Evaluation of In-Place Concrete Strength by Core Testing

by

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Abstract

When the average compressive strength of molded cylinder specimens do not meet the minimum required strength for a batch of concrete, testing must be performed on the in-place concrete to evaluate its strength. There are a variety of in-place testing methods used throughout the concrete industry. Of these, core testing is the most popular. Other methods, such as cast-in-place cylinders and pullout inserts can be used to evaluate the in-place strength of the concrete in question. The purpose of this project was to evaluate several different factors, such as aggregate type, strength level, restraint, concrete age and type of cementitious material, to determine their influence on the in-place concrete strength. Another goal of the project was to determine the relationship between core strength and molded cylinder strength. To do this, field slabs were cast with different aggregate types, supplemental cementitious materials, and strength levels. Testing was conducted near the edges of the slab as well as in the interior of the slab to determine if restraint had an impact on microcracking. Cores were recovered and tested in compression at ages of 28, 42, 91, and 365 days. Cast-in-place cylinders and pullout tests were also conducted at the same ages.

From the project, it was found that core strength is on average 87% of molded cylinder strength. Therefore, it is recommended that if the average core strength is 85% of the minimum design strength, the in-place concrete has satisfactory strength. This is accordance with ACI 318 (2011). It was also found that restraint and aggregate type had an impact on some test methods but not others.

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Table of Contents

Abstract ii

Acknowledgments iii

List of Tables ix

List of Figures xii

CHAPTER 1: INTRODUCTION

1.1 Background 1

1.2 Research Significance 3

1.3 Project Objectives 4

1.4 Report Scope 4

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction 6

2.2 Factors Affecting In-Place Strength 6

2.2.1 Age 7

2.2.2 Supplemental Cementing Materials 9

2.2.2.1 Fly Ash 10

2.2.2.2 Slag Cement 11

2.2.3 Moisture Conditioning 11

2.2.4 Coarse Aggregate Size and Type 13

2.2.5 Temperature Conditions 14

2.3 Strength Test Methods 15

2.3.1 Molded Cylinders 15

2.3.1.1 Summary of AASHTO T 23: The Making and Curing of Concrete Test Specimens in the Field ... 15
2.3.1.2 Strength Acceptance of Molded Cylinders ... 16
2.3.1.3 Variability ... 16
2.3.2 Cores ... 17
2.3.2.1 Summary of AASHTO T 24: Standard Method of Test for Obtaining and Testing Drilled Cores and Sawed Beams ... 18
2.3.2.2 Factors Affecting Apparent Strength of Cores ... 19
2.3.2.2.1 Length-to-Diameter Ratio ... 21
2.3.2.2.2 Core Diameter ... 22
2.3.2.2.3 Core Damage ... 24
2.3.2.2.4 Casting Direction ... 25
2.3.2.2.5 Presence of Embedded Steel Reinforcement in Core ... 27
2.3.2.3 Variability of Core Testing ... 27
2.3.2.4 Impact of Number of Cores Retrieved ... 28
2.3.3 Cast-In-Place Cylinders ... 30
2.3.3.1 Summary of ASTM C873 ... 31
2.3.3.2 Variability of Cast-in-Place Cylinders ... 33
2.3.4 Pullout Testing ... 33
2.3.4.1 Summary of ASTM C900 ... 34
2.3.4.2 Failure Mechanism ... 35
2.3.4.3 Variability of Pullout Testing ... 35
2.4 Summary of State DOT Payment Reduction Methods ... 38

2.4.1 Alabama Department of Transportation 38
2.4.2 Tennessee Department of Transportation 39
2.4.3 Florida Department of Transportation 41
2.4.4 Texas Department of Transportation 43
2.4.5 Comparison of State DOT Payment Reduction Methods 44
2.5 Summary 45
CHAPTER 3: EXPERIMENTAL PLAN
3.1 Introduction and Problem Definition 47
3.2 Development of Experimental Plan 49
3.3 Site Preparation 59
3.4 iButton Temperature Sensors 61
3.5 Casting 65
3.6 Finishing and Curing Methods 67
3.7 Molded Cylinders 69
3.8 Pullout Calibration Cubes 72
3.9 Cores 73
3.10 Cast-In-Place Cylinders 75
3.11 Pullout Inserts 78
3.12 Raw Materials 80
3.12.1 Coarse Aggregate and Fine Aggregate 80
3.12.2 Cement and Supplemental Cementing Materials 82
3.12.2.1 Class C Fly Ash 82
3.12.2.2 Class F Fly Ash 82

3.12.2.3 Slag Cement 82

3.12.2.4 Type I Cement 83

CHAPTER 4: PRESENTATION OF RESULTS

4.1 Introduction 84

4.2 Temperature Data 85

4.3 Effect of Cylinder Size 85

4.4 Verification of Pullout Table Provided by Germann Instruments 88

4.5 Effect of Aggregate Type 90

4.5.1 Effect of Aggregate Type on Strength of 6×12 in. Cylinders versus 4×8 in. Molded Cylinders 90

4.5.2 Effect of Aggregate Type on In-Place Testing 91

4.6 Effect of Restraint 93

4.7 Effect of Supplemental Cementing Materials and Strength Gain Over Time 98

4.8 Comparison of Core to Molded Cylinder Strengths 111

CHAPTER 5: DEVELOPMENT OF IMPLEMENTATION GUIDELINES FOR CORE TESTING

5.1 Introduction 118

5.2 Current ALDOT Coring and Evaluation Practice 118

5.3 Evaluation of Variables Affecting Core Strength 119

5.3.1 Core Length-to-Diameter Ratio 119

5.3.2 Core Diameter 120

5.3.3 Coring Direction Relative to Casting Direction 120

5.3.4 Aggregate Size 121

5.3.5 Moisture Conditioning 121

5.3.6 Damage 122

5.3.7 Aggregate Type 122

5.3.8 Effect of Restraint 123

5.3.9 Age 123

5.3.10 Presence of Steel Reinforcement 124

5.3.11 Relationship Between Core and Quality Assurance Cylinder Strengths 126

5.4 Recommended Procedure for Correcting Core Strength 126

CHAPTER 6: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary 130

6.2 Conclusions 133

6.3 Recommendations 134

References 136

Appendix A iButton Temperature Data 140

Appendix B Pullout Calibration Table from Germann Instruments 190

Appendix C Collected Strength Data 208

List of Tables

Table 2-1 Constants for ACI 209 Age Correction Equation.. 9

Table 2-2 Required Rod Diameter for the Consolidation of Cyliders....................................... 16

Table 2-3 Coefficients of Variation for Molded Cylinders 17

Table 2-4 Core Strength Correction Factors.. 21

Table 2-5 Recommended Correction Factors for Different L/D Ratios For Concrete Strengths Less Than 6000 psi 22

Table 2-6 Suggested Strength Correction Factors for Different Core Diameters....................... 24

Table 2-7 Strength Correction Factors for Steel Reinforcement Present in Core Sample.......... 27

Table 2-8 Probable Range of Core Strengths Due to Single-Operator Error 28

Table 2-9 Coefficient of Variation Due to In-Place Strength Variation Within a Structure 30

Table 2-10 Acceptable Pull-Out Test Range Based on Number of Tests.................................. 36

Table 2-11 Summary of Within-Test Coefficient of Variation of Pullout Test.......................... 37

Table 2-12 Tennessee Department of Transportation Classes of Concrete................................ 40

Table 2-13 Speficied Acceptable Average Strength Concrete Specimens for Given Class of Concrete based on Age of Specimen 40

Table 2-14 Price Adjustment for Tennessee Department of Transportation 41

Table 3-1 Abbreviations for Different Aggregate Types... 56

Table 3-2 Abbreviations for Different Supplemental Cementing Materials 56

Table 3-3 SSD Batch Weights for High-Strength Slabs.. 57

Table 3-4 SSD Batch Weights for Normal-Strength Slabs... 58

Table 3-5 Labeling of iButtons.. 64

Table 3-6 Coarse Aggregate Properties... 81

Table 4-1 Temperature Data for All Casts... 85

Table 4-2 Summary of P-values from ANOVA Analysis for In-Place Testing 91

Table 4-3 Summary of In-Place Strength to Molded Cylinder Strength Ratios by Coarse Aggregate Type for Normal-Strength Concrete 92

Table 4-4 P-values for Strength Level t-test 94

Table 4-5 P-values for t-tests Determining the Effect of Restraint 94

Table 4-6 Constants for ACI 209 Age Correction Equation... 99

Table 4-7 Summary of P-Values from ANOVA Analysis of Strength Gain for Different SCM Types.. 104

Table 4-8 P-Values of Paired t-tests Conducted on Portland Cement Specimens Versus Specimens Containing Supplemental Cementing Materials.. 105

Table 4-9 Summary of the Unbiased Estimate of the Standard Deviation for Strength Gain in Normal Strength Concrete 107

Table 4-10 Adjusted a and β Values for Different Testing Methods 108

Table 4-11 Comparison of Adjusted and Unadjusted ACI 209 Values of the Unbiased Estimate of the Standard Deviation 108

Table 4-12 Summary of P-Values Comparing Normal vs. High Strength Cores..................... 112

Table 4-13 P-values from Core versus Molded Cylinder t-test 113

Table 5-1 $T_{critical}$ Values for Given Number of Cores in a Sample 128

Table 5-2 Values of k based on the Number of Cores Extracted... 129

Table B-1 Pullout Table from Germann Instruments for 6/4/2013-6/4/2014....................191-198

Table B-2 Pullout Table from Germann Instruments for 6/5/2014-6/5/2015....................200-207

