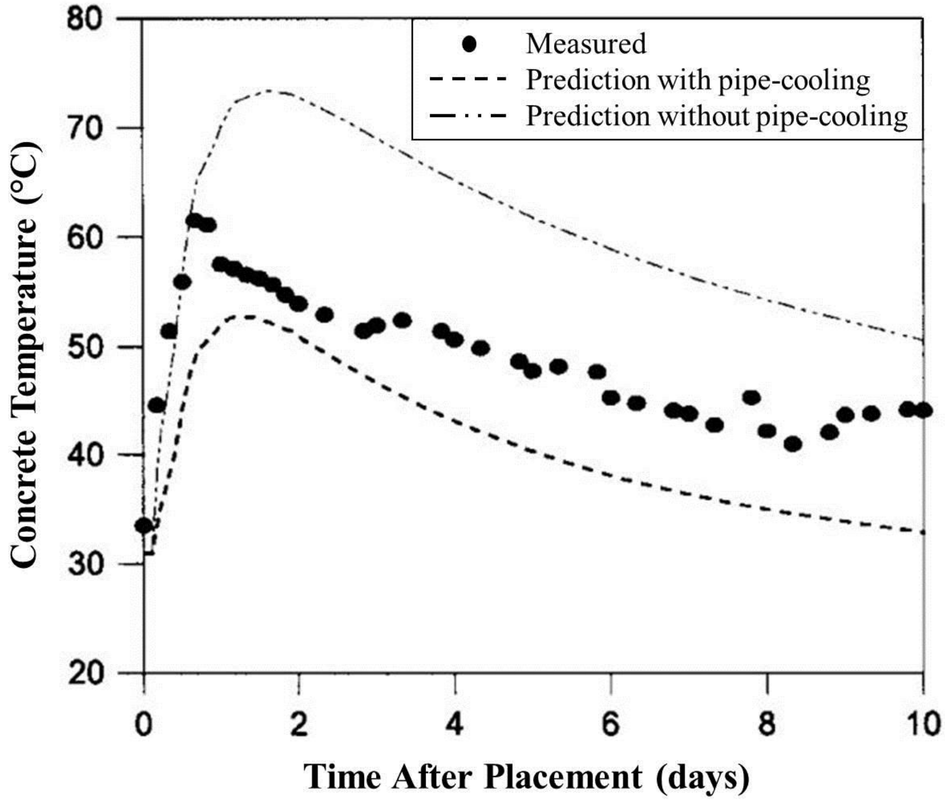

Figure 2-16: Effect of Cooling Pipes (adapted from Kim et al., 2000) ($°F = 1.8 \times °C + 32$)

2.4.3 Concrete Temperature Monitoring Methods

Concrete temperature monitoring is typically conducted from the time of placement to a specified number of days after placement. Temperature monitoring details are outlined in a mass concrete thermal control plan (TCP). In the TCP, the contractor describes the type of temperature monitoring systems to be used, the locations of the temperature sensors, and the duration of monitoring. Multiple monitoring systems or instruments can be installed to ensure that the temperature will still be monitored should one system fail (ACI 207, 2005). This also allows for the comparison of data in the case of an issue with the accuracy or calibration of a particular sensor.

Specifications typically require sensors at the element's core as well as the outer edges of the element. Core sensors are the most important, as they are essential in monitoring both the maximum in-place temperature and the temperature difference at various places in the concrete element. Edge sensors are typically placed at the location of the element most likely to experience thermal cracking. Sensors should be embedded deeply enough to protect the sensor from damage during construction as well as prevent the ambient temperature from disrupting concrete temperature measurements. It is often convenient to tie the sensor to the reinforcement. Obtaining temperature data for both the element core and the element edges allows for both the maximum concrete temperature and temperature difference to be monitored.

Types of temperature sensors vary from one project to another. Some temperature sensing devices included thermocouples, intelliRock sensors, and iButtons. Detailed information containing the capabilities and construction of temperature sensors used in this research project are shown in Section 4.4.

2.5 Thermal Control Plan

Contractors must work to develop a plan for maintaining concrete temperatures that meet the project specification requirements throughout construction. This sort of document is typically referred to as a Thermal Control Plan, or TCP. A TCP is ordinarily developed by the contractor and approved by the engineer prior to construction.

The following bulleted information is from the Georgia Department of Transportation (2013) and details information contained in a typical TCP:

- Concrete mixture proportions,
- Pre-placement temperature control techniques (i.e. placement time, placement temperature, pre-cooling, etc.),
- Curing duration and method,
- Estimated maximum concrete temperatures and temperature differences,

- Description of when and how mitigating measures will be taken if temperatures or temperature differences approach maximum limits,
- Thermal stress models from placement until 28 days after placement, and
- Temperature monitoring and recording system information, including location and type of temperature sensors.

2.6 ConcreteWorks

2.6.1 ConcreteWorks Background Information

ConcreteWorks is a user-friendly software program developed to aid engineers and contractors in predicting concrete temperatures in various concrete placements. ConcreteWorks allows the user to accurately model very specific details unique to a particular concrete element. Heat of hydration and heat transfer models provide the user with a temperature development profile across the entire element cross section during the early ages of a concrete placement. Multiple maturity functions can also be used to calculate stress and strength development in the concrete element. The ConcreteWorks software package is free and available on the TxDOT website (http://www.txdot.gov/inside-txdot/division/information-technology/engineering-software.html).

ConcreteWorks's extensive number of adjustable inputs enables the user to tailor the software model to a particular real-world concrete placement. ConcreteWorks also provides default input values that provide guidance in the event that a concrete placement has conditions that may be unknown. ConcreteWorks groups all of the inputs into nine categories (Concrete Durability Center, 2005). Each category and its contents are summarized in Table 2-4.

Table 2-4: ConcreteWorks Input Categories

Input Category	Contents
General Inputs	• Time, date, and location (city, state) of concrete placement • Duration of analysis (1-14 days)
Shape Inputs	• Element type and shape
Member Dimensions	• Member dimensions, specific the element shape
Mixture Proportions	• Mixture proportions and properties of mixture components
Material Properties	• Cement chemical composition and hydration properties • Coarse and fine aggregate type and concrete CTE
Mechanical Properties	• Maturity function inputs
Construction Inputs	• Fresh concrete temperature, form type, and method and duration of curing
Environment Inputs	• Weather data (temperature, wind speed, cloud cover, relative humidity)
Corrosion Inputs	• Type of reinforcing steel, reinforcement clear cover, and corrosion inhibitors used

In most cases, default values provide a reliable temperature analysis. However, as more accurate inputs are manually applied to the prediction model, the accuracy of the temperature predictions improves.

ConcreteWorks is capable of producing strength and maturity curves, temperature profiles, thermal cracking risk, and chloride diffusion service life. For mass concrete elements, the core temperature, temperature difference, and cracking risk are the most pertinent outputs. ConcreteWorks uses a plane strain finite-difference scheme to calculate the elastic stress in the member. Table 2-5 is adapted from the ConcreteWorks User Manual. It summarizes the outputs ConcreteWorks is capable of producing for various mass concrete members.

Table 2-5: ConcreteWorks Mass Concrete Outputs (adapted from Concrete Durability Center, 2005)

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

This research project focuses on the thermal cracking risk and temperature predictions. ConcreteWorks is capable of producing both of these outputs for all six of the ALDOT bridge elements instrumented with sensors and modeled in ConcreteWorks.

ConcreteWorks also enables the user to select and download the temperature profiles at various edge and core locations. This is particularly useful in analyzing the temperature differences at various edge locations during the periods of high cracking risk. Figure 2-17 shows an example of ConcreteWorks output information for a concrete pedestal in Scottsboro, Alabama. This element was modeled as a rectangular footing.

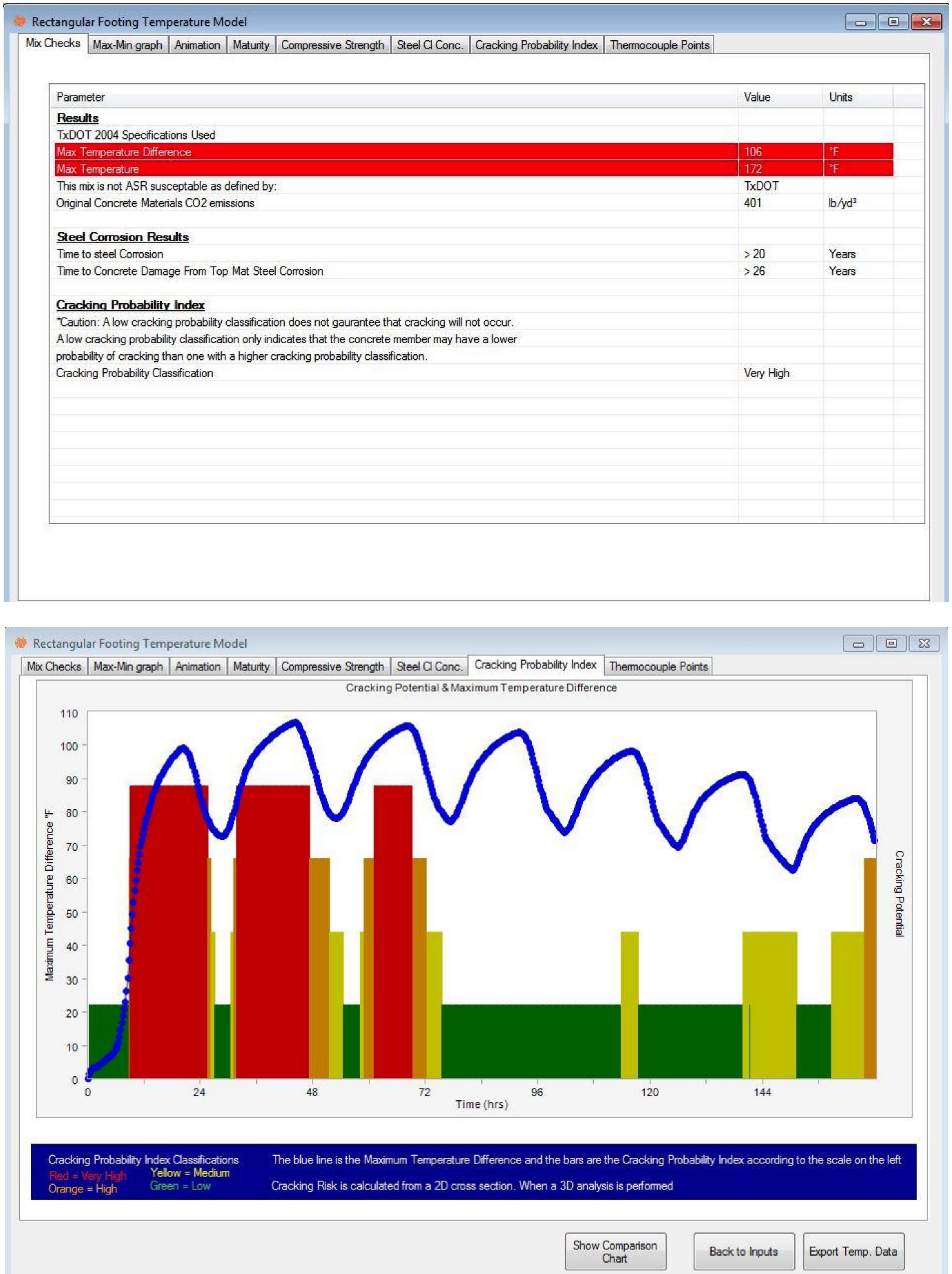

Figure 2-17: Output Summary (top) and Cracking Risk Profile (bottom) of Concrete Pedestal

The red, orange, yellow, and green bars on the cracking risk profile indicate the cracking risk of a concrete element during the first 168 hours after placement (Concrete Durability Center, 2005). Green indicates a low cracking probability and a tensile stress-strength ratio less than 0.60. Yellow indicates a medium cracking probability and a tensile stress-strength ratio from 0.60 to 0.67. Orange indicates a high cracking probability and a tensile stress-strength ratio from 0.67 to 0.72. Red indicates a very high cracking probability and a tensile stress-strength ratio greater than 0.72. Quantifying the risk of cracking allows the engineer to anticipate potential durability issues and take the necessary preventive measures. For more information regarding the models and methods used in ConcreteWorks predictions, the ConcreteWorks User Manual is available for free download at TxDOT online library.

CHAPTER 3: REVIEW OF MASS CONCRETE SPECIFICATIONS

3.1 Investigation of Current Mass Concrete Specifications

This section contains detailed information on how other agencies manage mass concrete concerns and regulate mass concrete construction. The groups discussed are the ACI, AASHTO, FHWA, various state DOTs, and industry consultants. Each sub-section in this chapter covers a different group.

3.1.1 ACI 301

ACI Committee 301 (2016) released a report containing useful information on the designation of mass concrete elements and requirements for those mass concrete elements. ACI 301 (2016) recommends applying the ACI 207 (2005) least dimension designation for mass concrete elements as 4 ft. Table 3-1 summarizes the requirements for mass concrete found in ACI 301 (2016).

Table 3-1: ACI 301 (2016) Mass Concrete Requirements

Material Requirements	**Temperature Monitoring System**	**Temperature Requirements**	**Plan**
• The use of Type III cement shall not be permitted unless approved by the engineer • Use hydraulic cement with low heat of hydration or portland cement with Class F fly ash or slag cement, or both	• Temperature sensors to be placed at the core and the edge, at a maximum of 2 inches from the center of the nearest concrete surface • 2 independent sets of sensors to be used • Temperatures should be recorded no less often than every 12 hours	• Maximum concrete placement temperature of 95°F • Maximum concrete temperature of 160°F • Maximum temperature difference of 35°F	• Thermal control plan must be submitted and approved by the engineer

Optional requirements for mass concrete in ACI 301 (2016) state that fly ash often makes up 40 to 50 percent of cementitious materials, whereas slag cement often makes up 65 to 75 percent.

3.1.2 AASHTO and FHWA

The FHWA does not have guidelines written specifically for mass concrete construction. Still, the FHWA Standard Specifications Book (FHWA, 2014) contains material requirements and temperature requirements for general concrete construction that applies to mass concrete. This information is shown in Table 3-2.

Table 3-2: FWHA FP-16 Specifications

Material Requirements	Temperature Monitoring System	Temperature Requirements
• Maximum fly ash replacement level 25% • Maximum slag cement replacement level 50%	• Maturity meter probes and temperature sensors to be installed in accordance with AASHTO T325	• Allowable concrete placement temperature 50 to 80 °F • Maximum concrete temperature 140 °F for high-strength concrete and 160 °F for prestressed concrete (no mass concrete designation) • Maximum temperature difference 35 °F • Concrete only to be placed when ambient temperature is between above 45 °F for cold-weather concreting and below 85 °F for hot-weather concreting

An investigation of mass concrete specifications and temperature requirements in the AASHTO Construction Specifications yielded no results. The only pertinent piece of information found was a maximum temperature requirement of 160 °F for precast construction (AASHTO, 2016).

3.1.3 U.S. States

A review of mass concrete specifications was performed for each state DOT. Table 3-3 summarizes which states contain specifications for mass concrete construction.

Table 3-3: U.S DOTs with Mass Concrete Specifications

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

Tables 3-4 through 3-10 summarize the mass concrete requirements for each individual DOT. Color-coded maps are also included to illustrate the mass concrete requirements across the U.S. DOTs. The following list highlights each pertinent topic in mass concrete specifications. This list also details which table or figure pertains to each topic:

- Type of document containing a mass concrete specification (Table 3-4),
- Least dimension for mass concrete elements (Table 3-5 and Figure 3-1),
- Cementitious material and chemical admixtures (Table 3-6),
- Temperature restrictions (Table 3-7),
    - Allowable temperature differences (Figure 3-2),
    - Maximum allowable concrete temperature (Figure 3-3),
- Temperature monitoring requirements (Table 3-8),
- Cooling system requirements (Table 3-9), and
- Thermal control plan (Table 3-10).

Table 3-4 – Mass Concrete Specification Reference Document

DOT	Reference Document
Arkansas	2014 Standard Specifications
California	2015 Standard Specifications
Florida	2013 Standard Specifications
Georgia	2013 Special Provision
Idaho	2012 Standard Specifications
Iowa	2012 Standard Specifications Amendment
Kentucky	2008 Standard Specifications (not found in 2012)
Louisiana	2013 Standard Specifications
Massachusetts	2012 Supplemental Specifications
Mississippi	2004 Standard Specifications
New Hampshire	2016 Standard Specifications
New Jersey	2007 Standard Specifications (updated 2016)
New York	2016 Standard Specifications
Ohio	2016 Construction and Material Specifications
Rhode Island	2013 Standard Specifications
South Carolina	2007 Standard Specifications
Texas	2014 Standard Specifications
Virginia	2007 Standard Specifications
West Virginia	2010 Special Provision

Table 3-5: Definition of Mass Concrete across U.S. States

DOT	Least dimension Requirement
California	• CIP pile with diameter > 8 ft (temperature monitoring only required if diameter > 14 ft) • All other elements with least dimension > 7 ft
Georgia	• Element with least dimension > 5 ft (> 6 ft for drilled shafts)
Idaho	• Element with least dimension > 4 ft
Iowa	• Footing with least dimension > 5 ft • Element with least dimension > 4 ft
Kentucky	• Element with least dimension > 5 ft (excluding drilled shafts)
Louisiana	• Element with least dimension > 4 ft
Ohio	• Drilled shaft with least dimension > 7 ft • Element with least dimension > 5 ft
Rhode Island	• Element with least dimension > 5 ft
South Carolina	• Circular element > 6 ft in length and > 5 ft in diameter • All other elements with least dimension > 5 ft
Texas	• Element with least dimension > 5 ft (excluding drilled shafts)
West Virginia	• Element with least dimension > 4 ft

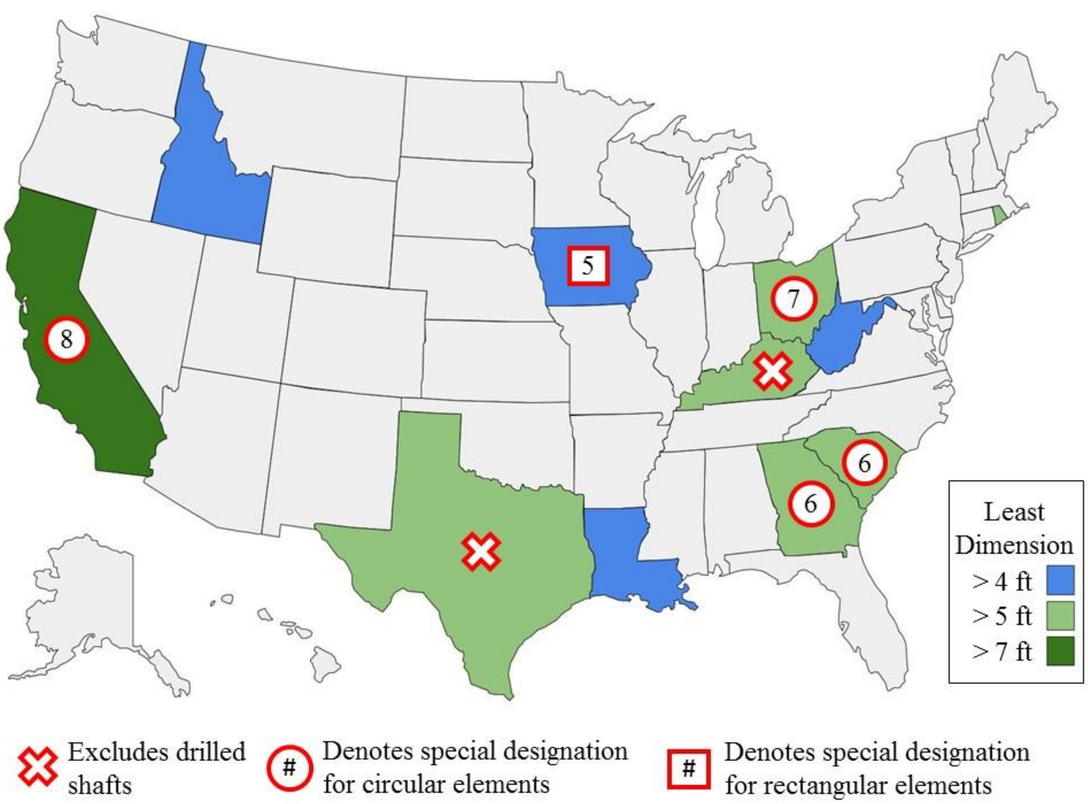

Figure 3-1: Mass Concrete Designation across U.S. DOTs

Table 3-6: State DOT Cementitious Material and Chemical Admixture Requirements

DOT	Material Requirements
Arkansas	• Type II (MH) cement • Maximum fly ash content of 120 pcy
California	• CIP piles with diameter > 8 ft require 25% fly ash replacement • Fly ash acceptable replacement range of 25-35% by weight • Slag cement acceptable replacement range of 50-75% by weight
Florida	• Type II MH cement • Fly ash acceptable replacement range 18-50% by weight (35-50% if core temperature expected to exceed 165°F) • Slag cement acceptable replacement range of 50-70% by weight • At least 20% fly ash and 40% portland cement required in a ternary mixture
Georgia	• Class F fly ash acceptable replacement range 25-40% • Slag cement replacement < 70% • Type III cement, Class C fly ash, silica fume, and metakaolin prohibited • Testing of aggregates for ASR susceptibility recommended
Iowa	• Minimum cementitious content of 560 pcy • w/c < 0.45 • Class C fly ash replacement < 20% • Total SCM replacement < 50% • Only Type I/II cement allowed • Air entrainment required
Kentucky	• Class F fly ash acceptable replacement range 25-30% • Slag cement replacement < 50%
Louisiana	• Fly ash replacement < 15% • Slag cement replacement < 50%
Massachusetts	• Recommends following mixing guide in special provision (not found)
Mississippi	• Class C concrete for massive reinforced section (w/c < 0.55)
New Hampshire	• Allowable slump = 1-3 inches
Ohio	• Minimum cementitious content of 470 pcy • Slag cement replacement < 50% • Type III cement not permitted • Accelerating admixtures not permitted
Virginia	• Minimum cementitious content of 494 pcy for massive lightly reinforced • Minimum cementitious content of 423 pcy for massive unreinforced
West Virginia	• Class F fly ash replacement < 25% • Slag cement replacement < 50% • Total SCM replacement < 50%

Table 3-7: State DOT Temperature Restrictions

DOT	Temperature Restrictions
Arkansas	• Allowable concrete placement temperature of 50-75 °F • Maximum temperature difference of 36 °F
California	• Maximum concrete temperature of 160 °F • Maximum temperature difference determined by TCP
Florida	• Maximum concrete temperature of 180 °F • Maximum temperature difference of 35 °F
Georgia	• Maximum concrete placement temperature of 85 °F unless approved by TCP • Maximum concrete temperature of 158 °F (should remain within 70 °F of mean annual ambient temperature) • Maximum temperature difference of 35 °F
Idaho	• Maximum temperature difference of 35 °F
Iowa	• Allowable concrete placement temperature of 40-70 °F • Maximum concrete temperature during heat dissipation 160 °F • Maximum temperature difference as a function of hours after placement of: 20 °F at 0-24 hrs, 30 °F at 24-48 hrs, 40 °F at 48-72 hrs, 50 °F after 72 hrs
Kentucky	• Maximum concrete placement temperature of 70 °F • Maximum temperature of 160 °F • Maximum temperature difference of 35 °F
Louisiana	• Maximum concrete placement temperature of 95 °F • Maximum concrete temperature of 160 °F • Maximum temperature difference of 35 °F
New Hampshire	• Maximum concrete temperature of 160 °F
New Jersey	• Maximum concrete temperature of 160 °F • Maximum temperature difference of 35 °F
New York	• Maximum temperature difference of 30 °F • Surface temperatures shall not drop more than 18 °F in a 24 hr period
Ohio	• Maximum concrete temperature of 160 °F • Maximum temperature difference of 36 °F
Rhode Island	• Maximum temperature difference of 70 °F
South Carolina	• Maximum concrete placement temperature of 80 °F • Maximum temperature difference of 35 °F
Texas	• Allowable concrete placement temperature of 50-75 °F • Maximum concrete temperature of 160 °F (TxDOT sponsored research by UT that determined 185 °F could be used for concrete containing SCMs) • Maximum temperature difference 35 °F
West Virginia	• Maximum temperature 160 °F • Maximum temperature difference 40 °F

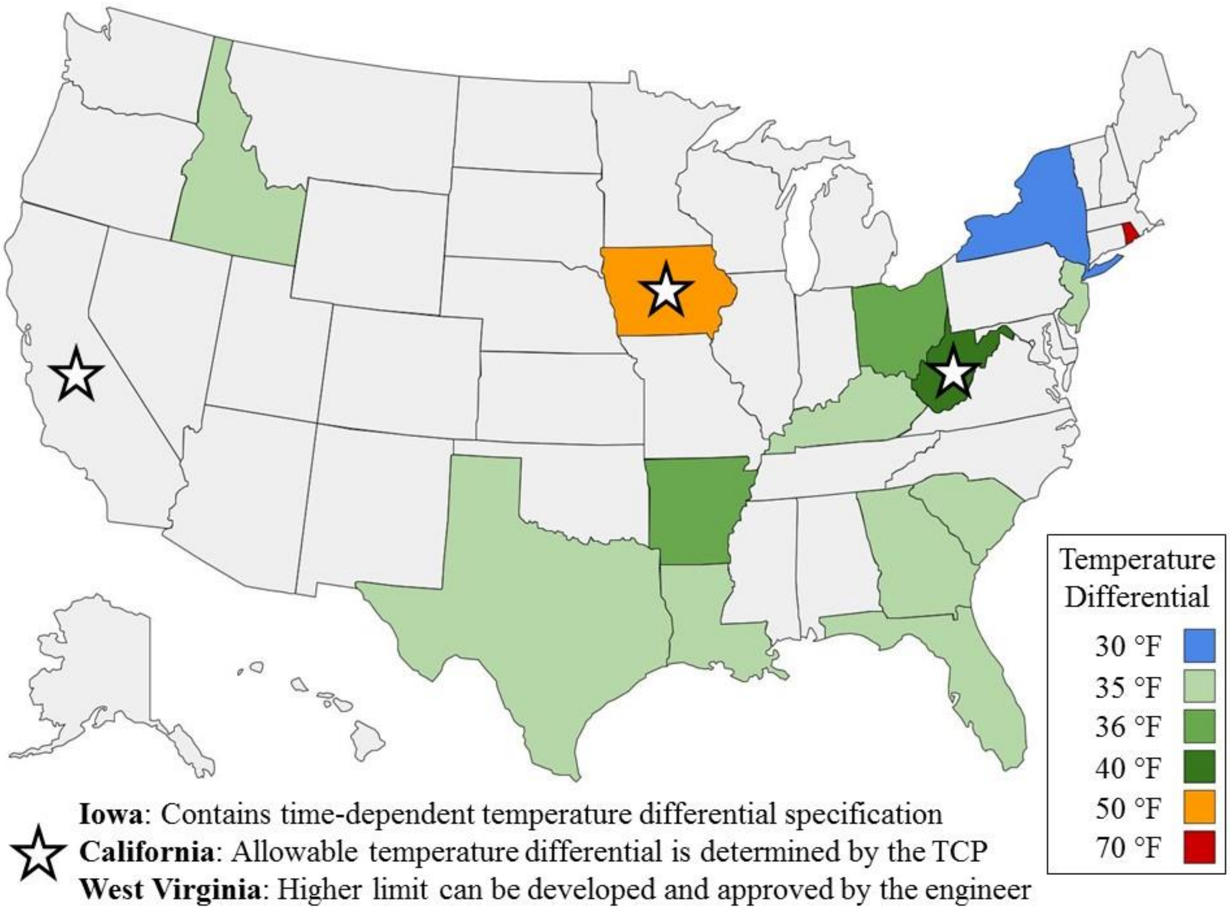

Figure 3-2: Allowable Temperature Differences across U.S. DOTs

The temperature difference limit imposed by the Iowa DOT, denoted by the star in Figure 3-2 and listed in Table 3-7, is particularly intriguing. Establishing a time-dependent temperature difference limit utilizes the increasing tensile strength of concrete. As the concrete matures and develops strength, it can withstand a higher temperature difference. A time-dependent temperature difference limit of this nature would ideally be included in a future ALDOT mass concrete specification. Research work performed to develop such a specification is detailed in this report.

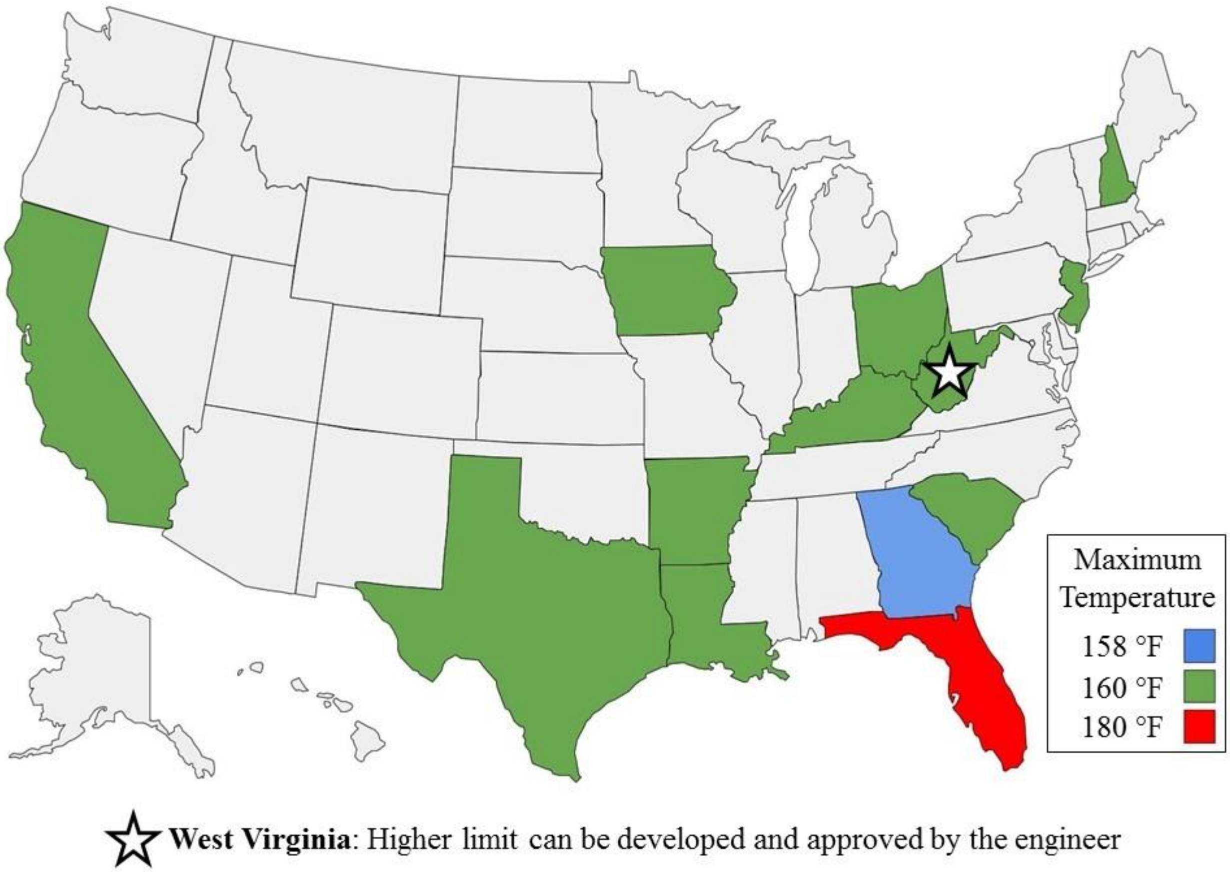

Figure 3-3: Maximum Temperature Limits across U.S. DOTs

Table 3-8: State DOT Temperature Monitoring Requirements

DOT	Temperature Monitoring Equipment
Arkansas	• Temperature to be monitored for 7 days
California	• Temperature to be recorded hourly until core temperature begins falling; at that time, temperature recording may be discontinued • Sensors placed in hottest location, outer edge, corner, and top
Florida	• Temperature recording interval no greater than 6 hours • Insulating materials outlined in TCP are not to be removed until core temperature is within 50 °F of ambient temperature • Measured concrete core and exterior surface temperatures approved by engineer
Georgia	• Temperatures to be recorded hourly (Engineer should be notified if core temperature reaches 140 °F or temperature difference reaches 30°F) • Sensor locations to be approved by the engineer • 2 independent sets of sensing devices in center, midpoint of side closest to center, midpoint of top surface, midpoint of bottom surface, and corner furthest from the center (edge sensors 2-6 inches from surface)
Idaho	• Temperature to be monitored for 7 days
Iowa	• Temperatures to be recorded every 4 hours and monitored by the engineer until core temperature is within 50 °F of ambient temperature • At least 10 sensors to be located at specified points and operating on 2 different systems (backup system required)
Kentucky	• Temperatures to be recorded every 4 hours and monitored by the engineer until core temperature is within 35 °F of ambient temperature • Temperature difference not to be recorded until 12 hrs after placement • 2 sensors installed in core and 2 sensors installed at the edge
Louisiana	• Temperature recording interval no greater than 6 hours • Sensors placed at center and edges using 2 independent systems
New Jersey	• Temperatures to be monitored for 15 days or until core temperature is within 35 °F of the lowest ambient temperature after placement
Ohio	• Temperatures to be monitored and regulated for 28 days after placement • 2 independent systems of sensors to be used
Rhode Island	• 2 independent systems of sensors to be used • Internal and external temperatures to be monitored
South Carolina	• Sensors placed at center of the placement and 2 inches from surface
Texas	• Temperature to be monitored for 4 days unless otherwise approved • 2 independent systems of sensors to be used • Edge sensors to be placed no more than 3 inches from the surface
West Virginia	• Temperature recording interval no greater than 6 hours • Sensors to be placed at the core, top, and sides of the placement

Table 3-9: State DOT In-Place Concrete Temperature Control Requirements

DOT	In-Place Concrete Temperature Control Requirements
Arkansas	• Forms should remain in place for at least 4 days anytime ambient temperature falls below 40 °F • All mass concrete shall be cured by free moisture; water curing shall be provided for all exposed surfaces for a period of 14 days
California	• Cooling pipes should be placed deeper than 4 inches from the surface
Rhode Island	• Side forms may be removed after 12 hrs, except when ambient temperature is below 50 °F
Texas	• Use only water curing for horizontal surfaces of mass concrete • Forms or insulating membranes must remain in place for 4 days
West Virginia	• Must be moist cured by means of moisture retention, water curing not permitted

Table 3-10: State DOT Thermal Control Plan

DOT	Thermal Control Plan
Arkansas	• TCP developed by contractor, approved by engineer
California	• TCP developed by contractor, approved/monitored by engineer
Florida	• TCP and mix designs developed by contractor, approved by engineer
Georgia	• TCP developed by contractor, approved by engineer 30 days prior to placement
Iowa	• TCP developed by contractor, approved by engineer (must be developed by licensed Temperature Control Engineer if element least dimension > 6.5 ft)
Kentucky	• TCP developed by contractor, approved/monitored by engineer
Louisiana	• TCP developed by contractor, approved by engineer
New Jersey	• TCP developed by contractor, approved by engineer 30 days prior to placement
Ohio	• TCP developed by contractor, approved by engineer 10 days prior to placement
Rhode Island	• TCP developed by contractor, approved by engineer
South Carolina	• Mass concrete placement plan prepared by contractor and submitted to engineer for approval (contains TCP, expected temperature development, and monitoring system details)
Texas	• Use ConcreteWorks or other approved method to develop TCP • If limits are exceeded, investigation must be performed by the engineer
West Virginia	• Temperature control requirements to be detailed by the engineer prior to construction

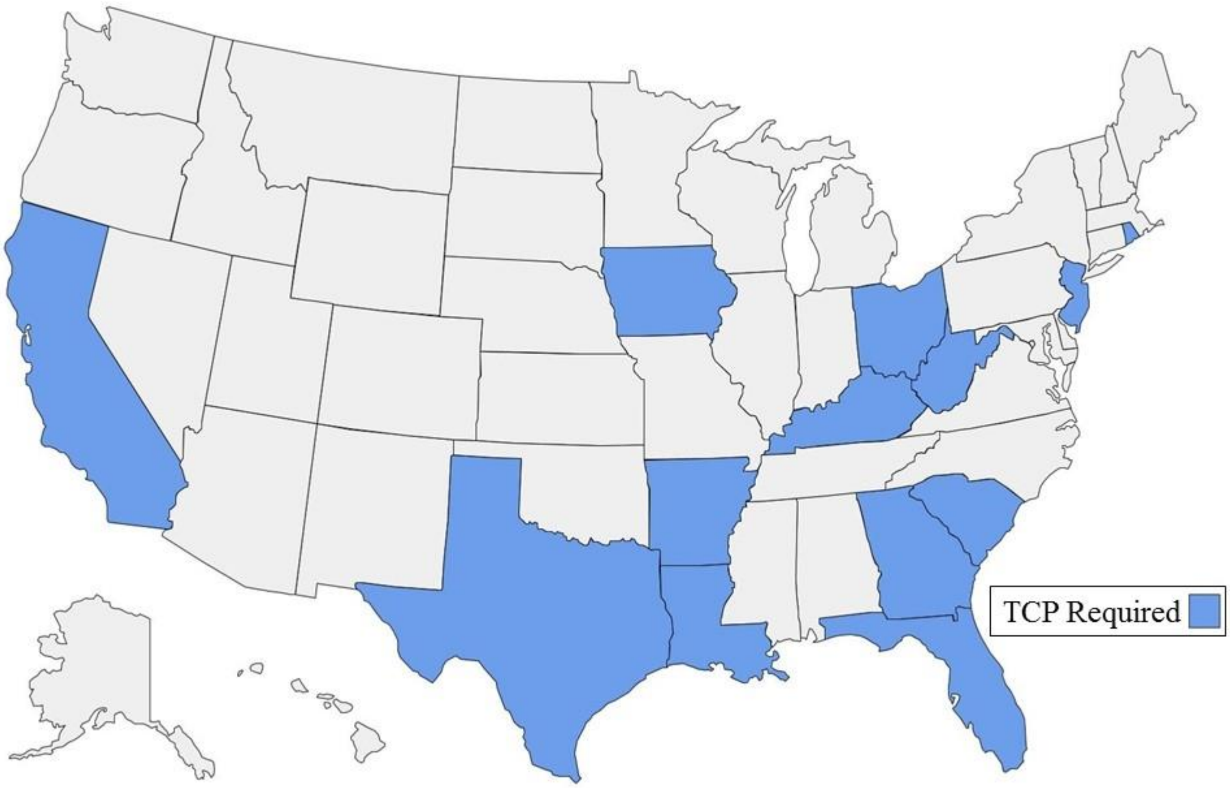

Figure 3-4: U.S. DOTs Requiring a TCP for Mass Concrete Construction

CHAPTER 4: EXPERIMENTAL PLAN

4.1 Introduction

The experimental plan for this project was developed to address each of the objectives outlined in Section 1.2. Based on the review of existing mass concrete literature and current ALDOT mass concrete practices, it was deemed worthwhile to provide guidance on how to effectively model, predict, and measure concrete temperatures in mass concrete elements. The research team developed effective methods and recommendations for accomplishing each of these tasks. This chapter details the software program, numerical models, laboratory mixing and testing procedures, and field instrumentation techniques used to execute the experimental plan.

4.2 Field Instrumentation

During this portion of the experimental plan, six ALDOT mass concrete bridge elements were instrumented with temperature sensors. The data from these temperature sensors aided in the completion of multiple project objectives from the Section 1.2. Most significantly, these elements were observed for signs of thermal cracking and early signs of DEF. Then, the temperature data from each element were compared with temperature predictions from ConcreteWorks. A study was performed on the accuracy of ConcreteWorks temperature predictions. Based on this information, temperature limits and temperature control requirements were recommended for potential inclusion in a future ALDOT mass concrete specification. The first ALDOT bridge element instrumented for this research was cast in July 2015. Final data recordings were obtained in May 2016.

The primary purpose of the field instrumentation phase is to measure concrete temperatures in ALDOT mass concrete elements with various shapes, sizes, coarse aggregate types, and placement conditions. Table 4-1 and Figure 4-1 summarize the location, size, and type of the elements instrumented for this project. The selection of elements was dictated by the availability of active ALDOT bridge construction projects. It was desired to obtain data for elements placed during both winter and summer conditions. This enabled researchers to assess the effect of placement temperature and ambient conditions on the temperature development of mass concrete elements. Additionally, elements with least dimension of 4 to 6 feet were targeted because this is the common range of mass concrete size designations in other DOT specifications (See Table 3-5 and Figure 3-1 of this report). Although the pedestal is much larger than 4 to 6 feet in least dimension, it provided a good example of a large element that would likely fall under a future ALDOT mass concrete designation.

Table 4-1: Size, Type, and Time of Year of Each Instrumented Element

City →			**Albertville**	**Harpersville**	**Scottsboro**		**Birmingham**	**Elba**
Placement Season	Summer		✓	✓	✓	✓		
	Winter						✓	✓
Element Type and Size	Footing	4-5 ft						
		5-6 ft						
		≥ 6 ft						
	Pedestal	10 ft			✓			
	Column	4-5 ft					✓	
		5-6 ft						
		≥ 6 ft						
	Bent Cap	4-5 ft						
		5-6 ft						✓
		≥ 6 ft	✓			✓		
	Wall	4 ft		✓				

Figure 4-1: Locations of Instrumented Elements

4.3 ConcreteWorks Analysis

4.3.1 Overview of ConcreteWorks Analysis

In this research project, ConcreteWorks was used to assess two main items. First, an analysis consisting of 480 placement scenarios was performed to determine which inputs had the greatest impact on maximum concrete temperature, maximum concrete temperature difference, and thermal cracking risk. For this analysis, rectangular columns were modeled because ConcreteWorks is capable of producing a cross sectional stress profile of rectangular columns by performing a 2-dimensional structural analysis on the cross section with non-material specific properties. A specialized version of ConcreteWorks was used that enables the user to download and analyze the 7-day stress profile at various locations along the cross section.

Second, each of the six ALDOT bridge elements instrumented with temperature sensors was modeled in ConcreteWorks. An assessment of the accuracy of ConcreteWorks predictions was performed based on the comparison of those predictions to the field instrumentation data.