Table C-1 Molded Cylinder Strengths from Cast RG4000CA.. 208

Table C-2 Molded Cylinder Strengths from Cast LS4000CT 209

Table C-3 Molded Cylinder Strengths from Cast RG4000CT 210

Table C-4 Molded Cylinder Strengths from Cast RG4000SC... 211

Table C-5 Molded Cylinder Strengths from Cast GR4000CT 212

Table C-6 Molded Cylinder Strengths from Cast RG4000FA 213

Table C-7 Molded Cylinder Strengths from Cast RG8000CT 214

Table C-8 Molded Cylinder Strengths from Cast LS8000CT 215

Table C-9 Cast-In-Place Cylinder Strengths from Cast RG4000SC 215

Table C-10 Cast-In-Place Cylinder Strengths from Cast GR4000CT 216

Table C-11 Cast-In-Place Cylinder Strengths from Cast RG4000FA 216

Table C-12 Cast-In-Place Cylinder Strengths from Cast RG8000CT 217

Table C-13 Cast-In-Place Cylinder Strengths from Cast LS8000CT 217

Table C-14 Core Strengths from Cast RG4000CA 218

Table C-15 Core Strengths from Cast LS4000CT 218

Table C-16 Core Strengths from Cast RG4000CT 219

Table C-17 Core Strengths from Cast RG4000SC 219

Table C-18 Core Strengths from Cast GR4000CT 220

Table C-19 Core Strengths from Cast RG4000FA 220

Table C-20 Core Strengths from Cast RG8000CT 221

Table C-21 Core Strengths from Cast LS8000CT 221

Table C-22 Pullout Strengths from Cast RG4000CA 222

Table C-23 Pullout Strengths from Cast LS4000CT 223

Table C-24 Pullout Strengths from Cast RG4000CT 224

Table C-25 Pullout Strengths from Cast RG4000SC 225

Table C-26 Pullout Strengths from Cast GR4000CT 226

Table C-27 Pullout Strengths from Cast RG4000FA 227

List of Figures

Figure 2-1 Compressive Strength vs. Age for Different Curing Conditions 12

Figure 2-2 Regression Plot of Core Strength vs. Moisture Gain 12

Figure 2-3 Normalized Average Core Strength versus Core Diameter 23

Figure 2-4 Effect of Coring Direction Relative to Casting Direction....................................... 26

Figure 2-5 *k* versus Number of Cores Given Coefficient of Variation.................................... 29

Figure 2-6 Schematic of Cast-In-Place Cylinder Mold Assembly 31

Figure 2-7 Schematic of LOK-Test Pullout Insert.. 34

Figure 2-8 Current ALDOT Price Adjustment 39

Figure 2-9: Summary of Payment Reduction Methods 44

Figure 3-1 Forces Affecting the Axial Restraint of a Slab 52

Figure 3-2 Typical Stress Distribution Between Slab and Sub-Base when Exposed to Shrinkage and Temperature Change Forces 52

Figure 3-3 Effect of Restraint on Slab Specimens.. 53

Figure 3-4 Typical Slab Layout 54

Figure 3-5 Testing Layout within a Typical Quadrant 55

Figure 3-6 Typical Slab Identification.. 57

Figure 3-7 Skid-Steer Used for Site Work.. 59

Figure 3-8 Second 15’ x 60’ Area Before Clearing.. 59

Figure 3-9 Second 15’ x 60’ Area After Clearing 60

Figure 3-10 Second 15’ x 60’ Area After Placement of Sub-Base.. 60

Figure 3-11 Typical Setup Before Placement... 61

Figure 3-12 Placement of iButton Sensors in a Slab 62

Figure 3-13 iButton Temperature Sensors in a 4×8 in. Cylinder... 62

Figure 3-14 iButon Held in a Clip 63

Figure 3-15 Soldered iButton........ 63

Figure 3-16 Telephone Plug-End of Communication Wire........ 64

Figure 3-17 Internal Vibration of a Slab........ 66

Figure 3-18 A Completed Slab 67

Figure 3-19 Curing Mats and Soaker Hoses 68

Figure 3-20 Complete Curing System 68

Figure 3-21 Making of 6×12 in. Molded Cylinders........ 69

Figure 3-22 Molded Cylinders in Their Initial Curing State 70

Figure 3-23 Molded Cylinders and Cubes Being Transported Back to AU Materials Lab........ 71

Figure 3-24 Typical Labeling of Molded Cylinders 71

Figure 3-25 Making of the LOK-Test Pullout Calibration Cubes 72

Figure 3-26 Drilling of a Core from the Exterior Region of a Slab........ 74

Figure 3-27 Filling of Cast-In-Place Cylinders 76

Figure 3-28 Removal of a Cast-In-Place Cylinder From a Slab 77

Figure 3-29 Typical L-50 and L-49 Inserts........ 78

Figure 3-30 Placement of L-49 LOK-Test Pullout Inserts Into a Slab 79

Figure 3-31 Pullout Test Being Performed........ 80

Figure 3-32 Coarse Aggregate Gradations 81

Figure 4-1 Comparison of the Average Strengths of 6×12 in. Cylinders versus 4×8 in. Cylinders 87

Figure 4-2 Average 6×12 in. Cylinders versus Average Calibration Pullout Cube Strength..... 89

Figure 4-3 Average Exterior Core Strength versus Average Interior Core Strength........ 95

Figure 4-4 Average Strength of Exterior Pullout Tests versus Average Strength of Interior Pullout Tests 96

Figure 4-5 Average Strength of Exterior Cast-in-Place Cylinders versus Average Strength of Interior Cast-in-Place Cylinders 97

Figure 4-6 Strength Gain of Concrete with Only Type I Portland Cement 100

Figure 4-7 Strength Gain of Concrete with 20% Class C Fly Ash 101

Figure 4-8 Strength Gain of Concrete with 20% Class F Fly Ash 102

Figure 4-9 Strength Gain of Concrete with 50% Slag Cement 103

Figure 4-10 Measured Strength Versus Estimated Strength Using ACI 209 for Molded 6×12 in. Cylinders with Unadjusted *a* and *β* Values 109

Figure 4-11 Measured Strength Versus Estimated Strength Using ACI 209 for Molded 6×12 in. Cylinders with Adjusted *a* and *β* Values 109

Figure 4-12 Measured Strength Versus Estimated Strength Using ACI 209 for Interior Cores with Unadjusted *a* and *β* Values 110

Figure 4-13 Measured Strength Versus Estimated Strength Using ACI 209 for Interior Cores with Adjusted *a* and *β* Values 110

Figure 4-14 Average 6×12 in. Molded Cylinder Strength versus Average Exterior Core Strength 114

Figure 4-15 Average 6×12 in. Molded Cylinder Strength versus Average Interior Core Strength 115

Figure 4-16 Average 6×12 in. Molded Cylinder Strength versus Average Core Strength 116

Figure 5-1 Average 6×12 in. Molded Cylinder Strength versus Average Core Strength 125

Figure 5-2 Average 6×12 in. Molded Cylinder Strength versus Average Core Strength 125

Figure 6-1 Current ALDOT Price Adjustment 131

Figure A-1 RG4000CA C612M 7-Day Temperature Data 140

Figure A-2 RG4000CA C612M 365-Day Temperature Data 141

Figure A-3 RG4000CA C612T 7-Day Temperature Data 141

Figure A-4 RG4000CA C612T 91-Day Temperature Data 142

Figure A-5 RG4000CA C48M 7-Day Temperature Data 142

Figure A-6 RG4000CA C48M 365-Day Temperature Data 143

Figure A-7 RG4000CA C48T 7-Day Temperature Data 143

Figure A-8 RG4000CA C48T 91-Day Temperature Data 144

Figure A-9 RG4000CA IM 7-Day Temperature Data 144

Figure A-10 RG4000CA IM 365-Day Temperature Data 145

Figure A-11 RG4000CA IT 7-Day Temperature Data 145

Figure A-12 RG4000CA IT 365-Day Temperature Data 146

Figure A-13 RG4000CA OT 7-Day Temperature Data 146

Figure A-14 RG4000CA OT 365-Day Temperature Data 147

Figure A-15 LS4000CT C48M 7-Day Temperature Data 147

Figure A-16 LS4000CT C48M 365-Day Temperature Data 148

Figure A-17 LS4000CT C48T 7-Day Temperature Data 148

Figure A-18 LS4000CT C48T 91-Day Temperature Data 149

Figure A-19 LS4000CT IM 7-Day Temperature Data 149

Figure A-20 LS4000CT IM 365-Day Temperature Data 150

Figure A-21 LS4000CT IT 7-Day Temperature Data 150

Figure A-22 LS4000CT IT 365-Day Temperature Data 151

Figure A-23 LS4000CT OM 7-Day Temperature Data 151

Figure A-24 LS4000CT OM 365-Day Temperature Data 152

Figure A-25 LS4000CT OT 7-Day Temperature Data 152

Figure A-26 LS4000CT OT 365-Day Temperature Data 153

Figure A-27 RG4000CT 7-Day Temperature Data 153
Figure A-28 RG4000CT C612M 365-Day Temperature Data 154
Figure A-29 RG4000CT C612T 7-Day Temperature Data 154
Figure A-30 RG4000CT C612T 365-Day Temperature Data 155
Figure A-31 RG4000CT C48M 7-Day Temperature Data 155
Figure A-32 RG4000CT C48M 365-Day Temperature Data 156
Figure A-33 RG4000CT IM 7-Day Temperature Data 156
Figure A-34 RG4000CT IM 365-Day Temperature Data 157
Figure A-35 RG4000CT IT 7-Day Temperature Data 157
Figure A-36 RG4000CT IT 365-Day Temperature Data 158
Figure A-37 RG4000CT OM 7-Day Temperature Data 158
Figure A-38 RG4000CT OM 365-Day Temperature Data 159
Figure A-39 RG4000CT OT 7-Day Temperature Data 159
Figure A-40 RG4000CT OT 365-Day Temperature Data 160
Figure A-41 RG4000SC C612M 7-Day Temperature Data 160
Figure A-42 RG4000SC C612M 365-Day Temperature Data 161
Figure A-43 RG4000SC C612T 7-Day Temperature Data 161
Figure A-44 RG4000SC C48M 7-Day Temperature Data 162
Figure A-45 RG4000SC C48M 91-Day Temperature Data 162
Figure A-46 RG4000SC C48T 7-Day Temperature Data 163
Figure A-47 RG4000SC C48T 365-Day Temperature Data 163
Figure A-48 RG4000SC IM 7-Day Temperature Data 164
Figure A-49 RG4000SC IM 365-Day Temperature Data 164

Figure A-50 RG4000SC IT 7-Day Temperature Data 165

Figure A-51 RG4000SC IT 365-Day Temperature Data 165

Figure A-52 RG4000SC OM 7-Day Temperature Data 166

Figure A-53 RG4000SC OM 91-Day Temperature Data 166

Figure A-54 RG4000SC OT 7-Day Temperature Data 167

Figure A-55 RG4000SC OT 365-Day Temperature Data 167

Figure A-56 GR4000CT C612M 7-Day Temperature Data 168

Figure A-57 GR4000CT C612M 91-Day Temperature Data 168

Figure A-58 GR4000CT C612T 7-Day Temperature Data 169

Figure A-59 GR4000CT C612T 365-Day Temperature Data 169

Figure A-60 GR4000CT C48M 7-Day Temperature Data 170

Figure A-61 GR4000CT C48M 91-Day Temperature Data 170

Figure A-62 GR4000CT C48T 7-Day Temperature Data 171

Figure A-63 GR4000CT C48T 91-Day Temperature Data 171

Figure A-64 GR4000CT IM 7-Day Temperature Data 172

Figure A-65 GR4000CT IM 365-Day Temperature Data 172

Figure A-66 GR4000CT IT 7-Day Temperature Data 173

Figure A-67 GR4000CT IT 91-Day Temperature Data 173

Figure A-68 GR4000CT OM 7-Day Temperature Data 174

Figure A-69 GR4000CT OM 365-Day Temperature Data 174

Figure A-70 GR4000CT OT 7-Day Temperature Data 175

Figure A-71 GR4000CT OT 365-Day Temperature Data 175

Figure A-72 RG4000FA C612M 7-Day Temperature Data 176

Figure A-73 RG4000FA C612T 7-Day Temperature Data 176

Figure A-74 RG4000FA C48M 7-Day Temperature Data 177

Figure A-75 RG4000FA C48T 7-Day Temperature Data 177

Figure A-76 RG4000FA IM 7-Day Temperature Data 178

Figure A-77 RG4000FA IT 7-Day Temperature Data 178

Figure A-78 RG4000FA OM 7-Day Temperature Data 179

Figure A-79 RG4000FA OT 7-Day Temperature Data 179

Figure A-80 RG8000CT C612M 7-Day Temperature Data 180

Figure A-81 RG8000CT C612M 365-Day Temperature Data 180

Figure A-82 RG8000CT C612T 7-Day Temperature Data 181

Figure A-83 RG8000CT C612T 365-Day Temperature Data 181

Figure A-84 RG8000CT C48M 7-Day Temperature Data 182

Figure A-85 RG8000CT C48T 7-Day Temperature Data 182

Figure A-86 RG8000CT C48T 365-Day Temperature Data 183

Figure A-87 RG8000CT IM 7-Day Temperature Data 183

Figure A-88 RG8000CT IM 91-Day Temperature Data 184

Figure A-89 RG8000CT IT 7-Day Temperature Data 184

Figure A-90 RG8000CT OM 7-Day Temperature Data 185

Figure A-91 LS8000CT C612M 7-Day Temperature Data 185

Figure A-92 LS8000CT C612T 7-Day Temperature Data 186

Figure A-93 LS8000CT C48M 7-Day Temperature Data 186

Figure A-94 LS8000CT C48T 7-Day Temperature Data 187

Figure A-95 LS8000CT IM 7-Day Temperature Data 187

Figure A-96 LS8000CT IT 7-Day Temperature Data 188

Figure A-97 LS8000CT OM 7-Day Temperature Data... 188

Figure A-98 LS8000CT OT 7-Day Temperature Data... 189

Figure B-1 Pullout Force vs. Predicted Molded Cylinder Strength for 6/4/2013-6/4/2014 190

Figure B-2 Pullout Force vs. Predicted Molded Cylinder Strength for 6/5/2014-6/5/2015 199

Chapter 1

Introduction

1.1 Background

For many years, concrete researchers have evaluated the relationship between molded cylinder strength and in-place concrete strength. It has been well documented that molded cylinders do not provide an accurate representation of the actual in-place strength found in structures. Molded cylinders are most often used as a measure of quality assurance and have long been the industry standard for determining the quality of the concrete delivered to the job site. If cylinder tests do not indicate strengths that satisfy the project requirements, then it is common practice to do in-place testing on the concrete in question.

Throughout the years, many different types of in-place tests have been developed. Some test methods induce more concrete damage than others. For many years, core testing has been the primary testing method to determine in-place strength. This process involves drilling a core from the concrete in question and testing it in compression. This is done by using an electric or gas powered core rig to cut out a cylindrical specimen, which is then trimmed to the appropriate length, capped, and tested in compression. There are many variables to consider when taking cores from concrete. ACI 214 (2010) discusses how apparent strength can be affected by core diameter, length-to-diameter ratio, moisture conditioning, damage, and steel reinforcement. The industry standard is to obtain cores and trim them to a length-to-diameter ratio of 2.0. If this is not possible, cores obtained from the in-place concrete are permitted to have a length-to-diameter ratio less than 2.0 but not less than 1.0. Normal-strength specimens with a length-to-diameter ratio between 1.0 and 1.75 must have a correction factor applied to test results as specified in AASHTO T24 (2009).

Currently, AASHTO T24 (2009) recommends a minimum core diameter of 3.70 inches be used. This is not always possible as steel reinforcement is often present in the in-place concrete and cannot be avoided unless a smaller core diameter is used. Similarly, if it is not possible to obtain a core with a length-to-diameter ratio of 1.0, then a smaller core diameter is also permitted. There have been studies conducted which suggest that there is a difference in apparent strength as core diameter decreases.

Moisture conditioning also has a significant effect on the apparent strength of drilled cores. Historically, there were many ways in which cores could be cured after being removed from a concrete specimen. In recent years, AASHTO T24 has specified that cores removed from a concrete member be sealed in plastic bags for at least five days after last coming in contact with moisture from either drilling or sawing.

Another common test method to evaluate in-place strength is the cast-in-place cylinder. This test method involves placing cylindrical molds, most commonly with length-to-diameter ratios of 2.0, within a support system to hold them in place while casting. While the concrete is being cast, the molds are filled and vibrated externally by touching a vibrator to the outside of the support system. The purpose of cast-in-place cylinders is to obtain a sample from the in-place concrete which matches the temperature and moisture history of the specimen while not inflicting damage upon the sample by cutting through the concrete matrix as with core testing.

A third common test method used to evaluate the in-place strength of concrete is the pullout test. This method requires that inserts be cast into the concrete at the time of placement. At the desired time, a jack is used to pull out the insert which was cast into the concrete. The jack system then returns a pullout force, which can be converted to an equivalent cylinder compressive strength using a calibration chart supplied by the pullout insert manufacturer.

1.2 Research Significance

Currently, the Alabama Department of Transportation (ALDOT) uses a strength-based concrete pay scale. If concrete cylinders exceed the specified minimum strength for the project, the party responsible for placing the concrete gets paid the full amount specified in the contract. If cylinders do not meet the required strength, cores are taken. This pay scale increases linearly from 50% of the specified pay if the average core strength is 85% of the required minimum strength to 100% of the specified pay if the average core strength equals or exceeds the required minimum strength. ACI 318 (2011) states that concrete shall be deemed structurally adequate if the average of three cores is equal to at least 85% of the specified minimum strength with no single value being less than 75% of the specified minimum strength. One of the primary goals of the research conducted was to assess the relationship between core strength and cylinder strength in order to evaluate the current pay scale being used by ALDOT.

The research conducted for this project also examined the effect of strength gain over time. It is widely known that concrete increases in strength as it ages. This rate of strength development is greatly affected by a number of factors, including placement temperature and type of cementing material used in casting. Supplemental cementing materials (SCMs) are commonly used in combination with portland cement. The reasons for partial replacement of cement with SCMs are numerous and include increased performance, greater sustainability, and cost savings. The use of SCMs can also have a significant impact on strength gain over time. This research looks to quantify the difference in strength gain between cementing materials through statistical analysis.

The strength of concrete can be greatly influenced by a number of factors. One factor which can adversely affect the compressive strength of concrete is microcracking. This phenomenon occurs most commonly in the interfacial transition zone (ITZ). The ITZ is the area most commonly found surrounding the coarse aggregate found in concrete. This thin layer develops as bleed water collects on the underside of coarse aggregate. This area is particularly susceptible to microcracking as it has an increased water-cement ratio as well as higher a permeability than the surrounding concrete matrix. This research aimed to conclude if restraint was a significant factor regarding compressive strength due to the development of microcracking in highly restrained areas within a concrete specimen.

1.3 Project Objectives

The following were the objectives for the project:

1.) Determine if there is a significant difference between core strength and molded cylinder strength and if there is, quantify this difference and provide recommendations.

2.) Develop recommendations for ALDOT specification regarding in-place core testing.

3.) Determine the effect of supplemental cementing materials on strength gain over time.

4.) Determine the effect of restraint on microcracking.

5.) Determine the effect of coarse aggregate type on measured in-place strength as it relates to molded, moist-cured cylinder strength.

6.) Provide guidance on how to deal with cores containing steel reinforcement.