4.3.2 Important Mass Concrete Variables

In mass concrete construction, certain variables have a more significant impact on concrete temperatures than others. For the purposes of this research, five inputs significant in mass concrete construction were isolated and varied. This allowed researchers to assess the effect of these five variables on temperature and cracking risk predictions:

- Coarse aggregate type,
- SCM type and replacement level,
- Placement location,
- Placement date, and
- Element size.

It is important to note that the concrete CTE was also varied according to the coarse aggregate type (see Section 2.2.4.3 of this report). A comprehensive mass concrete specification contains recommendations for materials, member size, temperature limits, and construction practices. By determining which inputs have the greatest impact on maximum concrete temperature, maximum concrete temperature difference, and cracking risk, ConcreteWorks can aid in the development of specification guidelines specific to different mass concrete placement scenarios. For instance, a mass concrete element cast during the winter may not have the same requirements as the same element cast during the summer, due to the fact that maximum concrete temperatures are likely to be lower for a winter placement.

4.4 Numerical Modeling

4.4.1 Introduction

As explained in Section 2.2, thermal cracks form when high temperature differences cause tensile stresses greater than the concrete's tensile strength. One objective of this project was to develop an improved method to determine the temperature difference to minimize the risk of thermal cracking. This portion of the experimental plan addresses that objective.

In order to develop a time-dependent temperature difference limit, a thermal stress equation was developed. This equation replaces the constant value for tensile strain capacity, as in Equation 2-1, with time-dependent functions for tensile strength and concrete modulus of elasticity. Thus, Equation 4-1 is an adapted version of Equation 2-1, isolating the allowable temperature difference and using concrete modulus of elasticity to convert tensile strain into tensile stress.

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

Where,

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

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

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

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

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

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

Instead of using a constant tensile strain capacity (ε_{tsc}) in Equation 2-1, Equation 4-1 uses time-dependent models for concrete tensile strength and stiffness. The creep function is used in Equation 4-1 to account for the fact that the effects of creep and relaxation also vary with time during the early ages after concrete placement. By replacing constant values for strain capacity and creep with time-dependent strength, stiffness, and creep functions, a time-dependent function for the maximum allowable concrete temperature difference was calculated.

For the purposes of this research, a time-dependent creep factor was calculated for comparison to the constant value of 0.65 used in the thermal cracking equation by Bamforth (2007). In order to calculate a time-dependent creep factor, the B3 model's compliance function described in Section 2.2.2.1 was used. The compliance function and stiffness function were then used to quantify the B3 creep coefficient. The B3 creep coefficient was then converted into a creep modification factor for application the thermal cracking equation. The creep modification factor adjusts the concrete modulus of elasticity to account for the effects of creep at early ages (ACI 209, 1982). This calculation is shown in Equation 4-2.

$$K(t) = \frac{1}{1 + \varphi(t, t_o)}$$ **Equation 4-2**

Where,

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

$K(t)$ = creep modification factor

As mentioned in Section 2.2.2.1, the Modified B3 model developed by Byard and Schindler (2015) was used to improve the accuracy of compliance calculations at early ages. Effectively modeling the creep effects using this modified B3 model was essential for thermal stress calculations, as thermal cracking is most prominent during concrete's early ages. Once time-dependent creep factors for both the original B3 model and modified B3 model were determined, time-dependent temperature difference limits for each model were then calculated and compared to the limits calculated when using a constant "K" value of 0.65.

After determining viable models for creep, strength, and stiffness, age-dependent temperature difference limits were produced based upon CTE values for concretes containing both river gravel and limestone coarse aggregates. Based on these limits, researchers were able to provide recommendations for a time-dependent temperature difference limit similar to that of the Iowa DOT (see Table 3-7) and specific to the coarse aggregate type.

4.5 Temperature Sensors

4.5.1 Assembling Temperature Sensors

Temperature sensors were assembled in the AU Structural and Concrete Materials Research Laboratory. The type of sensor selected was the DS1921G Thermochron iButton Device by Maxim Integrated. This particular device was selected because it is capable of both recording and storing up to 2048 data points. The iButton's stainless steel construction protects it against the forces and pressures experienced during concrete placement. The DS1921G operates within a range of -40 °F to 185 °F, which contains the range of concrete temperatures expected in the elements instrumented. The OneWire Viewer software associated with this sensor is free and easy to navigate. The software allows the user to program the start time and end time of temperature recording, as well as the frequency of recording. The "synchronize with real-time clock" feature associates an exact timestamp with each data point. This allows the user to examine the effects of ambient temperatures on concrete temperatures when processing the data.

Figure 4-2 displays each piece of the temperature sensor assembly. Pictured (from left to right) are a coated iButton sensor, U.S. nickel (for size reference), iButton, RJ-11 with two-wire cable connected, USB reader, and battery clip.

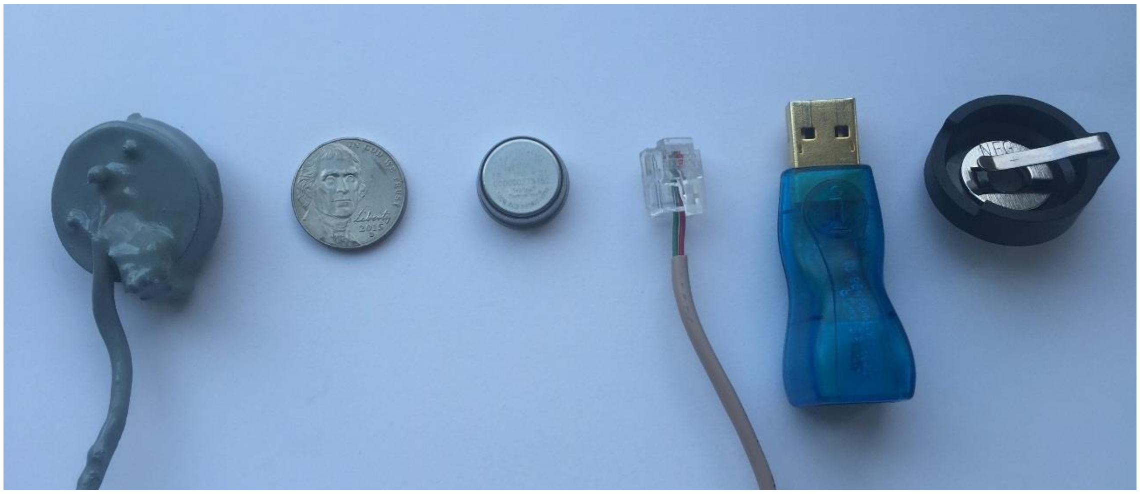

Figure 4-2: iButton Assembly

In order to access the data after the concrete was cast, a simple two-wire cable must be attached to the iButton. Because soldering to the stainless steel sensor is difficult, the sensors were placed in a Keystone Battery Holder used to hold 24 mm cell batteries. Once the sensor was placed in the holder clip, the wires were soldered to each side of the sensor. The other end of the wire was attached to a RJ-11 jack. The RJ-11 was then plugged directly in a USB Reader that is used to view the temperature data in the OneWire Viewer software.

To protect the sensor during fresh concrete placement, the sensor and clip are coated in a Static Control Epoxy Coating (GP 3525) by Sherwin Williams©. It is recommended to keep the epoxy from becoming too hot while curing. If high temperatures occur, the epoxy hardens before the sensors can be appropriately coated.

4.5.2 Programming and Installing Temperature Sensors

Once the sensors have been assembled in the laboratory, they are ready to be placed in the field. In this project, it is most effective to program the sensors before installing them in the field. This is possible because of a “mission start delay” feature in the One Wire Viewer software. A user can simply program the iButton to begin recording data shortly before the concrete is placed. If the placement time is changed, a user can simply re-program the iButton. Temperatures were recorded every 15 minutes for the first 14 days after placement. At this interval, the iButton has enough memory to store data for up to 21 days and 8 hours. After 14 days, researchers returned to the site to retrieve the 14-day data. Upon leaving the site, the recording interval was changed to 3 to 4 hours, ensuring that temperature was recorded at the time of peak temperature (approximately 3:00 PM) each day. At recording intervals of 3 and 4 hours there is enough memory to store data for up to 256 and 341 days, respectively.

Installation of the temperature sensors is unique to each element, as sensors were placed in different locations depending on the element type. It is easiest to install temperature sensors after the reinforcement cage has been tied, but before all sides of the formwork have been erected. For each element, two complete cross sections of sensors were installed to provide redundancy in the temperature monitoring system. Furthermore, two core sensors were installed in each cross section, as core temperature data are the most crucial to both maximum temperature and temperature difference calculations. In order to provide ALDOT recommendations on the proper placement methods and locations for “edge sensors,” iButtons were placed both on the reinforcing bars and in the cover area near the edge of the formwork. This was accomplished by extending edge sensors away from the rebar using pieces of FRP rebar from the laboratory. Figure 4-3 is an example of edge and rebar sensors installed in a crashwall in Harpersville, Alabama.

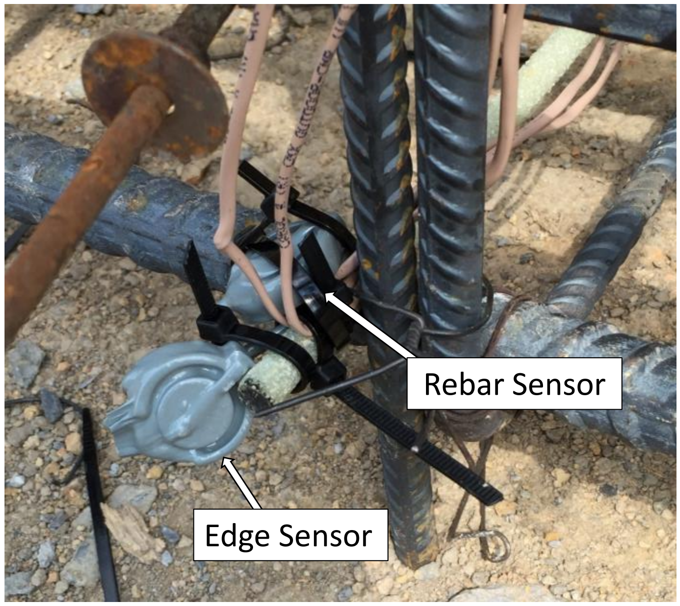

Figure 4-3: Rebar and Edge Sensors in Harpersville Crashwall

4.6 Laboratory Testing of Site Materials

Laboratory testing of raw concrete materials from each site was performed in the AU Structural and Concrete Materials Research Laboratory. Each of the tests described in this section was run to obtain an input necessary for ConcreteWorks. As discussed in Section 2.6.1, modifying default input values with values specific to each element can greatly improve the accuracy of ConcreteWorks temperature predictions. Each batch of field-collected raw materials was proportioned for a total of 1.5 ft^3 of concrete and mixed in a 3 ft^3 mixer.

4.6.1 Raw Materials

Raw materials were collected from the batch plant that supplied the concrete for each instrumented element. The concrete mixture proportions were the approved project mixture proportions obtained from the ALDOT engineer for each site. The materials were then sealed and stored in the lab until mixing.

4.6.2 Mixing Procedure

Each laboratory mixture was prepared in accordance with ASTM C 192 (2007) using the following summarized procedure:

- Prior to mixing, the mixer was thoroughly "buttered" with a layer of mortar,
- Once the mortar was emptied from the mixer, all aggregates and 80 percent of mix water was mixed for two minutes,
- Cementitious materials (cement and fly ash) were then added to the mixer along with the remaining 20 percent of mix water, and
- All materials were then mixed for three minutes, rested for three minutes, and mixed for a final two minutes.

A slump test was then performed according to AASHTO T 119 (2007) guidelines, and an air content test was run according to AASHTO T 152 (2005) guidelines. If these values were acceptably close to the values recorded on site during the concrete placement, the batch was used to produce cylinders for each specific test in accordance with ASTM C 192 (2010).

4.6.2.1 Semi-Adiabatic Calorimetry

Once the concrete was mixed, one 6 x 12 in. cylinder was prepared for semi-adiabatic calorimetry in a Q-drum. The temperature of the concrete and the weight of the cylinder were then recorded, and the cylinder was placed in the Q-drum testing apparatus. Each Q-drum test lasted a duration of seven days. At the end of the test, the adiabatic temperature data were saved. Finally, heat of hydration models developed by Schindler and Folliard (2005) were used to calculate the necessary ConcreteWorks hydration parameters.

The Q-drum test was performed for each batch in accordance with the guidelines provided by iQuadrel Services. The Q-drum testing apparatus used during this research is in Figure 4-4.

Figure 4-4: Q-Drum Testing Apparatus

4.6.2.2 Compressive Strength and Modulus of Elasticity Test

In order to determine the 28-day compressive strength and stiffness modulus, three 6 x 12 in. cylinders were prepared. At 28 days, they were tested in accordance with ASTM C 39 (2010) and ASTM C 469 (2010) to determine their compressive strength and modulus of elasticity, respectively.

4.6.2.3 CTE Test

The final test was the CTE test, run in accordance with AASHTO T 336-11 specifications. In order to perform this test, two 4 x 8 in. cylinders were prepared for each batch. In this test, the concrete samples' temperature is cycled between 50 °F (10°C) and 122 °F (50°C). The length change of the sample is measured, which allows the CTE to be calculated. Because this test is non-destructive, it can be run as many times as necessary to verify the results. Figure 4-5 shows the CTE testing apparatus using during this research.

Figure 4-5: CTE Testing Apparatus

CHAPTER 5: PRESENTATION AND DISCUSSION OF INSTRUMENTATION DATA

5.1 Introduction

Chapter 5 of this report focuses on project tasks associated with the instrumentation of ALDOT mass concrete elements with temperature sensors. The field-measured temperature data for six ALDOT mass concrete elements, as well as the discussion and analysis of these data are presented in this chapter.

5.2 Field Instrumentation Data

5.2.1 Field Instrumentation Overview

For each element, the following information is presented in either tabular or graphic form:

- General site information,
- Sensor layout diagram,
- 14-day concrete temperatures (core and edge),
- 14-day concrete temperature differences,
- Long-term concrete temperatures (core and edge), and
- Laboratory testing results

All of this information, as well as post-placement site observations, is included in this chapter. All site information, temperature data, and laboratory testing results not included in this chapter can be found in Appendix A through F. The following list contains the section and appendix designated for each instrumented element:

- Albertville Bent Cap (Section 5.2.2 and Appendix A),
- Harpersville Crashwall (Section 5.2.3 and Appendix B),
- Scottsboro Pedestal (Section 5.2.4 and Appendix C),

- Scottsboro Bent Cap (Section 5.2.5 and Appendix D),
- Elba Bent Cap (Section 5.2.6 and Appendix E), and
- Birmingham Column (Section 5.2.7 and Appendix F).

With regard to the sensor layout diagrams, the number of sensors placed into each element was determined based on the element type. The bent cap and column contained six sensors per cross section, while the crashwall and pedestal contained eight sensors per cross section. The two additional sensors in the crashwall and pedestal were installed in the bottom corner of the element, as these elements were cast on top of previous concrete placements that provided continuous external restraint along the bottom of the element. This concept is demonstrated in Figure 5-1.

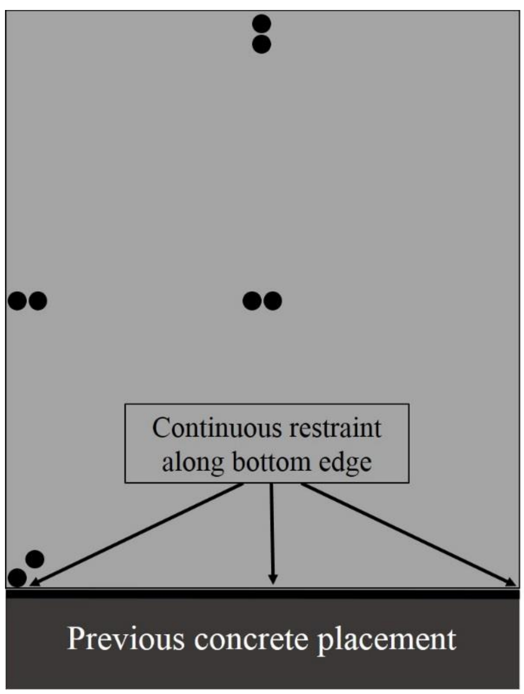

Figure 5-1: Continuous Restraint along Bottom Edge of Crashwall and Pedestal

In order to ensure the collection of data from at least one core sensor and one edge sensor, two identical cross sections of sensors were installed into each element. Additionally, because the core sensor is the most important for analysis purposes, two sensors were placed at the core of each cross section. A sensor layout diagram illustrates the sensor locations of one cross section of each element. When viewing each sensor layout diagram, it is important to note that the sensors with white fill are the sensors whose data are shown in the temperature graphs for the corresponding element.

5.2.2 Albertville Bent Cap

Figure 5-2: Photograph of Albertville Bent Cap

5.2.2.1 Site Details

Table 5-1: Albertville Bent Cap Site Information

Placement Date	7/31/2015
Placement Time	6:00 A.M.
Placement Location	Albertville, Alabama
Member Type	Rectangular bent cap
Member Dimensions	6.5’ × 6.5’ × 40’
Cement Type	I/II
Total Cementitious Materials Content (pcy)	567
SCM Type (% Replacement)	Class F Fly Ash (25%)
Coarse Aggregate Type	Limestone
Form Type	Wood

5.2.2.2 Sensor Layout

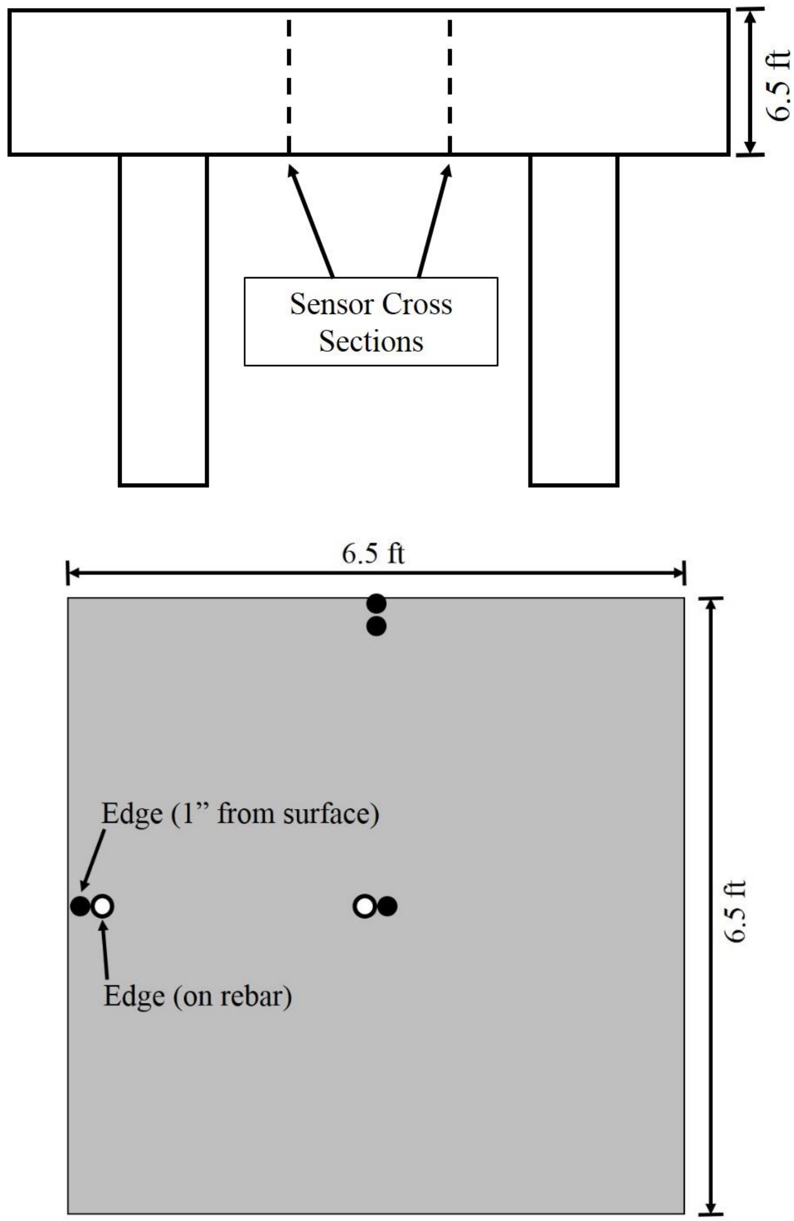

Figure 5-3: Albertville Bent Cap Elevation View (top) and Cross Section (bottom)

5.2.2.3 14-Day Temperature Data

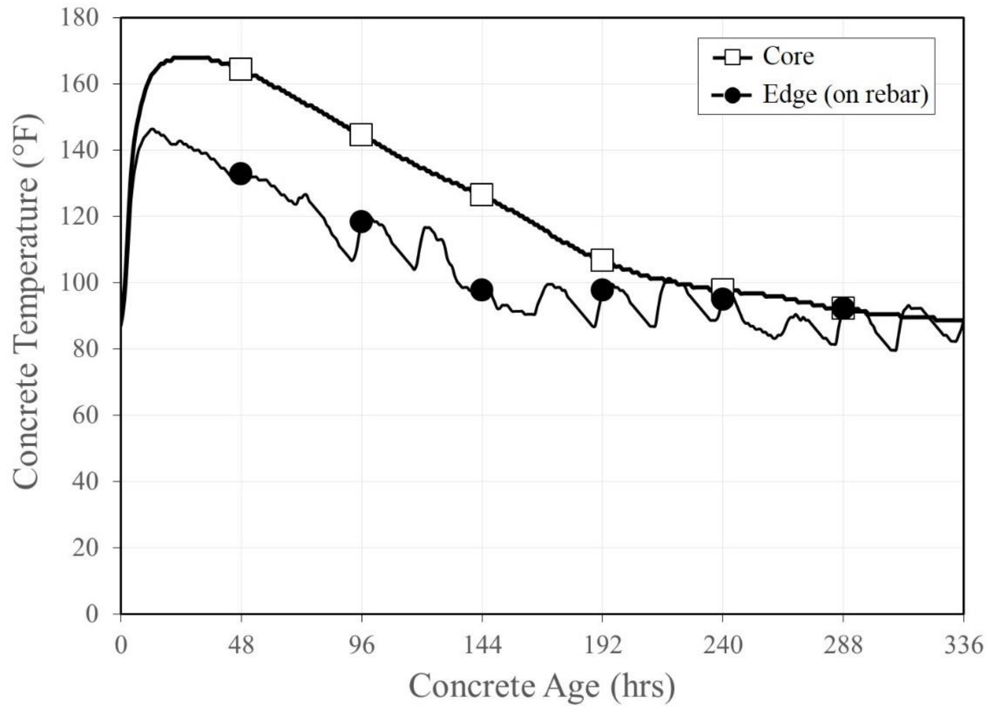

Figure 5-4: Albertville Bent Cap 14-Day Concrete Temperature Data

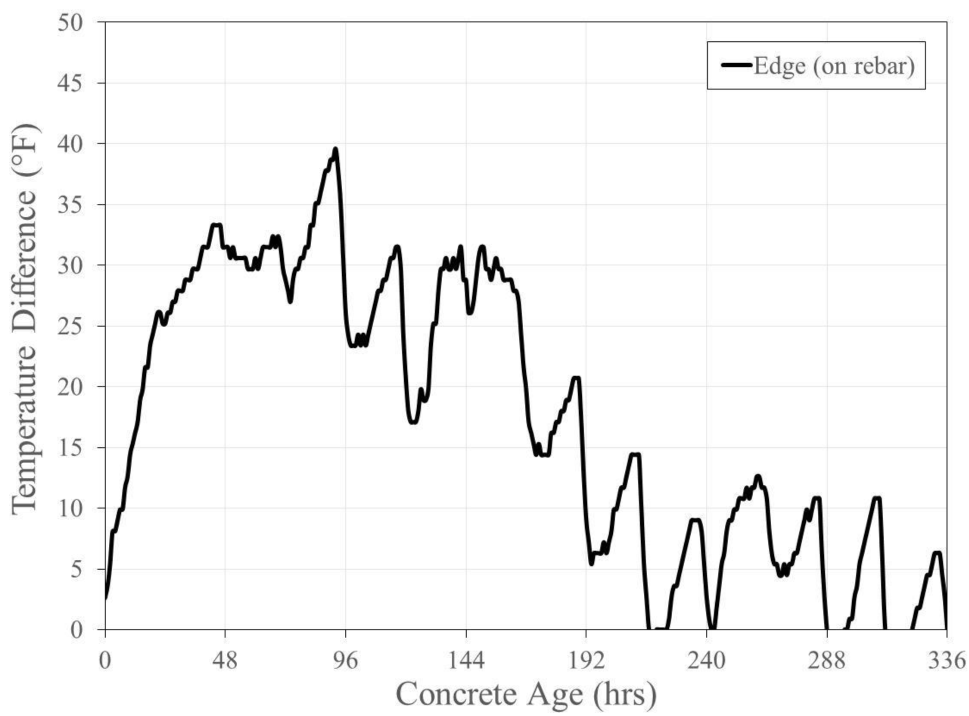

Figure 5-5: Albertville Bent Cap 14-Day Concrete Temperature Difference Data

5.2.2.4 Long-Term Temperature Data

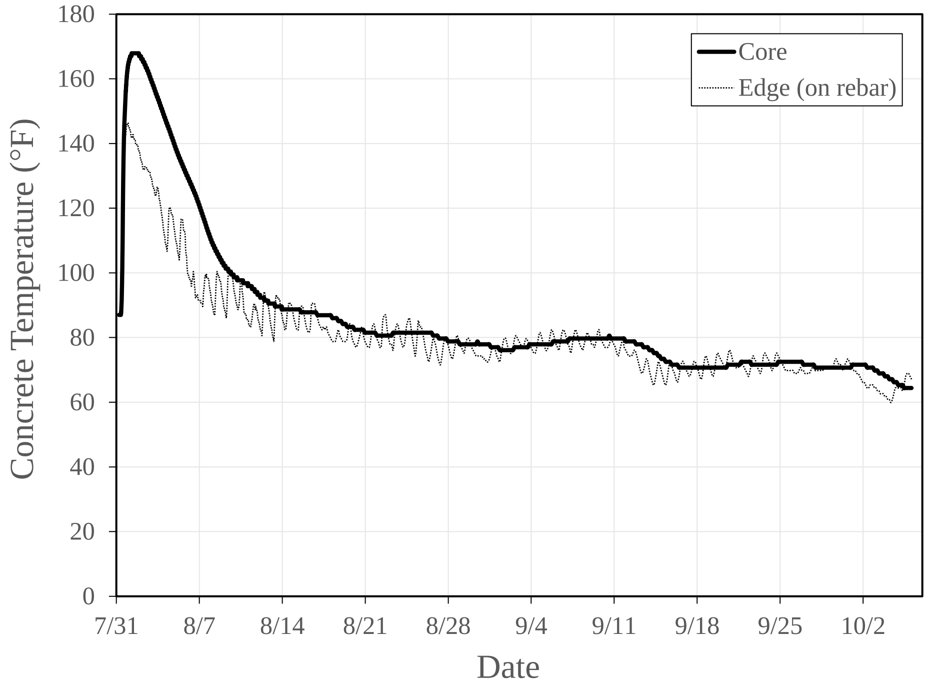

Figure 5-6: Albertville Bent Cap Long-Term Concrete Temperature Data

5.2.2.5 Post-Instrumentation Site Observations

Fourteen days after construction a site observation took place on August 14, 2015. At that time, no signs of thermal cracking or DEF were found. A final site visit took place on October 6, 2015. Again, no signs of temperature-related distresses were observed. The sensor lead-wires were cut during the final site visit, as there would soon be no man-lift on site and no access to the sensors.

5.2.2.6 Laboratory Testing Results

Table 5-2: Albertville Bent Cap Laboratory Testing Results

<table>
<tr><td rowspan="5">Q-Drum</td><td>Total Heat of Hydration, H_u (J/kg)</td><td>375,000</td></tr>
<tr><td>Activation Energy, E (J/mol)</td><td>31,800</td></tr>
<tr><td>Hydration Slope Parameter, β</td><td>1.574</td></tr>
<tr><td>Hydration Time Parameter, τ (hr)</td><td>15.74</td></tr>
<tr><td>Ultimate Degree of Hydration, α_u</td><td>0.846</td></tr>
<tr><td colspan="2">Concrete CTE (in./in./°F × 10^6)</td><td>4.82</td></tr>
<tr><td colspan="2">28-day Concrete Mean Compressive Strength, f_{cm28} (psi)</td><td>5,300</td></tr>
<tr><td colspan="2">28-day Concrete Modulus of Elasticity, E_{cm28} (psi)</td><td>4,750,000</td></tr>
</table>

5.2.3 Harpersville Crashwall

Figure 5-7: Photograph of Harpersville Crashwall

5.2.3.1 Site Details

Table 5-3: Harpersville Crashwall Site Information

Placement Date	8/24/2015
Placement Time	10:20 A.M.
Placement Location	Harpersville, Alabama
Member Type	Crashwall
Member Dimensions	48’ × 4’ × 10’
Cement Type	I/II
Total Cementitious Materials Content (pcy)	535
SCM Type (% Replacement)	Class C Fly Ash (20%)
Coarse Aggregate Type	Limestone
Form Type	Wood

5.2.3.2 Sensor Layout

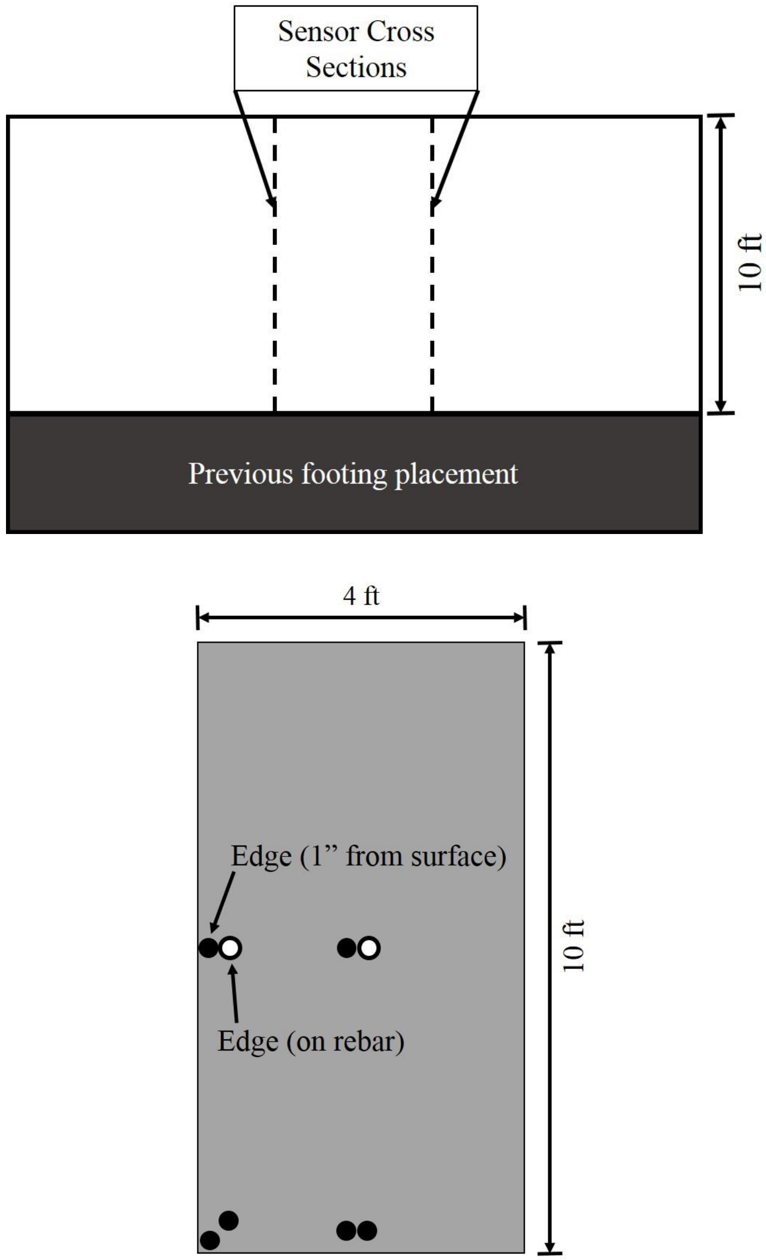

Figure 5-8: Harpersville Crashwall Elevation View (top) and Cross Section (bottom)

5.2.3.3 14-Day Temperature Data

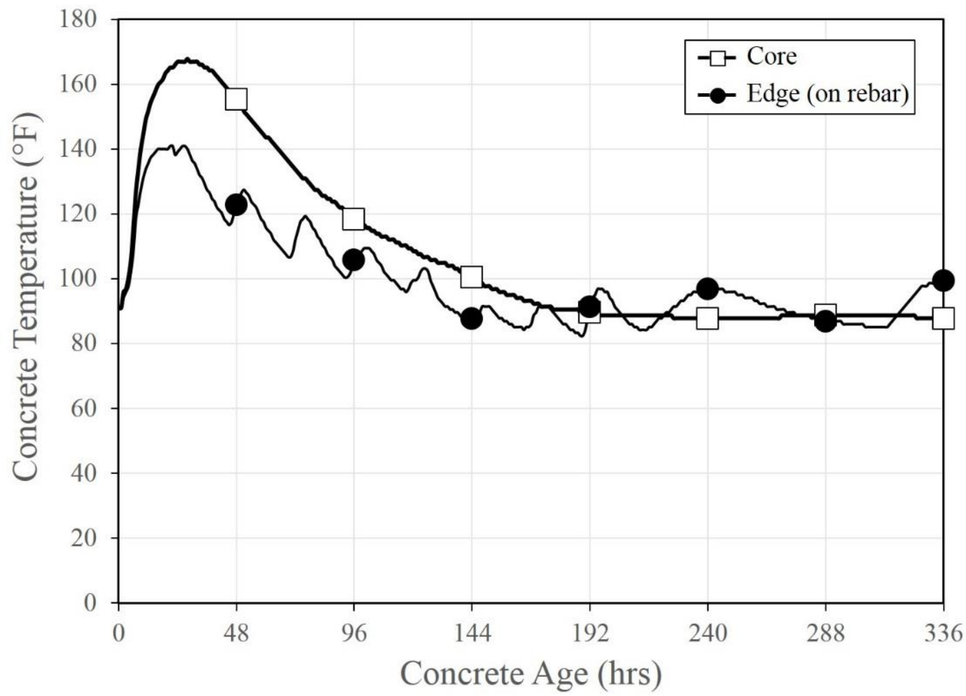

Figure 5-9: Harpersville Crashwall 14-Day Concrete Temperature Data

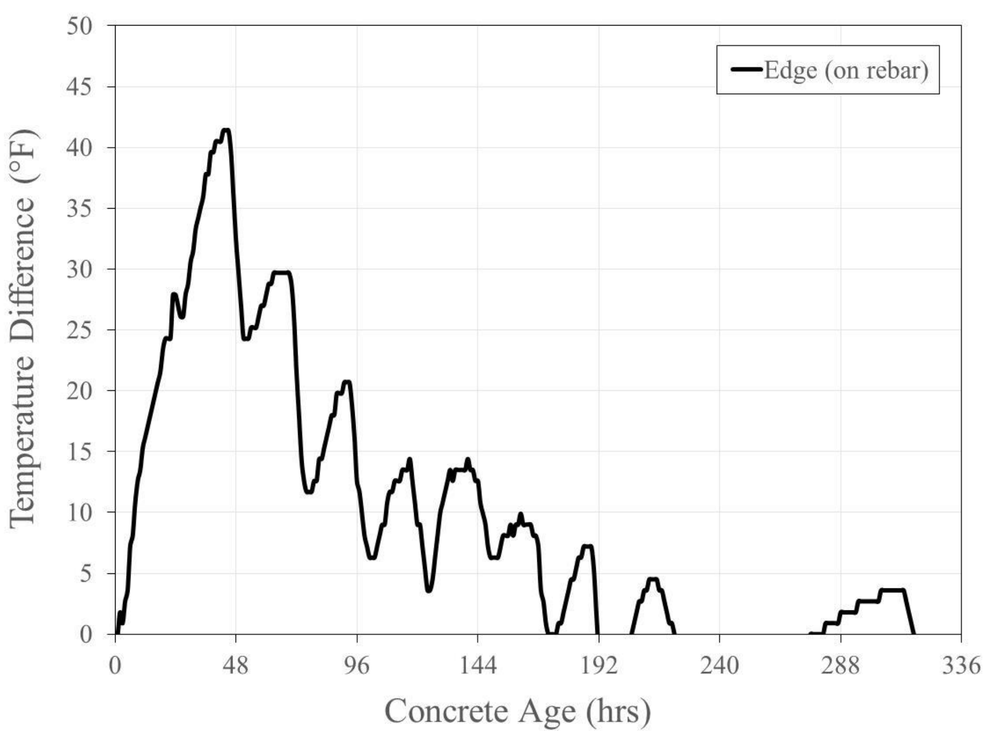

Figure 5-10: Harpersville Crashwall 14-Day Concrete Temperature Difference Data

5.2.3.4 Long-Term Temperature Data

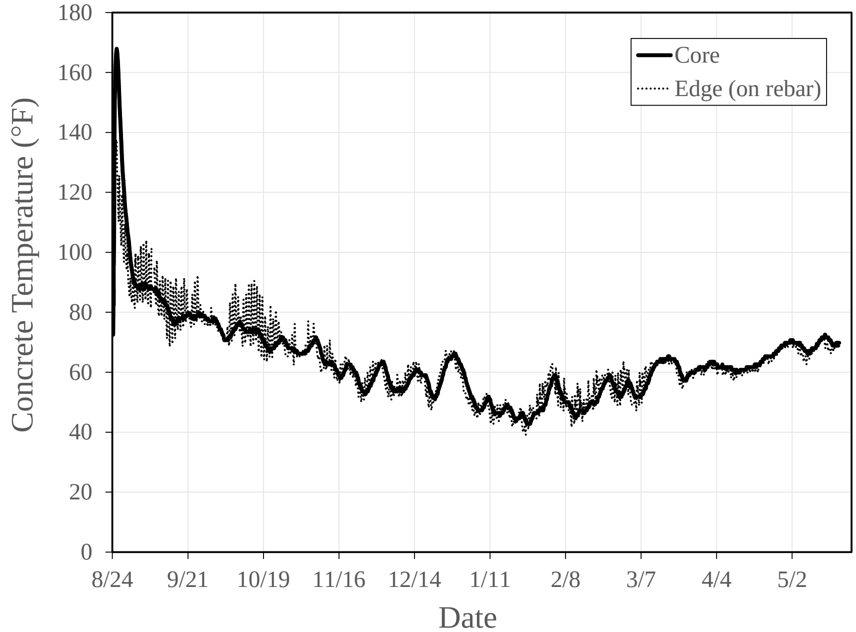

Figure 5-11: Harpersville Crashwall Long-Term Concrete Temperature Data

5.2.3.5 Post-Instrumentation Site Observations

Fourteen days after construction a site observation took place on September 7, 2015. At that time, no signs of thermal cracking or DEF were found. Additional visits took place on January 18, 2016 and May 19, 2016. Again, no signs of temperature-related distresses were observed during either of these visits. As of the last site visit, only one cross section of sensor lead-wires were accessible due to the addition of rip-rap at the base of the element.

5.2.3.6 Laboratory Testing Results

Table 5-4: Harpersville Crashwall Laboratory Testing Results

Q-Drum	Total Heat of Hydration, H_u (J/kg)	437,500
	Activation Energy, E (J/mol)	34,500
	Hydration Slope Parameter, β	1.138
	Hydration Time Parameter, τ (hr)	13.69
	Ultimate Degree of Hydration, α_u	0.758
Concrete CTE (in./in./°F × 10^6)		4.47
28-day Concrete Mean Compressive Strength, f_{cm28} (psi)		6,200
28-day Concrete Modulus of Elasticity, E_{cm28} (psi)		6,160,000

5.2.4 Scottsboro Pedestal

Figure 5-12: Photograph of Scottsboro Pedestal

5.2.4.1 Site Details

Table 5-5: Scottsboro Pedestal Site Information

Placement Date	9/3/2015
Placement Time	10:20 AM – 3:55 PM
Placement Location	Scottsboro, Alabama
Member Type	Pedestal
Member Dimensions	10’ × 12.5’ × 34’
Cement Type	I/II
Total Cementitious Materials Content (pcy)	620
SCM Type (% Replacement)	Class F Fly Ash (20%)
Coarse Aggregate Type	Limestone
Form Type	Steel

5.2.4.2 Sensor Layout

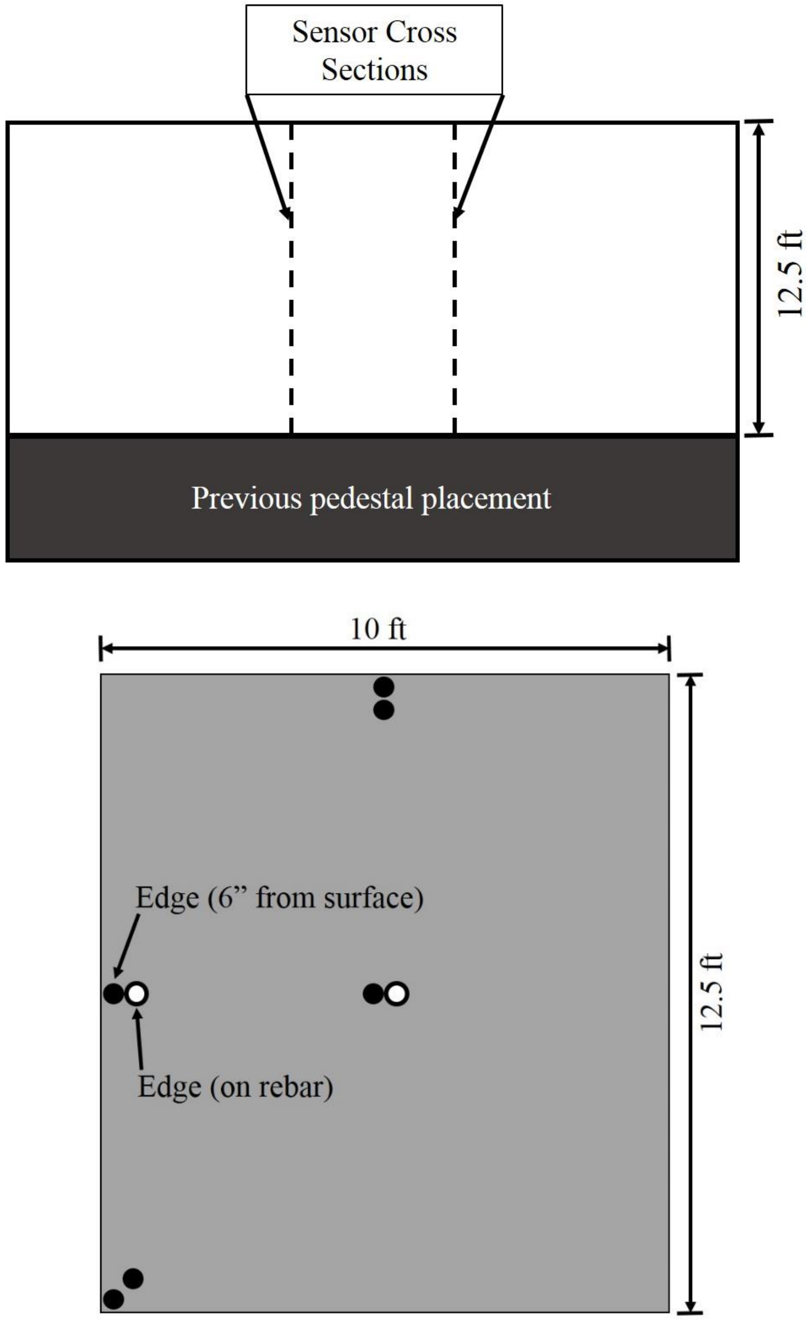

Figure 5-13: Scottsboro Pedestal Elevation View (top) and Cross Section (bottom)

5.2.4.3 14-Day Temperature Data

For scheduling reasons, the early age temperature data was actually obtained thirteen days after placement, instead of fourteen days.

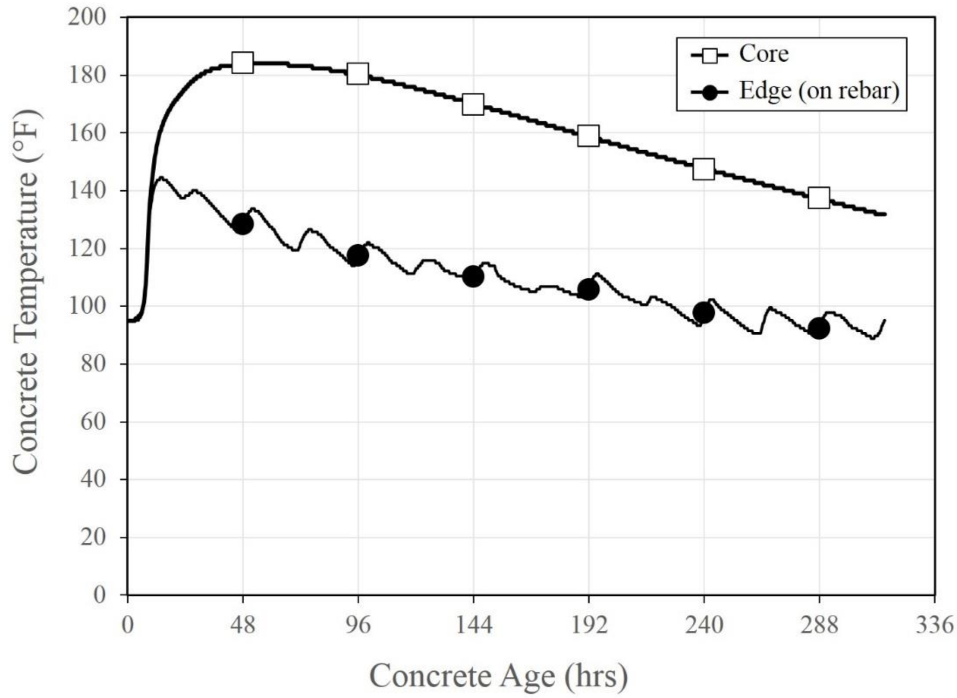

Figure 5-14: Scottsboro Pedestal 14-Day Concrete Temperature Data

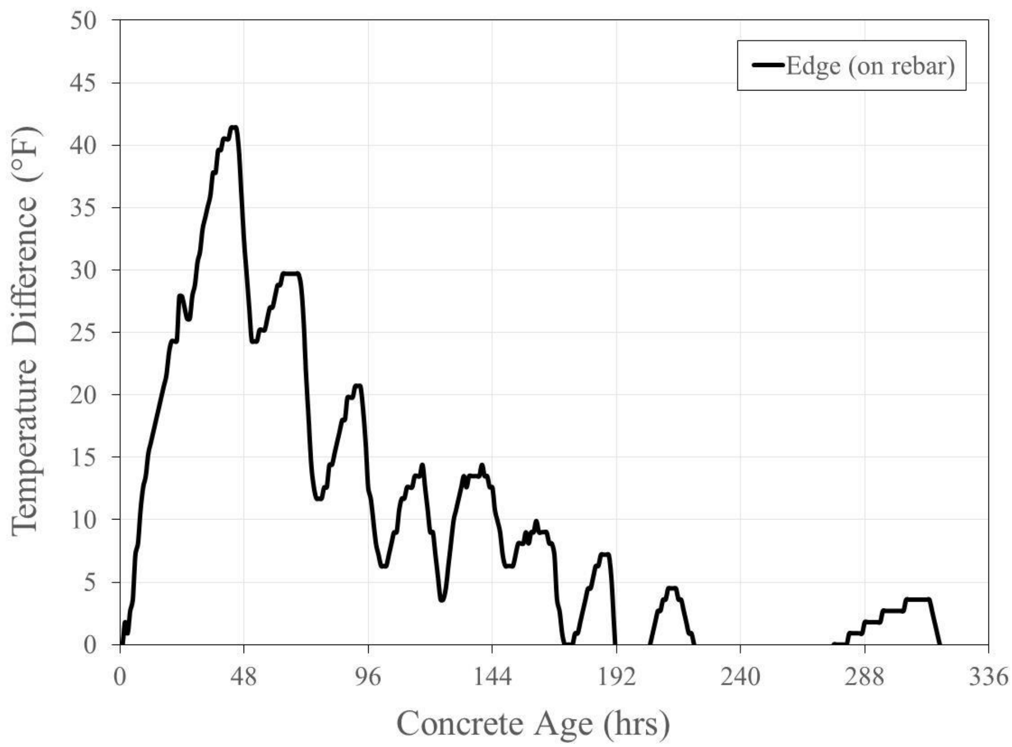

Figure 5-15: Scottsboro Pedestal 14-Day Concrete Temperature Difference Data

5.2.4.4 Long-Term Temperature Data

The lead wires were cut by construction workers sometime after the 14-day data were collected. For this reason, no long-term temperature data were collected for the Scottsboro pedestal.

5.2.4.5 Post-Instrumentation Site Observations

Thirteen days after construction a site observation took place on September 16, 2015. At that time, no signs of thermal cracking or DEF were found. A final site visit took place on May 19, 2016. Again, no signs of temperature-related distresses were observed. The sensor lead-wires were already cut by construction workers, preventing the collection of further long-term temperature data.

5.2.4.6 Laboratory Testing Results

Table 5-6: Scottsboro Pedestal Laboratory Testing Results

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

5.2.5 Scottsboro Bent Cap

Figure 5-16: Photograph of Scottsboro Bent Cap

5.2.5.1 Site Details

Table 5-7: Scottsboro Bent Cap Site Information

Placement Date	9/18/2015
Placement Time	11:00 A.M.
Placement Location	Scottsboro, Alabama
Member Type	Rectangular bent cap
Member Dimensions	6.5’ × 7.5’ × 41’
Cement Type	I/II
Total Cementitious Materials Content (pcy)	620
SCM Type (% Replacement)	Class F Fly Ash (20%)
Coarse Aggregate Type	Limestone
Form Type	Steel

5.2.5.2 Sensor Layout

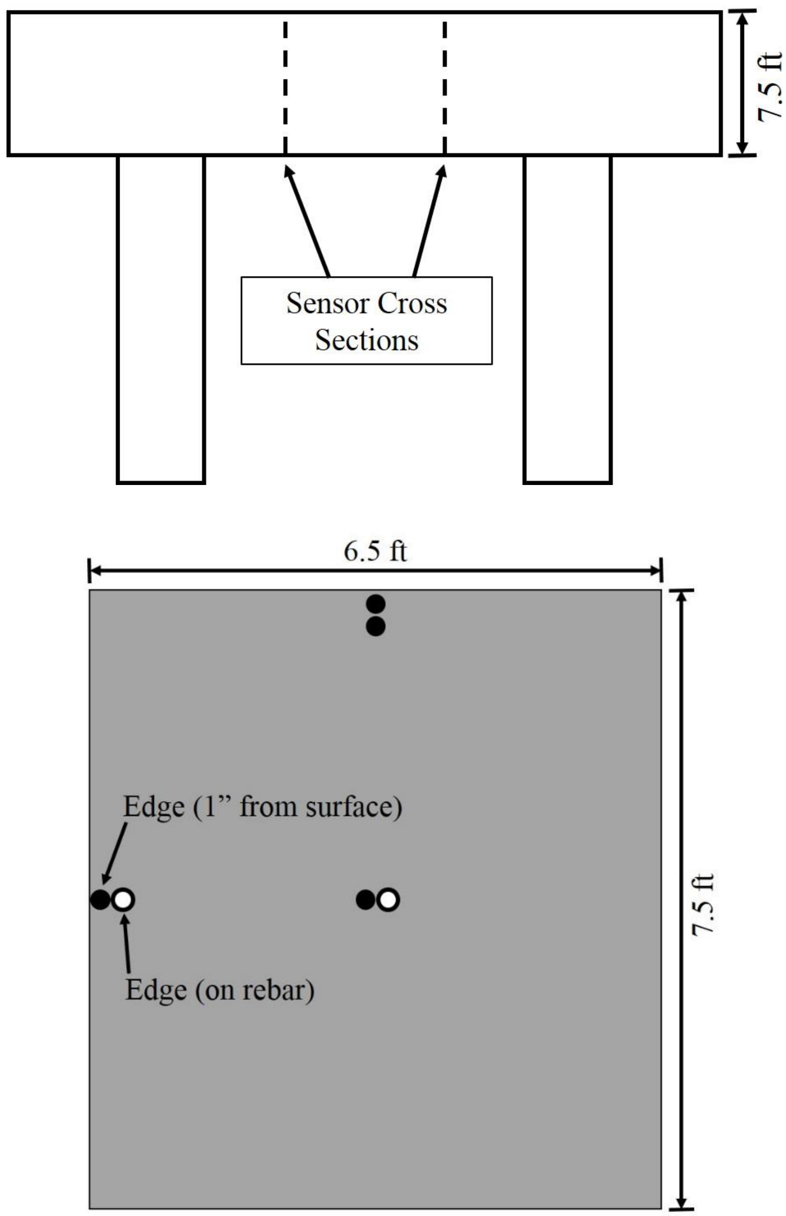

Figure 5-17: Scottsboro Bent Cap Elevation View (top) and Cross Section (bottom)

5.2.5.3 14-Day Temperature Data

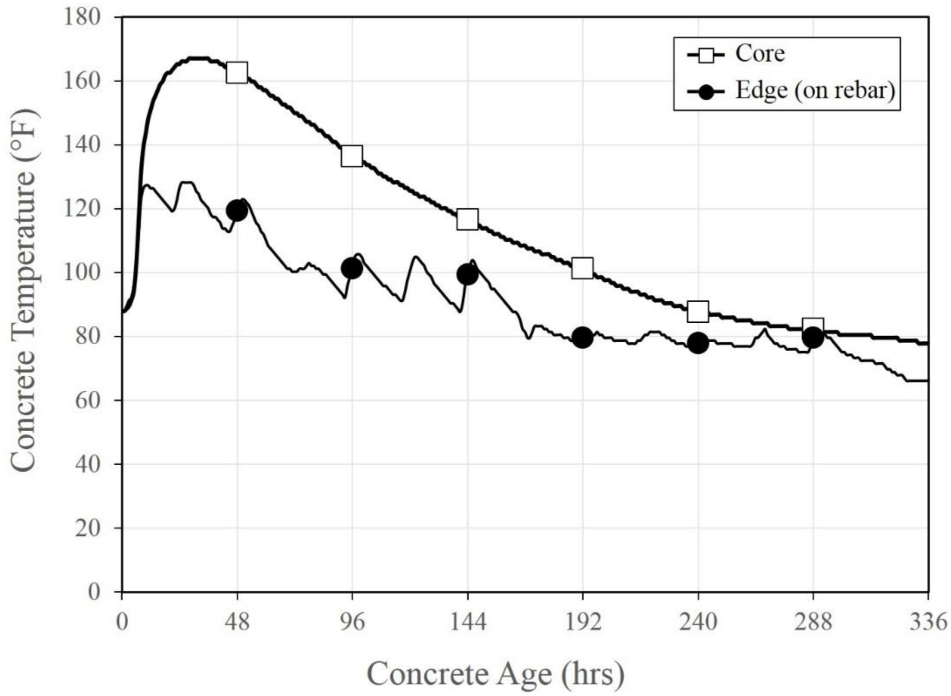

Figure 5-18: Scottsboro Bent Cap 14-Day Concrete Temperature Data

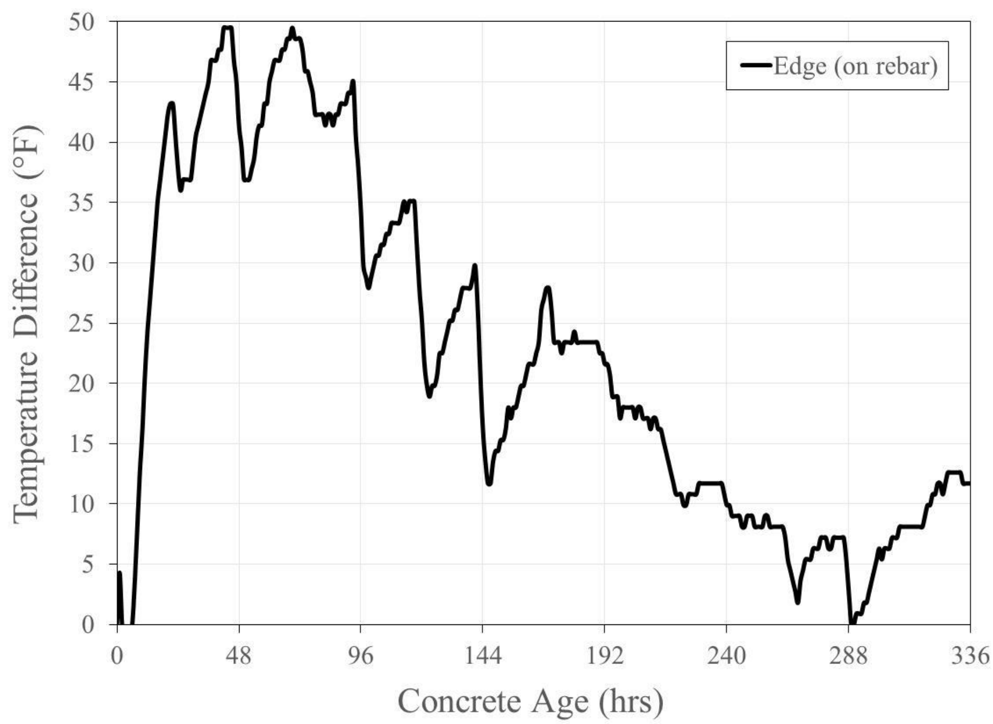

Figure 5-19: Scottsboro Bent Cap 14-Day Concrete Temperature Difference Data

5.2.5.4 Long-Term Temperature Data

The lead wires were cut by construction workers sometime after the 14-day data were collected. For this reason, no long-term temperature data were collected for the Scottsboro pedestal.

5.2.5.5 Post-Instrumentation Site Observations

Fourteen days after construction a site observation took place on October 6, 2015. At that time, no signs of thermal cracking or DEF were found. A final site visit took place on May 19, 2016. Again, no signs of temperature-related distresses were observed. The sensor lead-wires had already been cut by construction workers, preventing the collection of further long-term temperature data.

5.2.5.6 Laboratory Testing Results

Because the same concrete provider was used for both the Scottsboro pedestal and bent cap, only one set of materials was collected and only one set of laboratory tests was performed. For this reason, the values from Table 5-6 And Table 5-8 are identical.

Table 5-8: Scottsboro Bent Cap Laboratory Testing Results

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

5.2.6 Elba Bent Cap

Figure 5-20: Photograph of Elba Bent Cap

5.2.6.1 Site Details

Table 5-9: Elba Bent Cap Site Information

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

5.2.6.2 Sensor Layout

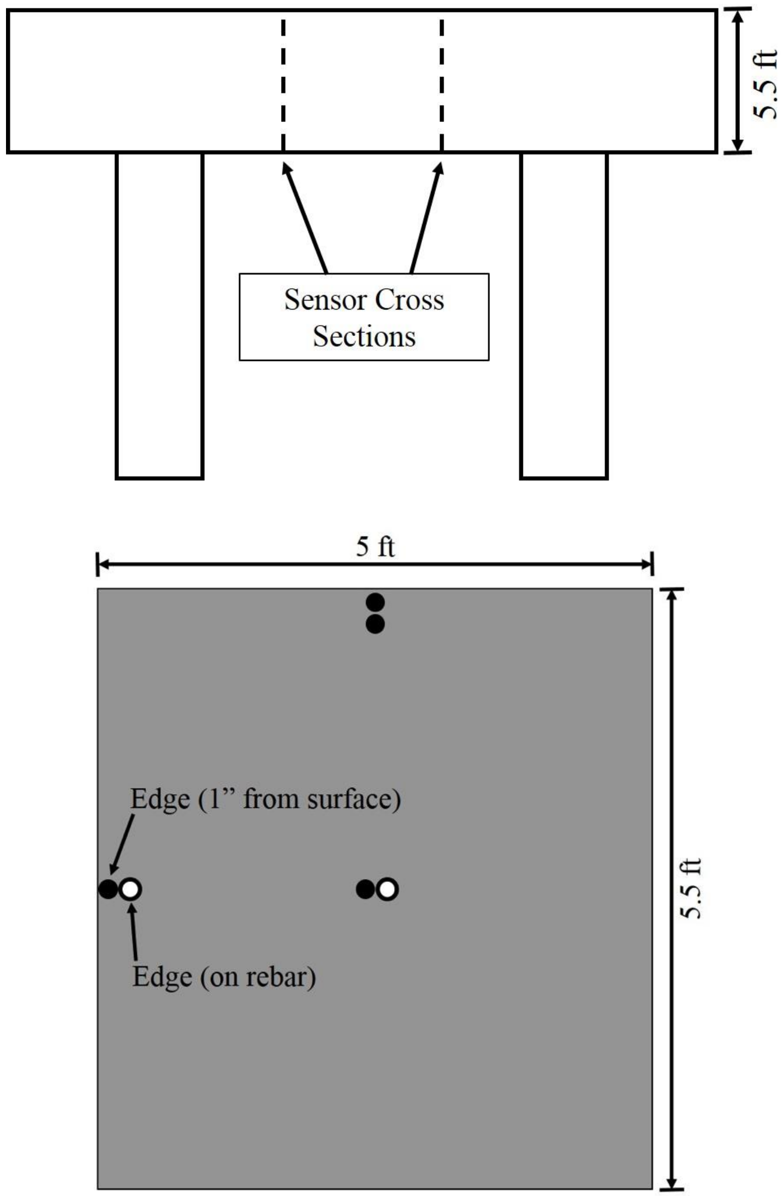

Figure 5-21: Elba Bent Cap Elevation View (top) and Cross Section (bottom)

5.2.6.3 14-Day Temperature Data

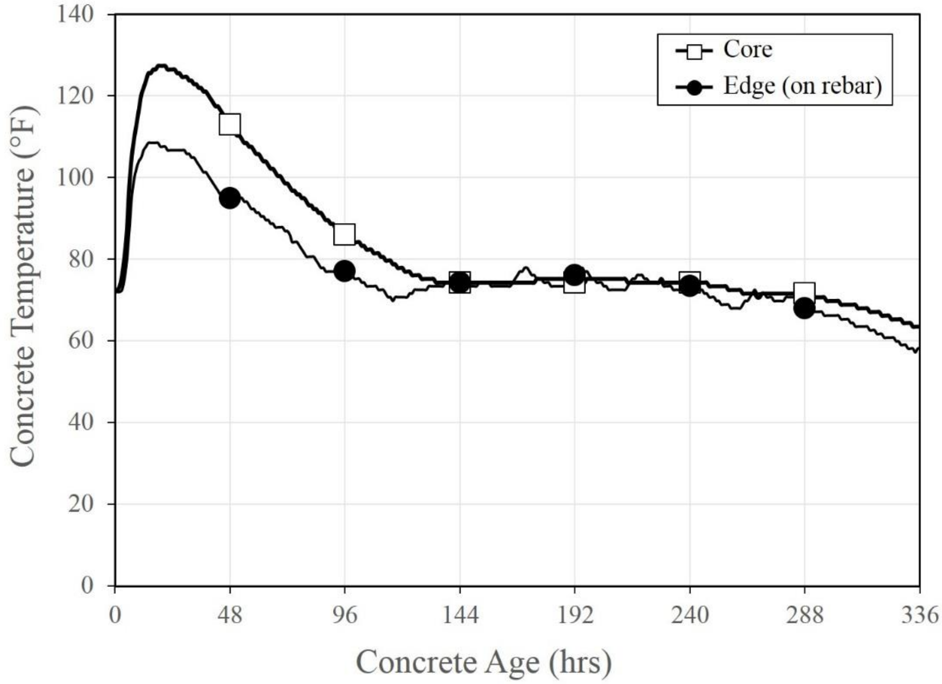

Figure 5-22: Elba Bent Cap 14-Day Concrete Temperature Data

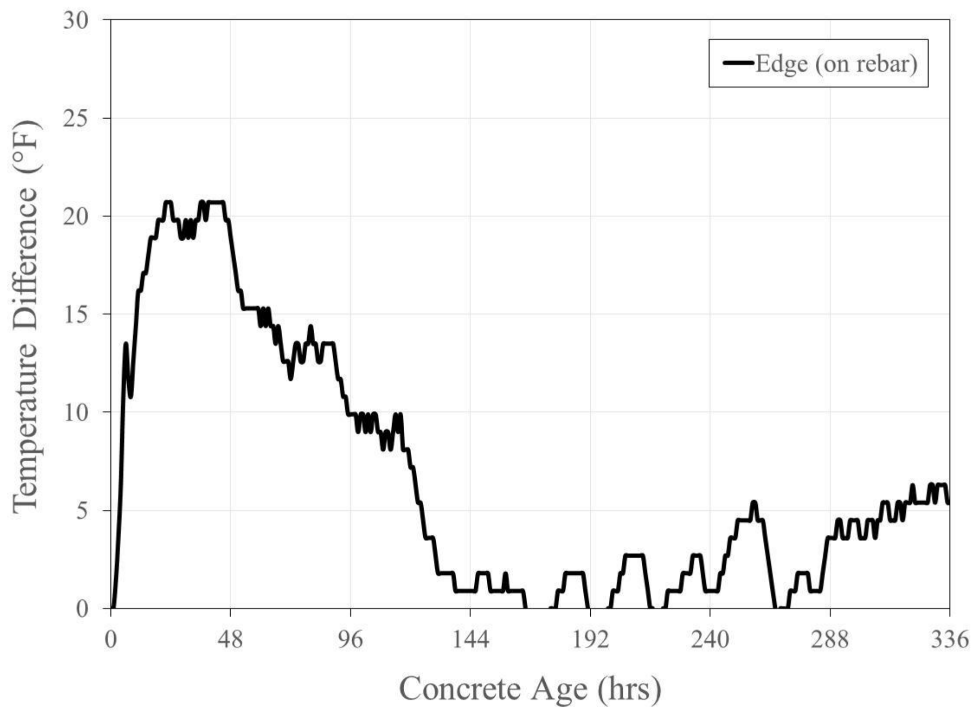

Figure 5-23: Elba Bent Cap 14-Day Concrete Temperature Difference Data

5.2.6.4 Long-Term Temperature Data

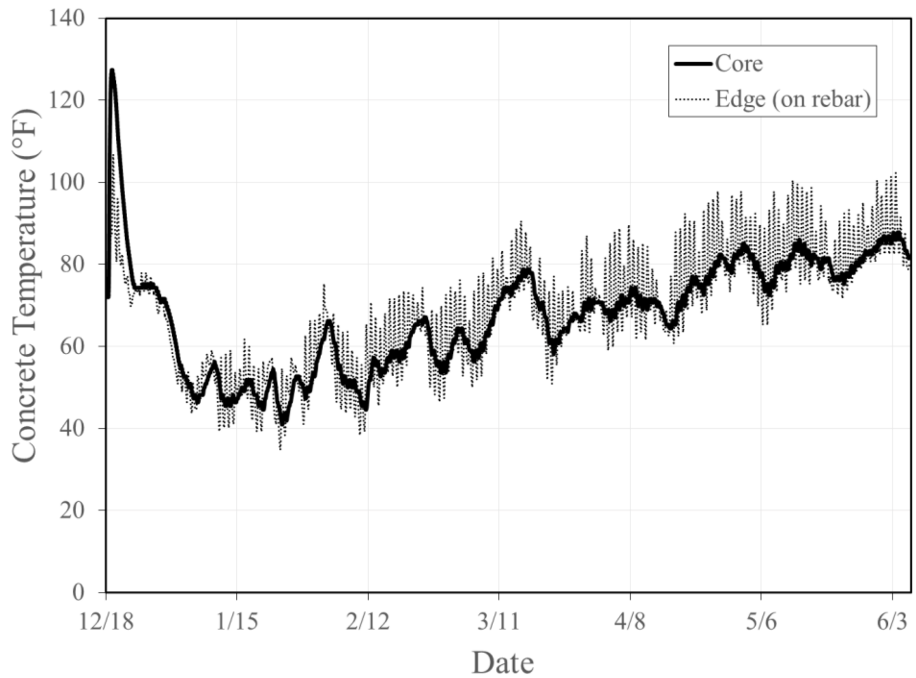

Figure 5-24: Elba Bent Cap Long-Term Concrete Temperature Data

5.2.6.5 Post-Instrumentation Site Observations

Fourteen days after construction a 14-day site observation took place on January 4, 2016. At that time, no signs of thermal cracking or DEF were found. A final site visit took place on June 6, 2016. Again, no signs of temperature-related distresses were observed. The sensor lead-wires are still accessible by man-lift, making it possible to obtain further long-term temperature data.

5.2.6.6 Laboratory Testing Results

Table 5-10: Elba Bent Cap Laboratory Testing Results

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

5.2.7 Birmingham Column

Figure 5-25: Photograph of Birmingham Column

5.2.7.1 Site Details

Table 5-11: Birmingham Column Site Information

Placement Date	1/21/2016
Placement Time	9:55 AM – 11:20 AM
Placement Location	Birmingham, Alabama
Member Type	Rectangular Column
Member Dimensions	4.5’ × 4.5’ × 20’
Cement Type	I/II
Total Cementitious Materials Content (pcy)	600
SCM Type (% Replacement)	Class C Fly Ash (20%)
Coarse Aggregate Type	Limestone
Form Type	Wood

5.2.7.2 Sensor Layout

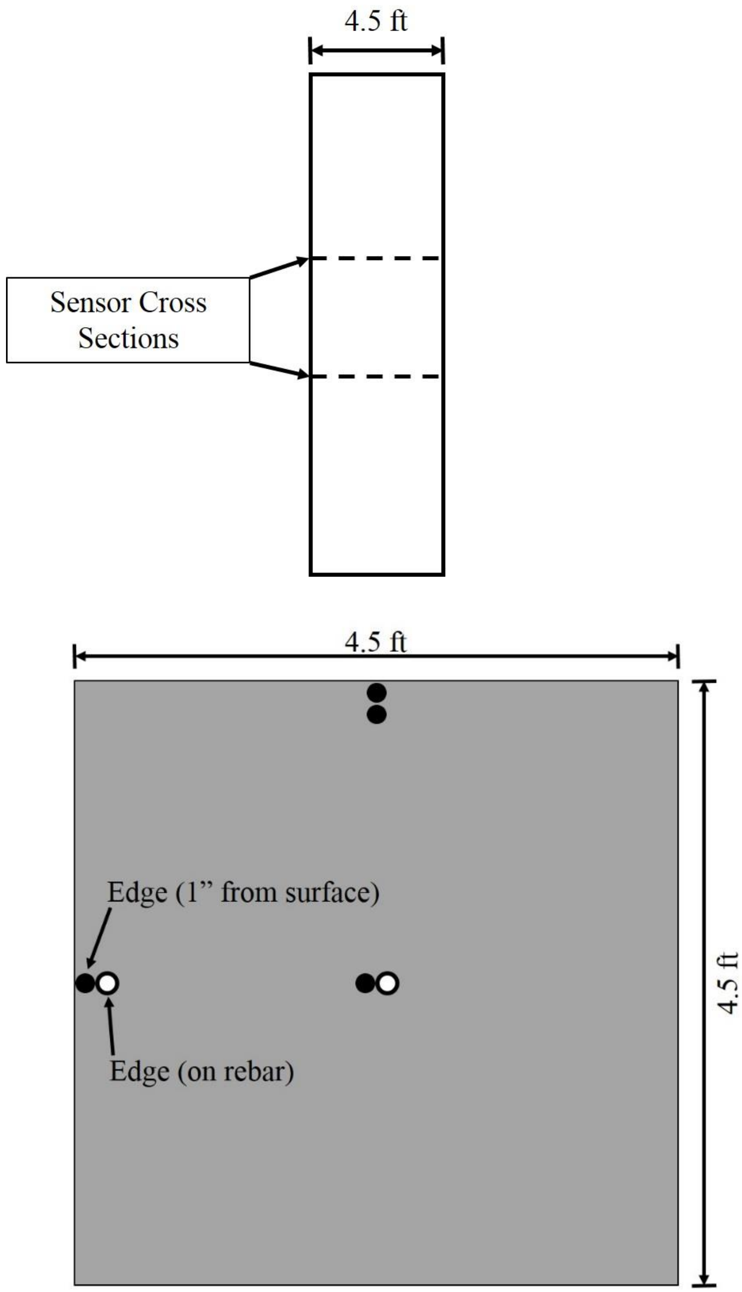

Figure 5-26: Birmingham Column Elevation View (top) and Cross Section (bottom)

5.2.7.3 14-Day Temperature Data

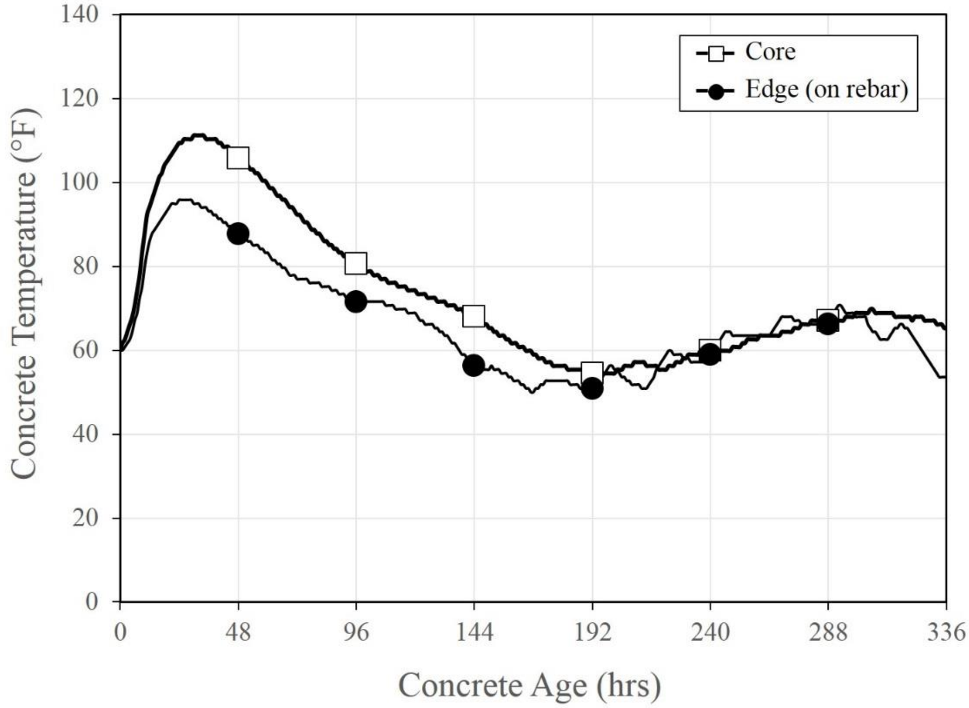

Figure 5-27: Birmingham Column 14-Day Concrete Temperature Data

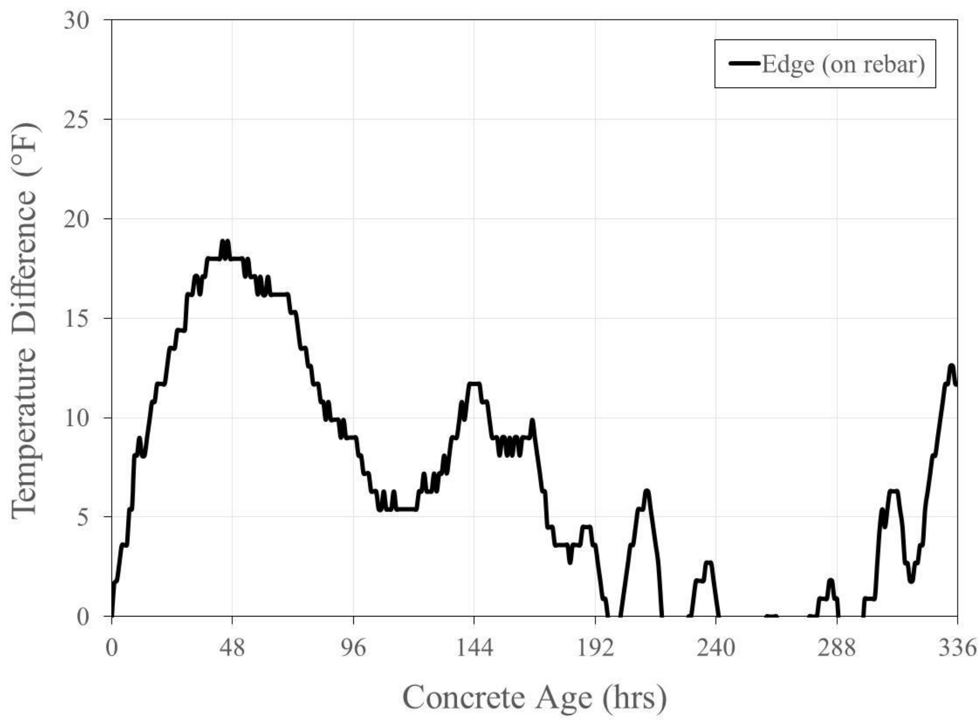

Figure 5-28: Birmingham Column 14-Day Concrete Temperature Difference Data

5.2.7.4 Long-Term Temperature Data

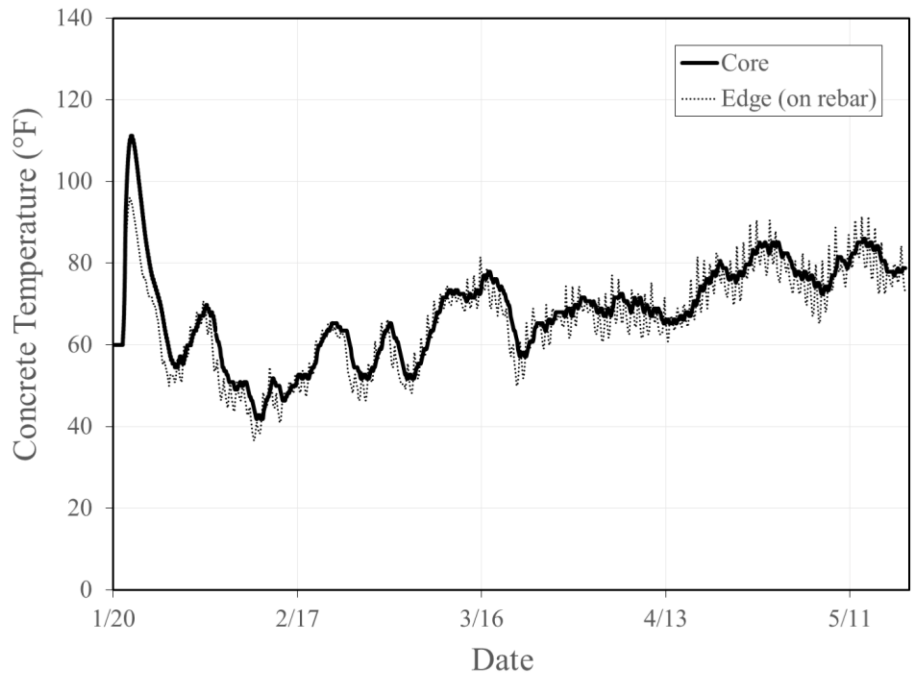

Figure 5-29: Birmingham Column Long-Term Concrete Temperature Data

5.2.7.5 Post-Instrumentation Site Observations

Fourteen days after construction a 14-day site observation took place on February 4, 2016. At that time, no signs of thermal cracking or DEF were found. A final site visit took place on May 19, 2016. Again, no signs of temperature-related distresses were observed. All sensor lead-wires were still accessible at this time, making the collection of further long-term temperature data possible.

5.2.7.6 Laboratory Testing Results

Table 5-12: Birmingham Column Laboratory Testing Results

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

5.3 Summary and Discussion of Temperature Data

In order to analyze the temperature data as a whole, the maximum concrete temperatures and maximum concrete temperature differences for each element are summarized in Table 5-13.

Table 5-13: Summary of Maximum Temperature Values from Instrumentation Data

Element	Least Cross section Dimension (ft)	Placement Date	Maximum Concrete Temperature	Maximum Concrete Temperature Difference
Albertville Bent Cap	6.5	7/31/15	168 °F	40 °F
Harpersville Crashwall	4	8/24/15	168 °F	41 °F
Scottsboro Pedestal	10	9/3/15	185 °F	68 °F
Scottsboro Bent Cap	6.5	9/18/15	167 °F	50 °F
Elba Bent Cap	5	12/18/15	127 °F	21 °F
Birmingham Column	4.5	1/21/16	111 °F	19 °F

The Elba Bent Cap and Birmingham Column experienced significantly lower maximum temperatures and maximum temperature differences than the other four elements. This is likely due to the fact that they were winter placements with smaller least dimensions than the other elements. Although 4 out of 6 elements exceeded the typical mass concrete temperature limits of 158 °F (maximum temperature) and 35 °F (maximum temperature difference), none of the elements showed signs of thermal cracking or DEF-related distress. It should be noted that even if DEF is occurring in these elements, signs of distress may not be visible during the first year after placement. These elements should continue to be monitored for signs of DEF for the first few years after placement.

Although not all temperature sensors functioned properly, using two cross sections of sensors allowed the data of at least one core sensor and at least one edge sensor to be retrieved at 14 days for all six elements.

CHAPTER 6: RESULTS AND DISCUSSION OF CONCRETEWORKS ANALYSIS

6.1 Assessment of the Accuracy of ConcreteWorks Predictions

Each ALDOT mass concrete element instrumented with temperature sensors was modeled in ConcreteWorks to obtain predictions of the development of in-place concrete temperatures. These temperature predictions were then compared to the measured temperatures in order to assess the accuracy of ConcreteWorks temperature predictions. All of the necessary input information, including nearby weather data (obtained from the NOAA) and laboratory testing results, was applied in order to thoroughly model the placement conditions of each element. Because ConcreteWorks can only calculate stresses for up to seven days after placement, only seven days of weather data were applied to each model. The following Sections 5.2.1 and 5.2.2 contain the data and results from the ConcreteWorks assessment.

6.1.1 ConcreteWorks 7-Day Concrete Temperature Predictions

This section contains graphical comparisons of measured concrete temperatures to ConcreteWorks predicted concrete temperatures. One of the most useful tools in ConcreteWorks is the ability to download a 7-day temperature history for any location within the cross section. For this research, temperature histories were analyzed for the locations where temperature sensors were placed. It should be noted that the locations of the sensors placed in the element are estimated based on the location of the sensors prior to concrete placement. Although the sensors were fastened to the rebar cage as securely as possible, it is possible that the sensors moved slightly during concrete placement. The cross sectional locations analyzed in ConcreteWorks were selected based on measurements taken prior to concrete placement. It should also be noted from Figure 5-1 that the sensors placed on the rebar for the Scottsboro pedestal were approximately 6 inches from the element surface, while the rebar sensors for the other elements were approximately 2 inches from the element surface. The closer the sensor is to the element surface, the greater the impact of formwork insulation and ambient weather conditions.

Figures 6-1 through 6-6 in this section show the comparison results for the six elements instrumented in this project. Each figure contains two graphs. The first (on top) displays concrete temperature data for the core and edge locations, and the second (on bottom) displays concrete temperature difference data between the edge and the core. Each graph shows 168 hours of temperature data, as this was the duration of the ConcreteWorks analysis. All gray lines in Figures 6-1 through 6-6 represent ConcreteWorks predictions, while all black lines represent measured temperatures data.

Any sudden increase or decrease in the edge temperature for the ConcreteWorks predictions occurs due to the removal of formwork. For each element, an approximate formwork removal time was obtained from the on-site engineer. In a few cases, such as the Harpersville crashwall, ConcreteWorks overestimates the effect of formwork removal on edge temperature change. Table 6-1 contains the approximate formwork removal times for each element. These values should be considered when examining the ConcreteWorks temperature data in Figures 6-1 through 6-6.