1.4 Report Scope

Chapter 2 of this document gives historical insight into the testing being conducted and draws recommendations and expectations from a review of published literature. Chapter 3 presents a summary of the experimental procedure used to accomplish the project objectives. This includes the description of test methods, experimental setup, testing techniques, and explanation of notation. Chapter 4 summarizes the results of the experimental work including statistical analysis of data and overall trends. Chapter 5 summarizes the work which was done, draws conclusions from the results presented in Chapter 4, and provides recommendations for ALDOT specifications as well as future research.

Chapter 2

Literature Review

2.1 Introduction

For many years, engineers have tried to conclude how to successfully compare test data collected from in-place strength testing to specified design strengths to determine if the strength of the in-place concrete is satisfactory. Different tests can produce different apparent strengths. This is due to both the variability of the in-place concrete as well as the test method and its variability. There is also a certain amount of variability that happens within in-place testing due to the operator of the test. Though it may be easy to obtain a value from a test method which predicts in-place strength, the real challenge of in-place strength testing is being able to translate these values into meaningful information which can be compared with the specified design strength in order to determine the adequacy of the concrete for the given application. Contractors who perform work for state departments of transportation also rely on these tests to accurately estimate the in-place compressive strength of concrete. This is because the amount for which they are compensated for their work is highly dependent on the strength level of the in-place concrete and if it meets the specified design strength for the given project.

2.2 Factors Affecting In-Place Strength

When samples are taken from the field, special care must be taken when interpreting the collected data in order to make valid conclusions. Many factors must be taken into account when conducting in-place strength testing. The manner in which testing is carried out will have a substantial effect on the apparent strength of the in-place concrete.

2.2.1 Age

Age is one of the most prominent factors that affects the strength of concrete. When hydraulic cement is hydrated, it gains strength. The primary gain in strength happens within the first 28 days after hydration. Mehta and Monteiro (2014) show that the rate of strength gain relative to age is dependent on many factors such as early-age temperature, cement type, and moisture conditions. Molded cylinders that are cured in accordance with AASHTO T23 (2009) are exposed to conditions that supply a constant supply of moisture to the specimens, ensuring that the hydration process continues until the specimens are tested. As stated by Price (1951), "where moisture is available for curing or where moisture contained in the concrete is not lost through drying, the strength development of the concrete will continue for a number of years."

Some correction factors have been developed that can take a strength obtained at any age and convert it to a 28-day strength. Yazdani and McKinnie (2004) conducted research for the Florida Department of Transportation (FDOT) in order to obtain strength correction factors based on concrete age using molded 6×12 in. cylinders that were made in accordance with ASTM C192-02 specifications. The 6×12 in. cylinders were cured by placing them in water tanks that were maintained at 73±3°F. The concretes which were tested in this study which contained SCMs were made with 20% Class F fly ash and 50% Grade 100 slag cement. Yazdani and McKinnie (2004) found that the strength relationship varied depending on the type of cementing materials which were used. The relationship between 28-day core strength and average core strength at a specific age as proposed by Yazdani and McKinnie (2004) can be seen in Equation 2-1. The strength conversion equations from Yazdani and McKinnie (2004) can be seen in Equation 2-2 through Equation 2-10. Note that different equations were developed for different cement types as well as different SCM types.

$$f'_c(28) = \frac{f_{core}*100}{F} \quad \text{Equation 2-1}$$

Where F is defined as:

$$F = 4.4 + 39.1(lnx) - 3.1(lnx)^2 \quad \text{(Type I Cement)} \quad \text{Equation 2-2}$$

$$F = -17.8 + 46.3(lnx) - 3.3(lnx)^2 \quad \text{(Type II Cement)} \quad \text{Equation 2-3}$$

$$F = 48.5 + 19.4(lnx) - 1.4(lnx)^2 \quad \text{(Type III Cement)} \quad \text{Equation 2-4}$$

Where: x = number of days since the concrete was placed

ln = natural log

Concretes with fly ash

Cement Type I: $f'c(28) = 0.490 * Exp(\frac{8.31}{t})^{0.276} * f'c(t)$ Equation 2-5

Cement Type II: $f'c(28) = 0.730 * Exp\left(\frac{2.89}{t}\right)^{0.514} * f'c(t)$ Equation 2-6

Cement Type III: $f'c(28) = 0.483 * Exp\left(\frac{5.38}{t}\right)^{0.191} * f'c(t)$ Equation 2-7

Concrete with slag cement

Cement Type I: $f'c(28) = 0.794 * Exp\left(\frac{7.06}{t}\right)^{1.06} * f'c(t)$ Equation 2-8

Cement Type II: $f'c(28) = 0.730 * Exp\left(\frac{6.02}{t}\right)^{0.747} * f'c(t)$ Equation 2-9

Cement Type III: $f'c(28) = 0.826 * Exp\left(\frac{2.36}{t}\right)^{0.672} * f'c(t)$ Equation 2-10

ACI 209.2R (2008) outlines a procedure for correcting compressive strength at any age back to an equivalent 28-day strength. This is done by using Equation 2-11. The value of a/β is defined as the time it takes for the concrete to reach half of its ultimate strength. Values for these constants can vary from 0.05 to 9.25 for a and 0.67 to 0.98 for β. The recommended values of the empirical constants for Equation 2-11 can be seen in Table 2-1.

$$f'_c(t) = f'_c(28) \times \left(\frac{t}{a+\beta \times t}\right) \quad \text{Equation 2-11}$$

Where:

t = time since casting (days)

a = empirical constant from Table 2-1 (days)

β = empirical constant from Table 2-1 (unitless)

Table 2-1: Constants for ACI 209 Age Correction Equation

Cement Type	**ACI 209 Empirical Constants for Equation 4-1**	
	***a* (days)**	***β***
Type I	4	0.85
Type III	2.3	0.92

Bartlett and MacGregor (1996) also did a statistical analysis of a number of data points collected over a number of years in Alberta, Canada. Their analysis showed that on average, in-place strength increased approximately 25% over the time period from 28 days to one year.

2.2.2 Supplementary Cementing Materials

Supplementary cementing materials (SCMs) are commonly used in today's concrete industry. They typically help decrease the overall cost of the mixture by reducing the amount of portland cement that is needed. SCMs are typically by-products of other industries and if they were not used in the concrete industry, they would be landfilled. Therefore the use of these materials also provides a more sustainable option. The use of these materials can also greatly improve the fresh and hardened properties of the concrete.

2.2.2.1 Fly Ash

Mehta and Monteiro (2014) show that the partial replacement of portland cement with fly ash can greatly improve both the fresh and hardened properties of concrete. Fly ash is typically produced from the burning of coal in electrical power plants. Joshi and Lohtia (1997) state that fly ash is comprised of fine, spherical, glassy particles which are collected in dust collection systems located within fossil fuel power plants. Fly ash particles are oftentimes finer than portland cement particles. Bijen (1996) concluded that the pore size distribution in concretes which contain fly ash is also substantially finer than concretes containing only portland cement.

There are two main classes of fly ash which are used in the United States: Class C fly ash and Class F fly ash. Class C fly ash has a higher calcium oxide content and therefore has both cementing and pozzolanic characteristics. Class F fly ash typically contains lower amounts of calcium oxide and therefore acts primarily as a pozzolan. Naik, et al. (2003) noted that the rate of early-strength gain in concretes containing Class C fly ash was higher than concretes containing Class F fly ash, which is mainly due to the greater reactivity of Class C ash.

In general, Xu (1997) says that concrete containing fly ash typically has a lower 28-day strength but higher long-term strength as compared to concretes using portland cement as the only cementitious material. Naik et al. (2003) also concluded that when moist cured, “The long-term pozzolanic strength contribution of Class F fly ash was somewhat greater compared to Class C fly ash. Consequently, long-term compressive strengths of Class F fly ash concrete mixtures were better than that for Class C fly ash concrete mixtures.”

2.2.2.2 Slag Cement

Slag cement, also called ground-granulated blast-furnace slag, can also be used in partial replacement of portland cement when batching concrete. Mehta and Monteiro (2014) conclude that one significant advantage of using slag cement is that it decreases the amount of heat generated when concrete is batched. This characteristic is ideal when placing mass concrete. Because of this, the use of slag cement also increases set time, which can also help when placing concrete in hot weather conditions. Bijen (1996) also states that one of the biggest advantages of using slag cement as partial replacement of portland cement is that it decreases the rate of penetration of chloride ions into the microstructure of the concrete as well as increasing the critical chloride concentration concerning chloride-induced corrosion. Oner and Akyuz (2007) concluded that the use of slag increased compressive strengths up to an optimal replacement percentage, which was determined to be 55-59%. From their experimental program, Oner and Akyuz (2007) also concluded that the use of slag in concrete increases workability but reduces the early-age strength of the concrete, but with proper curing, the strength increase was greater in concretes which contained slag cement because the pozzolanic reaction which converts calcium hydroxide into calcium-silicate-hydrate occurs slowly.

2.2.3 Moisture and Curing Conditions

The amount of moisture available to the concrete during the curing process has a significant effect on concrete strength and durability. When concrete is supplied with adequate moisture during curing, it allows the cement to hydrate continuously which produces higher strengths. Results from a study done by Popovics (1986) on molded cylinders can be seen in Figure 2-1.

Bartlett and MacGregor (1994a) did a study on the effect of moisture conditioning of cores after drilling. Their results from their study can be seen in the regression plot in Figure 2-2.