Table 6-1: Concrete Age of Instrumented Elements at the Time of Formwork Removal

Element	Formwork Removal Time
Albertville Bent Cap	72 hours
Harpersville Crashwall	18 hours
Scottsboro Pedestal	8 hours
Scottsboro Bent Cap	8 hours
Elba Bent Cap	72 hours
Birmingham Column	190 hours

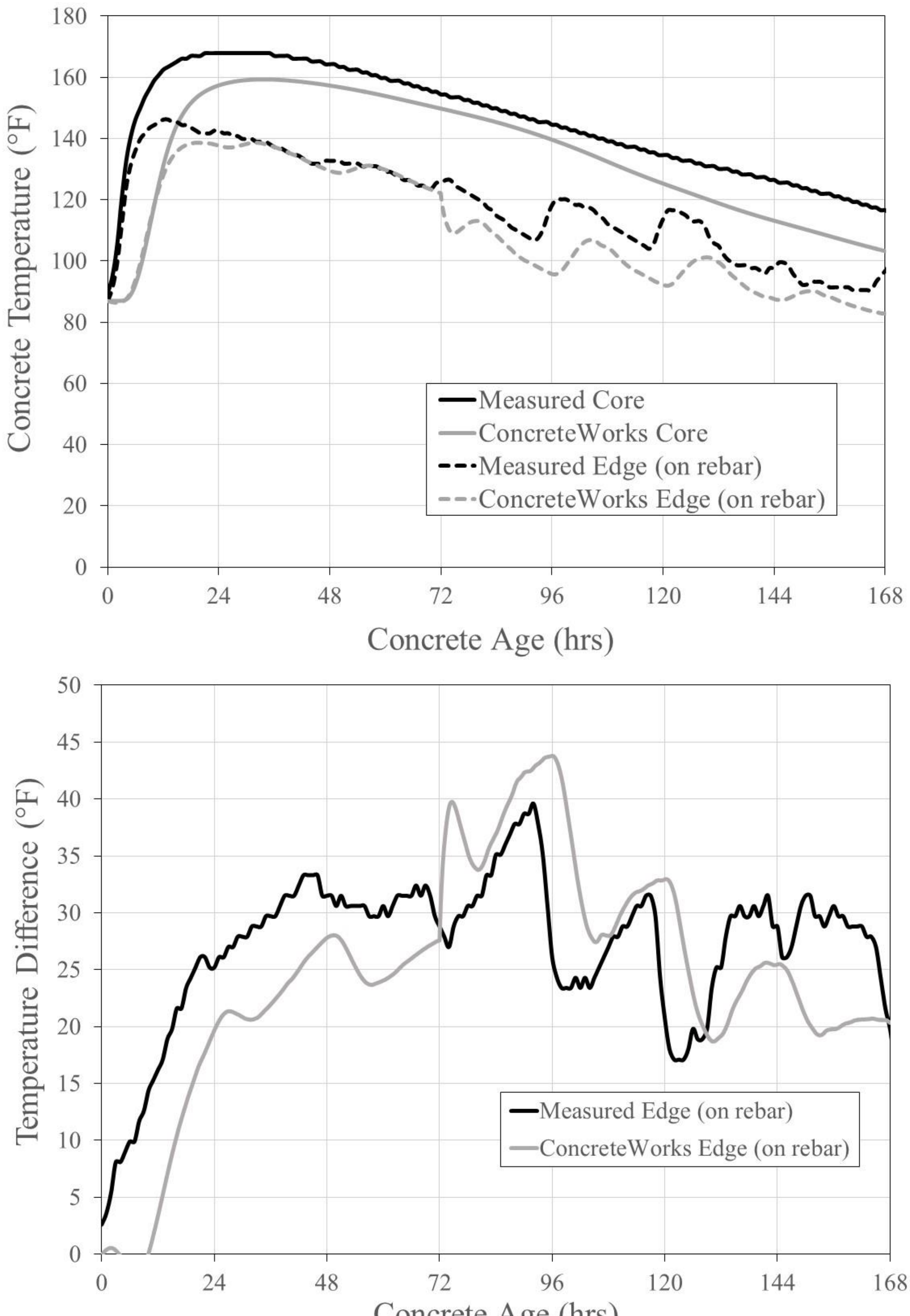

Figure 6-1: Albertville Bent Cap – ConcreteWorks Temperature Predictions versus Measured Concrete Temperatures

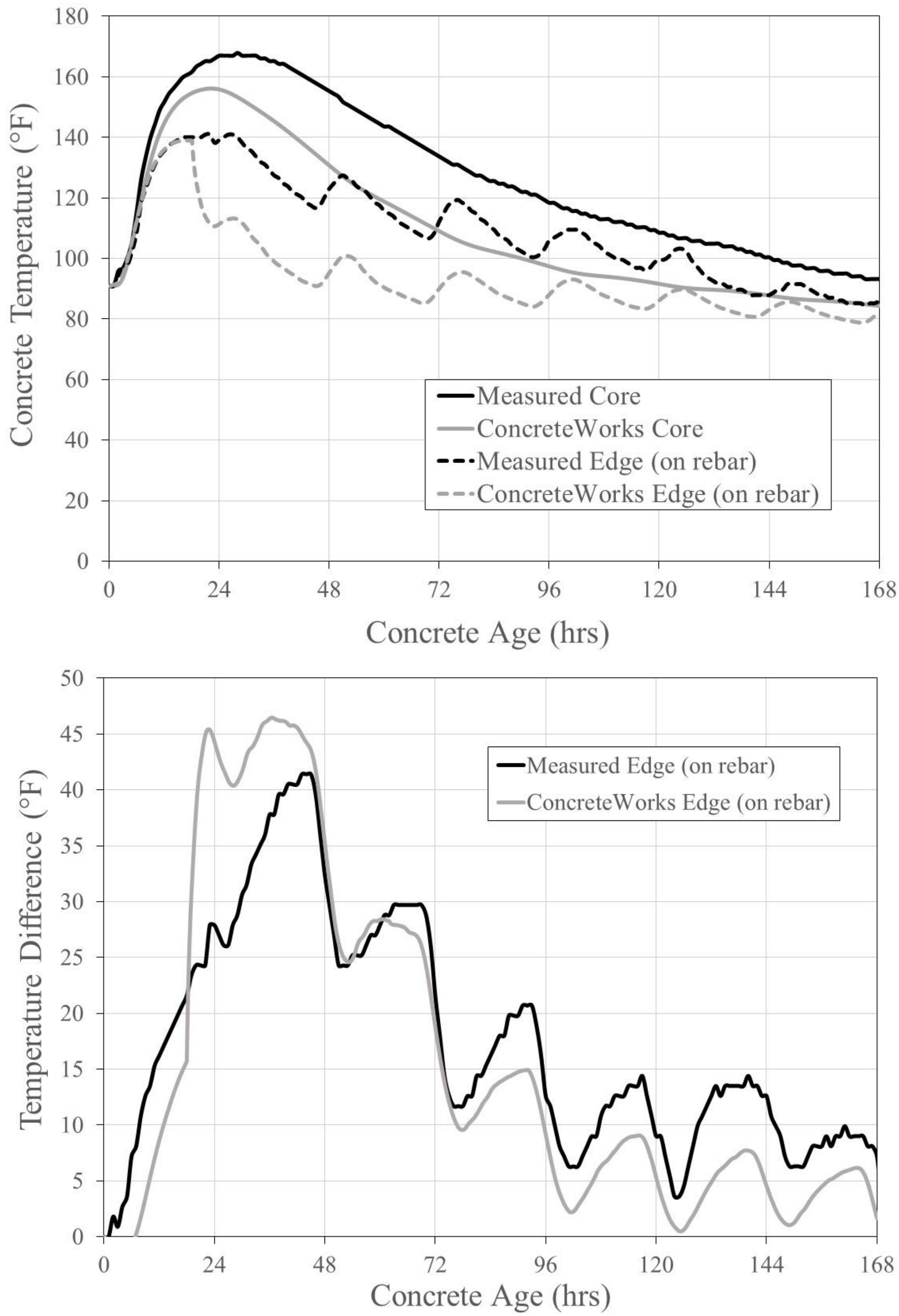

Figure 6-2: Harpersville Crashwall – ConcreteWorks Temperature Predictions versus Measured Concrete Temperatures

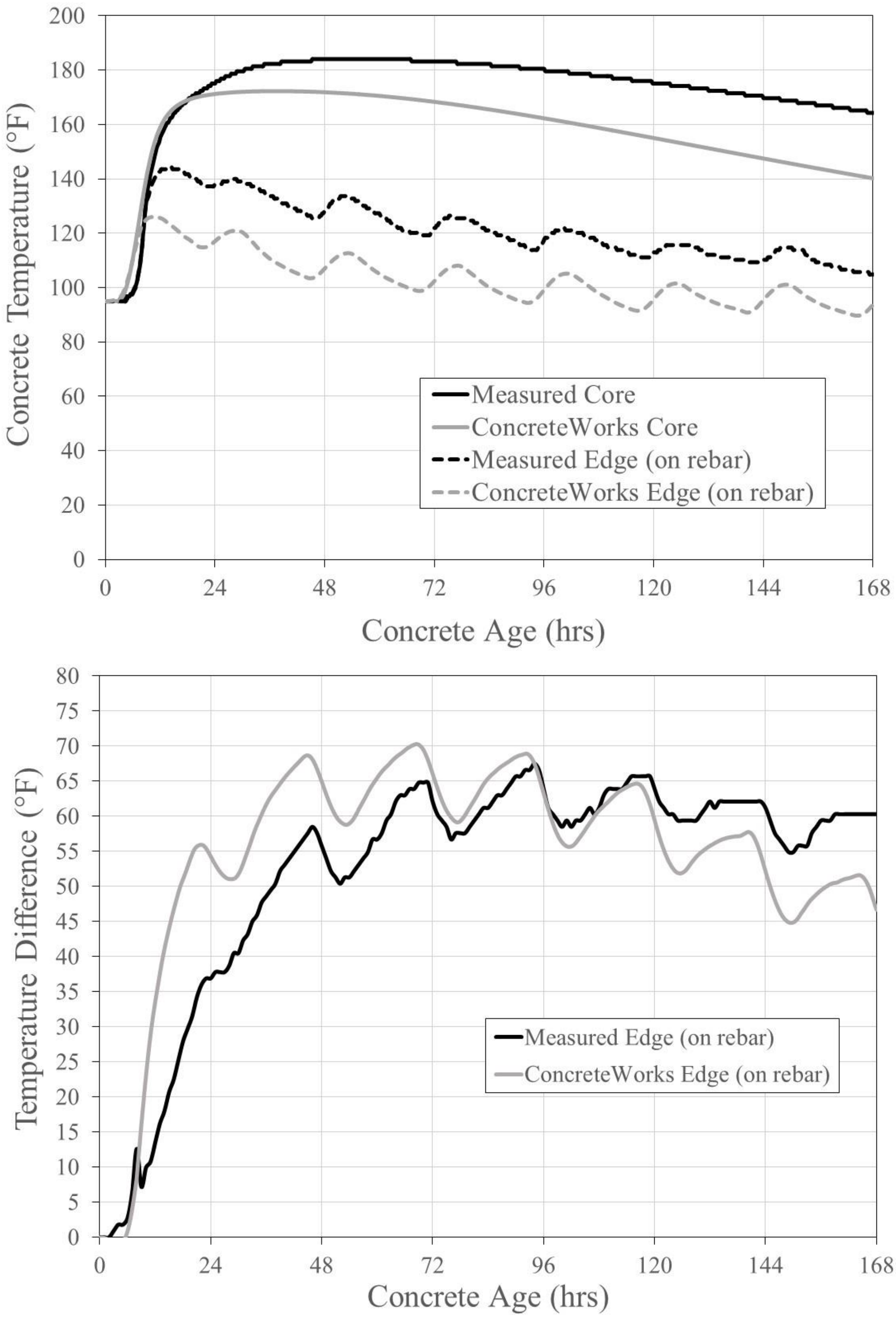

Figure 6-3: Scottsboro Pedestal - ConcreteWorks Temperature Predictions versus Measured Concrete Temperatures

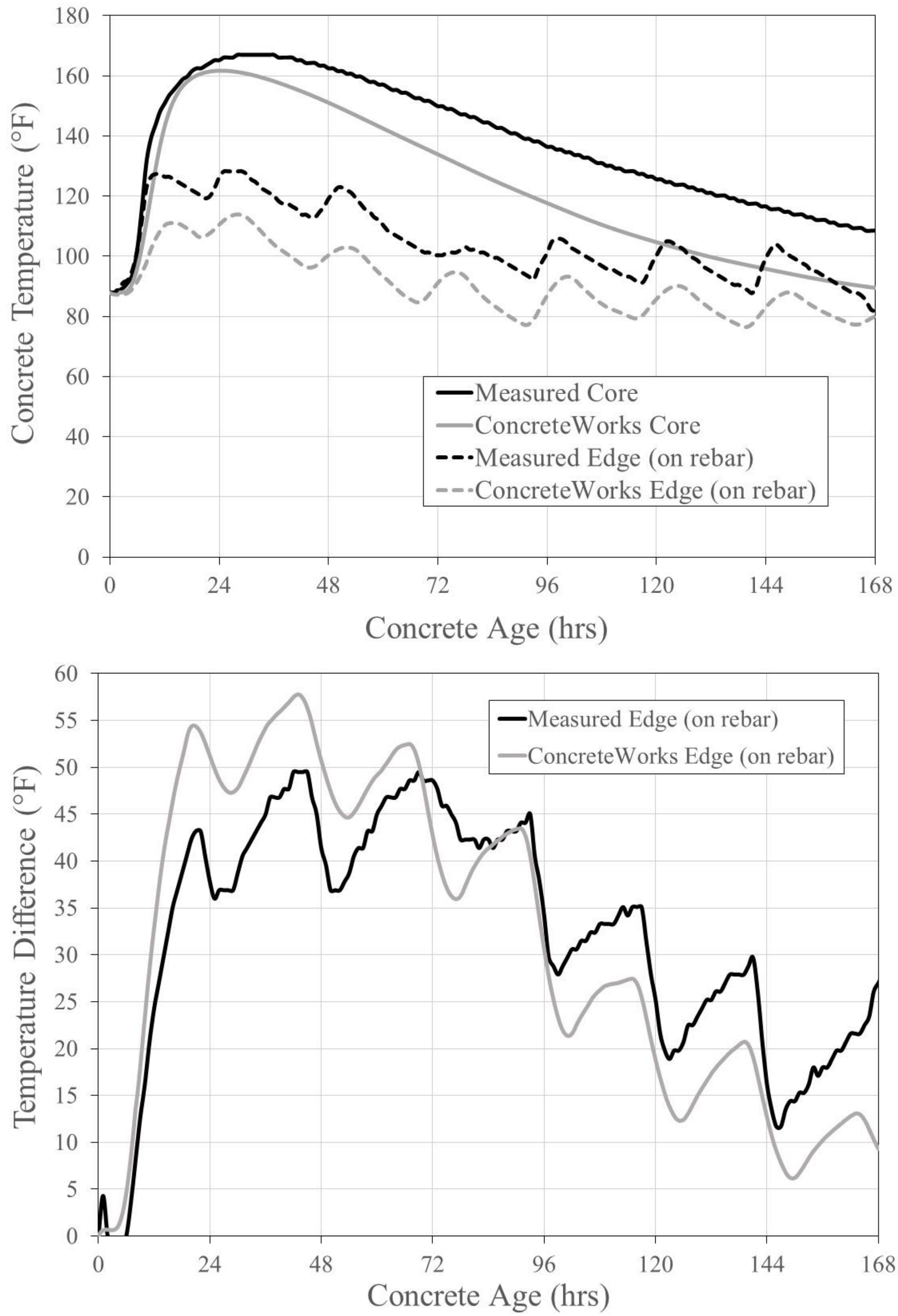

Figure 6-4: Scottsboro Bent Cap - ConcreteWorks Temperature Predictions versus Measured Concrete Temperatures

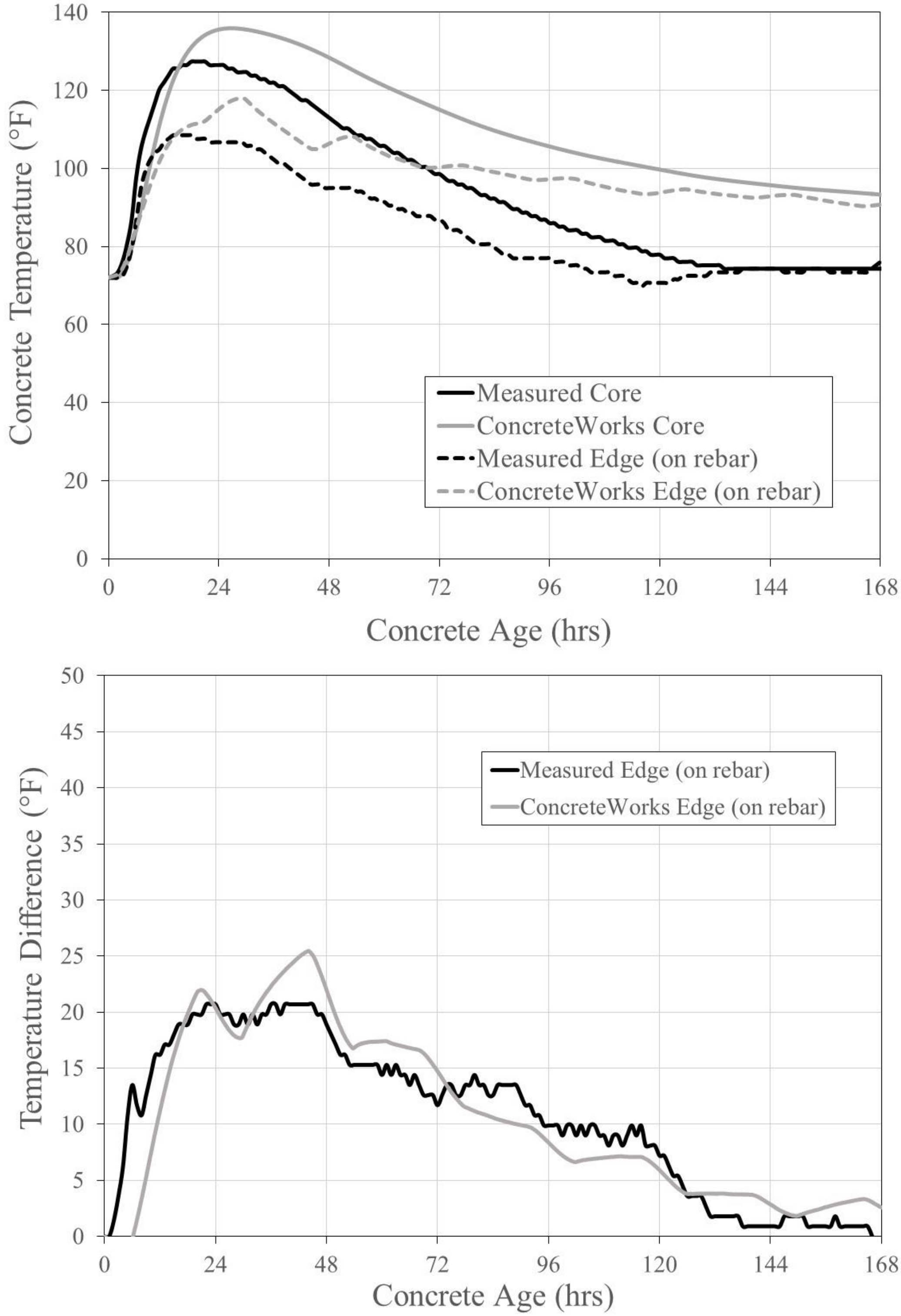

Figure 6-5: Elba Bent Cap - ConcreteWorks Temperature Predictions versus Measured Concrete Temperatures

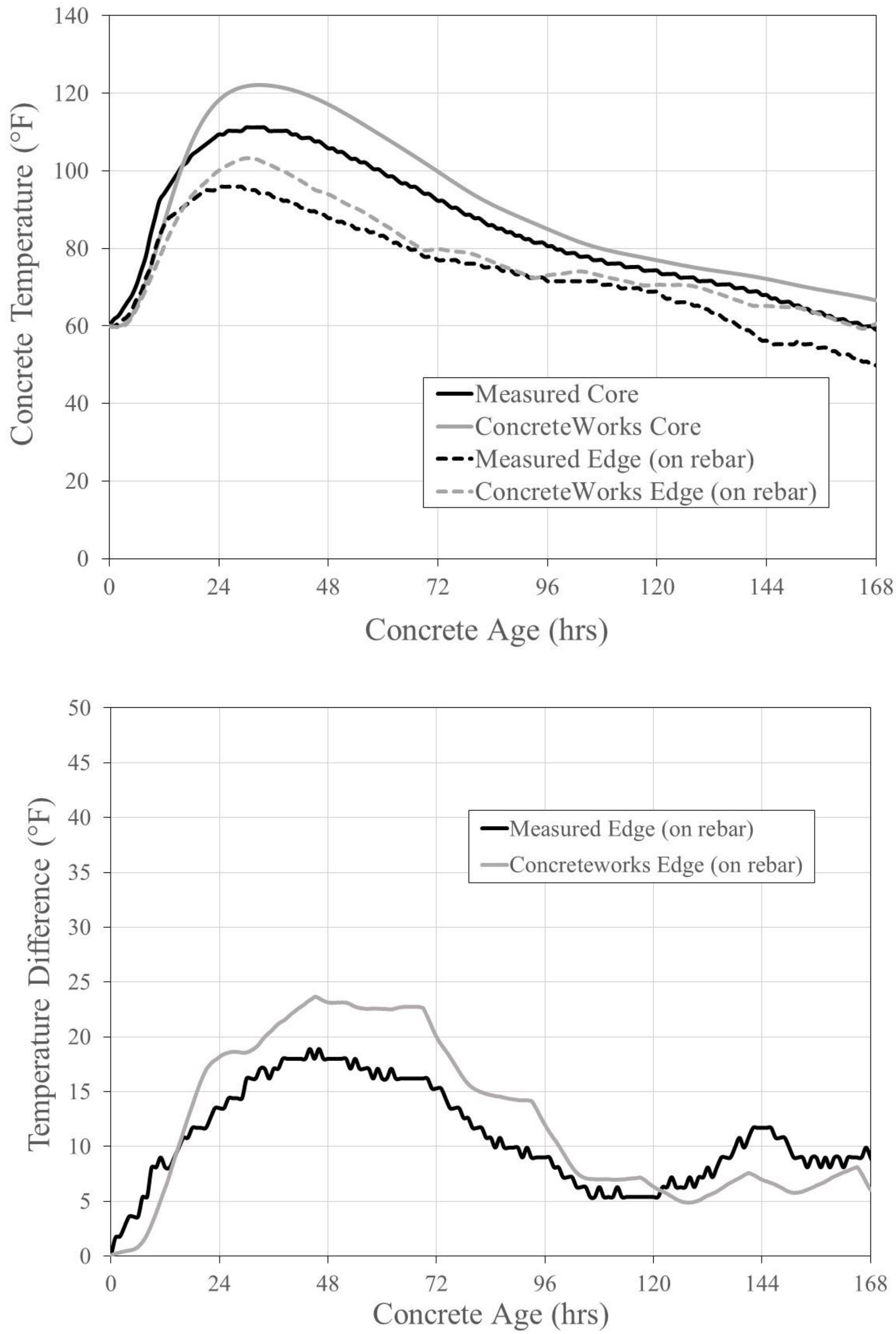

Figure 6-6: Birmingham Column - ConcreteWorks Temperature Predictions versus Measured Concrete Temperatures

6.1.2 Accuracy of ConcreteWorks Predictions

In order to quantify the accuracy of ConcreteWorks predictions, ranges were developed for acceptable error in maximum concrete temperature and maximum concrete temperature difference predictions. Only these two quantities were analyzed, as these were typically the values limited by a mass concrete specification. The acceptable error ranges reflect the variability in both the field temperature measurements and ConcreteWorks model predictions. The iButton sensors used for this research have a precision of ± 1.8 °F up to 158 °F (70 °C). When temperatures exceed 158 °F, the precision changes to ± 2.3 °F. Table 6-2 summarizes the ranges of acceptable error values for each predicted value assumed for this project. Each of the four accuracy categories is assigned a different color.

Table 6-2: Acceptable Amount of Error for ConcreteWorks Temperature Predictions

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

Table 6-3 contains a summarized comparison of measured temperature data and ConcreteWorks temperature predictions. Negative values of error indicate a ConcreteWorks under-prediction of the corresponding maximum temperature value. The error values are color-coded based on the corresponding error classification.

Table 6-3: Measured Maximum Temperature Values versus ConcreteWorks Predictions

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

All of the maximum concrete temperature difference predictions were classified as "acceptable" or better, and five were "good" or better. Based on Table 6-3, ConcreteWorks was fairly accurate in predicting concrete temperatures in the six ALDOT mass concrete elements instrumented during this research project.

6.2 Analysis with ConcreteWorks

6.2.1 Statistical Analysis of Results

One of the objectives listed in Chapter 1 involves the development of temperature control requirements for ALDOT mass concrete construction. In order to accomplish this objective, ConcreteWorks was used to determine which input variables (coarse aggregate type, SCM type, placement location, placement date, and element least dimension) had the greatest effect on the output variables (maximum temperature, maximum temperature difference, and cracking risk). In order to accomplish this task, a rectangular, mass concrete column was modeled for 480 placement scenarios, with each scenario representing a different set of input conditions.

The coarse aggregate types used in this analysis were limestone and river gravel. These aggregate types were selected because they are both commonly available in Alabama and used in concrete mixture proportions for ALDOT mass concrete elements. When adjusting the coarse aggregate type, it is important to change the concrete CTE as well. Based on previous CTE tests performed at Auburn University on Alabama concretes, values of 5.52×10^{-6} in./in./°F (limestone) and 6.95×10^{-6} in./in./°F (river gravel) were used (Schindler et al., 2010).

The placement locations used in this analysis were Mobile, Alabama and Huntsville, Alabama. Although ConcreteWorks also contains default weather data for Montgomery and Birmingham, Mobile and Huntsville were selected because they represent the warmest and coolest ambient conditions in Alabama.

The placement time was varied along with the placement date in order to account for the common construction practice of casting large elements during cooler hours of the day during summer months and during hotter hours of the day during winter months. ConcreteWorks automatically changes the default weather conditions based on the location, date, and time of the placement. The concrete placement temperature was also varied when adjusting the placement dates and locations. For each placement scenario, the concrete placement temperature was assumed to be equal to the ambient temperature at the time of placement.

The element least dimension was varied from 4 ft to 8 ft, using 1 ft increments. These five sizes were chosen because mass concrete size designations for other specifications typically fall within this range (see Table 3-5). It was expected at the time of this analysis that a future ALDOT specification would include a mass concrete size designation within this range of values.

The following list summarizes the different input variables and values of each variable used in this analysis:

- Coarse aggregate type (2 options) → limestone and siliceous river gravel;

- SCM Type (4 options) → 100% portland cement (PC), 30% Class F fly ash replacement, 30% Class C fly ash replacement, 50% slag cement replacement;
- Placement Location (2 options) → Huntsville, AL and Mobile, AL;
- Placement Date (6 options) → January 15 at 12:00 PM, March 15 at 10:00 AM, May 15 at 8:00 AM, July 15 at 8:00 AM, September 15 at 10:00 AM, November 15 at 12:00 PM; and
- Element Least Dimension (5 options) → 4 ft, 5 ft, 6 ft, 7 ft, and 8 ft.

In order to quantify the effect of each variable on the corresponding outputs, the ANOVA Single Factor Test was used. Table 6-4 contains the p-values for each input variable analyzed in this test. The closer the p-value is to zero, the greater impact that input variable has on a particular output variable. Based on a 95% confidence interval, a p-value of greater than 0.05 indicates statistical insignificance. Any variable that was determined to be statistically insignificant with respect to a particular output variable is denoted by bold red text.

Table 6-4: P-Values of Different Input Variables Based on ANOVA Test

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

Based on the data in Table 6-4, Table 6-5 was produced to show which inputs have the greatest impact on each output variable.

Table 6-5: Rankings of Statistical Impact of Input Variables Based on ANOVA Test

Maximum In-Place Concrete Temperature	**Maximum Concrete Temperature Difference**	**Maximum Concrete Cracking Risk**
Placement Date	Element Least Dimension	Element Least Dimension
↑	↑	↑
SCM Type	SCM Type	Coarse Aggregate Type
↑	↑	↑
Element Least Dimension	Placement Date	SCM Type
↑	↑	↑
Placement Location	Placement Location	Placement Date
↑	↑	↑
Coarse Aggregate Type	~~Coarse Aggregate Type~~	~~Placement Location~~

The arrows in Table 6-5 indicate the increasing significance of a particular input variable on the corresponding output variable. For example, based on this table, the placement date has the greatest impact of the five variables on the maximum concrete temperature, while the coarse aggregate type has the least. Any struck-through text, such as coarse aggregate type in the second column, indicates the statistical insignificance of that input variable on the corresponding output variable. Based on this information, coarse aggregate type has no significant impact on the maximum concrete temperature difference, while the placement location has no significant impact on the maximum concrete cracking risk.

6.2.2 Summary of ConcreteWorks Analysis

This section contains graphical information summarizing the impact of each designated input variable on the three output variables. In order to summarize the results of the analysis, only the two most impactful inputs variables, as determined from Table 6-5, were examined to demonstrate their effect on a particular output variable.

6.2.2.1 Maximum Concrete Temperatures

Figure 6-7 demonstrates the effect of placement date and SCM type on the maximum concrete temperature. This graph shows the ConcreteWorks maximum concrete temperature predictions from a 6 ft square column cast in Mobile using limestone aggregate.

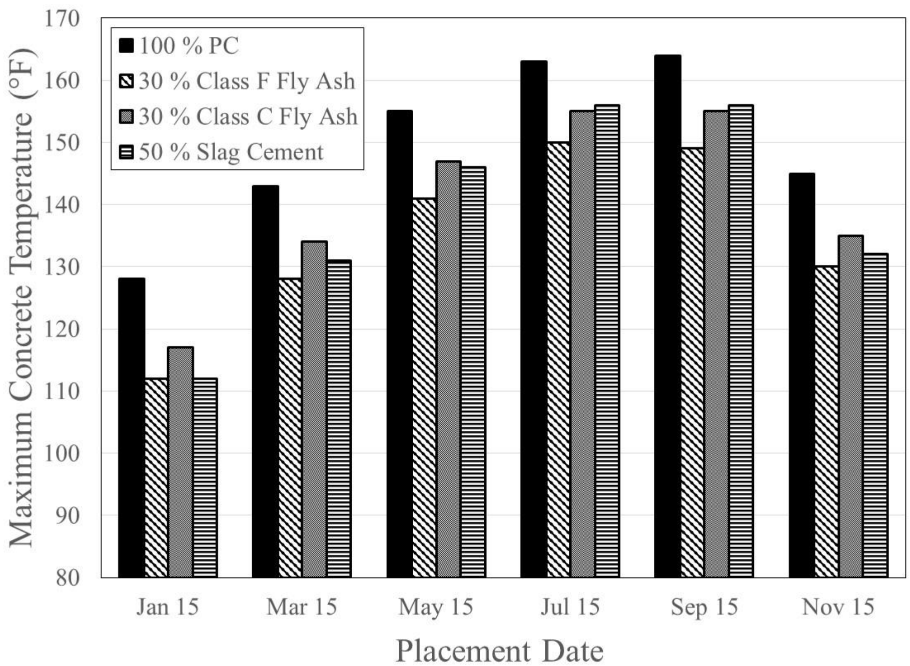

Figure 6-7: Effect of SCM Type and Placement Date on Maximum Concrete Temperature (Mobile, AL with limestone aggregate)

Based on Figure 6-7, it is clear that the use of SCMs, especially Class F fly ash, can reduce the maximum concrete temperature in a mass concrete element. Concretes containing these SCMs typically produce less heat during hydration than concretes containing only portland cement concrete (Schindler and Folliard, 2005).

Additionally, elements cast during warmer ambient conditions will experience higher maximum concrete temperatures. Based on the data in Figure 6-7, a higher concrete placement temperature will result in a higher in-place concrete temperature (Gajda and Vangeem, 2002).

This is why July and September placements in Figure 6-7 had the highest maximum temperatures.

Figure 6-8 demonstrates the effect of element least dimension on the maximum in-place concrete temperature. The data shown in this graph pertain to a concrete element cast in July containing 100% PC. It is clear from Figure 6-8 that an increase in element least dimension results in an increase in maximum concrete temperature. This supports the findings in Table 6-5.

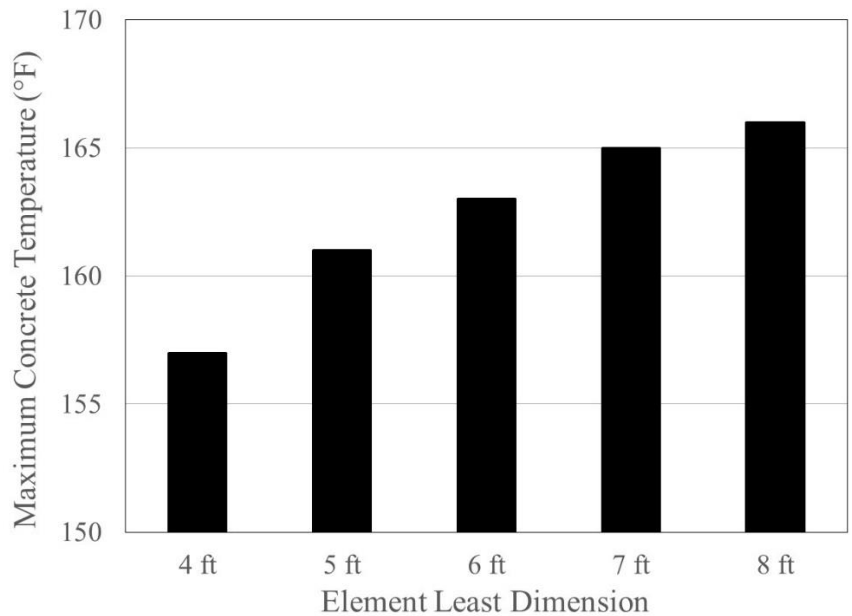

Figure 6-8: Effect of Element Least Dimension on Maximum Concrete Temperature (100% PC in July)

6.2.2.2 Maximum Concrete Temperature Difference

Figure 6-9 demonstrates the effect of element least dimension and SCM type on the maximum concrete temperature difference. This graph shows the ConcreteWorks maximum concrete temperature difference predictions from a square column cast in Mobile, AL on July 15th using limestone aggregate.

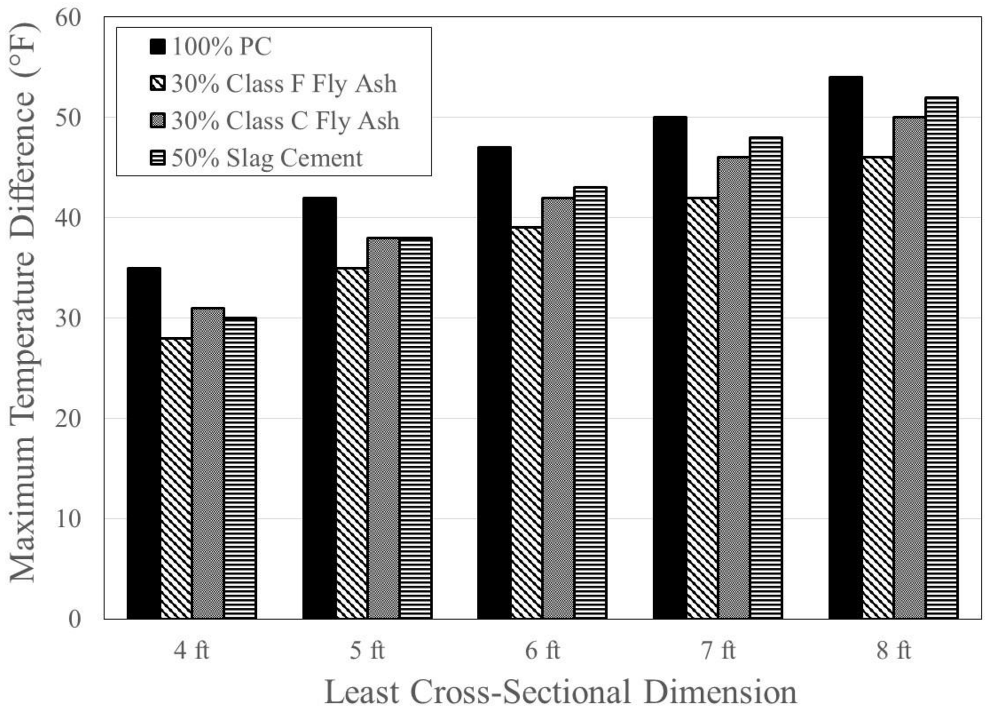

Figure 6-9: Effect of SCM Type and Element Least Dimension on Maximum Concrete Temperature Difference (Mobile, AL on July 15)

Based on Figure 6-9, it is clear that the use of SCMs can significantly reduce the maximum concrete temperature difference in a mass concrete element. Class F fly ash is the most effective in limiting the temperature difference, as it produces the lowest heat of hydration (Schindler and Folliard, 2005).

Additionally, as the element least dimension increases, so does the maximum temperature difference. This finding matches the results of the statistical analysis presented in Table 6-5. The large amount of interior concrete traps the heat in the element core, leading to larger temperature differences across the cross section. Based on the trends in this graph, the maximum temperature difference increases by approximately 5 °F for every 1 ft increase in element least dimension.

6.2.2.3 Maximum Concrete Cracking Risk

Figure 6-10 demonstrates the effect of placement date, element least dimension, and coarse aggregate type on the maximum concrete cracking risk. The cracking risk is obtained from an analysis that compares the development of tensile stress to tensile strength. This graph shows the ConcreteWorks maximum concrete cracking risk predictions from a square column with 100 % PC cast in Mobile.

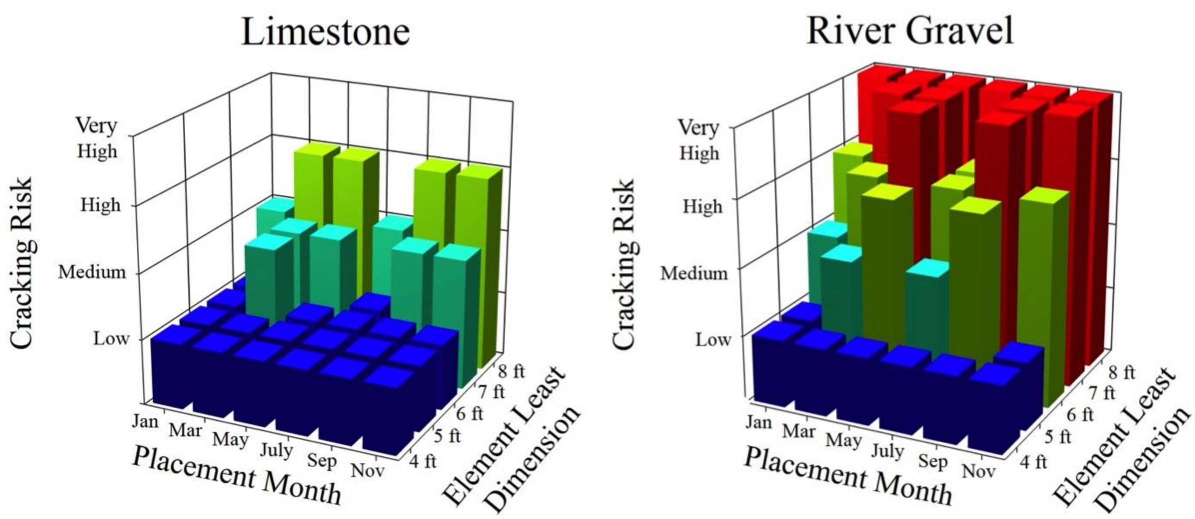

Figure 6-10: Effect of Placement Date, Element Size, and Aggregate Type on Maximum Concrete Cracking Risk (100% PC in Mobile, AL)

It is evident from Figure 6-10 that the use of limestone aggregate can significantly reduce the risk of thermal cracking in a mass concrete element. This supports the findings from Table 6-5. Those findings indicate that the coarse aggregate type has a significant effect on cracking risk, while having little to no effect on the concrete temperature difference. This is due to the fact that concrete with limestone aggregate has a much smaller CTE than concrete containing river gravel.

It is also evident from Table 6-5 that the use of SCMs can significantly reduce the cracking risk. This statement is reinforced by the graphs in Figure 6-11. Figure 6-11 displays the cracking risk categories of mass concrete columns cast in Mobile, AL with river gravel. Due to the higher temperatures produced in the 100% PC elements, the elements experience a higher risk of cracking.

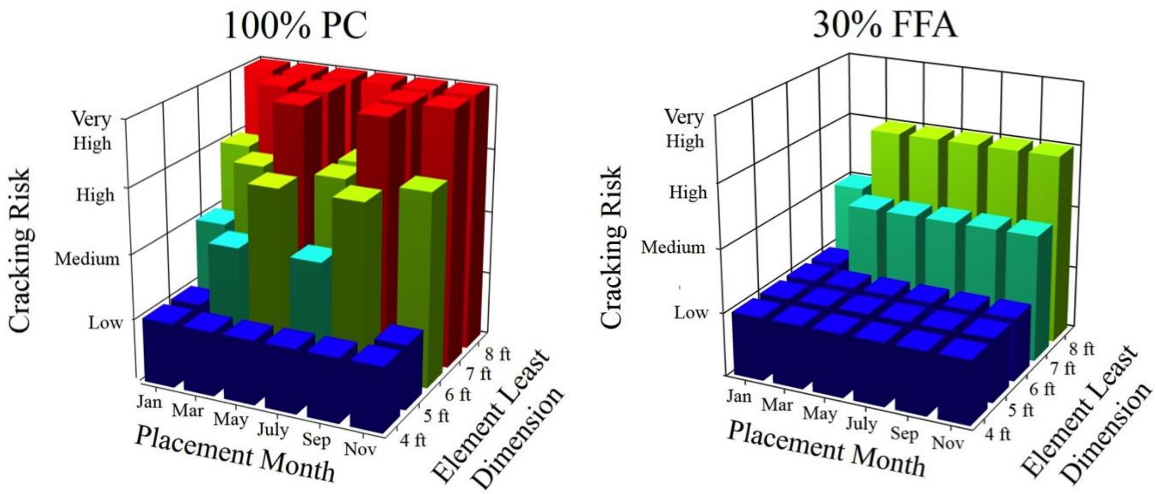

Figure 6-11: Effect of Placement Date, Element Size, and SCM Type on Maximum Concrete Cracking Risk (River Gravel in Mobile, AL)

It can also be determined from Figures 6-10 and 6-11 that the element least dimension has a significant impact on risk of thermal cracking. The larger the element least dimension is, the greater the risk of cracking in the element.