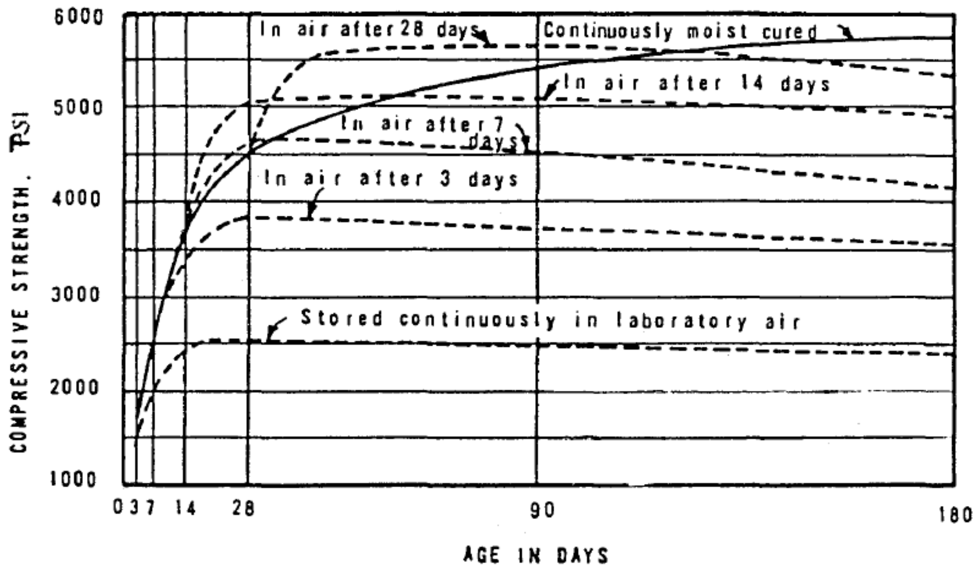

Figure 2-1: Compressive Strength vs. Age for Different Curing Conditions (Popovics, 1986)

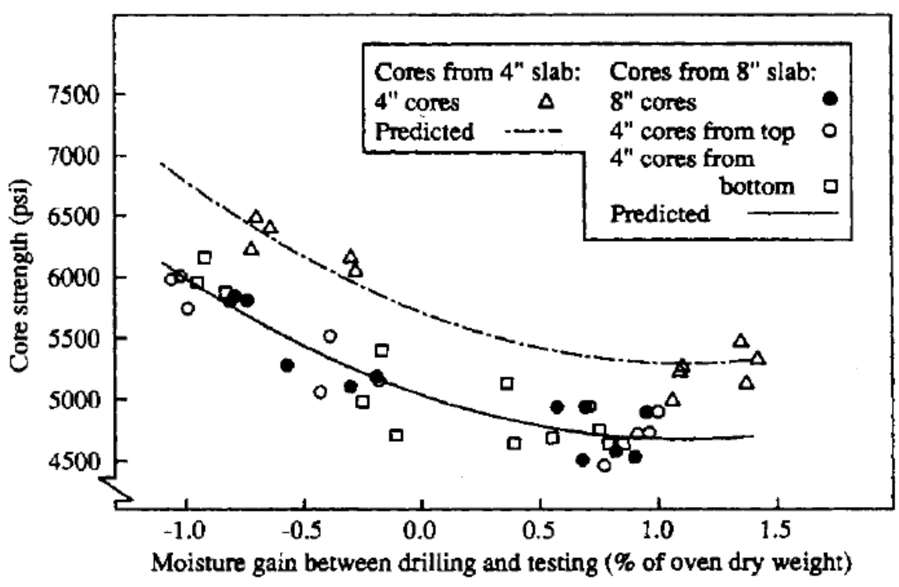

Figure 2-2: Regression Plot of Core Strength vs. Moisture Gain

(Bartlett and MacGregor, 1994a)

From Figure 2-1, it can be seen that continuously moist-cured cylinders perform the best in terms of long-term strength gain. It should also be noticed that when the cylinders were taken out of their moist-cured state and were kept in laboratory air, the overall strength of the cylinders decreased over this time period.

From Figure 2-2, it can be seen that Bartlett and MacGregor (1994a) concluded that the compressive strength of a core specimen is decreased when its moisture content is uniformly increased throughout its volume after it has been cored and, conversely, that the compressive strength of a core specimen is increased when its moisture content is uniformly decreased throughout its volume. Bartlett and MacGregor (1994a) explain by saying that when cores are soaked in water, the surface of the core swells. This swelling at the surface is restrained by the interior of the core, which does not experience any moisture gain. This in turn causes residual stresses to form and lowers the overall compressive strength of the core. Conversely, when cores are left to dry, this causes shrinkage to occur on the surface of the core which causes its overall compressive strength to increase. Because of this, Bartlett and MacGregor (1994a) concluded that the most accurate estimate of in-place strength will be generated from a core specimen which contains no moisture gradient. In an attempt to eliminate a moisture gradient as much as possible, AASHTO T 24 (2009) recommends that the surface moisture of cores be wiped off and left to dry until all surface moisture has evaporated but no longer than one hour. After this, cores should be sealed in plastic bags to avoid moisture loss and therefore not create a moisture gradient within the core.

2.2.4 Coarse Aggregate Size and Type

Aggregate type can play an important role when assessing the strength of in-place concrete. One of the biggest reasons why aggregate size and type affect the compressive strength of a concrete specimen so much is because of the relationship between coarse aggregate and the interfacial transition zone (ITZ). Ollivier, Maso, and Bourdette (1995) describe the ITZ as a water-cement ratio gradient which develops around coarse aggregate which results in a different microstructure of the hydrated cement paste which surrounds the coarse aggregate. Mehta and Monteiro (2014) explain that this happens because a film of water forms around the coarse aggregate particles which in turn causes an increase in the water-cement ratio around the aggregate. Due to the increased amount of water gathers around the surface of the aggregate, the ettringite and calcium hydroxide particles which form are larger and therefore form a layer around the aggregate which is weaker and more permeable. Mehta and Monteiro (2014) also conclude that the larger the coarse aggregate size, the higher the water-cement ratio in the ITZ will be, leading to a weaker and more permeable concrete. Arioz et al. (2007) showed that as the maximum aggregate size increased for cores with small diameters, the strength of the core decreased, but also noted that as core diameter increased, this effect was lessened. This means that larger specimens are impacted less by the size of the aggregate contained within them.

2.2.5 Temperature Conditions

Temperature conditions have a significant impact on the apparent strength of concrete. Mehta and Monteiro (2014) say that hot weather concreting increases slump loss, increases plastic-shrinkage cracking, and decreases the set time of freshly placed concrete. They go on to say that concrete that is placed under hot weather conditions has very rapid strength gain and will have greater 28-day strengths but lower long-term strengths than concrete which is cast at room temperature.

2.3 Strength Test Methods

In the concrete industry, there have been many methods devised to evaluate the in-place compressive strength of concrete. It is very important that these testing methods produce reliable and accurate results. In this section, the following test methods are discussed: molded cylinders, cores, cast-in-place cylinders, and pullout tests.

2.3.1 Molded Cylinders

For many years, molded cylinders have been the industry standard for measuring concrete strength. Though not a good indicator of in-place strength, molded cylinders are used to measure the consistency and quality of the concrete batch that was delivered to the site. AASHTO T23 (2009) states that "the results of this test method are used as a basis for quality control of concrete proportioning, mixing, and placing operation; determination of compliance with specifications; control for evaluating effectiveness of admixtures; and similar uses." In recent years, there has been a push to switch to 4x8 in. molded cylinders, especially for high-strength concretes. Day and Haque (1993) propose that this switch would pose numerous advantages, such as easier handling during transportation, smaller required storage spaces, lower required capacity of testing machines, and the reduced costs for molds, capping materials, and concrete.

2.3.1.1 Summary of AASHTO T23: The Making and Curing of Concrete Test Specimens in the Field

AASHTO T23 (2009), outlines the proper way to produce molded cylinders on site for compression testing. Molds which are used to form the cylinders must be non-absorbent, watertight, and must be able to retain their shape after being filled with concrete. The molds used for making the molded cylinders are also required to have a height twice that of their diameter as well as a diameter at least three time greater than the nominal maximum aggregate size (NMAS). AASHTO T23 (2009) also specifies that the rod used for consolidation of the concrete must have rounded ends, be smooth, straight, and conform to the diameter specifications listed in Table 2-2. AASHTO T23 (2009) also specifies that a rubber or rawhide mallet weighing 1.25 ± 0.50 lb shall be used to tap the sides after rodding.

Table 2-2: Required Rod Diameter for the Consolidation of Cyliders (AASHTO T23 2009)

Diameter of Cylinder (in.)	Rod Dimensions	
	Diameter (in.)	Length of Rod (in.)
< 6	3/8	12
6	5/8	20
9	5/8	26

2.3.1.2 Strength Acceptance of Molded Cylinders

Molded cylinders are often used as a method of quality control. In order for concrete to be accepted, ACI 318 (2011) states that the following requirements must be met:

1. The average of three consecutive tests ≥ f'_c
2. For $f'_c \leq 5000$ psi: No result more than 500 psi below f'_c

   For $f'_c \geq 5000$ psi: No result more than $0.1 \times f'_c$ below f'_c

2.3.1.3 Variability

AASHTO T22 (2009) gives expected coefficients of variation for molded cylinders which can be seen in Table 2-3. These coefficients of variation are for cylinders made under both laboratory and field conditions and tested at the same age by the same laboratory. These coefficients are valid for

6×12 in. cylinders with compressive strengths between 2,000 and 8,000 psi and 4×8 in. cylinders with compressive strengths between 2,500 and 4,700 psi.

Table 2-3: Coefficients of Variation for Molded Cylinders (AASHTO T22 2009)

Specimen Type	Coefficient of Variation	Acceptable Range of Individual Cylinder Strengths	
		2 Cylinders	3 Cylinders
6 × 12 in. Cylinder - Laboratory Conditions	2.4 %	6.6 %	7.8 %
6 × 12 in. Cylinder - Field Conditions	2.9 %	8.0 %	9.5 %
4 × 8 in. Cylinder - Laboratory Conditions	3.2 %	9.0 %	10.6 %

When concrete is cast, three 6×12 in. cylinders are often made for quality assurance which therefore implies that the range of the compressive strengths of these cylinders under the conditions expressed by AASHTO T22 (2009) should not exceed 9.5 %.