As shown in Table 6-5, the placement date has little impact on cracking risk. Variables such as least dimension, coarse aggregate type, and use of SCMs have a stronger influence on concrete cracking risk.

6.2.2.4 Temperature Difference versus Cracking Risk

While large temperature differences can lead to thermal cracking, the concrete CTE is a major determining factor in the risk of thermal cracking. A concrete with a low CTE can withstand a larger temperature difference without developing thermal cracking. Some mass concrete specifications implement thermal cracking mitigation measures dependent upon the temperature difference alone, without considering the concrete CTE. As demonstrated by Table 2-1 in this report, the allowable temperature difference limit of 35 °F developed by Bamforth (1995) was developed for concrete containing river gravel. Concrete containing river gravel typically has a higher CTE than concrete containing limestone (see Figure 2-3). The ConcreteWorks analysis performed for this research supports the conclusion that the risk of thermal risk can be decreased by using a lower CTE concrete. Figure 6-11 supports this conclusion. This plot compares the maximum temperature difference and thermal cracking risk for a 7 ft square column with various placement dates and placement locations. The x-axis is organized based on SCM type and coarse aggregate type. PC represents portland cement. FFA represents Class F fly ash. SL represents slag cement. CFA represents Class C fly ash. LS represents limestone aggregate. RG represents river gravel. In Figure 6-12, the shaded gray plot represents the maximum temperature difference for each placement scenario, plotted on the primary y-axis. The black line represents the thermal cracking risk, plotted on the secondary y-axis. The dashed blue line corresponds to a limit of 35 °F.

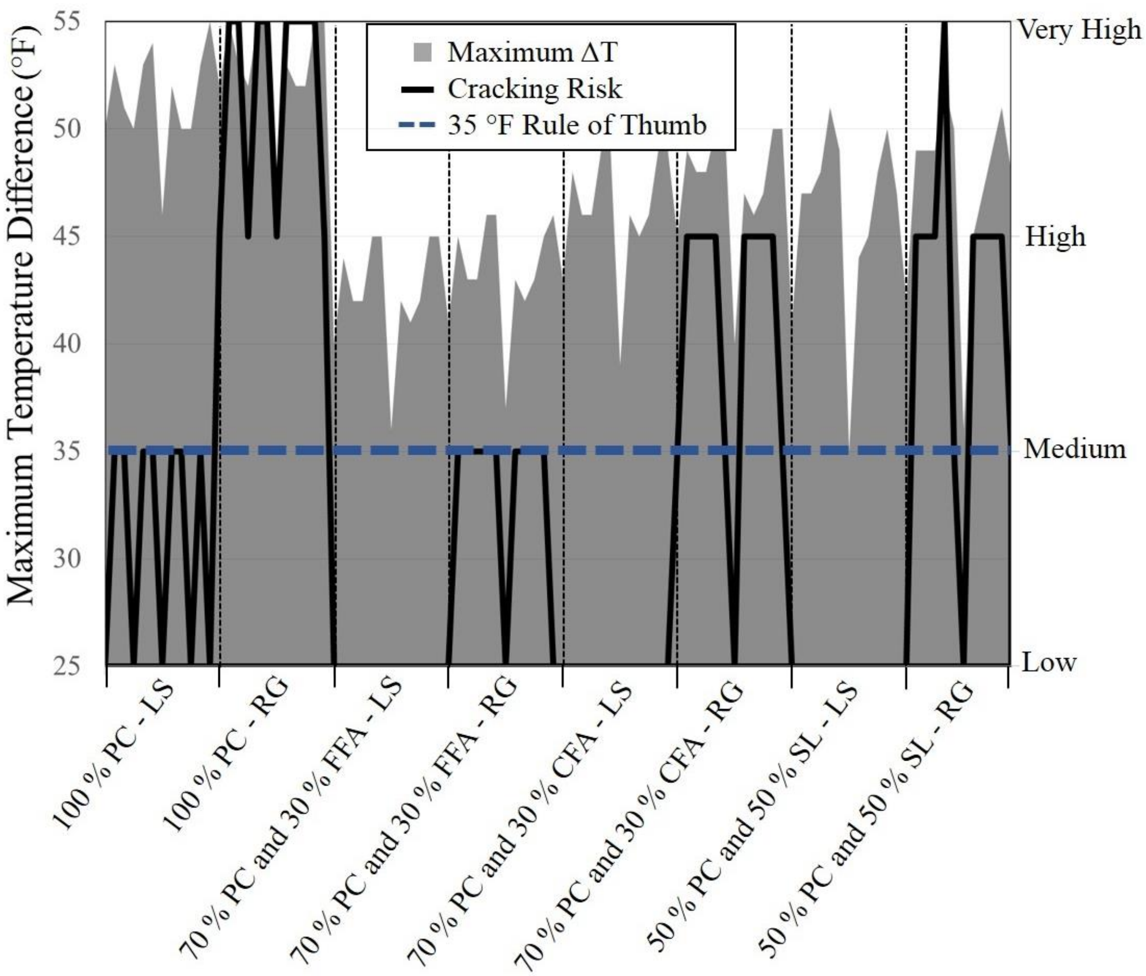

Figure 6-12: Cracking Risk versus Maximum Temperature Difference (7-ft least cross sectional dimension)

As shown in Figure 6-12, for each of the four different cement mixtures, limestone concretes have similar temperature difference profiles as river gravel concretes, but experience a much lower risk of thermal cracking. The sudden drop in temperature difference from 100% PC to 30% FFA occurs because the use of Class F Fly Ash significantly reduces the temperature difference in mass concrete elements, as shown in Figure 6-9. It should be noted that concretes with SCMs and limestone aggregate never exceed a “low” thermal cracking risk. Based on Figure 6-12, the worst case scenario for thermal cracking in mass concrete elements is plain portland cement concrete with river gravel. Using SCMs can lower the temperature differences, thus lowering cracking risk. However, using a lower CTE coarse aggregate, such as limestone, is the most effective strategy in reducing the risk of thermal cracking.

In summary, the results from this section indicate that a cracking risk analysis is a better way to determine maximum allowable temperature difference, rather than using a fixed 35 °F temperature difference for all concrete placements.

6.2.2.5 Mass Concrete Size Designation

In order to determine an appropriate mass concrete size designation, the ConcreteWorks analysis data were analyzed to determine what size member would exceed the allowable maximum concrete temperature or experience thermal cracking. Because SCMs and coarse aggregate type have major impact on maximum concrete temperature and thermal cracking, respectively, these variables were used to determine what size member would require temperature control. The results of this study are summarized in Table 6-6. In this table it is assumed that a concrete element that experiences a concrete temperature greater than 158 °F (the maximum temperature limit for DEF) or a cracking risk of “high” or “very high” (see Figure 2-16) would be recognized as mass concrete, and thus require a TCP.

As shown in Table 6-6, the use of SCMs and low-CTE coarse aggregate, such as limestone, can significantly reduce concrete maximum temperatures and cracking risk, thus increasing the element least dimension requiring a TCP. For instance, consider the members with 100% PC and river gravel. With these mixture proportions, an element with least cross sectional dimension of 4 ft would require a TCP. However, replacing the river gravel with limestone aggregate would increase that least dimension to 5 ft. In the same way, replacing 30% of the PC with FFA would increase that least dimension to 8 ft. For this reason, a mass concrete size designation should reflect the use of SCMs and the type of coarse aggregate in the concrete.

Table 6-6: ConcreteWorks Analysis of an Appropriate Mass Concrete Size Designation

SCM Type	Coarse Aggregate Type	Element Least Dimension	T_{max} greater than 158 °F?	Cracking Risk High or Very High?	TCP Needed?
100% PC	Limestone	4 ft			
		5 ft	✓		✓
		6 ft	✓		✓
		7 ft	✓		✓
		8 ft	✓	✓	✓
	River Gravel	4 ft	✓		✓
		5 ft	✓	✓	✓
		6 ft	✓	✓	✓
		7 ft	✓	✓	✓
		8 ft	✓	✓	✓
30% FFA	Limestone	4 ft			
		5 ft			
		6 ft			
		7 ft			
		8 ft			
	River Gravel	4 ft			
		5 ft			
		6 ft			
		7 ft			
		8 ft	✓	✓	✓
30% CFA	Limestone	4 ft			
		5 ft			
		6 ft			
		7 ft			
		8 ft	✓		✓
	River Gravel	4 ft			
		5 ft			
		6 ft			
		7 ft	✓	✓	✓
		8 ft	✓	✓	✓
50% SL	Limestone	4 ft			
		5 ft			
		6 ft			
		7 ft	✓		✓
		8 ft	✓	✓	✓
	River Gravel	4 ft			
		5 ft			
		6 ft	✓	✓	✓
		7 ft	✓	✓	✓
		8 ft	✓	✓	✓

CHAPTER 7: DEVELOPMENT OF AN ALDOT MASS CONCRETE SPECIFICATION

7.1 Introduction

The purpose of this chapter is to provide recommendations for an ALDOT mass concrete specification. These include recommendations include guidelines for a mass concrete size designation, maximum concrete temperature limit, and maximum concrete temperature difference limit. This chapter will also provide recommendations for temperature monitoring techniques.

7.2 Mass Concrete Size Designation

Based on the ConcreteWorks analysis detailed in Section 6.2, it was determined that the element least dimension has a significant effect on the maximum concrete temperature, maximum concrete temperature difference, and maximum cracking risk. For this reason, it is important to determine what element least dimension should designate a member as mass concrete. As shown in Figure 3-1, eleven other state DOTs provide mass concrete size designations. Of those eleven states, 10 out of the 11 designate mass concrete members as any concrete element whose least dimension is greater than 4 ft or 5 ft.

The purpose of the mass concrete size designation is to inform the contractor when measures should be taken to monitor and regulate concrete temperatures to prevent DEF and thermal cracking. This is often accomplished by the development of a TCP. Based on the ConcreteWorks analysis data in Section 6.2.2.5, the use of SCMs and low-CTE coarse aggregates can significantly reduce the risk of DEF or thermal cracking in a mass concrete element. A concrete element of a certain least cross section dimension containing 100% PC and river gravel may have a high risk of thermal cracking. At the same time, another element with the same dimensions but containing limestone would have a lower risk of thermal cracking. It is important to take into account the cement composition and coarse aggregate type when determining whether or not a concrete element should be considered mass concrete. For this reason, a tiered mass concrete size designation is developed based on the cementitious materials composition and coarse aggregate type. A flow chart detailing this tiered specification is shown in Figure 7-1. Since coarse aggregate types other than limestone and river gravel could be used in Alabama, the coarse aggregate selection in Figure 7-1 is setup for limestone and other (river gravel). The values in this figure were determined using Table 6-6.

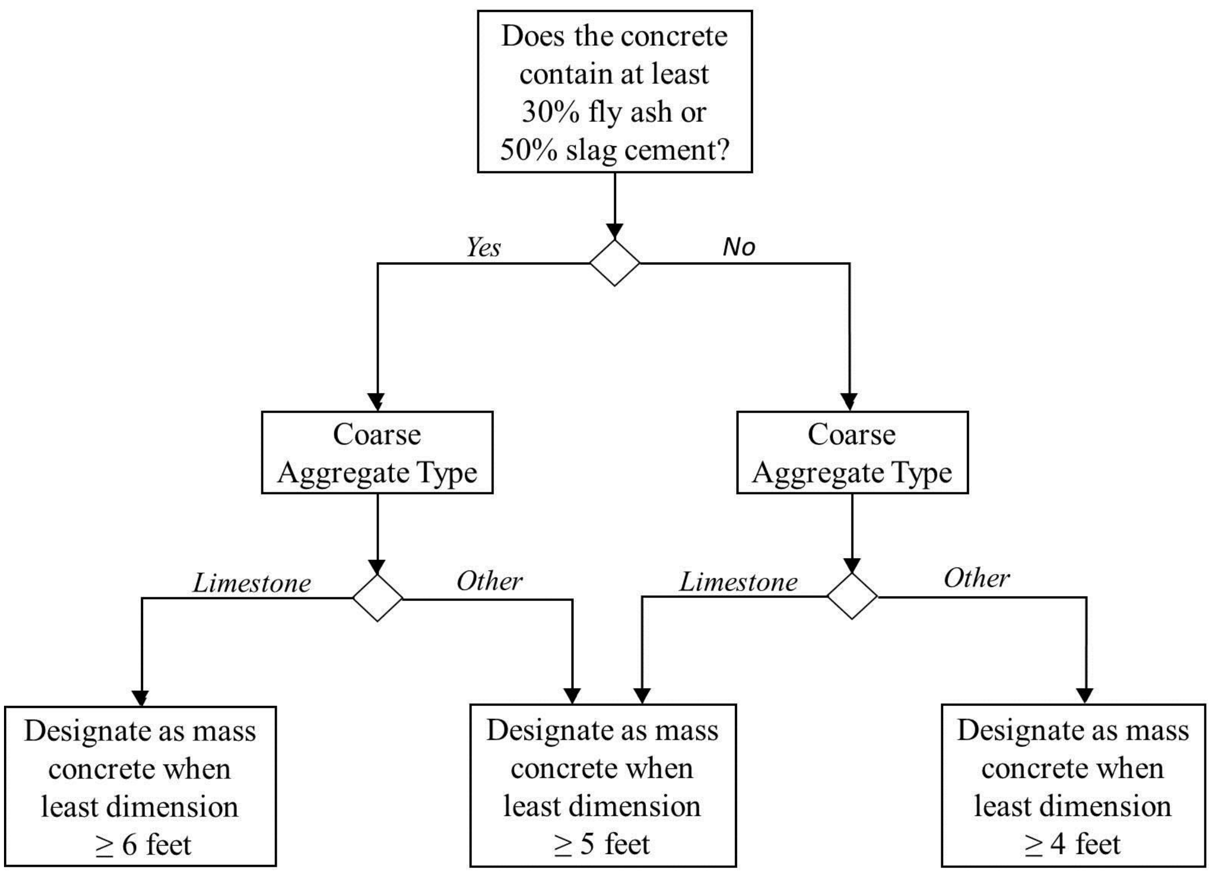

Figure 7-1: Mass Concrete Size Designation Flow Chart

If the cementitious materials or coarse aggregate type are unknown, a least dimension of 4 feet should be used to designate mass concrete members, as this corresponds to the least conservative case of other aggregate (river gravel) with no SCMs. The value of 4 ft also corresponds to the value specified by ACI 301 (2016) and some other state DOTs.

7.3 Maximum Concrete Temperature Limit

In order to prevent DEF in mass concrete elements, a maximum concrete temperature limit is required. As discussed in Section 2.3.3.1, the use of SCMs such as Class F fly ash and slag cement can aid in mitigating DEF in concrete. Although the expansion test data in the TxDOT sponsored research (Folliard et al., 2006) supports this conclusion, TxDOT did not adopt any elevated maximum concrete temperature limits for mass concrete applications. Because no testing was done during this research project regarding the use of SCMs to prevent DEF, it is recommended that ALDOT implement a maximum concrete temperature limit (T_{max}) of 158 °F. This value reflects the limit recommended by Taylor et al. (2001) described in Section 2.3.2. This value of 158 °F also corresponds to the majority of maximum temperature limits implemented by other U.S. DOTs, as previously demonstrated in Figure 3-3.

7.4 Maximum Concrete Temperature Difference Limit

7.4.1 Thermal Cracking Analysis to Develop an Age-Dependent Maximum Temperature Difference Limit

It is necessary for any mass concrete specification to define requirements to minimize the risk of thermal cracking. This is typically accomplished through a maximum concrete temperature difference limit. One of the primary objectives of this research was to investigate methods for predicting tensile strength development and creep effects in order to develop an age-dependent temperature difference limit.

In order to accomplish this objective, the time-dependent quantities from the thermal cracking equation (Equation 4-1) were calculated using the appropriate models previously defined in this report. This strength-dependent thermal cracking equation is shown in Equation 7-1, and the supporting strength and stiffness models are shown in Equations 7-2 through 7-5.

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

Where,

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

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

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

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

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

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

In order to calculate the concrete tensile strength, the concrete compressive strength must first be determined. Equation 7-2 contains the compressive strength model used in the B3 Model (Bazant and Baweja, 2000).

$$f_{cmt} = \left(\frac{t}{a + bt}\right) f_{cm28}$$ **Equation 7-2**

Where,

f_{cmt} = concrete mean compressive strength at any time t (psi),

t = age of the concrete (days),

a = 4.0 for moist-cured concrete with Type I cement (days),

b = 0.85 for moist-cured concrete with Type I cement (unitless), and

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

Equation 7-3 summarizes Raphael's (1984) tensile stress equation that allows for the conversion of concrete compressive strength to concrete tensile strength. This tensile strength relationship was selected because it was developed for mass concrete applications using splitting tension tests, which is the primary mechanism of thermal cracking in mass concrete elements.

$$f_t = 1.7 f_c^{2/3} \qquad \textbf{Equation 7-3}$$

Where,

f_t = concrete tensile strength (psi), and

f_c = concrete compressive strength (psi).

The concrete stiffness can also be calculated as a function of the concrete compressive strength. ACI 318 (2014) provides guidance on how to calculate the concrete modulus of elasticity, as shown in Equation 7-4.

$$E_C = 33(w_c)^{1.5}\sqrt{f_c} \qquad \textbf{Equation 7-4}$$

Where,

E_c = concrete modulus of elasticity (psi),

w_c = concrete unit weight (pcf), and

f_c = concrete compressive strength (psi).

For this research, the use of normalweight concrete in mass concrete applications was assumed. When using normalweight concrete, Equation 7-4 can be simplified into Equation 7-5 (ACI 318, 2014).

$$E_C = 57{,}000\sqrt{f_c} \qquad \textbf{Equation 7-5}$$

Once the strength and stiffness have been calculated, the final time-dependent variable to be quantified is the creep modification factor. Equation 7-6 contains the creep coefficient equation used in the B3 Model (Bazant and Baweja, 2000).

$$\varphi(t, t_o) = E(t_o)J(t, t_o) - 1 \qquad \textbf{Equation 7-6}$$

Where,

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

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

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

t = age of concrete, and

t_o = age of concrete loading.

In the B3 model, the creep coefficient is directly calculated from the compliance function. The compliance function found in the B3 model is shown in Equation 7-7.

$$J(t, t_o) = q_1 + C_o(t, t_o) + C_d(t, t_o, t_c) \qquad \textbf{Equation 7-7}$$

Where,

q_1 = instantaneous strain due to unit stress,

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

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

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

Finally, the B3 creep coefficient can be converted into a creep modification factor (ACI 209, 1982) using Equation 7-8.

$$K(t) = \frac{1}{1 + \varphi(t, t_o)} \qquad \textbf{Equation 7-8}$$

Where,

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

$K(t)$ = creep modification factor.

Effectively modeling the creep effects in concrete at early ages is essential in assessing the risk of thermal cracking in a concrete member. For this reason, the constant creep coefficient of 0.65 used by Bamforth (1995) was tested against time-dependent creep modification factors calculated using the B3 and Modified B3 models. Table 7-1 displays the input variables used in the calculation of the B3 and Modified B3 compliance values. As explained in Section 2.2.2.2, the original Bazant-Baweja (2000) B3 model significantly underestimates creep effects at early ages (Byard and Schindler, 2015). This claim is demonstrated by the graphs in Figure 7-2 and Figure 7-3, which display the compliance values and creep modification factors, respectively, calculated using both the B3 and Modified B3 models.

Table 7-1: Summary of Input Values for B3 Model Compliance Calculations

Input	**Symbol**	**Value**
Mean 28-day compressive strength	f'_{cm28}	5,200 psi
Cement Content	c	620 lb/yd^3
Empirical Constants	m	0.5
	n	1
Water-cement ratio	w/c	0.44
Aggregate-cement ratio	a/c	4.79

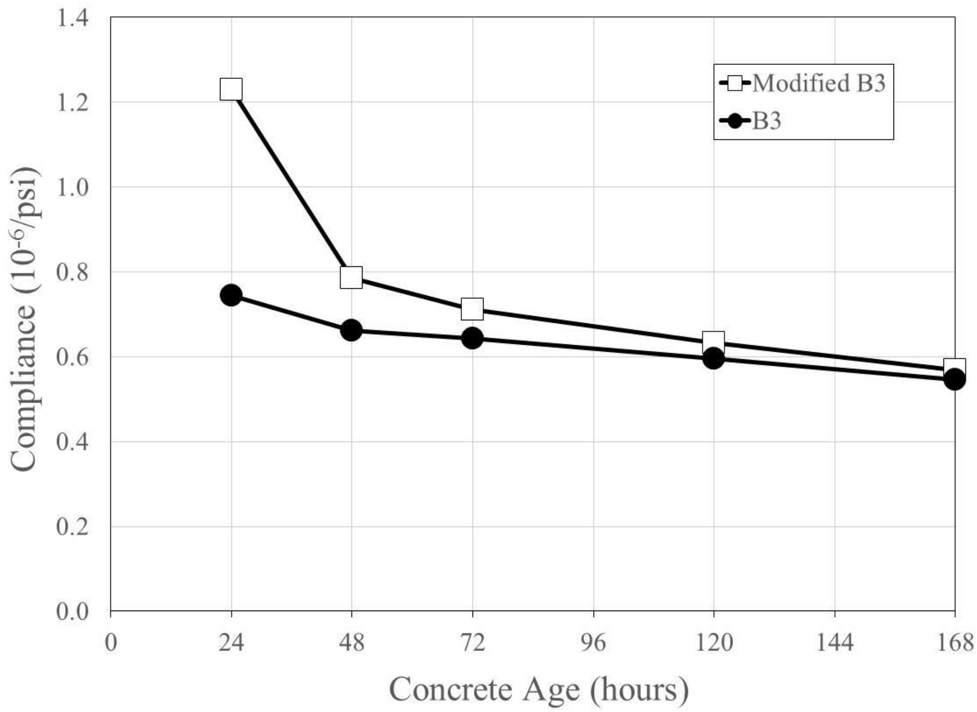

Figure 7-2: Compliance Values – B3 versus Modified B3

Figure 7-2 shows the compliance values calculated using time steps of 0-24 hours, 24-48 hours, 48-72 hours, 72-120 hours, and 120-168 hours. It can be noted from this figure that the compliance calculations for the B3 model and Modified B3 model differ greatly during early concrete ages, and converge after approximately seven days. Evidence for this statement can also be found in the calculation of creep factors during the first seven days, shown in Figure 7-3.

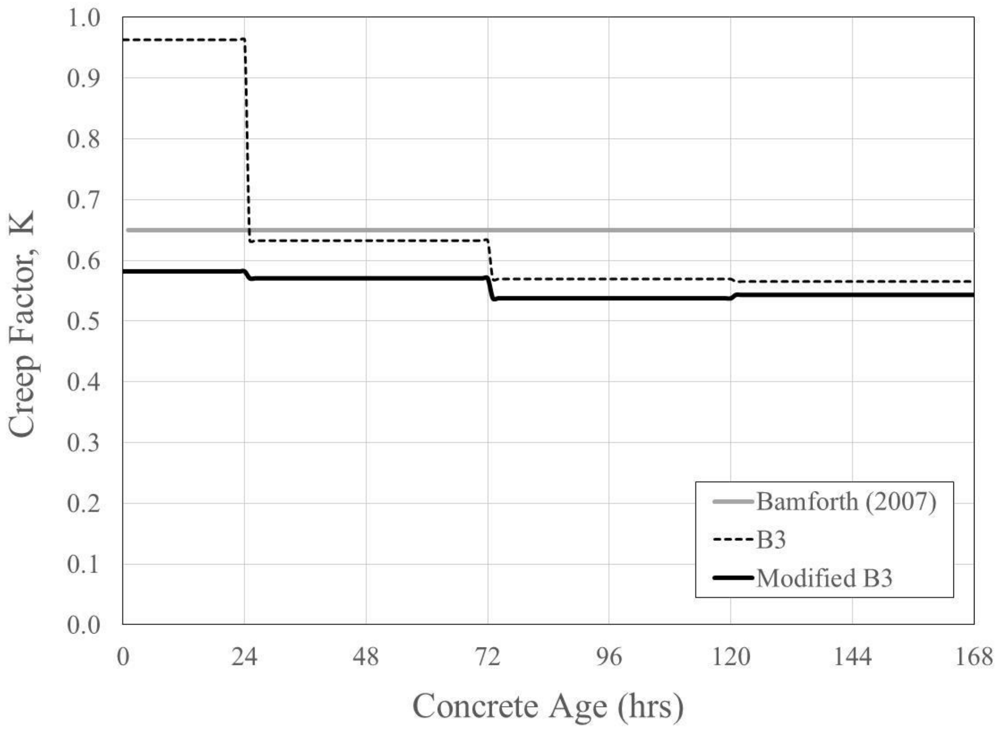

Figure 7-3: Creep Factors – Bamforth versus B3 versus Modified B3

In Figure 7-3, the sudden jumps in creep coefficient at 24 hours, 72 hours, and 120 hours are the ages where the loads were applied (t_o).These creep coefficients make it clear that for an early-age thermal cracking analysis, a creep coefficient from the Modified B3 model should be used in lieu of the B3 model (Byard and Schindler, 2015).

Age-dependent temperature difference limits calculated using Equation 7-1 and the strength, stiffness, and creep models from Equations 7-2 through 7-5. These limits are plotted in Figures 7-4 and 7-5 for limestone and river gravel, respectively. These figures were developed by assuming a restraint factor (R) of 0.42 and a specified 28-day compressive strength (f'_c) of 4,000 psi. Creep coefficients from both the Bamforth and the Modified B3 Model were used for the sake of comparison. A sample calculation of the maximum allowable temperature difference at 24 hours for limestone using the Modified B3 Model's creep coefficient is shown for reference.

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

$$= \frac{178\, psi}{1887000\, psi \times\ 0.582 \times 5.52 \times 10^{-6}\ \ in./in./°\text{F} \times 0.42} = \mathbf{70}\ °\text{F}$$

The sudden jumps in the Modified B3 lines shown in Figures 7-4 and 7-5 indicate a change in loading age (t_o).

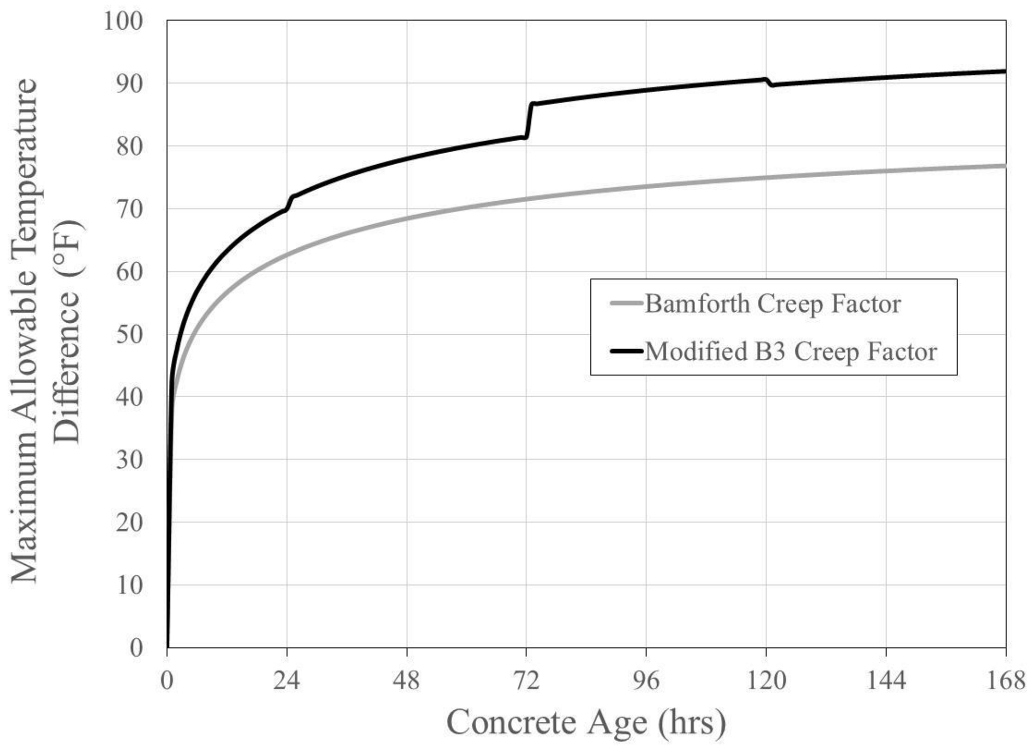

Figure 7-4: 7-day Maximum Allowable Temperature Difference Limits – Limestone

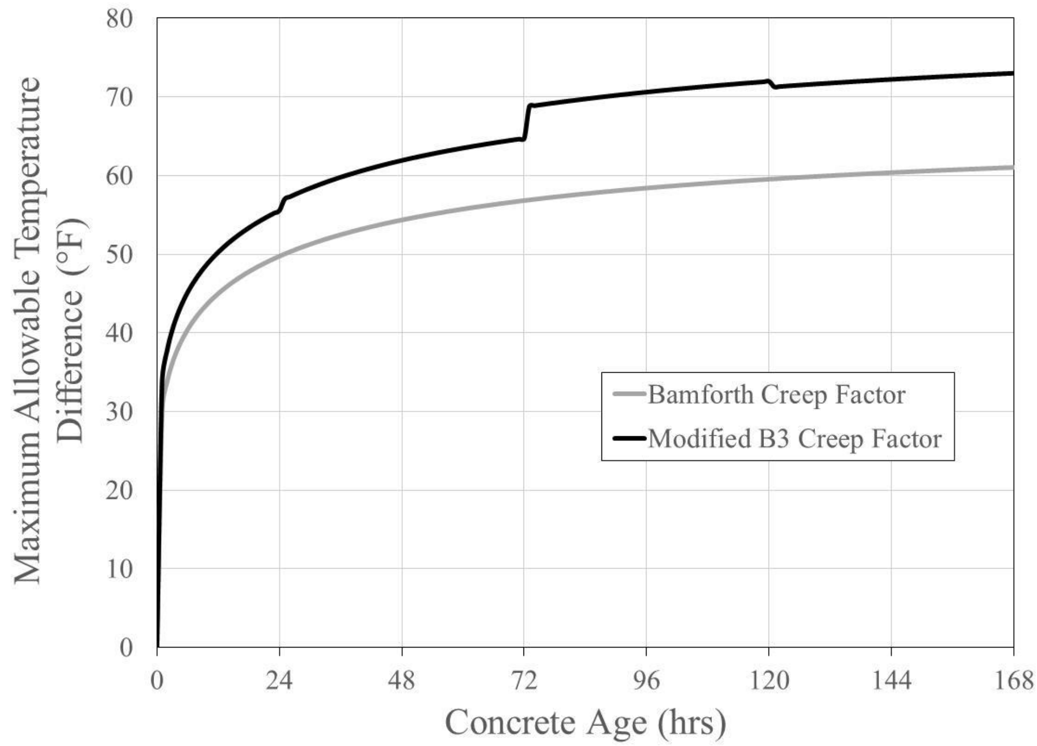

Figure 7-5: 7-day Maximum Allowable Temperature Difference Limits – River Gravel

Based on Figures 7-4 and 7-5, the use of the Bamforth creep factor of 0.65 produced similar or lower (more conservative) maximum allowable temperature difference limits when compared to the results when using the Modified B3 creep factors for different time steps. During the first 168 hours, the Modified B3 creep factor temperature difference limit remains within approximately 10 °F of the Bamforth creep factor limit. Because it is proven to be both simple, accurate, and conservative, it is recommended that the Bamforth creep coefficient of 0.65 be used for all age-dependent temperature difference limit calculations in ALDOT specification.

7.4.2 Development of Maximum Concrete Temperature Difference Limit Using the Bamforth Creep Coefficient

After identifying the appropriate method for determining the creep factor, a summary graph was developed showing the age-dependent maximum temperature difference limits for both limestone and river gravel concretes. This graph is shown in Figure 7-6. The limestone concrete

CTE was assumed to be 5.52 × 10^{-6} in./in./°F, while the river gravel concrete CTE was assumed to be 6.95 × 10^{-6} in./in./°F (Schindler et al., 2010).

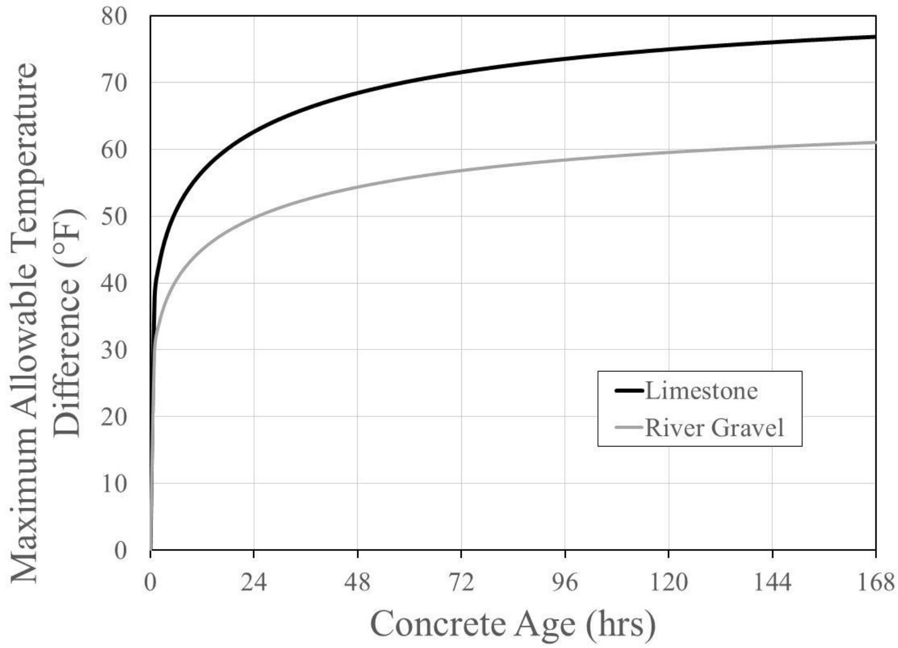

Figure 7-6: Allowable Temperature Difference Limits – Limestone and River Gravel

The limits in Figure 7-6 reflect calculations performed using a restraint factor (R) of 0.42 and a concrete mean 28-day compressive strength (f_{cm28}) of 5,200 psi. The concrete mean compressive strength is not the same as the specified concrete compressive strength (ACI 214, 2011). The mean compressive strength of a concrete element is typically higher than the specified 28-day strength. To account for this, ACI 214 (2011) provides guidance on how to calculate the concrete mean compressive strength as a function of the specified 28-day strength. The information from this report is shown in Table 7-2.

Table 7-2: Calculation of Actual Concrete Compressive Strength (ACI 214, 2011)

Required average compressive strength	**Specified compressive strength**
$f'_{cm} = f'_c + 1{,}000$ psi	when $f'_c < 3{,}000$ psi
$f'_{cm} = f'_c + 1{,}200$ psi	when $f'_c \geq 3{,}000$ psi and $f'_c \leq 5{,}000$ psi
$f'_{cm} = 1.10 f'_c + 700$ psi	when $f'_c > 5{,}000$ psi

Because a typical 28-day specified strength for ALDOT mass concrete elements is 4,000 psi, a value of 5,200 psi for f_{cm28} was used in this analysis based on Table 7-2.

For the sake of simplification, a table of values was assembled containing the temperature difference limit for the river gravel concrete. These values are displayed in Table 7-3.

Table 7-3: Allowable Maximum Temperature Differences with River Gravel Concrete

Concrete Age, *t* (hours)	**Maximum Allowable Temperature Difference, $\Delta T_{max}(t)$ (°F)**
12	45
24	50
36	53
48	54
60	56
72	57
84	58
96	58
108	59
120	60
132	60
144	60
156	61
168	61

It should be noted, however, that the limits in Table 7-3 can be adjusted proportionally to account for other values of CTE. An increase in CTE will result in a decrease in maximum allowable concrete temperature difference. In order to easily modify the temperature difference values in Table 7-3 for concrete CTE values other than 6.95×10^{-6} in./in./°F (default used for river gravel in Figure 7-6), a CTE modification factor is introduced. This modification factor is shown in Equation 7-9.

$$F_{CTE} = \frac{CTE_{assumed}}{CTE_{actual}}$$ **Equation 7-9**

Where,

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

$CTE_{assumed}$ = assumed concrete CTE (6.95×10^{-6} in./in./°F in Table 7-3), and

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

After determining the appropriate CTE modification factor, the temperature difference values from Table 7-3 can be modified according to the Equation 7-10.

$$\Delta T_{max,modified}(t) = F_{CTE} \times \Delta T_{max}(t) \qquad \textbf{Equation 7-10}$$

Where,

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

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

$\Delta T_{max}(t)$ = allowable maximum temperature limit as a function of concrete age (°F) (listed in Table 7-3).

Equation 7-10, in conjunction with the values in Table 7-3, can be used to easily and accurately determine an age-dependent temperature difference limit for any mass concrete member with a known concrete CTE. The concrete CTE can be determined by testing the project concrete in accordance with AASHTO T336 (2011).

It is best to test the concrete CTE using AASHTO T336 (2011). However, if the concrete CTE is not tested, default CTE values can be used. The default CTE values recommended for specification are included in Table 7-4.

Table 7-4: Default CTE Values for Concretes Made with Various Coarse Aggregate Types (Schindler et al., 2010)

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

If the coarse aggregate type is unknown, and thus no default CTE value is available, an age-dependent allowable temperature difference limit should not be used.

To account for all conditions, a tiered temperature difference specification is provided in Table 7-5. This table should provide the engineer with a clear direction for an appropriate mass concrete temperature difference specification, whether the coarse aggregate type and concrete CTE are known or unknown.

Table 7-5: Tiered Maximum Temperature Difference Limit for a Mass Concrete Specification

Tier	Requirement	Specification
I	CTE of project concrete is tested according to AASHTO T336	Use ***known*** CTE value to calculate ***age-dependent ΔT limit*** in accordance with Table 7-3 and modified with Equation 7-10
II	Coarse aggregate type of project concrete is known, but concrete CTE has not been tested	Use ***default*** CTE value from Table 7-4 to calculate ***age-dependent ΔT limit*** in accordance with Table 7-3 and modified with Equation 7-10
III	Coarse aggregate type of project concrete is unknown	Use maximum ΔT limit of ***35 °F***

7.4.3 Application of the Tiered Maximum Temperature Difference Limits on Instrumented ALDOT Mass Concrete Elements

In order to demonstrate the impact of the tiered specification on ALDOT mass concrete construction, the limits from Table 7-5 were calculated for each of the six ALDOT mass concrete members instrumented during this project. These limits were then plotted against the measured concrete temperature difference data for each of the six instrumented elements. The laboratory-tested CTE values for each element used to calculate the Tier I maximum temperature difference limits are summarized in Table 7-6.

Table 7-6: CTE Values for the Instrumented ALDOT Mass Concrete Elements

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

The maximum concrete temperature difference limit lines and measured concrete temperature data for each of the six ALDOT mass concrete elements are plotted in Figures 7-7 through 7-12.

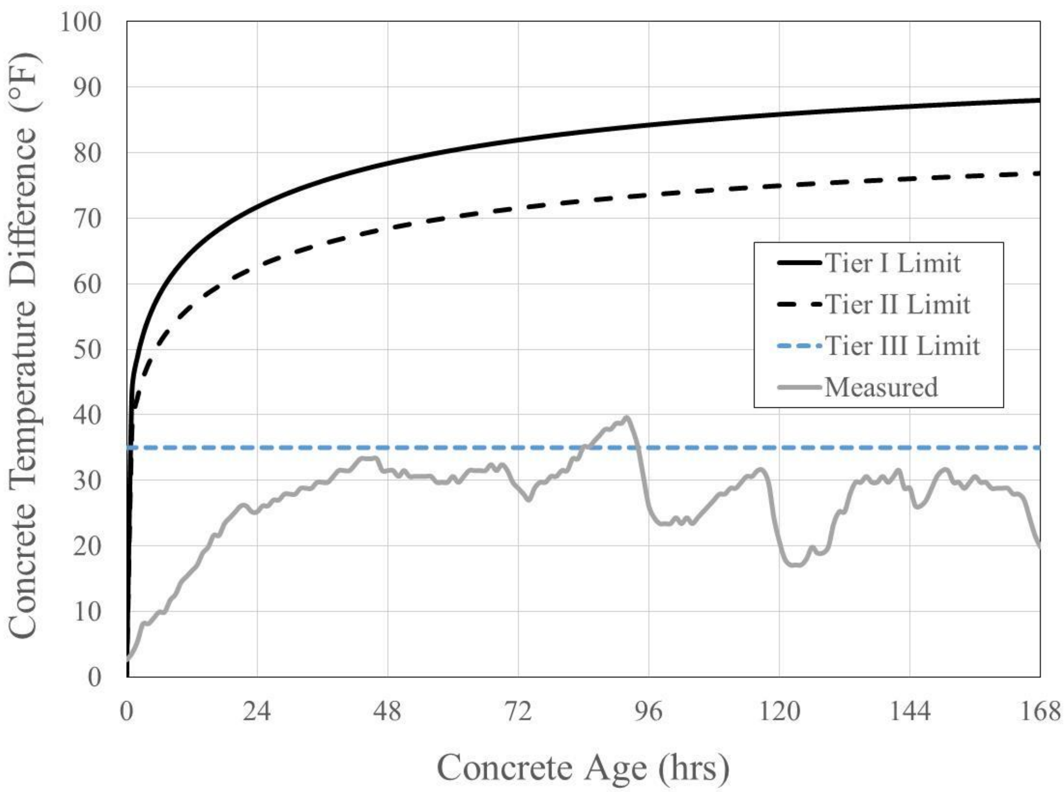

Figure 7-7: Albertville Bent Cap Temperature Difference Data versus Potential Limits

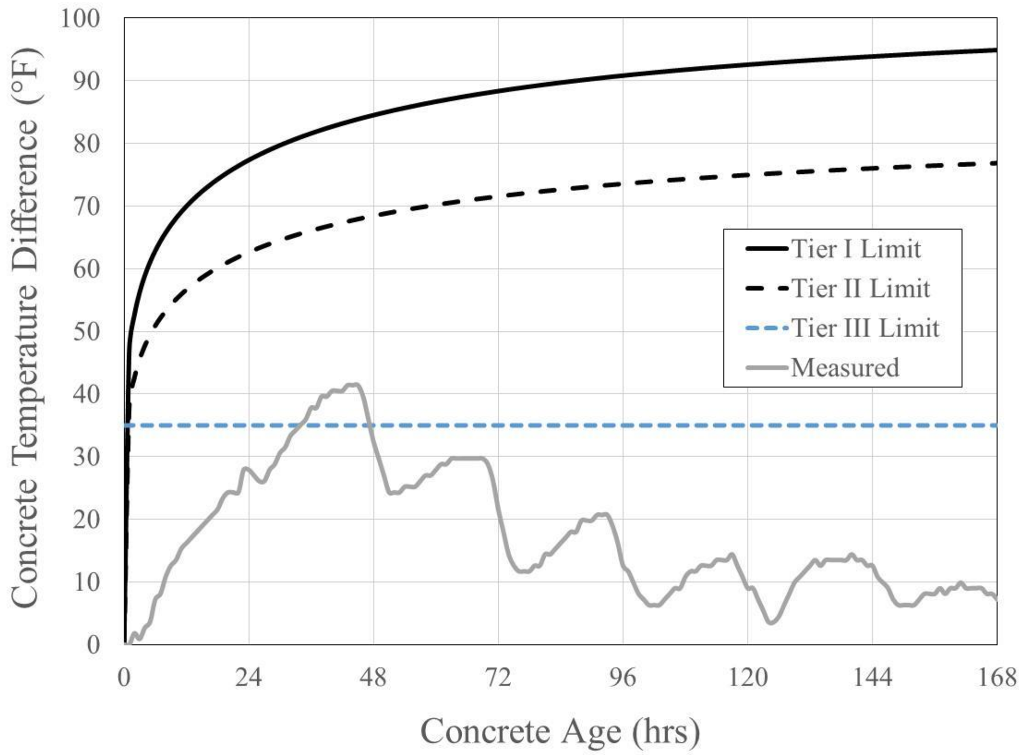

Figure 7-8: Harpersville Crashwall Temperature Difference Data versus Potential Limits

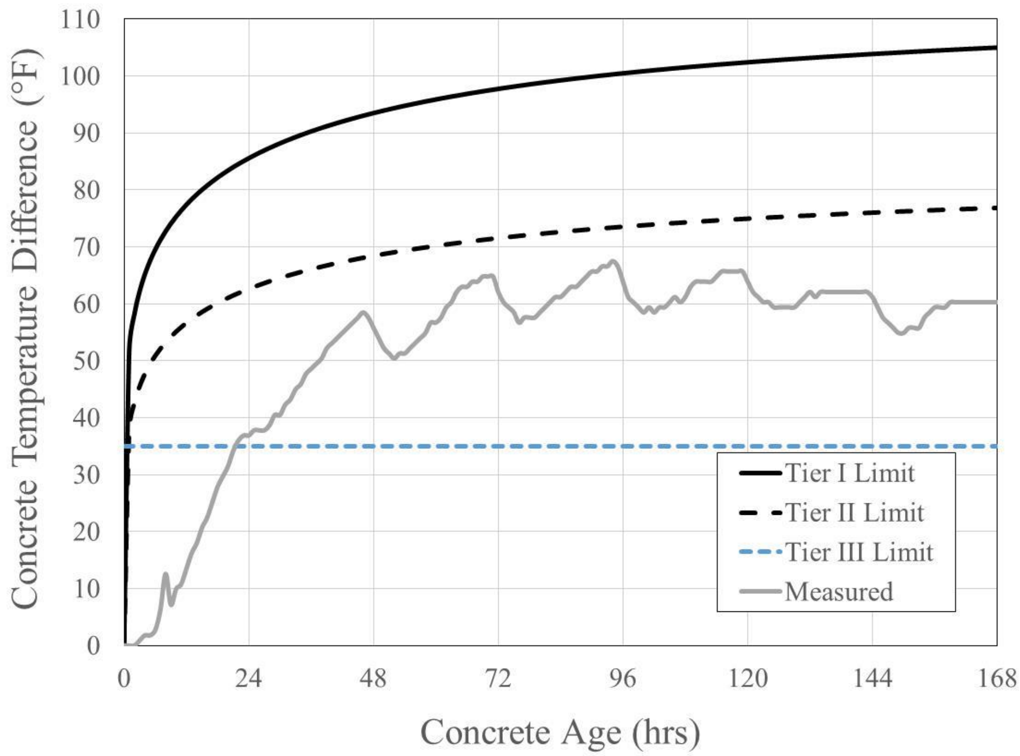

Figure 7-9: Scottsboro Pedestal Temperature Difference Data versus Potential Limits

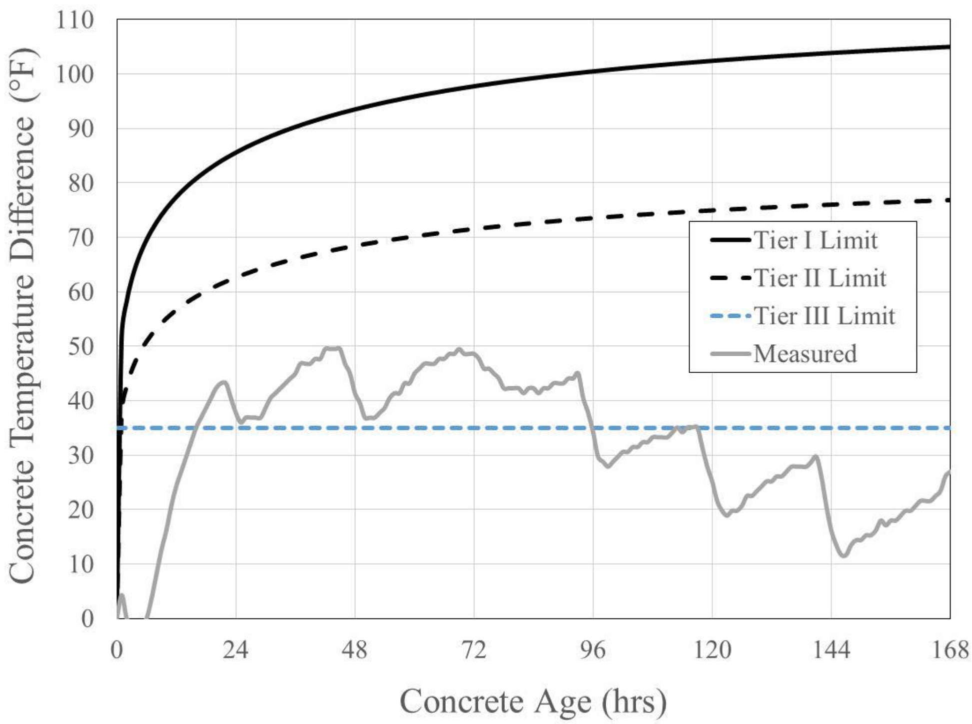

Figure 7-10: Scottsboro Bent Cap Temperature Difference Data versus Potential Limits

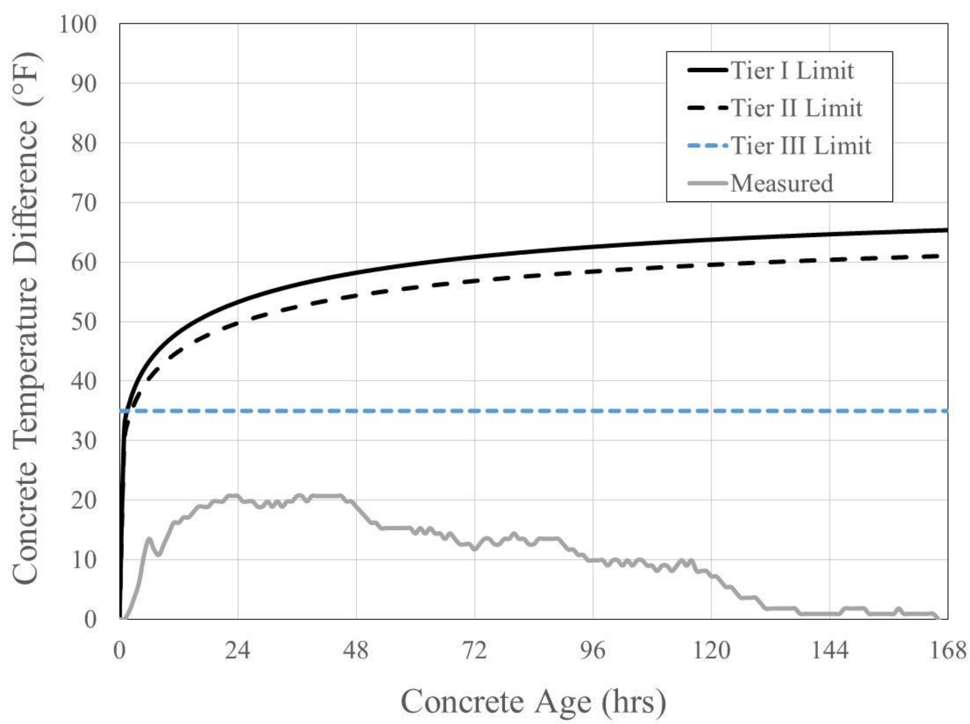

Figure 7-11: Elba Bent Cap Temperature Difference Data versus Potential Limits

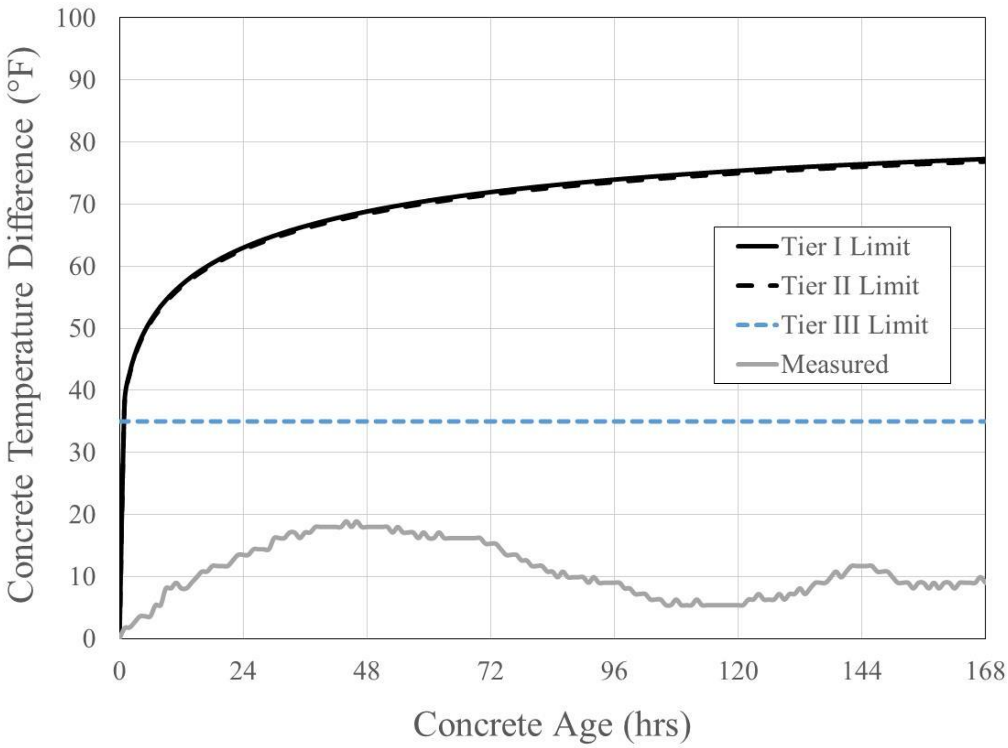

Figure 7-12: Birmingham Column Temperature Difference Data versus Potential Limits

As shown in Figures 7-7 through 7-12, the measured maximum temperature differences for four out of the six elements exceed the 35 °F limit (Tier III). In the case of the Scottsboro pedestal, the temperature difference exceeded the 35 °F limit at approximately 24 hours, reached a maximum of 68 °F at 94 hours, and was still well over 35 °F at 7 days. However, the temperature difference never exceeded the limit lines for Tier I or Tier II at any point in the first 168 hours after placement. Based on the 35 °F limit, extensive thermal cracking would be expected in this element. However, no thermal cracking was observed. This supports the validity of the age-dependent temperature difference limits and affirms the overly conservative nature of the 35 °F limit. In summary, none of the six elements showed signs of early-age thermal cracking. Correspondingly, the temperature difference data for all six elements never exceeded the Tier I or Tier II concrete temperature difference limits. This leads to the conclusion that the use of the

Tier I and Tier II approach are more representative of in-place behavior than the Tier III approach. A maximum concrete temperature difference limit that is calculated as a function of time, rather than is a constant 35 °F limit, is also used by the Iowa DOT mass concrete specification.

7.5 Temperature Monitoring Recommendations

7.5.1 Duration of Temperature Monitoring

In mass concrete construction, it is important to monitor temperatures and ensure that temperature limits defined in the specification are not exceeded. Temperature monitoring in mass concrete elements should be performed for a duration of 7 days after concrete placement, as the maximum concrete temperature and maximum concrete temperature difference are likely to occur during the first 7 days after placement. For each of the mass concrete elements instrumented and modeled during this research, the maximum concrete temperature and maximum concrete temperature difference was reached within 7 days after concrete placement. This claim is evidenced by the measured temperature data and the ConcreteWorks analyses presented in Chapter 5 and Chapter 6, respectively. The Arkansas DOT and Idaho DOT mass concrete specifications also require temperature monitoring for a duration of 7 days after concrete placement.

7.5.2 Placement Location of Temperature Monitoring Sensors

Before installing a temperature monitoring system in a mass concrete element, it is important to know the proper cross section at which to install sensors, as well as the proper locations within each cross section. Redundancy is important when installing temperature sensors, as it is not uncommon for sensors be damaged during concrete placement. For this reason, it is recommended that multiple sensor cross sections be instrumented within each mass concrete element.

Within each instrumented cross section, sensors should be installed at the element core, where the maximum concrete temperature is expected to occur. It is recommended that at least two sensors be installed at the element core for each cross section, as the core temperature data are essential for both the maximum concrete temperature and concrete temperature difference data.

Thermal cracks typically originate at the concrete surface, as this is the location of the largest concrete temperature difference. For this reason, sensors should also be installed at the element edge, where thermal cracks are most likely to develop. Thermal cracks are likely to develop at the face of the element, rather than the corner of the element, because internal restraint is the highest at the element face. If edge sensors are installed too deeply beneath the surface of the concrete element, the sensors no longer reflect a reasonably accurate measurement of the concrete temperature at the element edge. This means that the measured concrete temperature difference between the core and the edge would appear to be significantly lower than it actually is. Despite this, it is often most practical to install sensors on the reinforcement cage before the erection of the side forms. For this reason, the exact depth of the edge sensor beneath the surface of the element may be difficult to predict. Thus, it is recommended to simply install the edge sensors on the rebar cage of the element, not at the surface. When mounted on the reinforcement cage, the sensors should always be installed facing the concrete surface, rather than the concrete interior.

During this research, sensors were installed both at the surface and on the rebar of six ALDOT mass concrete elements. All of the elements had a reinforcement clear cover of approximately 2 inches, except for the Scottsboro Pedestal, which had a clear cover of approximately 6 inches. A larger reinforcement clear cover means a larger distance between the rebar sensor and the edge sensors. Figures 7-13 and 7-14 contain the measured 7-day temperature difference data between sensors placed on the rebar and the edge of the element for all six elements instrumented during this project. Figure 7-13 contains the data for the Albertville Bent Cap, Scottsboro Bent Cap, and Elba Bent Cap. Figure 7-14 contains the data for the Harpersville Crashwall, Scottsboro Pedestal, and Birmingham Column. Separate graphs were created in order to make the lines for each element more distinguishable.

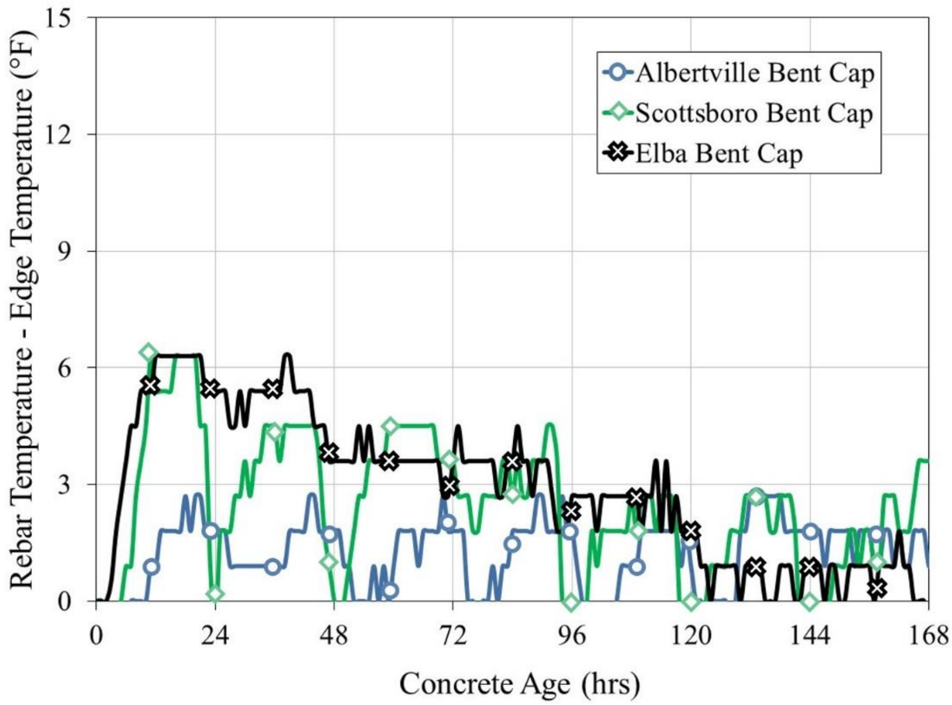

Figure 7-13: Temperature Difference between Edge and Rebar Sensors Measured for the Albertville Bent Cap, Scottsboro Bent Cap, and Elba Bent Cap

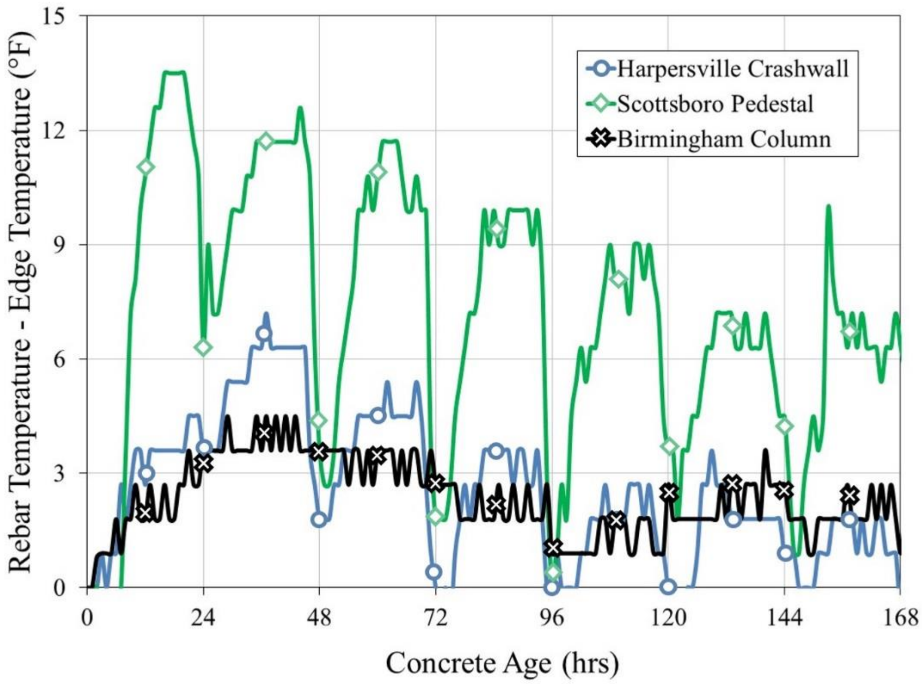

Figure 7-14: Temperature Difference between Edge and Rebar Sensors Measured for the Harpersville Crashwall, Scottsboro Pedestal, and Birmingham Column

As evidenced by Figures 7-13 and 7-14, the further the rebar sensor is from the edge sensor, the larger the temperature difference between the two locations. In the scenario where the reinforcement clear cover at the location of the edge sensor is greater than 6 in., measures should be taken to ensure that the edge sensor remains within 6 in. of the concrete surface to obtain a temperature difference that is not too different from the maximum possible. This can be accomplished by installing additional reinforcement that extends into the cover area at the location of the edge sensor.

The proper cross section for installing temperatures sensors is dependent upon the element type and placement condition. In general, sensors should be installed at the location of the maximum concrete temperature and the location of the highest risk of thermal cracking. For rectangular bent caps, rectangular columns, and rectangular elements placed on top of previous concrete placements, the sensor cross sections used during the instrumentation phase of this project may be referenced. Figure 7-15 displays the proper location of sensor cross sections for a rectangular bent cap cast on top of two piers. Schematics displaying the proper location of sensors within each cross section of a horizontal rectangular element (e.g. bent cap), a vertical rectangular element (e.g. square column), and a vertical circular element (e.g. circular pier) are shown in Figures 7-18, 7-19 and 7-20, respectively.

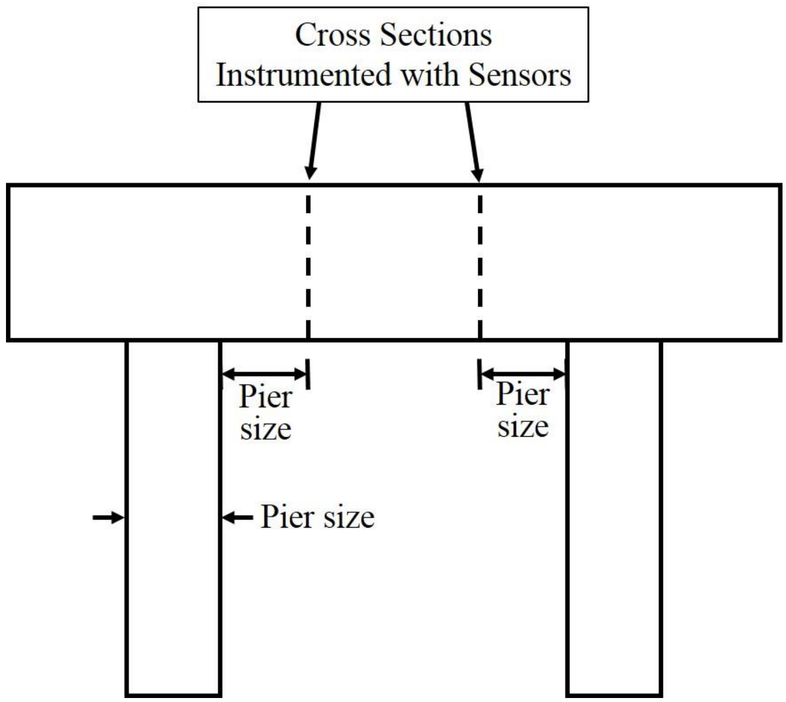

Figure 7-15: Recommended Location of Sensor Cross Sections in a Rectangular Bent Cap

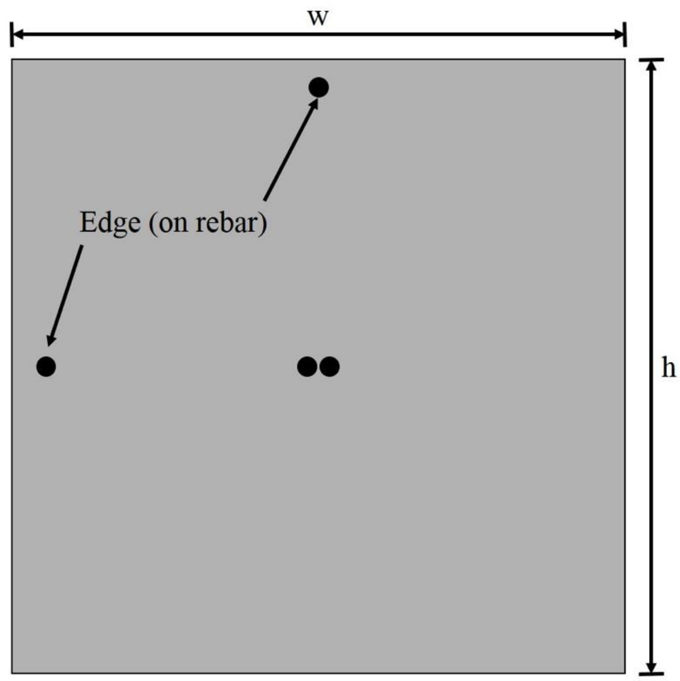

Figure 7-16: Recommended Sensor Locations in a Horizontal Rectangular Concrete Element (e.g. Bent Cap)

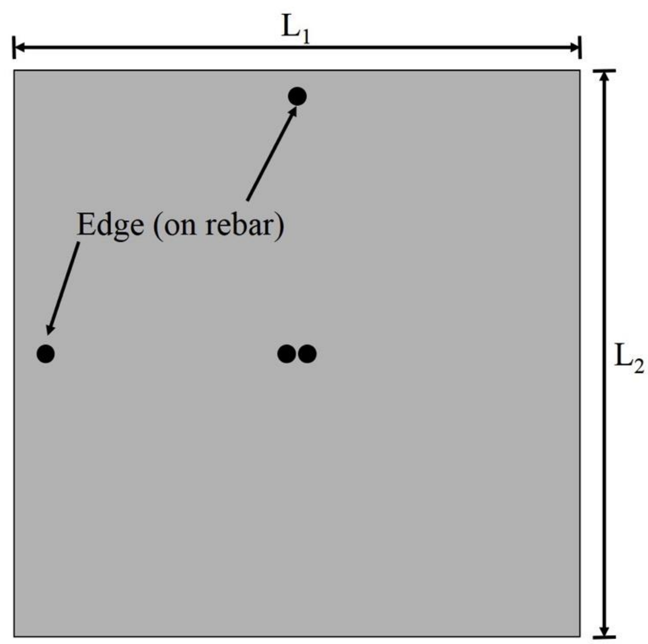

Figure 7-17: Recommended Sensor Locations in a Vertical Rectangular Concrete Element (e.g. Square Column)

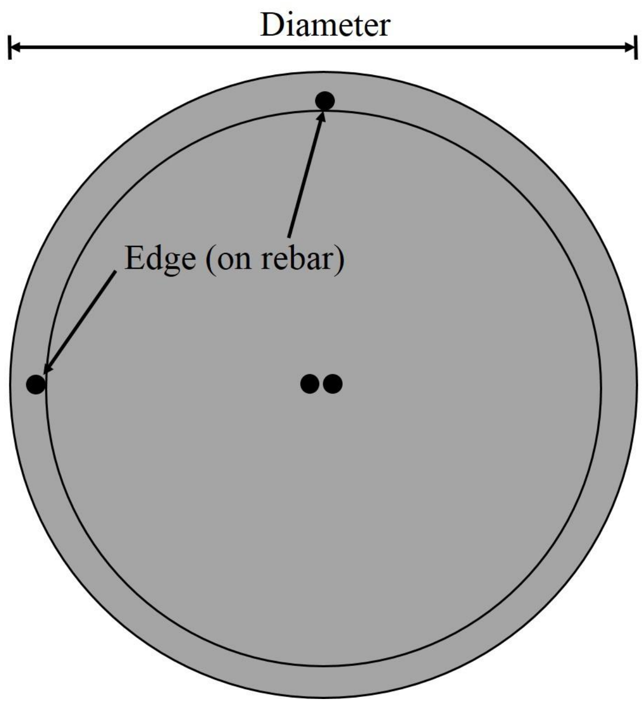

Figure 7-18: Recommended Sensor Locations in a Vertical Circular Concrete Element (e.g. Circular Pier)

CHAPTER 8: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

8.1 Summary of Project

The research described in this thesis was performed as a part of larger research project funded by ALDOT in order to develop a specification for ALDOT mass concrete construction. This phase of the project focused primarily on establishing a size designation and appropriate concrete temperature limits for ALDOT mass concrete construction. In order to accomplish these tasks, ConcreteWorks software analyses, field instrumentation of ALDOT mass concrete elements, and numerical modeling of thermal cracking in mass concrete elements were performed.

In order to produce a thorough ConcreteWorks software analysis of Alabama mass concrete elements, 480 mass concrete placements were modeled to examine the effect of various placement variables on the temperature development and cracking risk. The input variables examined include placement date/time, placement location, cementitious materials, coarse aggregate type, and element least dimension. The three output variables recorded include the maximum concrete temperature, maximum concrete temperature difference, and maximum concrete cracking risk. The output data from each placement model were compiled, and statistical analysis was performed to determine which input variables had the greatest impact on each output variable. These data aid in the development of an appropriate least dimension designation for an ALDOT mass concrete specification.

During the field instrumentation phase of this project, six ALDOT bridge elements were instrumented with temperature sensors. Sensors were installed in the element core and at various locations along the element edge where thermal cracks were most likely to develop. Installing sensors in these locations enabled researchers to determine the maximum concrete temperature and maximum concrete temperature difference during the early ages after placement. When possible, sensors were left active after the collection of early-age data so that long-term data may be obtained and analyzed for future research. Concrete materials were obtained for each element so that laboratory testing may be performed. Through laboratory testing, the hydration parameters, concrete strength, and concrete CTE were determined. These values enabled researchers to produce an accurate ConcreteWorks inputs for each of the six instrumented elements. Next, the ConcreteWorks temperature predictions were compared to measured temperature data in order to assess the accuracy of ConcreteWorks temperature predictions for Alabama mass concrete applications. Multiple site visits have taken place in order to examine each element for signs of DEF and thermal cracking.

The thermal cracking equation of Bamforth (2007) was modified to produce an age-dependent maximum allowable concrete temperature difference limit that more accurately reflects the observed response of Alabama concrete. In order to accomplish this task, methods for predicting the concrete strength, stiffness, and creep effects were analyzed. By modeling each of these factors as age-dependent quantities, an age-dependent temperature difference limit can be produced for concrete placements with various restraint conditions and concrete CTE values.

Finally, all of the data obtained through each phase of this project were used to develop appropriate concrete temperature requirements and a mass concrete size designation for inclusion in a future ALDOT mass concrete specification.

8.2 Research Conclusions

- ConcreteWorks is sufficiently accurate in predicting temperatures for Alabama mass concrete elements, as long as the proper hydration parameters are applied to the model. In some cases, the default hydration parameters calculated by ConcreteWorks are sufficiently accurate.

- During the field instrumentation phase of this project, maximum concrete temperatures were measured above 158 °F. Though no signs of DEF were observed, it is too early to see signs of DEF.
- During the field instrumentation phase of this project, maximum concrete temperature differences were measured above 35 °F with no signs of thermal cracking. This observation supports the hypothesis the 35 °F limit is over-conservative for many mass concrete elements, particularly those containing limestone aggregate in the concrete.
- Based on a ConcreteWorks analysis, it was determined that while the coarse aggregate type does have a significant effect on the risk of thermal cracking, it has little to no effect on the maximum concrete temperature difference.
- In order to mitigate DEF and thermal cracking in mass concrete elements, the element least dimension, placement temperature, use of SCMs, and coarse aggregate type are the most significant variables.
- For ALDOT construction, a maximum concrete temperature limit of 158 °F should be applied to all elements classified as mass concrete to minimize the risk of DEF occurring.
- For ALDOT construction, the cementitious materials and coarse aggregate type should be used to determine the appropriate least dimension designation for mass concrete elements, as shown in Figure 7-1. If either of these raw concrete material

attributes are unknown, a least dimension of 4 ft should be used to designate members as mass concrete.

- For ALDOT construction, the concrete CTE should be used to determine an age-dependent temperature difference limit for the first 7 days after placement. The maximum allowable temperature difference limits calculated using a river gravel concrete CTE of 6.95×10^{6} in./in./°F are shown in Table 8-1. If the concrete CTE is tested according to AASHTO T336 (2011), the values in Table 8-1 can be adjusted using Equations 8-1 and 8-2. If the concrete CTE is unknown, default CTE values for Alabama concrete with different coarse aggregate types can be found in Table 8-2. If the coarse aggregate type is unknown, a maximum allowable temperature of 35 °F should be used. Table 8-3 summarizes the tiered temperature difference limit specification developed during this research.

Table 8-1: Allowable Maximum Temperature Differences with River Gravel Concrete

Concrete Age, t (hours)	**Maximum Allowable Temperature Difference, $\Delta T_{max}(t)$ (°F)**
12	45
24	50
36	53
48	54
60	56
72	57
84	58
96	58
108	59
120	60
132	60
144	60
156	61
168	61

$$F_{CTE} = \frac{CTE_{assumed}}{CTE_{actual}}$$ **Equation 8-1**

Where,

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

$CTE_{assumed}$ = assumed concrete CTE (6.95×10^{-6} in./in./°F in Table 8-1), and

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

$$\Delta T_{max,modified}(t) = F_{CTE} * \Delta T_{max}(t)$$ **Equation 8-2**

Where,

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

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

$\Delta T_{max}(t)$ = allowable maximum temperature limit as a function of concrete age (°F) (listed in Table 8-1).

Table 8-2: Default CTE Values for Concretes Made with Various Coarse Aggregate Types (Schindler et al., 2010)

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

Table 8-3: Tiered Maximum Temperature Difference Limit for a Mass Concrete Specification

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

- For all ALDOT concrete elements classified as mass concrete, temperature monitoring sensors should be installed to ensure that the temperature requirements outlined in the ALDOT mass concrete specification are met for a duration of 7 days after placement.
- Edge temperature sensors should be installed on the rebar cage at no more than 6 in. from the concrete surface.

8.3 Research Recommendations

- If accessible, the six ALDOT bridge elements instrumented with temperature sensors during this project should be monitored for signs of DEF and thermal cracking.
- Temperature data should be obtained for ALDOT mass concrete elements of different types, sizes, and placement conditions than those elements whose data are presented in this project. Ideally, each new element instrumented with temperature sensors should check an empty box in Table 4-1.
- Diagrams should be developed that detail the proper temperature sensor installment locations for mass concrete element types not covered by this research project.

- The maturity method can be used to account for the effect of in-place temperatures on concrete properties. The temperature data in this project reflect that the edge temperatures exceeded 73 °F, even in winter applications. Because of this, maturity is currently neglected in the recommendations made in this work, which should result in conservative maximum allowable temperature differential limits. It is recommended that the use of the maturity method be investigated to determine the maximum allowable temperature difference limit.
- Before implementing an ALDOT mass concrete specification, developers of the specification should train the necessary ALDOT personnel in order to understand and implement the temperature control requirements detailed in the specification.

REFERENCES

AASHTO. 2016. LRFD Bridge Construction Specifications, 3rd Edition, with 2016 Interim Revisions. American Association of State Highway and Transportation Officials. Washington, D

AASHTO T 119. 2007. Slump of Hydraulic Cement Concrete. American Association of State Highway and Transportation Officials. Washington D.C.

AASHTO T 152. 2005. Air Content of Freshly Mixed Concrete by the Pressure Method. American Association of State Highway and Transportation Officials. Washington D.C.

AASHTO T325. 2004. Standard Method of Test for Estimating the Strength of Concrete in Transportation Construction by Maturity Tests. American Association of State Highway and Transportation Officials. Washington D.C.

AASHTO T 336. 2011. Coefficient of Thermal Expansion of Hydraulic Cement Concrete. American Association of State Highway and Transportation Officials. Washington D.C.

ACI. 2016. *Technical Committee Manual*. American Concrete Institute. Farmington Hills, MI.

ACI 207. 2005. *Guide to Mass Concrete.* American Concrete Institute. Farmington Hills, MI.

ACI 207. 2005. *Cooling and Insulating Systems for Mass Concrete*. American Concrete Institute. Farmington Hills, MI.

ACI 207. 2007. *Report on Thermal and Volume Change Effects on Cracking of Mass Concrete*. American Concrete Institute. Farmington Hills, MI.

ACI 209. 1982. Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures. American Concrete Institute. Farmington Hills, MI.

ACI 209. 1992. *Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures*. American Concrete Institute. Farmington Hills, MI.

ACI 209. 2008. *Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete*. American Concrete Institute. Farmington Hills, MI.

ACI 211. 1991. *Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete*. American Concrete Institute. Farmington Hills, MI.

ACI 214. 2011. *Guide to Evaluation of Strength Test Results of Concrete*. American Concrete Institute. Farmington Hills, MI.

ACI 301. 2016. *Specifications for Structural Concrete*. American Concrete Institute. Farmington Hills, MI.

ACI 318. 2014. *Building Code Requirements for Structural Concrete and Commentary*. American Concrete Institute. Farmington Hills, MI.

Alabama DOT. 2008. Maturity Method to Determine Early-Age Strengths of Concrete. Alabama Department of Transportation. Montgomery, AL.

Arkansas DOT. 2014. Standard Specifications for Highway Construction. Arkansas State Highway and Transportation Department. Little Rock, AR.

ASTM C 39. 2015. Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens. *ASTM International*. West Conshohocken, PA.

ASTM C 192. 2015. Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory. *ASTM International*. West Conshohocken, PA.

ASTM C 469. 2014. Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression. *ASTM International*. West Conshohocken, PA.

Bamforth, P.B., and W.F. Price. 1995. Concreting Deep Lifts and Large Volume Pours. Construction Industry Research and Information Association.

Bamforth, P.B. 2007. *Early-age thermal crack control in concrete*. Ciria.

Bazant, Z.P. and S. Baweja. 2000. Creep and Shrinkage Prediction Model for Analysis and Design of Concrete Structures: Model B3. *Adam Neville Symposium: Creep and Shrinkage – Structural Design Effects*. American Concrete Institute. Farmington Hills, MI. 1-83.

Browne, R.D. 1972. Thermal movement in concrete. *Concrete* 6, no. 11. 51-53.

Byard, B.E. and A.K. Schindler. 2015. Modeling early-age stress development of restrained concrete. *Materials and Structures* 48, no. 1-2: 435-450.

California DOT. 2010. Standard Specifications. California Department of Transportation. Sacramento, CA.

Concrete Durability Center. 2005. Concrete Works Version 2.0 User's Manual. Sponsored by Texas Department of Transportation: Project 0-4563.

Emmons, P.H. 1993. *Concrete Repair and Maintenance Illustrated*. RSMeans Inc.

FHWA. 2014. Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects. U.S. DOT Federal Highway Administration. Washington D.C.

Folliard, K.J., R. Barborak, T. Drimalas, L. Du, S. Garber, J. Ideker, T. Ley, S. Williams, M. Juenger, B. Fournier, M.D.A. Thomas. 2006. Preventing ASR/DEF in New Concrete: Final Report. Sponsored by Texas DOT: Project 0-4085.

Folliard, K.J., M. Juenger, A.K. Schindler, K. Riding, J. Poole, L.F. Kallivokas, S. Slatnick, J. Whigham, and J.L. Meadows. 2008. Prediction Model for Concrete Behavior – Final Report. Sponsored by Texas DOT: Project 0-4563.

Florida DOT. 2013. Standard Specifications for Road and Bridge Construction. Florida Department of Transportation. Tallahassee, FL

Gajda, J., and E. Alsamsam. 2006. Engineering Mass Concrete Structures. *Structural Engineer, November*.

Gajda, J., A. Kaufman, and F. Sumodjo. 2005. Precooling Mass Concrete. *Concrete Construction*: 36-40.

Gajda, J. and M. Vangeem. 2002. Controlling Temperatures in Mass Concrete. Concrete International: 59-62.

Georgia DOT. 2013. Mass Concrete. Special Provision to Standard Specifications Construction of Transportation Systems. Georgia Department of Transportation. Atlanta, GA.

Idaho DOT. 2012. Standard Specifications for Highway Construction. Idaho Department of Transportation. Boise, ID.

Iowa DOT. 2012. Developmental Specifications for Mass Concrete – Control of Heat of Hydration. Iowa Department of Transportation. Ames, IA.

Kentucky DOT. 2008. Standard Specifications for Road and Bridge Construction. Kentucky Transportation Cabinet. Frankfort, KY.

Kim, J.K., K.H. Kim, and J.K. Yang. 2001. Thermal analysis of hydration heat in concrete structures with pipe-cooling system. *Computers and Structures* 79, no.2: 163-171.

Kurtis, K.E., L. Kahn, and A. Jayapalan, 2012. Cracking in Massive Concrete Seals. Sponsored by Georgia DOT: Project TO 02-115, RP 12-11.

Louisiana DOT. 2016. Standard Specifications for Roads and Bridges. Louisiana Department of Transportation and Development Baton Rouge, LA.

Massachusetts DOT. 1995. Standard Specifications for Highways and Bridges. Massachusetts Highway Department. Boston, MA.

McCall, W.C. 2013. DEF: Field Case Study of Deep Water Foundations at US Route 378 Bridge over Savannah River (Presentation).

Mehta, P.K. and P.J.M. Monteiro. 2006. *Concrete: Microstructure, Properties, and Materials, 3rd Edition*. McGraw Hill.

Mississippi DOT. 2004. Standard Specifications for Road and Bridge Construction. Mississippi Department of Transportation. Jackson, MS.

Myuran, K., N. S. A. Wanigaratne, and M. T. R. Jayasinghe. 2015. Strategies for Prevention of Delayed Ettringite Formation in large Concrete Sections. *Engineer: Journal of the Institution of Engineers, Sri Lanka 48*, no. 2. 1-13.

New Hampshire DOT. 2016. Standard Specifications for Road and Bridge Construction. New Hampshire Department of Transportation. Concord, NH.

New Jersey DOT. 2007. Standard Specifications for Road and Bridge Construction. New Jersey Department of Transportation. Trenton, NJ.

New York State DOT. 2014. Standard Specifications. State of New York Department of Transportation. Albany, NY.

Ohio DOT. 2013. Construction and Material Specifications. Ohio Department of Transportation. Columbus, Ohio.

Pavoine, A., L. Divet, and S. Fenouillet. 2006. A Concrete Performance Test for Delayed Ettringite Formation: Part I Optimisation. *Cement and Concrete Research* 36, no. 12: 2138-2143.

Raphael, J.M. 1984. Tensile strength of concrete. *Journal Proceedings*. Vol. 81, no. 2: 158-165.