2.3.2 Cores

When moist cured, molded quality assurance cylinders are tested in compression and the resulting strength does not exceed the compressive design strength (f'_c) set forth by the design engineer, then in-place strength testing must be done on the in-place concrete to determine if it has adequate strength. Neville (2001) states that when concrete cylinders break low, it can be caused by a number of reasons including inadequate strength, poor consolidation, incurring damage during transit, freeze-thaw damage, improper curing, and improper testing methods. When quality assurance cylinders have an average strength below the specified compressive design strength (f'_c), core testing is most often used to assess the strength of the in-place concrete.

2.3.2.1 Summary of AASHTO T24: Standard Method of Test for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

When taking cores, care must be taken to ensure that as little damage as possible is inflicted upon the core, both while drilling and while transporting the core back to the laboratory for testing. When drilling, it is important that the core rig is securely fastened to the surface from which the core is being taken to ensure that there is as little core barrel wobble as possible. Also, specimens should be secured during transportation so that they do not incur any damage to impact. AASHTO T24 (2009) specifies that a minimum core diameter of 3.75 in. should be used when retrieving cores to evaluate in-place strength. After cores have been drilled, the excess surface water should be wiped off and the surface of the core should be allowed to dry, but should not be exposed longer than one hour after drilling. Cores should then be placed in sealed plastic bags to prevent moisture loss and to ensure that no temperature gradients develop. Cores should be kept in the sealed plastic bags at all times except during trimming and end preparation. In order to be tested, the ends of the core must not have any projections greater than 0.2 in. above the end surfaces and shall not depart from perpendicularity to the longitudinal axis by a slope of more than 1 : 8d where d is the average core diameter in inches. If water is used during the trimming or grinding of the core ends during trimming, these operations should be done as soon as possible after the core has been removed from the in-place concrete. After the end preparation has been completed, the core should be wiped of all excess water and allowed to let all surface water evaporate, but not be exposed for more than one hour.

AASHTO T24 (2009) also states that the length-to-diameter ratio of the obtained core should be between 1.9 and 2.1. If the length-to-diameter ratio is greater than 2.1, it must be trimmed in order to meet the specification. If a core has a length-to-diameter ratio less than 1.75, correction factors must be applied to correct its apparent strength. Also, a core's height must be at least 95% of its diameter before capping and at least greater than or equal to its diameter after capping. Once cores are exposed to wetting due to drilling or trimming, they must be bagged in sealed plastic bags for at least five days to ensure that no moisture gradients are present in the core specimen. Once the five-day period has passed, the ends of the core must be either trimmed or ground to the required planeness or be capped in accordance with AASHTO T231 (2009). If the trimming or grinding involves exposure to moisture, this process should occur before this five-day period.

The initial length of the drilled core should be measured and recorded to the nearest 0.2 inches. If bonded caps are applied to the specimens, the length of the specimens should be recorded both before and after capping to the nearest 0.1 inch. The length of the core which was taken after end preparation should be used to calculate the length-to-diameter ratio of the core. The diameter of the core should also be measured and recorded to the nearest 0.01 inch. This is done by taking at least 2 measurements at the mid-height of the core at right angles to one another. Once these data are recorded, the cores are tested in accordance to AASHTO T22 (2009). Cores must be tested within seven days of being drilled.

2.3.2.2 Factors Affecting Apparent Strength of Cores

Before cores are taken, parties involved in the design and construction of the concrete structure must agree on certain details. There are many factors which have an effect on the apparent strength of the cores obtained from the in-place structure. Studies have also been conducted to determine the effects of different core diameters, length-to-diameter ratios, the amount of damage imparted on a core, core moisture conditioning, effect of reinforcement, and direction of coring relative to casting direction.

Bartlett and MacGregor (1995) proposed that the strength of a core should be converted into an equivalent in-place strength using Equation 2-12, where $f_{c,is}$ is the equivalent in-place concrete strength, $F_{l/d}$ is the strength correction factor for length-to-diameter ratio, F_{dia} is the correction factor for core diameter, F_r is the correction factor for cores containing reinforcing bars at right angles to the central axis of the core, F_{mc} is the correction factor for moisture conditions, F_d is the correction factor for core damage, and f_c is the measured strength of the core. Bartlett and MacGregor (1995) also provide a table which shows how these values are calculated. This table can be seen in Table 2-4. The factors which are obtained from Table 2-4 are then substituted into Equation 2-12 to calculate the equivalent core strength.

$$f_{c,is} = F_{l/d} F_{dia} F_r F_{mc} F_d f_c \quad \textbf{Equation 2-12}$$

Table 2-4: Core Strength Correction Factors (ACI 214 2010)

	Factor	Mean value	Coefficient of variation V, %
$F_{\ell/d}$: ℓ/d ratio†	Standard treatment‡:	$1-\{0.130-\alpha f_{core}\}\left(2-\frac{\ell}{d}\right)^2$	$2.5\left(2-\frac{\ell}{d}\right)^2$
	Soaked 48 hours in water:	$1-\{0.117-\alpha f_{core}\}\left(2-\frac{\ell}{d}\right)^2$	$2.5\left(2-\frac{\ell}{d}\right)^2$
	Dried§:	$1-\{0.144-\alpha f_{core}\}\left(2-\frac{\ell}{d}\right)^2$	$2.5\left(2-\frac{\ell}{d}\right)^2$
F_{dia}: core diameter	2 in. (50 mm)	1.06	11.8
	4 in. (100 mm)	1.00	0.0
	6 in. (150 mm)	0.98	1.8
F_{mc}: core moisture content	Standard treatment‡:	1.00	2.5
	Soaked 48 hours in water:	1.09	2.5
	Dried§:	0.96	2.5
F_d: damage due to drilling		1.06	2.5

To obtain equivalent in-place concrete strength, multiply the measured core strength by appropriate factor(s) in accordance with Equation 2-11.
†Constant α equals 3(10–6) 1/psi for *fcore* in psi, or 4.3(10–4) 1/MPa for *fcore* in MPa.
‡Standard treatment specified in ASTM C42/C42M.
§Dried in air at 60 to 70°F (16 to 21°C) and relative humidity less than 60% for 7 days.

2.3.2.2.1 Length-to-Diameter Ratio

Much research has been done on the effect of length-to-diameter ratio on core testing. Bartlett and MacGregor (1994d) state that "short specimens fail at greater loads because the steel loading platens of the testing machine restrain lateral expansion throughout the specimen more effectively." Therefore, the smaller the length-to-diameter ratio, the larger the apparent strength of the core will be. When assessing the in-place strength of concrete, AASHTO T24 (2009) defines correction factors that must be applied to cores which have length-to-diameter ratios from 1.0 to 1.75 which can be seen below in Table 2-5. Arioz et al. (2007) concluded that the effect of the length-to-diameter ratio was more significant as the diameter of the specimen decreased.

Table 2-5: Recommended Correction Factors for Different L/D Ratios For Concrete Strengths Less Than 6000 psi (AASHTO T24 2009)

Core L/D	AASHTO T24 Strength Correction Factor
1.75	0.98
1.50	0.96
1.25	0.93
1.00	0.87

AASHTO T24 (2009) does not list recommended values for length-to-diameter strength correction factors for concretes with strengths higher than 6000 psi. Similarly, AASHTO T24 (2009) notes that for strengths above 10,000 psi that correction factors may be higher that what is listed in Table 2-5, and that these factors should be applied to high strength concretes with caution. AASHTO T24 (2009) makes no recommendation about what should be done for strength correction factors for concrete with compressive strengths between 6,000 and 10,000 psi. Similarly, Bartlett and MacGregor (1994d) say that there is some indication that as concrete strength increases, the strength correction factors for length-to-diameter ratio begin to increase, which implies that as concrete strength increases, length-to-diameter ratio has less of an impact on apparent strength.

2.3.2.2.2 Core Diameter

There are conflicting opinions when it comes to the effect of diameter on core strength. For cores with the same length-to-diameter ratio, Meininger (1968) found that the core diameter does not have an effect on the core's apparent strength in cores with length-to-diameter ratios of 2.0. Bartlett and MacGregor (1994c) noticed that the strength of a 2-inch diameter core was approximately 94% of a 4-inch diameter core and 92% of a 6-inch diameter core. This trend can be observed in Figure 2-3.

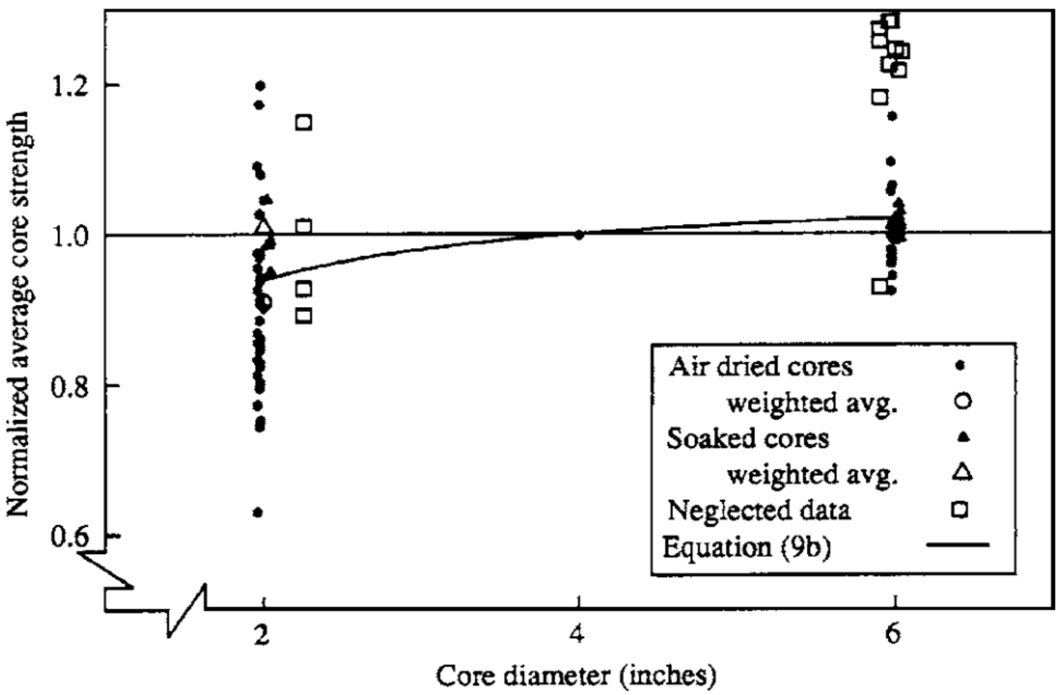

Figure 2-3: Normalized Average Core Strength versus Core Diameter

(Barlett and MacGregor 1994c)

From this data, it is easy to see that Barlett and MacGregor (1994c) found that the diameter of a cored specimen does have an impact on the apparent strength of the core. The data in Figure 2-3 has been normalized so that the standard core diameter is 4 inches.