Rhode Island DOT. 2010. Standard Specifications for Road and Bridge Construction. Rhode Island Department of Transportation. Providence, RI.

Rostasy, F.S., T. Tanabe, and M. Laube. 1998. Assessment of External Restraint. *Prevention of thermal cracking in concrete at early ages,* edited by R. Springenschmid. Vol. 15. CRC Press. 149-151.

Scherer, G.W. 1999. Crystallization in Pores. *Cement and Concrete Research* 29, no. 8: 1347-1358.

Schindler, A.K., and B.F. McCullough. 2002. Importance of Concrete Temperature Control during Concrete Pavement Construction in Hot Weather Conditions. *Transportation Research Record: Journal of the Transportation Research Board*: 3-10

Schindler, A.K. and K.J. Folliard. 2005. Heat of Hydration Models for Cementitious Materials. *ACI Materials Journal* 102, no.1: 24-33.

Schindler, A.K., M.L. Hughes, R.W. Barnes, and B.E. Byard. 2010. Evaluation of cracking of the US 331 bridge deck. Sponsored by Alabama DOT: Project 930-645.

South Carolina DOT. 2007. Standard Specifications for Highway Construction. South Carolina Department of Transportation. Columbia, SC.

Tankasala, A., A.K. Schindler, and K. Riding. 2017. Thermal Cracking Risk Using Lightweight Aggregate in Mass Concrete. Manuscript submitted for publication to the Journal of the Transportation Research Board, Washington, D.C.

Taylor, H.F.W., C. Famy, and K.L. Scrivener. Delayed Ettringite Formation. Cement and Concrete Research 31, no. 5: 683-693

Texas DOT. 2014. Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges. Texas Department of Transportation. Austin, TX.

Thomas, M.D.A., K. Folliard, T. Drimalas, and T. Ramlochan. 2008. Diagnosing Delayed Ettringite Formation in Concrete Structures. Cement and Concrete Research 38, no. 6: 841-847.

Virginia DOT. 2007. Road and Bridge Specifications. Virginia Department of Transportation. Richmond, VA.

West Virginia DOT. 2010. Standards Specifications for Roads and Bridges. West Virginia Department of Transportation Division of Highways. Charleston, WV.

APPENDIX A: ALBERTVILLE BENT CAP

Table A-1: Concrete Mixture Proportions – Albertville Bent Cap

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

Table A-2: Cement Composition – Albertville Bent Cap

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

Table A-3: Fly Ash Composition – Albertville Bent Cap

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

Table A-4: 7-Day Weather Data – Albertville Bent Cap

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

Table A-5: ALDOT Test Results of Project Concrete Specimens – Albertville Bent Cap

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

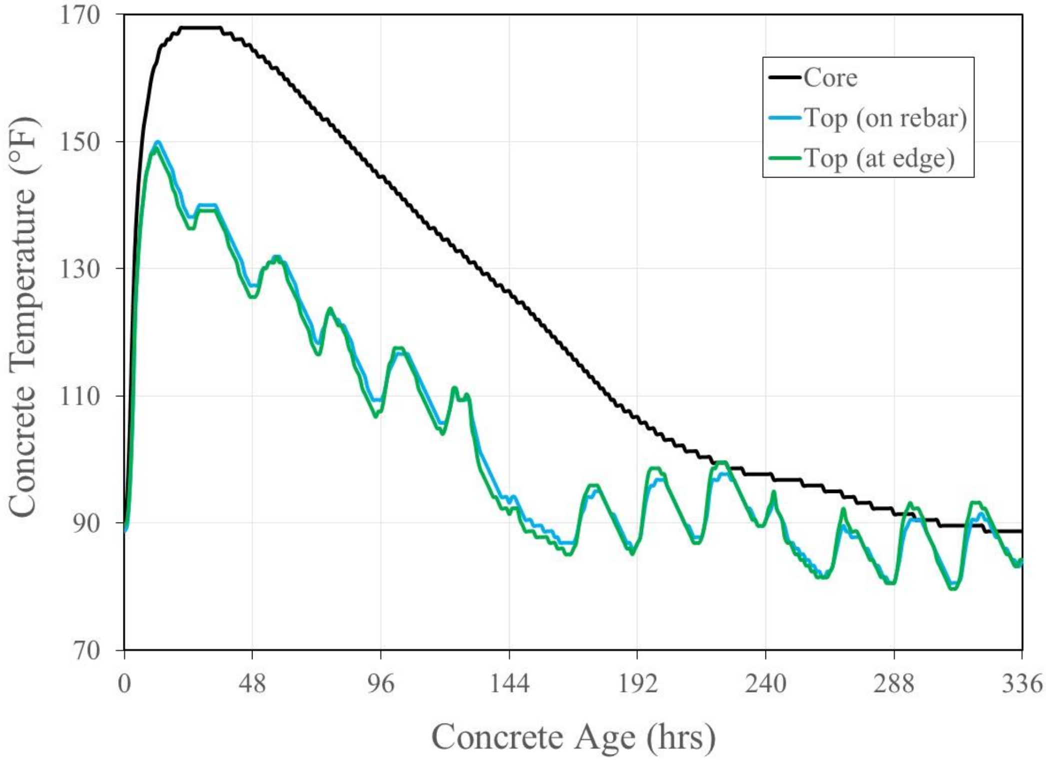

Figure A-1: Cross-Section #1 Temperature Data – Albertville Bent Cap

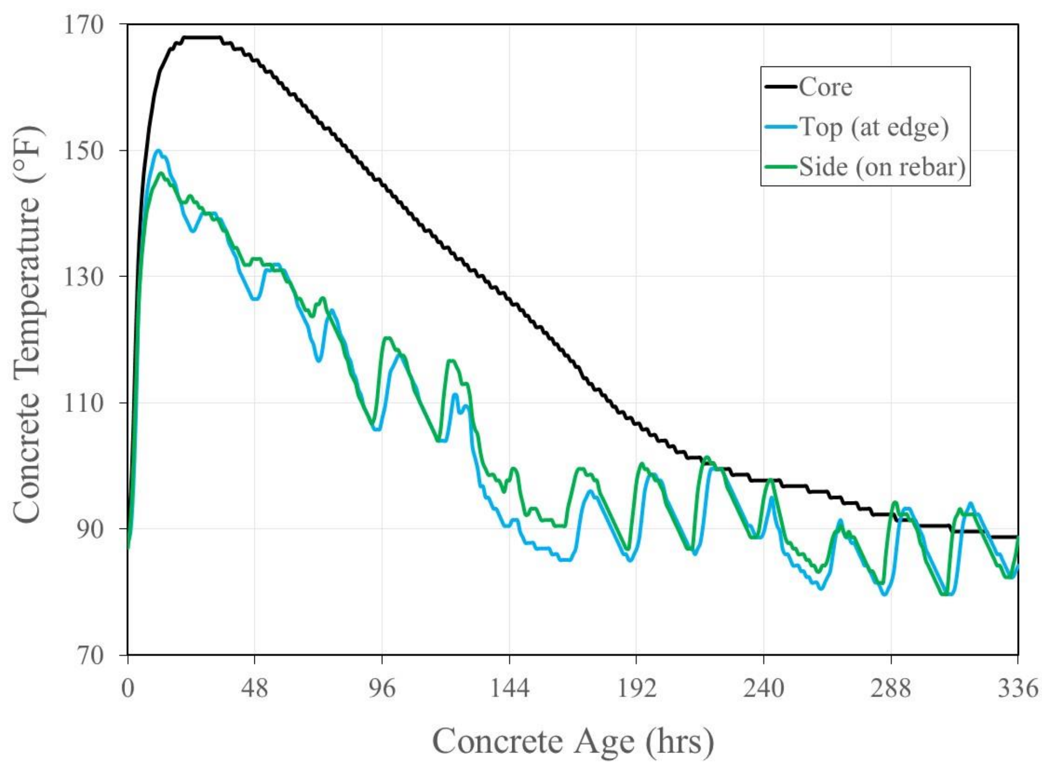

Figure A-2: Cross-Section #2 Temperature Data – Albertville Bent Cap

APPENDIX B: HARPERSVILLE CRASHWALL

Table B-1: Concrete Mixture Proportions – Harpersville Crashwall

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

Table B-2: Cement Composition – Harpersville Crashwall

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

Table B-3: Fly Ash Composition – Harpersville Crashwall

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

Table B-4: 7-Day Weather Data – Harpersville Crashwall

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

Table B-5: ALDOT Test Results of Project Concrete Specimens – Harpersville Crashwall

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

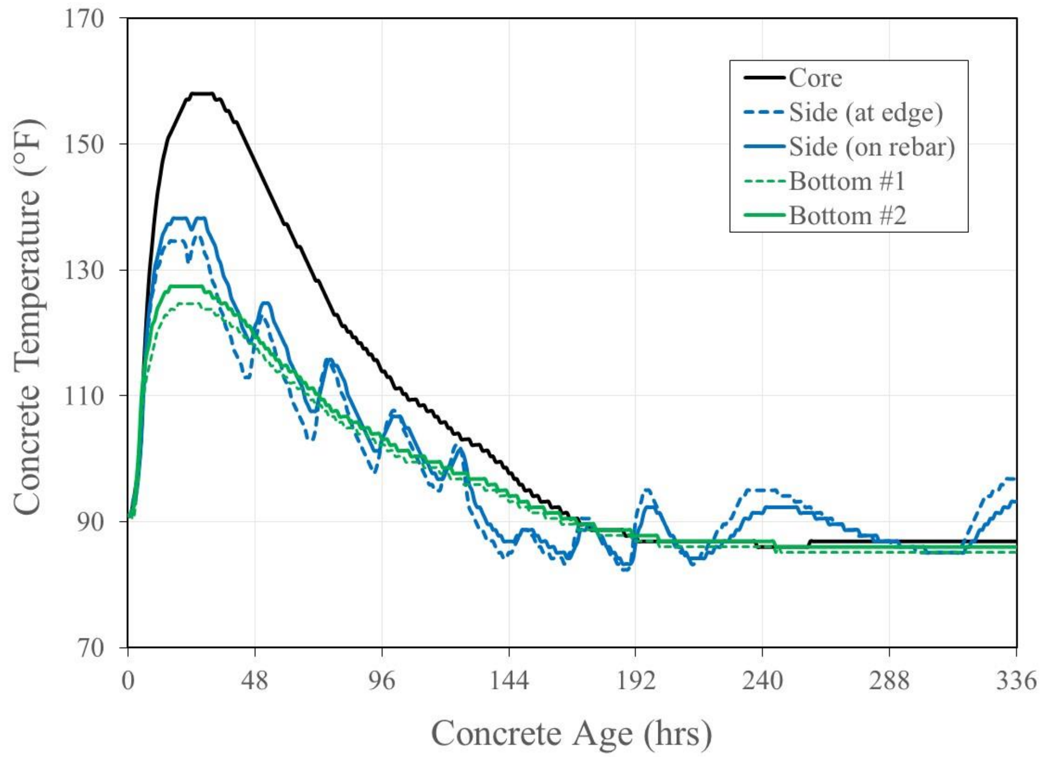

Figure B-1: Cross-Section #1 Temperature Data – Harpersville Crashwall

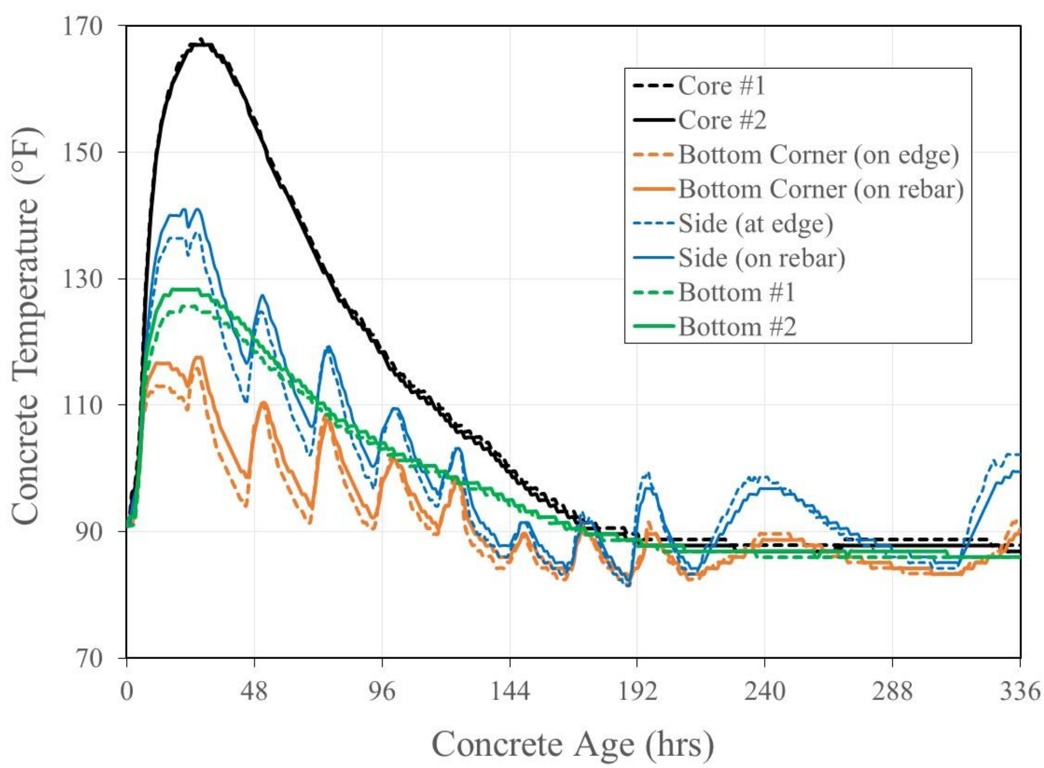

Figure B-2: Cross-Section #2 Temperature Data – Harpersville Crashwall

APPENDIX C: SCOTTSBORO PEDESTAL

Table C-1: Concrete Mixture Proportions – Scottsboro Pedestal

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

Table C-2: Cement Composition – Scottsboro Pedestal

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

Table C-3: Fly Ash Composition – Scottsboro Pedestal

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

Table C-4: 7-Day Weather Data – Scottsboro Pedestal

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

Table C-5: ALDOT Test Results of Project Concrete Specimens – Scottsboro Pedestal

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

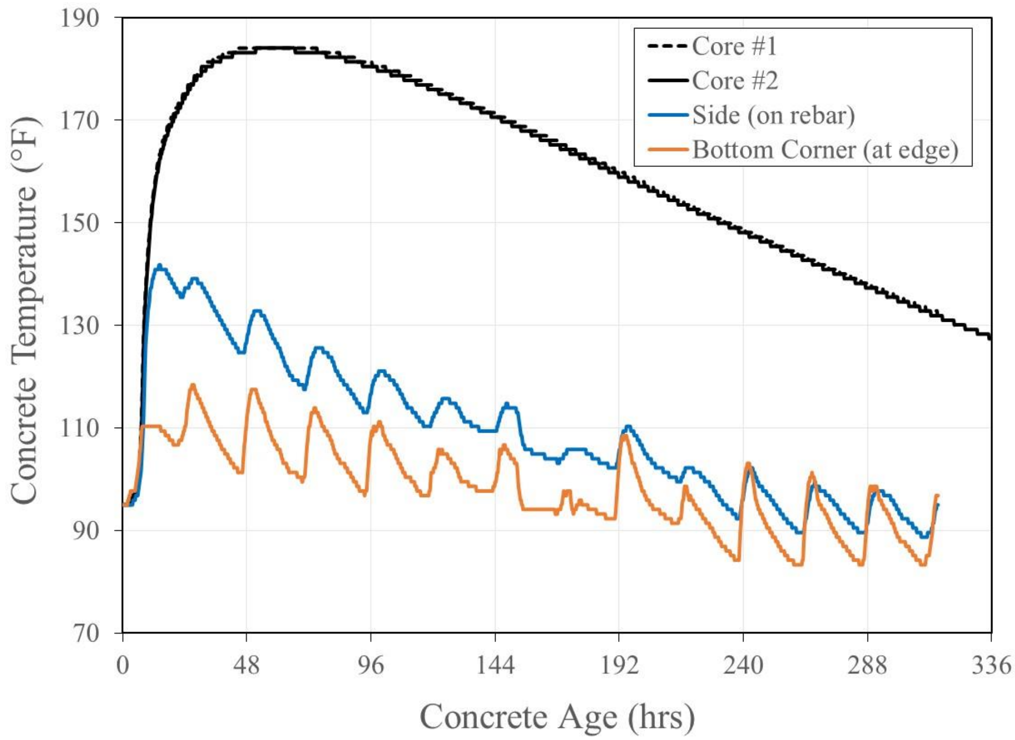

Figure C-1: Cross-Section #1 Temperature Data – Scottsboro Pedestal

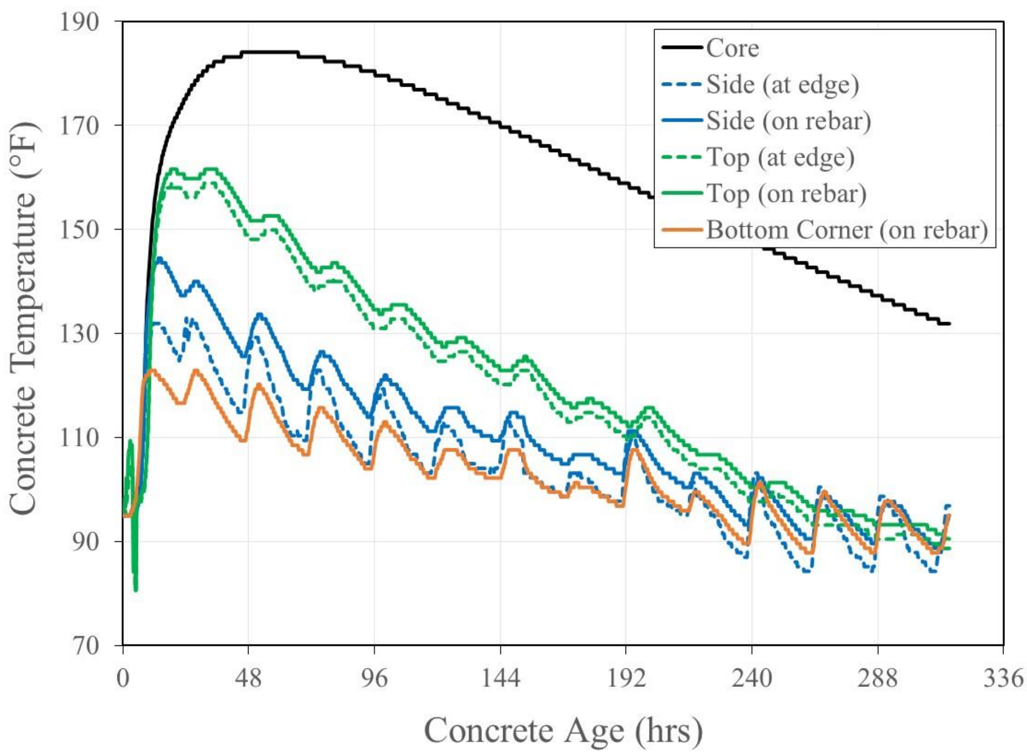

Figure C-2: Cross-Section #2 Temperature Data – Scottsboro Pedestal

APPENDIX D: SCOTTSBORO BENT CAP

Table D-1: Concrete Mixture Proportions – Scottsboro Bent Cap

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

Table D-2: Cement Composition – Scottsboro Bent Cap

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

Table D-3: Fly Ash Composition – Scottsboro Bent Cap

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

Table D-4: 7-Day Weather Data – Scottsboro Bent Cap

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

Table D-5: ALDOT Test Results of Project Concrete Specimens – Scottsboro Bent Cap

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

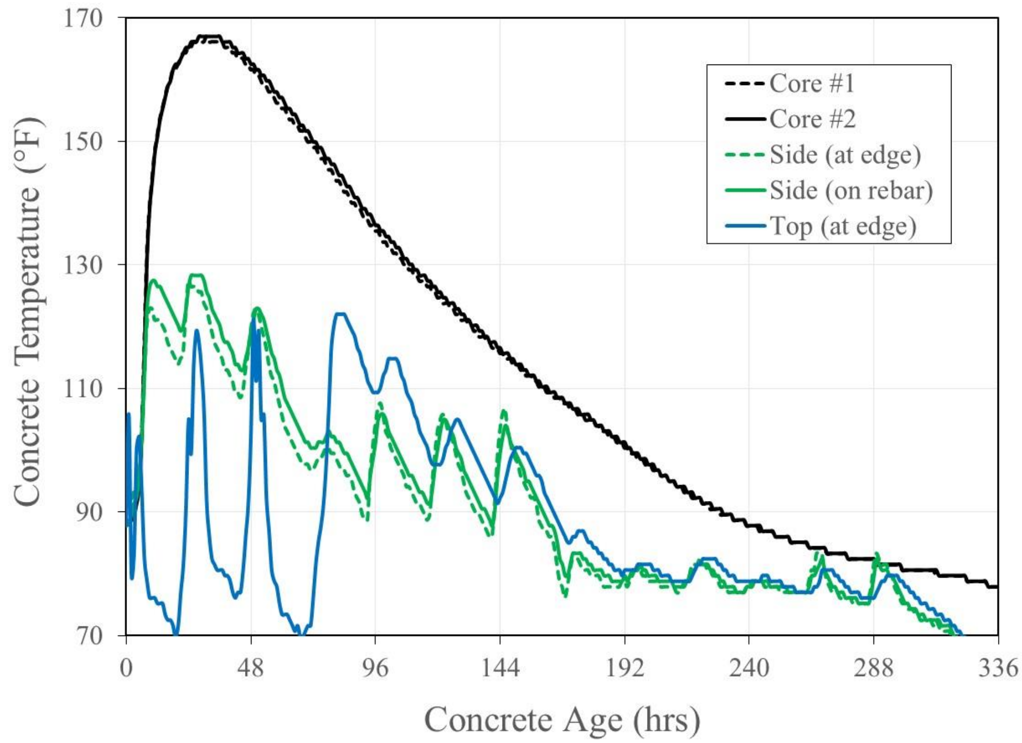

Figure D-1: Cross-Section #1 Temperature Data – Scotttsboro Bent Cap

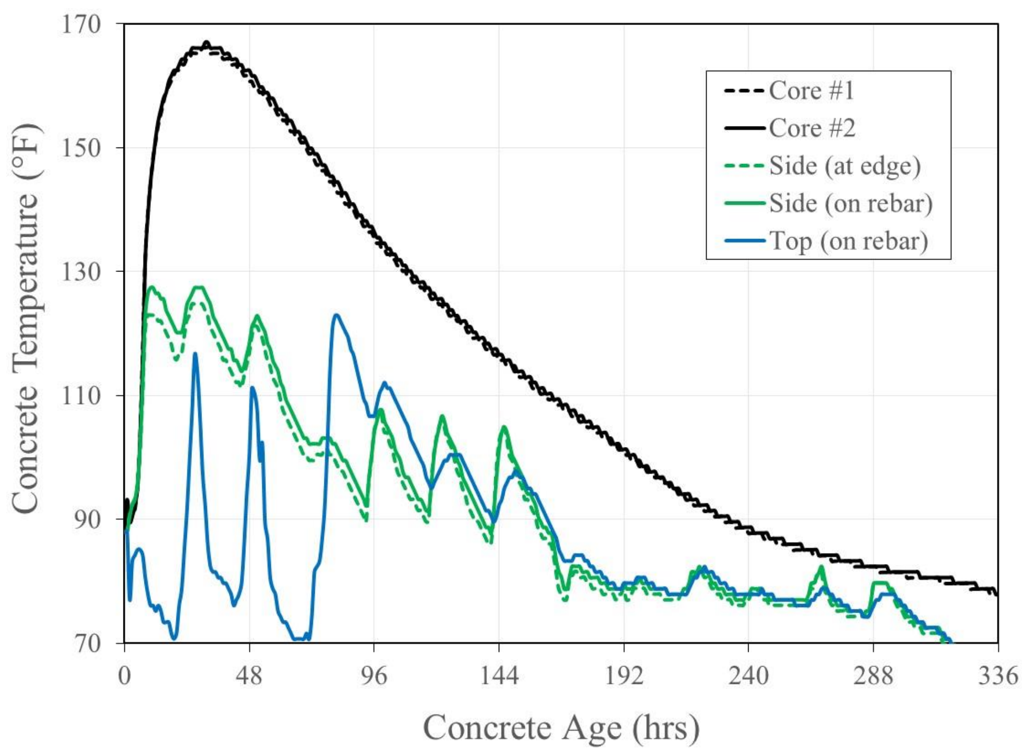

Figure D-2: Cross-Section #2 Temperature Data – Scottsboro Bent Cap

APPENDIX E: ELBA BENT CAP

Table E-1: Concrete Mixture Proportions – Elba Bent Cap

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

Table E-2: Cement Composition – Elba Bent Cap

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

Table E-3: Fly Ash Composition – Elba Bent Cap

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

Table E-4: 7-Day Weather Data – Elba Bent Cap

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

Table E-5: ALDOT Test Results of Project Concrete Specimens – Elba Bent Cap

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

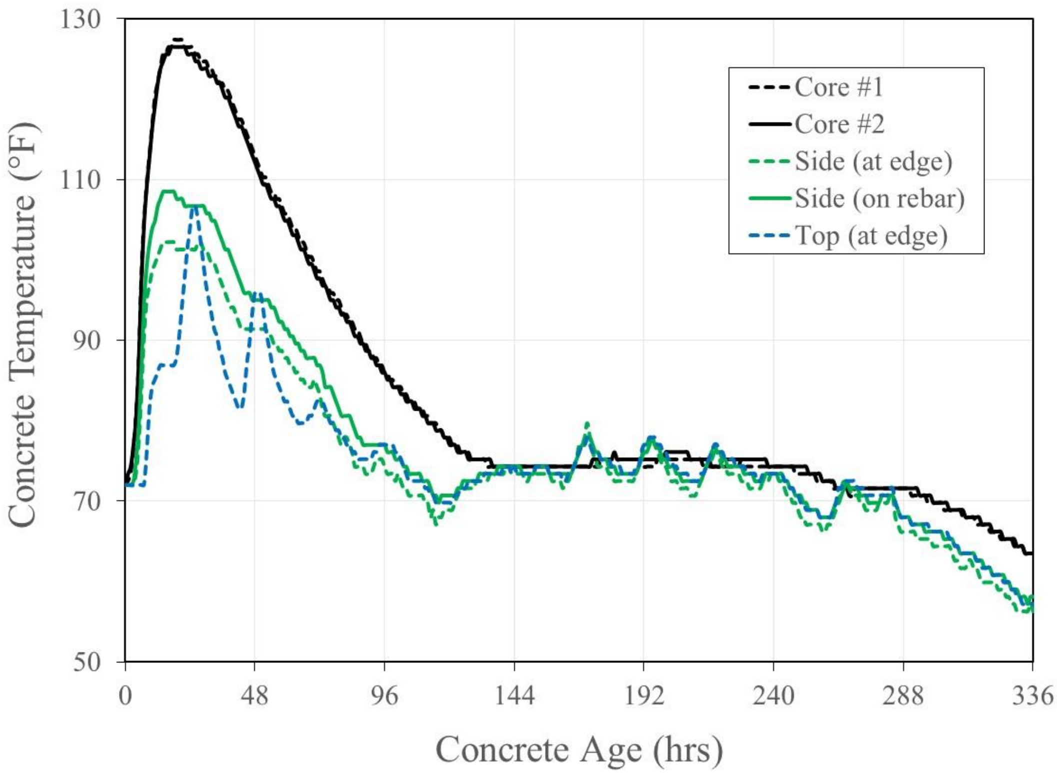

Figure E-1: Cross-Section #1 Temperature Data – Elba Bent Cap

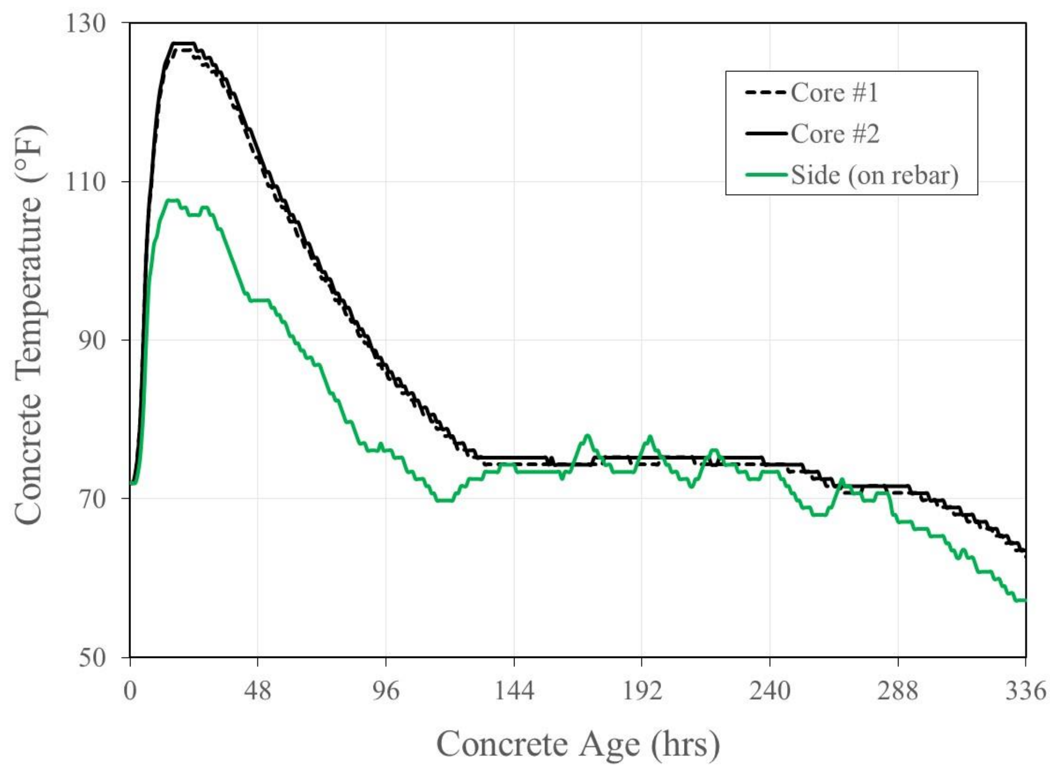

Figure E-2: Cross-Section #2 Temperature Data – Elba Bent Cap

APPENDIX F: BIRMINGHAM COLUMN

Table F-1: Concrete Mixture Proportions – Birmingham Column

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

Table F-2: Cement Composition – Birmingham Column

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

Table F-3: Fly Ash Composition – Birmingham Column

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

Table F-4: 7-Day Weather Data – Birmingham Column

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

Table F-5: ALDOT Test Results of Project Concrete Specimens – Birmingham Column

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

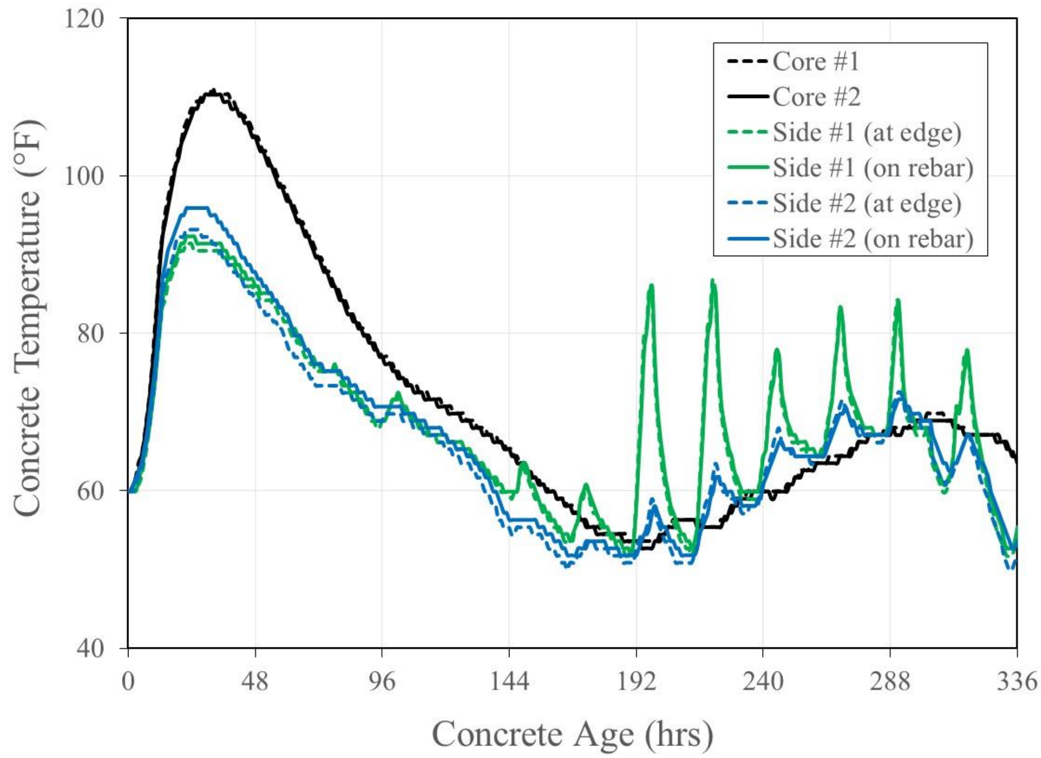

Figure F-1: Cross-Section #1 Temperature Data – Birmingham Column

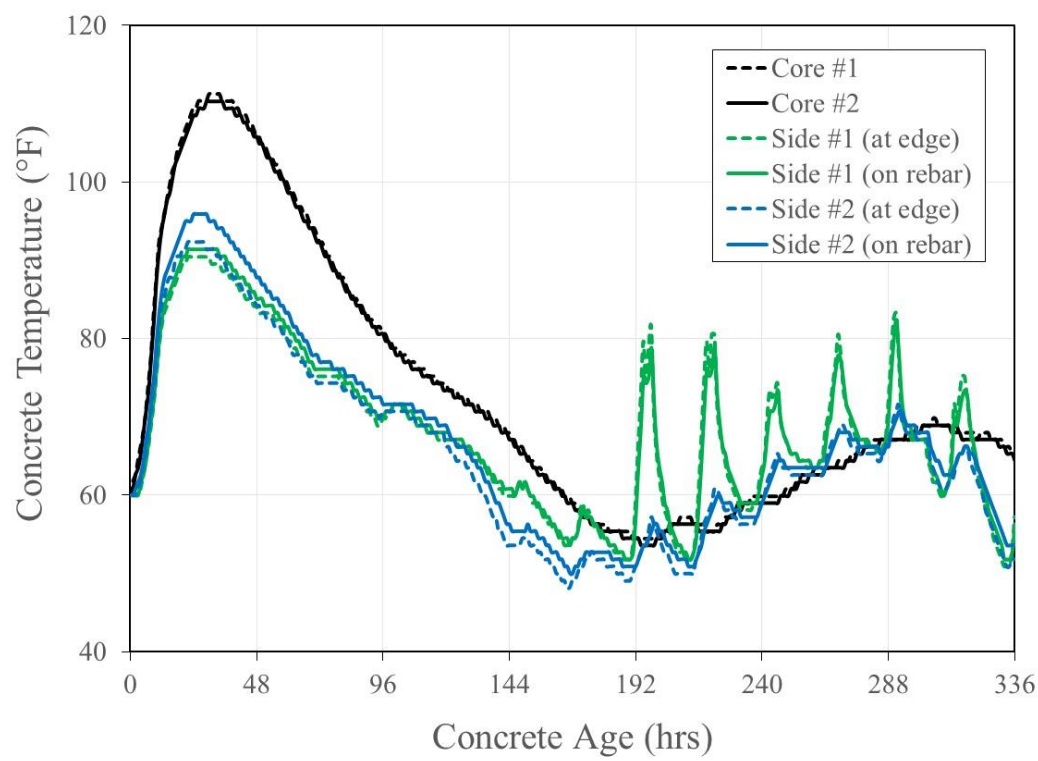

Figure F-2: Cross-Section #2 Temperature Data – Birmingham Column

APPENDIX G: CONCRETEWORKS ANALYSIS

Table G-1: ConcreteWorks Analysis Output Data

Run	Cementitious Materials	Coarse Aggregate Type	City	Placement Date/Time	Placement Temp (°F)	Minimum Dimension (ft)	Maximum Concrete Temperature Differential (°F)	Maximum Concrete Temperature (°F)	Cracking Risk
1	100% PC	Limestone	Mobile	Jan 15 - 12 pm	54	4	35	117	Low
2	100% PC	Limestone	Mobile	Jan 15 - 12 pm	54	5	42	123	Low
3	100% PC	Limestone	Mobile	Jan 15 - 12 pm	54	6	47	128	Low
4	100% PC	Limestone	Mobile	Jan 15 - 12 pm	54	7	50	132	Low
5	100% PC	Limestone	Mobile	Jan 15 - 12 pm	54	8	54	134	Medium
6	100% PC	Limestone	Mobile	March 15 - 10 am	64.4	4	37	134	Low
7	100% PC	Limestone	Mobile	March 15 - 10 am	64.4	5	44	140	Low
8	100% PC	Limestone	Mobile	March 15 - 10 am	64.4	6	50	143	Medium
9	100% PC	Limestone	Mobile	March 15 - 10 am	64.4	7	53	146	Medium
10	100% PC	Limestone	Mobile	March 15 - 10 am	64.4	8	57	148	High
11	100% PC	Limestone	Mobile	May 15 - 8 am	73.6	4	35	148	Low
12	100% PC	Limestone	Mobile	May 15 - 8 am	73.6	5	43	152	Low
13	100% PC	Limestone	Mobile	May 15 - 8 am	73.6	6	48	155	Low
14	100% PC	Limestone	Mobile	May 15 - 8 am	73.6	7	51	157	Medium
15	100% PC	Limestone	Mobile	May 15 - 8 am	73.6	8	55	159	High
16	100% PC	Limestone	Mobile	July 15 - 8 am	80.6	4	35	157	Low
17	100% PC	Limestone	Mobile	July 15 - 8 am	80.6	5	42	161	Low
18	100% PC	Limestone	Mobile	July 15 - 8 am	80.6	6	47	163	Low
19	100% PC	Limestone	Mobile	July 15 - 8 am	80.6	7	50	165	Low
20	100% PC	Limestone	Mobile	July 15 - 8 am	80.6	8	54	166	Medium
21	100% PC	Limestone	Mobile	Sept 15 - 10 am	81.3	4	37	157	Low
22	100% PC	Limestone	Mobile	Sept 15 - 10 am	81.3	5	44	161	Low

23	100% PC	Limestone	Mobile	Sept 15 - 10 am	81.3	6	50	164	Low
24	100% PC	Limestone	Mobile	Sept 15 - 10 am	81.3	7	53	165	Medium
25	100% PC	Limestone	Mobile	Sept 15 - 10 am	81.3	8	58	167	High
26	100% PC	Limestone	Mobile	Nov 15 - 12 pm	66.4	4	37	135	Low
27	100% PC	Limestone	Mobile	Nov 15 - 12 pm	66.4	5	45	141	Low
28	100% PC	Limestone	Mobile	Nov 15 - 12 pm	66.4	6	50	145	Low
29	100% PC	Limestone	Mobile	Nov 15 - 12 pm	66.4	7	54	147	Medium
30	100% PC	Limestone	Mobile	Nov 15 - 12 pm	66.4	8	59	150	High
31	100% PC	Limestone	Huntsville	Jan 15 - 12 pm	41.4	4	30	99	Low
32	100% PC	Limestone	Huntsville	Jan 15 - 12 pm	41.4	5	36	105	Low
33	100% PC	Limestone	Huntsville	Jan 15 - 12 pm	41.4	6	42	111	Low
34	100% PC	Limestone	Huntsville	Jan 15 - 12 pm	41.4	7	46	115	Low
35	100% PC	Limestone	Huntsville	Jan 15 - 12 pm	41.4	8	50	118	Medium
36	100% PC	Limestone	Huntsville	March 15 - 10 am	55.9	4	36	122	Low
37	100% PC	Limestone	Huntsville	March 15 - 10 am	55.9	5	43	128	Low
38	100% PC	Limestone	Huntsville	March 15 - 10 am	55.9	6	48	132	Medium
39	100% PC	Limestone	Huntsville	March 15 - 10 am	55.9	7	52	135	Medium
40	100% PC	Limestone	Huntsville	March 15 - 10 am	55.9	8	56	138	High
41	100% PC	Limestone	Huntsville	May 15 - 8 am	67.5	4	34	140	Low
42	100% PC	Limestone	Huntsville	May 15 - 8 am	67.5	5	42	145	Low
43	100% PC	Limestone	Huntsville	May 15 - 8 am	67.5	6	47	148	Low
44	100% PC	Limestone	Huntsville	May 15 - 8 am	67.5	7	50	150	Medium
45	100% PC	Limestone	Huntsville	May 15 - 8 am	67.5	8	54	152	High
46	100% PC	Limestone	Huntsville	July 15 - 8 am	78.6	4	34	155	Low
47	100% PC	Limestone	Huntsville	July 15 - 8 am	78.6	5	42	159	Low
48	100% PC	Limestone	Huntsville	July 15 - 8 am	78.6	6	47	161	Low
49	100% PC	Limestone	Huntsville	July 15 - 8 am	78.6	7	50	163	Low
50	100% PC	Limestone	Huntsville	July 15 - 8 am	78.6	8	54	164	Medium
51	100% PC	Limestone	Huntsville	Sept 15 - 10 am	75.9	4	36	150	Low
52	100% PC	Limestone	Huntsville	Sept 15 - 10 am	75.9	5	44	154	Low

53	100% PC	Limestone	Huntsville	Sept 15 - 10 am	75.9	6	49	157	Low
54	100% PC	Limestone	Huntsville	Sept 15 - 10 am	75.9	7	53	159	Medium
55	100% PC	Limestone	Huntsville	Sept 15 - 10 am	75.9	8	57	161	High
56	100% PC	Limestone	Huntsville	Nov 15 - 12 pm	60.1	4	36	126	Low
57	100% PC	Limestone	Huntsville	Nov 15 - 12 pm	60.1	5	44	132	Low
58	100% PC	Limestone	Huntsville	Nov 15 - 12 pm	60.1	6	49	136	Low
59	100% PC	Limestone	Huntsville	Nov 15 - 12 pm	60.1	7	55	139	Low
60	100% PC	Limestone	Huntsville	Nov 15 - 12 pm	60.1	8	59	142	Medium
61	100% PC	River Gravel	Mobile	Jan 15 - 12 pm	54	4	34	121	Low
62	100% PC	River Gravel	Mobile	Jan 15 - 12 pm	54	5	43	128	Low
63	100% PC	River Gravel	Mobile	Jan 15 - 12 pm	54	6	49	133	Medium
64	100% PC	River Gravel	Mobile	Jan 15 - 12 pm	54	7	52	137	High
65	100% PC	River Gravel	Mobile	Jan 15 - 12 pm	54	8	56	140	Very High
66	100% PC	River Gravel	Mobile	March 15 - 10 am	64.4	4	36	139	Low
67	100% PC	River Gravel	Mobile	March 15 - 10 am	64.4	5	45	145	Medium
68	100% PC	River Gravel	Mobile	March 15 - 10 am	64.4	6	51	149	High
69	100% PC	River Gravel	Mobile	March 15 - 10 am	64.4	7	55	152	Very High
70	100% PC	River Gravel	Mobile	March 15 - 10 am	64.4	8	59	154	Very High
71	100% PC	River Gravel	Mobile	May 15 - 8 am	73.6	4	37	153	Low
72	100% PC	River Gravel	Mobile	May 15 - 8 am	73.6	5	43	158	High
73	100% PC	River Gravel	Mobile	May 15 - 8 am	73.6	6	49	161	Very High
74	100% PC	River Gravel	Mobile	May 15 - 8 am	73.6	7	53	163	Very High
75	100% PC	River Gravel	Mobile	May 15 - 8 am	73.6	8	56	165	Very High
76	100% PC	River Gravel	Mobile	July 15 - 8 am	80.6	4	36	162	Low
77	100% PC	River Gravel	Mobile	July 15 - 8 am	80.6	5	41	166	Medium
78	100% PC	River Gravel	Mobile	July 15 - 8 am	80.6	6	48	169	High
79	100% PC	River Gravel	Mobile	July 15 - 8 am	80.6	7	52	171	High
80	100% PC	River Gravel	Mobile	July 15 - 8 am	80.6	8	55	173	Very High
81	100% PC	River Gravel	Mobile	Sept 15 - 10 am	81.3	4	38	162	Low
82	100% PC	River Gravel	Mobile	Sept 15 - 10 am	81.3	5	44	167	High

83	100% PC	River Gravel	Mobile	Sept 15 - 10 am	81.3	6	51	169	Very High
84	100% PC	River Gravel	Mobile	Sept 15 - 10 am	81.3	7	55	171	Very High
85	100% PC	River Gravel	Mobile	Sept 15 - 10 am	81.3	8	59	173	Very High
86	100% PC	River Gravel	Mobile	Nov 15 - 12 pm	66.4	4	36	140	Low
87	100% PC	River Gravel	Mobile	Nov 15 - 12 pm	66.4	5	45	146	Low
88	100% PC	River Gravel	Mobile	Nov 15 - 12 pm	66.4	6	51	150	High
89	100% PC	River Gravel	Mobile	Nov 15 - 12 pm	66.4	7	55	153	Very High
90	100% PC	River Gravel	Mobile	Nov 15 - 12 pm	66.4	8	60	156	Very High
91	100% PC	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	4	31	102	Low
92	100% PC	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	5	37	109	Low
93	100% PC	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	6	42	115	Medium
94	100% PC	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	7	48	120	High
95	100% PC	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	8	52	123	Very High
96	100% PC	River Gravel	Huntsville	March 15 - 10 am	55.9	4	35	127	Low
97	100% PC	River Gravel	Huntsville	March 15 - 10 am	55.9	5	44	133	Low
98	100% PC	River Gravel	Huntsville	March 15 - 10 am	55.9	6	50	138	Very High
99	100% PC	River Gravel	Huntsville	March 15 - 10 am	55.9	7	53	141	Very High
100	100% PC	River Gravel	Huntsville	March 15 - 10 am	55.9	8	58	144	Very High
101	100% PC	River Gravel	Huntsville	May 15 - 8 am	67.5	4	33	146	Low
102	100% PC	River Gravel	Huntsville	May 15 - 8 am	67.5	5	42	150	Medium
103	100% PC	River Gravel	Huntsville	May 15 - 8 am	67.5	6	48	154	High
104	100% PC	River Gravel	Huntsville	May 15 - 8 am	67.5	7	52	156	Very High
105	100% PC	River Gravel	Huntsville	May 15 - 8 am	67.5	8	55	158	Very High
106	100% PC	River Gravel	Huntsville	July 15 - 8 am	78.6	4	36	160	Low
107	100% PC	River Gravel	Huntsville	July 15 - 8 am	78.6	5	42	164	Medium
108	100% PC	River Gravel	Huntsville	July 15 - 8 am	78.6	6	48	167	High
109	100% PC	River Gravel	Huntsville	July 15 - 8 am	78.6	7	52	169	Very High
110	100% PC	River Gravel	Huntsville	July 15 - 8 am	78.6	8	56	171	Very High
111	100% PC	River Gravel	Huntsville	Sept 15 - 10 am	75.9	4	38	155	Low
112	100% PC	River Gravel	Huntsville	Sept 15 - 10 am	75.9	5	44	160	High

113	100% PC	River Gravel	Huntsville	Sept 15 - 10 am	75.9	6	51	163	Very High
114	100% PC	River Gravel	Huntsville	Sept 15 - 10 am	75.9	7	55	165	Very High
115	100% PC	River Gravel	Huntsville	Sept 15 - 10 am	75.9	8	59	167	Very High
116	100% PC	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	4	36	130	Low
117	100% PC	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	5	44	137	Low
118	100% PC	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	6	51	141	Medium
119	100% PC	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	7	55	145	High
120	100% PC	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	8	61	148	Very High
121	70% PC + 30% FFA	Limestone	Mobile	Jan 15 - 12 pm	54	4	27	102	Low
122	70% PC + 30% FFA	Limestone	Mobile	Jan 15 - 12 pm	54	5	32	108	Low
123	70% PC + 30% FFA	Limestone	Mobile	Jan 15 - 12 pm	54	6	36	112	Low
124	70% PC + 30% FFA	Limestone	Mobile	Jan 15 - 12 pm	54	7	40	116	Low
125	70% PC + 30% FFA	Limestone	Mobile	Jan 15 - 12 pm	54	8	43	119	Low
126	70% PC + 30% FFA	Limestone	Mobile	March 15 - 10 am	64.4	4	30	119	Low
127	70% PC + 30% FFA	Limestone	Mobile	March 15 - 10 am	64.4	5	36	124	Low
128	70% PC + 30% FFA	Limestone	Mobile	March 15 - 10 am	64.4	6	40	128	Low
129	70% PC + 30% FFA	Limestone	Mobile	March 15 - 10 am	64.4	7	44	131	Low
130	70% PC + 30% FFA	Limestone	Mobile	March 15 - 10 am	64.4	8	48	134	Low
131	70% PC + 30% FFA	Limestone	Mobile	May 15 - 8 am	73.6	4	29	133	Low
132	70% PC + 30% FFA	Limestone	Mobile	May 15 - 8 am	73.6	5	35	137	Low
133	70% PC + 30% FFA	Limestone	Mobile	May 15 - 8 am	73.6	6	39	141	Low
134	70% PC + 30% FFA	Limestone	Mobile	May 15 - 8 am	73.6	7	42	143	Low
135	70% PC + 30% FFA	Limestone	Mobile	May 15 - 8 am	73.6	8	46	145	Low
136	70% PC + 30% FFA	Limestone	Mobile	July 15 - 8 am	80.6	4	28	142	Low
137	70% PC + 30% FFA	Limestone	Mobile	July 15 - 8 am	80.6	5	35	146	Low
138	70% PC + 30% FFA	Limestone	Mobile	July 15 - 8 am	80.6	6	39	150	Low
139	70% PC + 30% FFA	Limestone	Mobile	July 15 - 8 am	80.6	7	42	152	Low
140	70% PC + 30% FFA	Limestone	Mobile	July 15 - 8 am	80.6	8	46	154	Low
141	70% PC + 30% FFA	Limestone	Mobile	Sept 15 - 10 am	81.3	4	30	141	Low
142	70% PC + 30% FFA	Limestone	Mobile	Sept 15 - 10 am	81.3	5	37	146	Low

143	70% PC + 30% FFA	Limestone	Mobile	Sept 15 - 10 am	81.3	6	41	149	Low
144	70% PC + 30% FFA	Limestone	Mobile	Sept 15 - 10 am	81.3	7	45	151	Low
145	70% PC + 30% FFA	Limestone	Mobile	Sept 15 - 10 am	81.3	8	49	153	Low
146	70% PC + 30% FFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	4	30	120	Low
147	70% PC + 30% FFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	5	36	126	Low
148	70% PC + 30% FFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	6	41	130	Low
149	70% PC + 30% FFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	7	45	133	Low
150	70% PC + 30% FFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	8	49	135	Low
151	70% PC + 30% FFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	4	22	84	Low
152	70% PC + 30% FFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	5	27	90	Low
153	70% PC + 30% FFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	6	32	95	Low
154	70% PC + 30% FFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	7	36	99	Low
155	70% PC + 30% FFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	8	40	103	Low
156	70% PC + 30% FFA	Limestone	Huntsville	March 15 - 10 am	55.9	4	29	108	Low
157	70% PC + 30% FFA	Limestone	Huntsville	March 15 - 10 am	55.9	5	34	113	Low
158	70% PC + 30% FFA	Limestone	Huntsville	March 15 - 10 am	55.9	6	38	117	Low
159	70% PC + 30% FFA	Limestone	Huntsville	March 15 - 10 am	55.9	7	42	120	Low
160	70% PC + 30% FFA	Limestone	Huntsville	March 15 - 10 am	55.9	8	45	123	Low
161	70% PC + 30% FFA	Limestone	Huntsville	May 15 - 8 am	67.5	4	28	126	Low
162	70% PC + 30% FFA	Limestone	Huntsville	May 15 - 8 am	67.5	5	33	130	Low
163	70% PC + 30% FFA	Limestone	Huntsville	May 15 - 8 am	67.5	6	37	134	Low
164	70% PC + 30% FFA	Limestone	Huntsville	May 15 - 8 am	67.5	7	41	136	Low
165	70% PC + 30% FFA	Limestone	Huntsville	May 15 - 8 am	67.5	8	44	138	Low
166	70% PC + 30% FFA	Limestone	Huntsville	July 15 - 8 am	78.6	4	28	140	Low
167	70% PC + 30% FFA	Limestone	Huntsville	July 15 - 8 am	78.6	5	34	144	Low
168	70% PC + 30% FFA	Limestone	Huntsville	July 15 - 8 am	78.6	6	38	147	Low
169	70% PC + 30% FFA	Limestone	Huntsville	July 15 - 8 am	78.6	7	42	150	Low
170	70% PC + 30% FFA	Limestone	Huntsville	July 15 - 8 am	78.6	8	46	151	Low
171	70% PC + 30% FFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	4	30	135	Low
172	70% PC + 30% FFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	5	36	140	Low

173	70% PC + 30% FFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	6	40	143	Low
174	70% PC + 30% FFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	7	45	146	Low
175	70% PC + 30% FFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	8	49	148	Low
176	70% PC + 30% FFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	4	29	111	Low
177	70% PC + 30% FFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	5	35	116	Low
178	70% PC + 30% FFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	6	41	121	Low
179	70% PC + 30% FFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	7	45	124	Low
180	70% PC + 30% FFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	8	49	127	Low
181	70% PC + 30% FFA	River Gravel	Mobile	Jan 15 - 12 pm	54	4	27	104	Low
182	70% PC + 30% FFA	River Gravel	Mobile	Jan 15 - 12 pm	54	5	33	111	Low
183	70% PC + 30% FFA	River Gravel	Mobile	Jan 15 - 12 pm	54	6	37	116	Low
184	70% PC + 30% FFA	River Gravel	Mobile	Jan 15 - 12 pm	54	7	41	120	Low
185	70% PC + 30% FFA	River Gravel	Mobile	Jan 15 - 12 pm	54	8	45	123	Medium
186	70% PC + 30% FFA	River Gravel	Mobile	March 15 - 10 am	64.4	4	30	122	Low
187	70% PC + 30% FFA	River Gravel	Mobile	March 15 - 10 am	64.4	5	36	128	Low
188	70% PC + 30% FFA	River Gravel	Mobile	March 15 - 10 am	64.4	6	41	132	Low
189	70% PC + 30% FFA	River Gravel	Mobile	March 15 - 10 am	64.4	7	45	136	Medium
190	70% PC + 30% FFA	River Gravel	Mobile	March 15 - 10 am	64.4	8	49	138	High
191	70% PC + 30% FFA	River Gravel	Mobile	May 15 - 8 am	73.6	4	28	136	Low
192	70% PC + 30% FFA	River Gravel	Mobile	May 15 - 8 am	73.6	5	35	141	Low
193	70% PC + 30% FFA	River Gravel	Mobile	May 15 - 8 am	73.6	6	40	145	Low
194	70% PC + 30% FFA	River Gravel	Mobile	May 15 - 8 am	73.6	7	43	148	Medium
195	70% PC + 30% FFA	River Gravel	Mobile	May 15 - 8 am	73.6	8	48	150	High
196	70% PC + 30% FFA	River Gravel	Mobile	July 15 - 8 am	80.6	4	28	146	Low
197	70% PC + 30% FFA	River Gravel	Mobile	July 15 - 8 am	80.6	5	35	151	Low
198	70% PC + 30% FFA	River Gravel	Mobile	July 15 - 8 am	80.6	6	40	154	Low
199	70% PC + 30% FFA	River Gravel	Mobile	July 15 - 8 am	80.6	7	43	157	Medium
200	70% PC + 30% FFA	River Gravel	Mobile	July 15 - 8 am	80.6	8	47	159	High
201	70% PC + 30% FFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	4	30	146	Low
202	70% PC + 30% FFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	5	37	150	Low

203	70% PC + 30% FFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	6	43	154	Low
204	70% PC + 30% FFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	7	46	157	Medium
205	70% PC + 30% FFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	8	51	159	High
206	70% PC + 30% FFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	4	30	123	Low
207	70% PC + 30% FFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	5	36	129	Low
208	70% PC + 30% FFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	6	41	134	Low
209	70% PC + 30% FFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	7	46	137	Medium
210	70% PC + 30% FFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	8	51	140	High
211	70% PC + 30% FFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	4	22	86	Low
212	70% PC + 30% FFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	5	27	92	Low
213	70% PC + 30% FFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	6	33	98	Low
214	70% PC + 30% FFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	7	37	103	Low
215	70% PC + 30% FFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	8	41	106	Low
216	70% PC + 30% FFA	River Gravel	Huntsville	March 15 - 10 am	55.9	4	28	110	Low
217	70% PC + 30% FFA	River Gravel	Huntsville	March 15 - 10 am	55.9	5	34	116	Low
218	70% PC + 30% FFA	River Gravel	Huntsville	March 15 - 10 am	55.9	6	39	121	Medium
219	70% PC + 30% FFA	River Gravel	Huntsville	March 15 - 10 am	55.9	7	43	124	Medium
220	70% PC + 30% FFA	River Gravel	Huntsville	March 15 - 10 am	55.9	8	47	127	High
221	70% PC + 30% FFA	River Gravel	Huntsville	May 15 - 8 am	67.5	4	28	129	Low
222	70% PC + 30% FFA	River Gravel	Huntsville	May 15 - 8 am	67.5	5	34	134	Low
223	70% PC + 30% FFA	River Gravel	Huntsville	May 15 - 8 am	67.5	6	39	138	Low
224	70% PC + 30% FFA	River Gravel	Huntsville	May 15 - 8 am	67.5	7	42	141	Medium
225	70% PC + 30% FFA	River Gravel	Huntsville	May 15 - 8 am	67.5	8	46	143	High
226	70% PC + 30% FFA	River Gravel	Huntsville	July 15 - 8 am	78.6	4	28	144	Low
227	70% PC + 30% FFA	River Gravel	Huntsville	July 15 - 8 am	78.6	5	35	148	Low
228	70% PC + 30% FFA	River Gravel	Huntsville	July 15 - 8 am	78.6	6	40	152	Low
229	70% PC + 30% FFA	River Gravel	Huntsville	July 15 - 8 am	78.6	7	43	154	Medium
230	70% PC + 30% FFA	River Gravel	Huntsville	July 15 - 8 am	78.6	8	47	156	High
231	70% PC + 30% FFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	4	29	138	Low
232	70% PC + 30% FFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	5	36	143	Low

233	70% PC + 30% FFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	6	42	147	Low
234	70% PC + 30% FFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	7	45	150	Medium
235	70% PC + 30% FFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	8	50	153	High
236	70% PC + 30% FFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	4	29	113	Low
237	70% PC + 30% FFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	5	35	119	Low
238	70% PC + 30% FFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	6	40	124	Low
239	70% PC + 30% FFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	7	46	128	Low
240	70% PC + 30% FFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	8	50	132	High
241	50% PC + 50% Slag	Limestone	Mobile	Jan 15 - 12 pm	54	4	25	99	Low
242	50% PC + 50% Slag	Limestone	Mobile	Jan 15 - 12 pm	54	5	30	106	Low
243	50% PC + 50% Slag	Limestone	Mobile	Jan 15 - 12 pm	54	6	36	112	Low
244	50% PC + 50% Slag	Limestone	Mobile	Jan 15 - 12 pm	54	7	41	117	Low
245	50% PC + 50% Slag	Limestone	Mobile	Jan 15 - 12 pm	54	8	46	122	Low
246	50% PC + 50% Slag	Limestone	Mobile	March 15 - 10 am	64.4	4	30	119	Low
247	50% PC + 50% Slag	Limestone	Mobile	March 15 - 10 am	64.4	5	36	125	Low
248	50% PC + 50% Slag	Limestone	Mobile	March 15 - 10 am	64.4	6	42	131	Low
249	50% PC + 50% Slag	Limestone	Mobile	March 15 - 10 am	64.4	7	47	135	Low
250	50% PC + 50% Slag	Limestone	Mobile	March 15 - 10 am	64.4	8	52	139	Medium
251	50% PC + 50% Slag	Limestone	Mobile	May 15 - 8 am	73.6	4	30	134	Low
252	50% PC + 50% Slag	Limestone	Mobile	May 15 - 8 am	73.6	5	37	141	Low
253	50% PC + 50% Slag	Limestone	Mobile	May 15 - 8 am	73.6	6	42	146	Low
254	50% PC + 50% Slag	Limestone	Mobile	May 15 - 8 am	73.6	7	47	150	Low
255	50% PC + 50% Slag	Limestone	Mobile	May 15 - 8 am	73.6	8	52	153	Medium
256	50% PC + 50% Slag	Limestone	Mobile	July 15 - 8 am	80.6	4	30	145	Low
257	50% PC + 50% Slag	Limestone	Mobile	July 15 - 8 am	80.6	5	38	151	Low
258	50% PC + 50% Slag	Limestone	Mobile	July 15 - 8 am	80.6	6	43	156	Low
259	50% PC + 50% Slag	Limestone	Mobile	July 15 - 8 am	80.6	7	48	160	Low
260	50% PC + 50% Slag	Limestone	Mobile	July 15 - 8 am	80.6	8	52	163	Medium
261	50% PC + 50% Slag	Limestone	Mobile	Sept 15 - 10 am	81.3	4	32	144	Low
262	50% PC + 50% Slag	Limestone	Mobile	Sept 15 - 10 am	81.3	5	40	151	Low

263	50% PC + 50% Slag	Limestone	Mobile	Sept 15 - 10 am	81.3	6	46	156	Low
264	50% PC + 50% Slag	Limestone	Mobile	Sept 15 - 10 am	81.3	7	51	160	Low
265	50% PC + 50% Slag	Limestone	Mobile	Sept 15 - 10 am	81.3	8	56	163	Medium
266	50% PC + 50% Slag	Limestone	Mobile	Nov 15 - 12 pm	66.4	4	30	120	Low
267	50% PC + 50% Slag	Limestone	Mobile	Nov 15 - 12 pm	66.4	5	36	127	Low
268	50% PC + 50% Slag	Limestone	Mobile	Nov 15 - 12 pm	66.4	6	43	132	Low
269	50% PC + 50% Slag	Limestone	Mobile	Nov 15 - 12 pm	66.4	7	49	137	Low
270	50% PC + 50% Slag	Limestone	Mobile	Nov 15 - 12 pm	66.4	8	53	141	Medium
271	50% PC + 50% Slag	Limestone	Huntsville	Jan 15 - 12 pm	41.4	4	19	80	Low
272	50% PC + 50% Slag	Limestone	Huntsville	Jan 15 - 12 pm	41.4	5	25	86	Low
273	50% PC + 50% Slag	Limestone	Huntsville	Jan 15 - 12 pm	41.4	6	30	92	Low
274	50% PC + 50% Slag	Limestone	Huntsville	Jan 15 - 12 pm	41.4	7	35	98	Low
275	50% PC + 50% Slag	Limestone	Huntsville	Jan 15 - 12 pm	41.4	8	39	102	Low
276	50% PC + 50% Slag	Limestone	Huntsville	March 15 - 10 am	55.9	4	27	105	Low
277	50% PC + 50% Slag	Limestone	Huntsville	March 15 - 10 am	55.9	5	33	112	Low
278	50% PC + 50% Slag	Limestone	Huntsville	March 15 - 10 am	55.9	6	39	117	Low
279	50% PC + 50% Slag	Limestone	Huntsville	March 15 - 10 am	55.9	7	44	122	Low
280	50% PC + 50% Slag	Limestone	Huntsville	March 15 - 10 am	55.9	8	49	127	Medium
281	50% PC + 50% Slag	Limestone	Huntsville	May 15 - 8 am	67.5	4	29	126	Low
282	50% PC + 50% Slag	Limestone	Huntsville	May 15 - 8 am	67.5	5	34	132	Low
283	50% PC + 50% Slag	Limestone	Huntsville	May 15 - 8 am	67.5	6	40	137	Low
284	50% PC + 50% Slag	Limestone	Huntsville	May 15 - 8 am	67.5	7	45	142	Low
285	50% PC + 50% Slag	Limestone	Huntsville	May 15 - 8 am	67.5	8	49	145	Medium
286	50% PC + 50% Slag	Limestone	Huntsville	July 15 - 8 am	78.6	4	30	143	Low
287	50% PC + 50% Slag	Limestone	Huntsville	July 15 - 8 am	78.6	5	37	149	Low
288	50% PC + 50% Slag	Limestone	Huntsville	July 15 - 8 am	78.6	6	43	153	Low
289	50% PC + 50% Slag	Limestone	Huntsville	July 15 - 8 am	78.6	7	48	157	Low
290	50% PC + 50% Slag	Limestone	Huntsville	July 15 - 8 am	78.6	8	52	160	High
291	50% PC + 50% Slag	Limestone	Huntsville	Sept 15 - 10 am	75.9	4	31	136	Low
292	50% PC + 50% Slag	Limestone	Huntsville	Sept 15 - 10 am	75.9	5	38	143	Low

293	50% PC + 50% Slag	Limestone	Huntsville	Sept 15 - 10 am	75.9	6	45	148	Low
294	50% PC + 50% Slag	Limestone	Huntsville	Sept 15 - 10 am	75.9	7	50	152	Low
295	50% PC + 50% Slag	Limestone	Huntsville	Sept 15 - 10 am	75.9	8	54	156	High
296	50% PC + 50% Slag	Limestone	Huntsville	Nov 15 - 12 pm	60.1	4	28	109	Low
297	50% PC + 50% Slag	Limestone	Huntsville	Nov 15 - 12 pm	60.1	5	35	116	Low
298	50% PC + 50% Slag	Limestone	Huntsville	Nov 15 - 12 pm	60.1	6	42	122	Low
299	50% PC + 50% Slag	Limestone	Huntsville	Nov 15 - 12 pm	60.1	7	47	127	Low
300	50% PC + 50% Slag	Limestone	Huntsville	Nov 15 - 12 pm	60.1	8	52	131	Low
301	50% PC + 50% Slag	River Gravel	Mobile	Jan 15 - 12 pm	54	4	25	101	Low
302	50% PC + 50% Slag	River Gravel	Mobile	Jan 15 - 12 pm	54	5	31	109	Low
303	50% PC + 50% Slag	River Gravel	Mobile	Jan 15 - 12 pm	54	6	36	115	Low
304	50% PC + 50% Slag	River Gravel	Mobile	Jan 15 - 12 pm	54	7	42	121	Low
305	50% PC + 50% Slag	River Gravel	Mobile	Jan 15 - 12 pm	54	8	47	126	Medium
306	50% PC + 50% Slag	River Gravel	Mobile	March 15 - 10 am	64.4	4	30	122	Low
307	50% PC + 50% Slag	River Gravel	Mobile	March 15 - 10 am	64.4	5	37	129	Low
308	50% PC + 50% Slag	River Gravel	Mobile	March 15 - 10 am	64.4	6	43	135	Medium
309	50% PC + 50% Slag	River Gravel	Mobile	March 15 - 10 am	64.4	7	49	140	High
310	50% PC + 50% Slag	River Gravel	Mobile	March 15 - 10 am	64.4	8	54	144	Very High
311	50% PC + 50% Slag	River Gravel	Mobile	May 15 - 8 am	73.6	4	30	138	Low
312	50% PC + 50% Slag	River Gravel	Mobile	May 15 - 8 am	73.6	5	38	145	Low
313	50% PC + 50% Slag	River Gravel	Mobile	May 15 - 8 am	73.6	6	44	150	Medium
314	50% PC + 50% Slag	River Gravel	Mobile	May 15 - 8 am	73.6	7	49	155	High
315	50% PC + 50% Slag	River Gravel	Mobile	May 15 - 8 am	73.6	8	54	159	Very High
316	50% PC + 50% Slag	River Gravel	Mobile	July 15 - 8 am	80.6	4	30	149	Low
317	50% PC + 50% Slag	River Gravel	Mobile	July 15 - 8 am	80.6	5	38	156	Low
318	50% PC + 50% Slag	River Gravel	Mobile	July 15 - 8 am	80.6	6	44	161	Medium
319	50% PC + 50% Slag	River Gravel	Mobile	July 15 - 8 am	80.6	7	49	165	High
320	50% PC + 50% Slag	River Gravel	Mobile	July 15 - 8 am	80.6	8	54	168	Very High
321	50% PC + 50% Slag	River Gravel	Mobile	Sept 15 - 10 am	81.3	4	32	148	Low
322	50% PC + 50% Slag	River Gravel	Mobile	Sept 15 - 10 am	81.3	5	40	155	Low

323	50% PC + 50% Slag	River Gravel	Mobile	Sept 15 - 10 am	81.3	6	47	161	High
324	50% PC + 50% Slag	River Gravel	Mobile	Sept 15 - 10 am	81.3	7	52	165	Very High
325	50% PC + 50% Slag	River Gravel	Mobile	Sept 15 - 10 am	81.3	8	58	168	Very High
326	50% PC + 50% Slag	River Gravel	Mobile	Nov 15 - 12 pm	66.4	4	30	123	Low
327	50% PC + 50% Slag	River Gravel	Mobile	Nov 15 - 12 pm	66.4	5	37	130	Low
328	50% PC + 50% Slag	River Gravel	Mobile	Nov 15 - 12 pm	66.4	6	43	137	Low
329	50% PC + 50% Slag	River Gravel	Mobile	Nov 15 - 12 pm	66.4	7	50	142	Medium
330	50% PC + 50% Slag	River Gravel	Mobile	Nov 15 - 12 pm	66.4	8	55	146	Very High
331	50% PC + 50% Slag	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	4	20	81	Low
332	50% PC + 50% Slag	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	5	25	89	Low
333	50% PC + 50% Slag	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	6	30	95	Low
334	50% PC + 50% Slag	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	7	36	101	Low
335	50% PC + 50% Slag	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	8	41	106	Low
336	50% PC + 50% Slag	River Gravel	Huntsville	March 15 - 10 am	55.9	4	27	108	Low
337	50% PC + 50% Slag	River Gravel	Huntsville	March 15 - 10 am	55.9	5	34	115	Low
338	50% PC + 50% Slag	River Gravel	Huntsville	March 15 - 10 am	55.9	6	39	121	Medium
339	50% PC + 50% Slag	River Gravel	Huntsville	March 15 - 10 am	55.9	7	45	127	High
340	50% PC + 50% Slag	River Gravel	Huntsville	March 15 - 10 am	55.9	8	50	131	Very High
341	50% PC + 50% Slag	River Gravel	Huntsville	May 15 - 8 am	67.5	4	29	130	Low
342	50% PC + 50% Slag	River Gravel	Huntsville	May 15 - 8 am	67.5	5	35	136	Low
343	50% PC + 50% Slag	River Gravel	Huntsville	May 15 - 8 am	67.5	6	41	142	Medium
344	50% PC + 50% Slag	River Gravel	Huntsville	May 15 - 8 am	67.5	7	47	146	High
345	50% PC + 50% Slag	River Gravel	Huntsville	May 15 - 8 am	67.5	8	51	150	Very High
346	50% PC + 50% Slag	River Gravel	Huntsville	July 15 - 8 am	78.6	4	30	147	Low
347	50% PC + 50% Slag	River Gravel	Huntsville	July 15 - 8 am	78.6	5	38	153	Low
348	50% PC + 50% Slag	River Gravel	Huntsville	July 15 - 8 am	78.6	6	44	158	Medium
349	50% PC + 50% Slag	River Gravel	Huntsville	July 15 - 8 am	78.6	7	49	162	High
350	50% PC + 50% Slag	River Gravel	Huntsville	July 15 - 8 am	78.6	8	54	166	Very High
351	50% PC + 50% Slag	River Gravel	Huntsville	Sept 15 - 10 am	75.9	4	31	140	Low
352	50% PC + 50% Slag	River Gravel	Huntsville	Sept 15 - 10 am	75.9	5	39	147	Low

353	50% PC + 50% Slag	River Gravel	Huntsville	Sept 15 - 10 am	75.9	6	45	153	Medium
354	50% PC + 50% Slag	River Gravel	Huntsville	Sept 15 - 10 am	75.9	7	51	157	High
355	50% PC + 50% Slag	River Gravel	Huntsville	Sept 15 - 10 am	75.9	8	57	161	Very High
356	50% PC + 50% Slag	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	4	28	111	Low
357	50% PC + 50% Slag	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	5	35	119	Low
358	50% PC + 50% Slag	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	6	42	126	Low
359	50% PC + 50% Slag	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	7	48	131	Medium
360	50% PC + 50% Slag	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	8	54	136	High
361	70% PC + 30% CFA	Limestone	Mobile	Jan 15 - 12 pm	54	4	28	105	Low
362	70% PC + 30% CFA	Limestone	Mobile	Jan 15 - 12 pm	54	5	34	112	Low
363	70% PC + 30% CFA	Limestone	Mobile	Jan 15 - 12 pm	54	6	39	117	Low
364	70% PC + 30% CFA	Limestone	Mobile	Jan 15 - 12 pm	54	7	43	121	Low
365	70% PC + 30% CFA	Limestone	Mobile	Jan 15 - 12 pm	54	8	47	125	Low
366	70% PC + 30% CFA	Limestone	Mobile	March 15 - 10 am	64.4	4	32	124	Low
367	70% PC + 30% CFA	Limestone	Mobile	March 15 - 10 am	64.4	5	38	129	Low
368	70% PC + 30% CFA	Limestone	Mobile	March 15 - 10 am	64.4	6	43	134	Low
369	70% PC + 30% CFA	Limestone	Mobile	March 15 - 10 am	64.4	7	48	137	Low
370	70% PC + 30% CFA	Limestone	Mobile	March 15 - 10 am	64.4	8	52	140	Medium
371	70% PC + 30% CFA	Limestone	Mobile	May 15 - 8 am	73.6	4	31	138	Low
372	70% PC + 30% CFA	Limestone	Mobile	May 15 - 8 am	73.6	5	38	143	Low
373	70% PC + 30% CFA	Limestone	Mobile	May 15 - 8 am	73.6	6	42	147	Low
374	70% PC + 30% CFA	Limestone	Mobile	May 15 - 8 am	73.6	7	46	149	Low
375	70% PC + 30% CFA	Limestone	Mobile	May 15 - 8 am	73.6	8	50	151	Medium
376	70% PC + 30% CFA	Limestone	Mobile	July 15 - 8 am	80.6	4	31	147	Low
377	70% PC + 30% CFA	Limestone	Mobile	July 15 - 8 am	80.6	5	38	152	Low
378	70% PC + 30% CFA	Limestone	Mobile	July 15 - 8 am	80.6	6	42	155	Low
379	70% PC + 30% CFA	Limestone	Mobile	July 15 - 8 am	80.6	7	46	158	Low
380	70% PC + 30% CFA	Limestone	Mobile	July 15 - 8 am	80.6	8	50	160	Medium
381	70% PC + 30% CFA	Limestone	Mobile	Sept 15 - 10 am	81.3	4	33	147	Low
382	70% PC + 30% CFA	Limestone	Mobile	Sept 15 - 10 am	81.3	5	40	152	Low

383	70% PC + 30% CFA	Limestone	Mobile	Sept 15 - 10 am	81.3	6	45	155	Low
384	70% PC + 30% CFA	Limestone	Mobile	Sept 15 - 10 am	81.3	7	49	158	Low
385	70% PC + 30% CFA	Limestone	Mobile	Sept 15 - 10 am	81.3	8	53	160	Medium
386	70% PC + 30% CFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	4	32	124	Low
387	70% PC + 30% CFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	5	38	130	Low
388	70% PC + 30% CFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	6	44	135	Low
389	70% PC + 30% CFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	7	49	139	Low
390	70% PC + 30% CFA	Limestone	Mobile	Nov 15 - 12 pm	66.4	8	53	141	Low
391	70% PC + 30% CFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	4	23	87	Low
392	70% PC + 30% CFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	5	29	94	Low
393	70% PC + 30% CFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	6	34	99	Low
394	70% PC + 30% CFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	7	39	104	Low
395	70% PC + 30% CFA	Limestone	Huntsville	Jan 15 - 12 pm	41.4	8	43	108	Low
396	70% PC + 30% CFA	Limestone	Huntsville	March 15 - 10 am	55.9	4	30	111	Low
397	70% PC + 30% CFA	Limestone	Huntsville	March 15 - 10 am	55.9	5	36	117	Low
398	70% PC + 30% CFA	Limestone	Huntsville	March 15 - 10 am	55.9	6	41	122	Low
399	70% PC + 30% CFA	Limestone	Huntsville	March 15 - 10 am	55.9	7	46	126	Low
400	70% PC + 30% CFA	Limestone	Huntsville	March 15 - 10 am	55.9	8	49	129	Medium
401	70% PC + 30% CFA	Limestone	Huntsville	May 15 - 8 am	67.5	4	30	130	Low
402	70% PC + 30% CFA	Limestone	Huntsville	May 15 - 8 am	67.5	5	36	135	Low
403	70% PC + 30% CFA	Limestone	Huntsville	May 15 - 8 am	67.5	6	40	139	Low
404	70% PC + 30% CFA	Limestone	Huntsville	May 15 - 8 am	67.5	7	45	142	Low
405	70% PC + 30% CFA	Limestone	Huntsville	May 15 - 8 am	67.5	8	48	144	Low
406	70% PC + 30% CFA	Limestone	Huntsville	July 15 - 8 am	78.6	4	31	145	Low
407	70% PC + 30% CFA	Limestone	Huntsville	July 15 - 8 am	78.6	5	37	150	Low
408	70% PC + 30% CFA	Limestone	Huntsville	July 15 - 8 am	78.6	6	42	153	Low
409	70% PC + 30% CFA	Limestone	Huntsville	July 15 - 8 am	78.6	7	46	156	Low
410	70% PC + 30% CFA	Limestone	Huntsville	July 15 - 8 am	78.6	8	50	158	Medium
411	70% PC + 30% CFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	4	32	139	Low
412	70% PC + 30% CFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	5	39	145	Low

413	70% PC + 30% CFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	6	44	149	Low
414	70% PC + 30% CFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	7	49	152	Low
415	70% PC + 30% CFA	Limestone	Huntsville	Sept 15 - 10 am	75.9	8	53	154	Medium
416	70% PC + 30% CFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	4	31	114	Low
417	70% PC + 30% CFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	5	37	121	Low
418	70% PC + 30% CFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	6	44	126	Low
419	70% PC + 30% CFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	7	49	130	Low
420	70% PC + 30% CFA	Limestone	Huntsville	Nov 15 - 12 pm	60.1	8	52	133	Low
421	70% PC + 30% CFA	River Gravel	Mobile	Jan 15 - 12 pm	54	4	29	108	Low
422	70% PC + 30% CFA	River Gravel	Mobile	Jan 15 - 12 pm	54	5	35	116	Low
423	70% PC + 30% CFA	River Gravel	Mobile	Jan 15 - 12 pm	54	6	40	122	Low
424	70% PC + 30% CFA	River Gravel	Mobile	Jan 15 - 12 pm	54	7	45	126	Medium
425	70% PC + 30% CFA	River Gravel	Mobile	Jan 15 - 12 pm	54	8	49	130	High
426	70% PC + 30% CFA	River Gravel	Mobile	March 15 - 10 am	64.4	4	32	128	Low
427	70% PC + 30% CFA	River Gravel	Mobile	March 15 - 10 am	64.4	5	40	134	Low
428	70% PC + 30% CFA	River Gravel	Mobile	March 15 - 10 am	64.4	6	45	139	Medium
429	70% PC + 30% CFA	River Gravel	Mobile	March 15 - 10 am	64.4	7	49	143	High
430	70% PC + 30% CFA	River Gravel	Mobile	March 15 - 10 am	64.4	8	54	146	Very High
431	70% PC + 30% CFA	River Gravel	Mobile	May 15 - 8 am	73.6	4	31	142	Low
432	70% PC + 30% CFA	River Gravel	Mobile	May 15 - 8 am	73.6	5	39	148	Low
433	70% PC + 30% CFA	River Gravel	Mobile	May 15 - 8 am	73.6	6	44	152	Medium
434	70% PC + 30% CFA	River Gravel	Mobile	May 15 - 8 am	73.6	7	48	155	High
435	70% PC + 30% CFA	River Gravel	Mobile	May 15 - 8 am	73.6	8	52	158	Very High
436	70% PC + 30% CFA	River Gravel	Mobile	July 15 - 8 am	80.6	4	30	152	Low
437	70% PC + 30% CFA	River Gravel	Mobile	July 15 - 8 am	80.6	5	38	157	Low
438	70% PC + 30% CFA	River Gravel	Mobile	July 15 - 8 am	80.6	6	44	161	Medium
439	70% PC + 30% CFA	River Gravel	Mobile	July 15 - 8 am	80.6	7	48	164	High
440	70% PC + 30% CFA	River Gravel	Mobile	July 15 - 8 am	80.6	8	52	166	Very High
441	70% PC + 30% CFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	4	32	152	Low
442	70% PC + 30% CFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	5	41	157	Low

443	70% PC + 30% CFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	6	47	161	Medium
444	70% PC + 30% CFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	7	50	164	High
445	70% PC + 30% CFA	River Gravel	Mobile	Sept 15 - 10 am	81.3	8	55	166	Very High
446	70% PC + 30% CFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	4	32	128	Low
447	70% PC + 30% CFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	5	39	135	Low
448	70% PC + 30% CFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	6	45	140	Low
449	70% PC + 30% CFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	7	50	144	Medium
450	70% PC + 30% CFA	River Gravel	Mobile	Nov 15 - 12 pm	66.4	8	55	147	High
451	70% PC + 30% CFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	4	23	89	Low
452	70% PC + 30% CFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	5	29	97	Low
453	70% PC + 30% CFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	6	35	103	Low
454	70% PC + 30% CFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	7	40	108	Low
455	70% PC + 30% CFA	River Gravel	Huntsville	Jan 15 - 12 pm	41.4	8	45	113	Medium
456	70% PC + 30% CFA	River Gravel	Huntsville	March 15 - 10 am	55.9	4	31	115	Low
457	70% PC + 30% CFA	River Gravel	Huntsville	March 15 - 10 am	55.9	5	37	122	Low
458	70% PC + 30% CFA	River Gravel	Huntsville	March 15 - 10 am	55.9	6	42	127	Medium
459	70% PC + 30% CFA	River Gravel	Huntsville	March 15 - 10 am	55.9	7	47	131	High
460	70% PC + 30% CFA	River Gravel	Huntsville	March 15 - 10 am	55.9	8	52	134	Very High
461	70% PC + 30% CFA	River Gravel	Huntsville	May 15 - 8 am	67.5	4	30	134	Low
462	70% PC + 30% CFA	River Gravel	Huntsville	May 15 - 8 am	67.5	5	37	140	Low
463	70% PC + 30% CFA	River Gravel	Huntsville	May 15 - 8 am	67.5	6	42	145	Medium
464	70% PC + 30% CFA	River Gravel	Huntsville	May 15 - 8 am	67.5	7	46	148	High
465	70% PC + 30% CFA	River Gravel	Huntsville	May 15 - 8 am	67.5	8	51	151	Very High
466	70% PC + 30% CFA	River Gravel	Huntsville	July 15 - 8 am	78.6	4	31	150	Low
467	70% PC + 30% CFA	River Gravel	Huntsville	July 15 - 8 am	78.6	5	38	155	Low
468	70% PC + 30% CFA	River Gravel	Huntsville	July 15 - 8 am	78.6	6	44	159	Medium
469	70% PC + 30% CFA	River Gravel	Huntsville	July 15 - 8 am	78.6	7	47	162	High
470	70% PC + 30% CFA	River Gravel	Huntsville	July 15 - 8 am	78.6	8	52	164	Very High
471	70% PC + 30% CFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	4	34	144	Low
472	70% PC + 30% CFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	5	40	150	Low

473	70% PC + 30% CFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	6	46	154	Medium
474	70% PC + 30% CFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	7	50	157	High
475	70% PC + 30% CFA	River Gravel	Huntsville	Sept 15 - 10 am	75.9	8	55	160	Very High
476	70% PC + 30% CFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	4	31	118	Low
477	70% PC + 30% CFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	5	38	125	Low
478	70% PC + 30% CFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	6	44	131	Low
479	70% PC + 30% CFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	7	50	135	Medium
480	70% PC + 30% CFA	River Gravel	Huntsville	Nov 15 - 12 pm	60.1	8	55	139	High