Bartlett and MacGregor (1994c) also concluded that the variability was much larger in specimens with smaller diameters. They suggest that cores with smaller dimensions are much more susceptible to being impacted by through-thickness variation of the in-place concrete, especially in slabs. Arioz et al. (2007) concluded that as core diameter decreased, strength decreased as well.

ACI 214.4R (2010) gives recommended values for correction factors for cores based on their diameters which can be seen below in Table 2-6.

Table 2-6: Suggested Strength Correction Factors for Different Core Diameters (ACI 241.4R 2010)

Diameter (in.)	Strength Correction Factors
2	1.06
4	1.00
6	0.98

Bartlett and MacGregor (1994c) also concluded that the effect of core damage increases as the size of the specimen decreases. This is especially important with respect to the effect of core diameter on apparent strength. This implies that the smaller the diameter of the specimen, the greater effect that damage will have on its apparent strength.

2.3.2.2.3 Core Damage

When obtaining concrete cores, there is inherit damage that the cores are subjected to due to the destructive nature of the drilling process. As can be seen in Table 2-4 from Bartlett and MacGregor (1995), a strength correction factor of 1.06 is to be applied when a core is damaged during drilling. Bartlett and MacGregor (1994c) explain that cores can be damaged due to microcracking, cutting through coarse aggregate, and undulations at the drilled surface, but no clarification is made on what specifically constitutes enough damage for this factor of 1.06 to be applied. Arioz, et al. (2007) found that strength correction factors for core damage decreased in concretes with higher strengths and hypothesized that the reason for this is that high strength cores are subjected to less damage during the coring process.

Aggregate type also has an effect on the amount of damage imparted on the core during drilling. Khoury, Aliabdo and Ghazy (2014) found that cores from concrete containing natural coarse aggregate, such as river gravel, have a more harmful affect on the amount of damage imparted onto the core during drilling than concretes made from other aggregates such as limestone. Khoury, Aliabdo and Ghazy (2014) also concluded that cores taken from higher strength concretes, in general, have less damage imparted on them than cores taken from lower strength concretes.

2.3.2.2.4 Casting Direction

There has been great debate over whether the direction of coring relative to the casting direction has an impact on the apparent strength of the core. The primary reason why there is suspicion that coring direction with respect to casting direction has an effect on the apparent strength of a core is because of the ITZ. Mehta and Monteiro (2014) write that the ITZ is most prominent around the bottom of the coarse aggregate due to bleed water which creates a plane of weakness in one direction. Suprenant (1985) concluded that due to the plane of weakness which is formed around the bottom of the coarse aggregate relative to casting direction, the direction in which the core is drilled is significant. An illustration of this effect can be seen below in Figure 2-4. From Figure 2-4, Suprenant (1985) illustrates the plane of weakness around the bottom of the coarse aggregate. When cores which are drilled parallel to the casting direction are tested, this plane of weakness is perpendicular to the applied test load. However, if a core is drilled perpendicular to the casting direction, this plane of weakness is now parallel to the applied force when the core is tested in compression. Neville (2001) also offers another explanation of why by stating that "whereas drilling downward allows the drill to be fixed and held firmly in position, drilling horizontally almost inevitably permits a slight movement of the drilling barrel." Munday and Dhir (1984) conducted research on coring direction versus casting direction and suggest that cores taken parallel to the casting direction will have strengths approximately 8% greater than cores drilled perpendicularly to the casting direction.

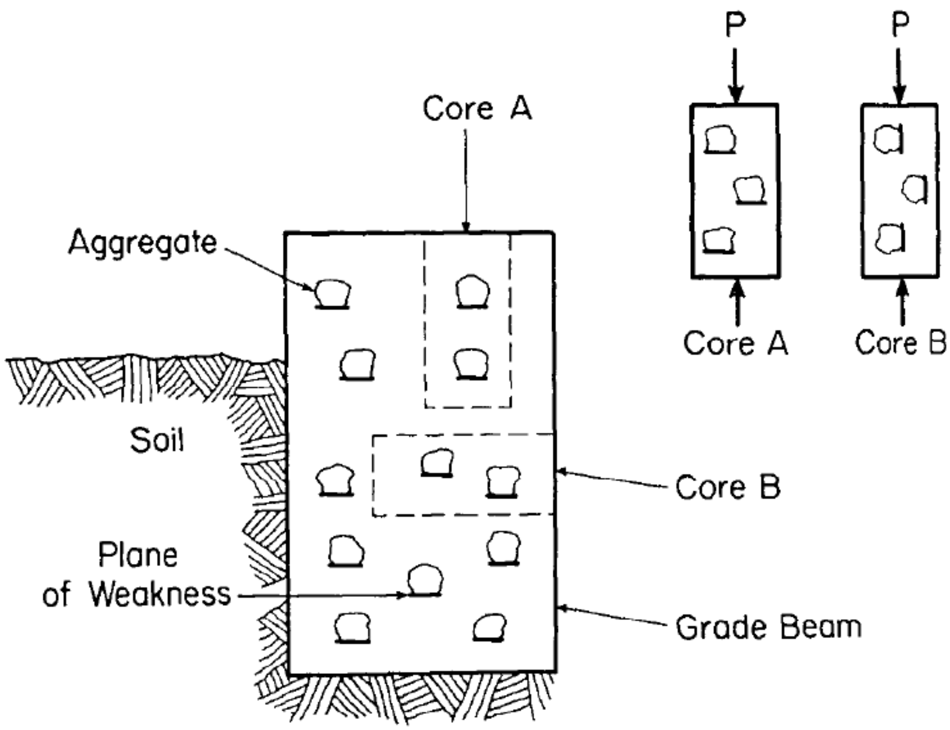

Figure 2-4: Effect of Coring Direction Relative to Casting Direction (Suprenant 1985)

There are other studies though, such as the one conducted by Bloem (1965), which have concluded that coring direction relative to casting direction does not produce statistically significant differences in apparent strengths. Bartlett and MacGregor (1994b) also conclude that there was not a significant difference in their data between cores which were drilled parallel versus perpendicular to the casting direction.

2.3.2.2.5 Presence of Embedded Steel Reinforcement in Core

Sometimes in members with a congested steel reinforcement layout, it is difficult to recover a core without hitting steel reinforcement. Because of this, guidance must be given on what to do if a core contains steel reinforcement within it. AASHTO T24 (2009) recommends that core specimens containing embedded reinforcement not be used to determine compressive, splitting tensile, or flexural strength.

Bartlett and MacGregor (1995) recommended the correction factors shown in Table 2-7 to correct the compressive strength of a core containing steel reinforcement. It should be noted that the correction factors presented by Bartlett and Macgregor (1995) are for steel reinforcement which runs perpendicular to the axis of drilling. The strength correction factors shown in Table 2-7 would be used in Equation 2-11 to correct a core's compressive strength. No guidance is given on how much of the steel reinforcing bar must be contained within the core for the correction factors to be applied or if bar size has an impact on core compressive strength.

Table 2-7: Strength Correction Factors for Steel Reinforcement Present in Core Sample (Bartlett and MacGregor 1995)

Number of Reinforcing Bars Present in Core	Strength Correction Factor
1	1.08
2	1.13

2.3.2.3 Variability of Core Testing

Bartlett and MacGregor (1994c) found that the variability of cores greatly depended on the through thickness variation within the concrete itself. They also noted that "the variability of measured strengths of small-diameter cores is particularly sensitive to being inflated by the variability of the in situ strength across the dimension of the element being cored." Therefore, if through-thickness variation develops in concrete members, the variability of cores with smaller diameters are likely to be more susceptible to its effects, but if through-thickness variation is not significantly present within a member, then the coefficient of variation will be fairly similar for cores of all diameters. If through-thickness variability is not present, Bartlett and MacGregor (1994c) concluded that the variability of the in situ concrete from members of moderate size from one batch of concrete is approximately 5 %. Arioz et al. (2007) also found that variability also increased as core diameter decreased. ACI 214.4R (2010) gives a table of expected range for core strengths based on the number of specimens collected which can be seen below in Table 2-8.

Table 2-8: Probable Range of Core Strengths Due to Single-Operator Error (ACI 214.4R 2010)

Number of cores	Expected range of core strength as percent of average core strength	Range with 5% chance of being exceeded as percent of average core strength
3	5.4	10.6
4	6.6	11.6
5	7.2	12.4
6	8.1	12.9
7	8.6	13.3
8	9.1	13.7
9	9.5	14.1
10	9.8	14.3

2.3.2.4 Impact of Number of Cores Retrieved

In order to meet the strength acceptability standards of ACI 318 (2011) for the strength acceptance based on core test results, the average of the three cores taken must be greater than 85 % of the specified design strength while also not having a single core with a strength lower than 75 % of the specified design strength. Although it is common that three cores are taken in order to evaluate in-place strength, there are times in which more than three cores are taken. Bartlett and Lawler (2011) point out that an increase in the number of cores would not impact the mean-strength criterion, but it would have an impact on the requirement that no single test result could have a strength lower than 75 % of the speficied design strength. Inherently, if the amount of specimens increases, the liklihood that a specimen with a strength lower than 75 % of the specified design strength would increase as well. If the coefficient of variation, *V*, is known, then the graph seen in Figure 2-5 from Bartlett and Lawler (2011) can be used to determine the acceptable value of *k*, which is defined as the lowest acceptable ratio between a single core strength and specified design strength.

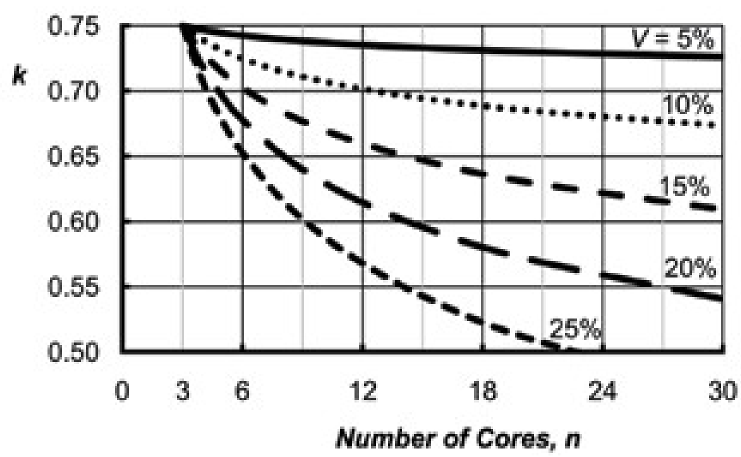

Figure 2-5: *k* versus Number of Cores Given Coefficient of Variation

(Bartlett and Lawler 2011)

From the plot in Figure 2-5, it can be seen that the specified value for *k* given V decreases as the number of cores taken increases. This implies that the change in single core strength criterion should be taken into account when more than three core specimens are retrieved and tested.

Bartlett and Lawler (2011) outline a statistical method which can be used to determine the acceptable value of *k* based on the number of cores extracted from the concrete in question. First, a value of P_1 must be specified. P_1 represents the chance that a core from a set of three cores will have a strength less than $0.75 \times f'_c$. ACI 214 (2010) recommends this value be 10 %. Once this value is established, the value of P_2 can be calculated using Equation 2-13 based on the number of cores which are retrieved. P_2 is defined as the corresponding probability to P_1, but with a sample size of n instead of 3.

$$P_2 = 1 - (1 - P_1)^{3/n} \quad \text{Equation 2-13}$$

Once P_2 is calculated, then the corresponding value of c_1, which is the number of standard deviations below the mean that P_2 occurs, can be found using the standard normal distribution function. An allowable coefficient of variation, V_{ws}, is then chosen. ACI 214 (2010) contains Table 2-9, which can be used to choose an appropriate coefficient of variation. Once this is done, c_1 and V_{ws} are inserted into Equation 2-14 and the appropriate value of k is calculated.

Table 2-9: Coefficient of Variation Due to In-Place Strength Variation Within a Structure

Structure composition		One member	Many members
One batch of concrete		7%	8%
Many batches of concrete	Cast-in-place	12%	13%
	Precast	9%	10%

$$k = 0.85 - c_1 \times V_{ws} \quad \text{Equation 2-14}$$

2.3.3 Cast-In-Place Cylinders

With the cast-in-place cylinder testing method, a cylinder mold is held in place by a support system and filled with concrete as the member is being cast. A detailed setup for a cast-in-place cylinder can be seen below in Figure 2-6. After remaining in the structure for some time, the specimen is removed from the structure, transported to the testing lab, and tested. ACI 228.1R (2003) outlines the benefits of using cast-in-place cylinders and states that the advantages of using this method over coring include no damage being imparted on the specimen due to coring while also matching the thermal history of the specimen to the in-place concrete.

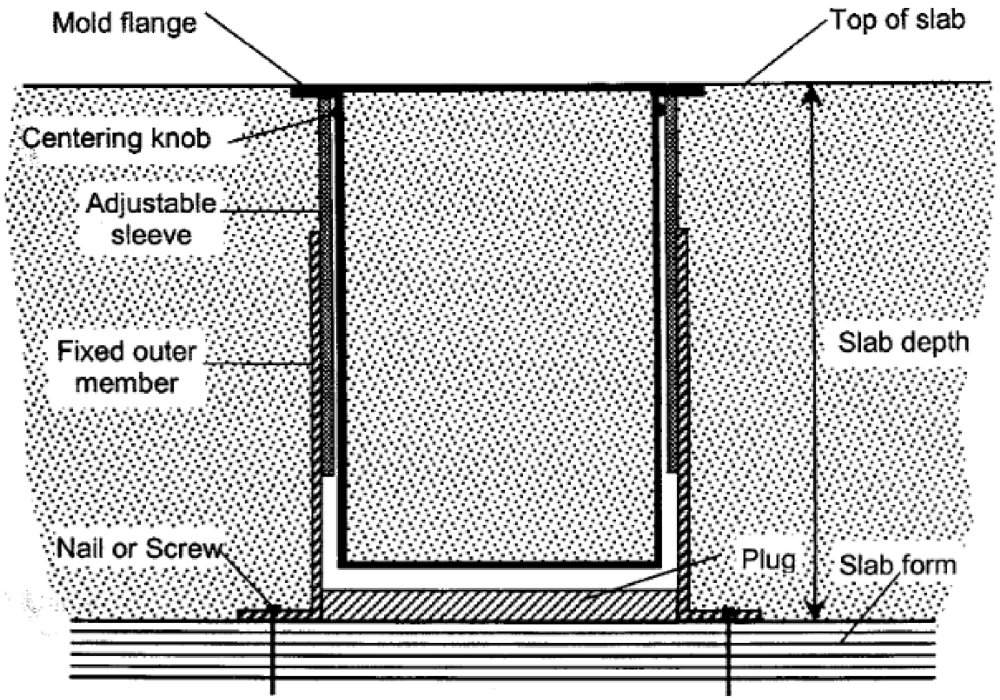

Figure 2-6: Schematic of Cast-In-Place Cylinder Mold Assembly (ASTM C873, 2011)

2.3.3.1 Summary of ASTM C873

Cast-in-place cylinders are regulated by ASTM C873. ASTM C873 (2011) specifies that the diameter of the mold must be at least 3 times the size of the nominal maximum aggregate size of the coarse aggregate gradation used in the concrete while also specifying that the mold must have a length-to-diameter ratio of at least 1.0 as well as a minimum diameter of 4 inches. Molds used to make cast-in-place cylinder specimens must conform to the water leakage specification in ASTM C470 while also being rigid enough to maintain their shape after being filled with concrete. The molds must also have a lip or ledge for the mold to sit on top of the support system as well as to seal the gap between the support system and the mold. The support members used to hold the concrete molds in place must be right circular cylinders which must have a diameter which accomodates the concrete molds while also being rigid enough to resistance deformation. Once any steel reinforcement is put in place, the support system should be secured to the formwork being used by using nails or screws. Upon installation, the top of the molds shall be even with the top of the formwork for the member. When filling the cylinder with concrete, the consolidation methods used for the member should also be used for the cast-in-place cylinders. If internal vibration is used, then the vibrator should be used externally on the specimens, briefly touching the vibrator to the outside of the support member. Internal consolidation should not be used for the specimens except under special instructions. The surface of the specimens should be finished the same as the surrounding concrete. The specimens should be exposed to the same curing conditions as the surrounding concrete while recording the maximum and minimum temperatures of the slab surface during the curing period.

The specimens should remain fully in place until recovered. After the specimens are removed from the slab, they must be kept at ± 10°F of the slab surface temperature at the time of removal until they are tested. Specimens must be transported back to the testing facility within 4 hours of removal from the concrete member. Caution must be used during transportation so that the specimens are not damaged. Also, insulation must be provided to prevent extreme temperature variation as well as moisture loss. Once the specimens have reached the testing facility, molds must be stripped. The average diameter of the specimen must be determined by taking the average of two measurements at mid-height of the specimen perpendicular to one another. If the specimens are to be capped, the length of the specimen should be recorded after capping. Compression testing of the members shall be done according to ASTM C39. The specimens should be testing in the moisture condition in which they were received from the field. Compressive strength should be determined using the specimen cross-sectional area obtained from using the specimen's average diameter. If the specimen has a length-to-diameter ratio less than 1.75, the appropriate correction factor should be applied to obtain an equivalent strength.

2.3.3.2 Variability of Cast-in-Place Cylinders

ASTM C873 (2011) states that the single-operator coefficient of variation for cast-in-place cylinder specimens is 3.5 % for a range of concrete strengths from 1500 to 6000 psi. This means that the results from two tests which were correctly performed should not differ by more than 10.0 % of their average.

2.3.4 Pullout Testing

Carino (1997) states that the idea for the pullout test was established in the Soviet Union. Kierkegaard-Hansen and Bickley (1978) state that the LOK-Test pullout method was first developed in Denmark in the 1960s in order to develop a method of measuring the in-place strength of hardened concrete. Hubler (1982) states that "in operation, the LOK-TEST device, a calibrated screw-actuated hydraulic jack, non-destructively pulls pre-positioned bolts embedded in the concrete." The force which is required to crack the concrete is recorded and then converted to a compressive strength. Bickley (1982) states that by using the pullout test method, "variations in the strength of in-place concrete can be measured and the minimum strength in a placement calculated by the standard statistical methods to high degrees of confidence." Stone, Carino and Reeve (1986) state that nondestructive methods for determining the in-place strength of concrete are becoming increasingly popular due to their simplicity, cost, and time effectiveness. A schematic of a cast-in insert can be seen in Figure 2-7.

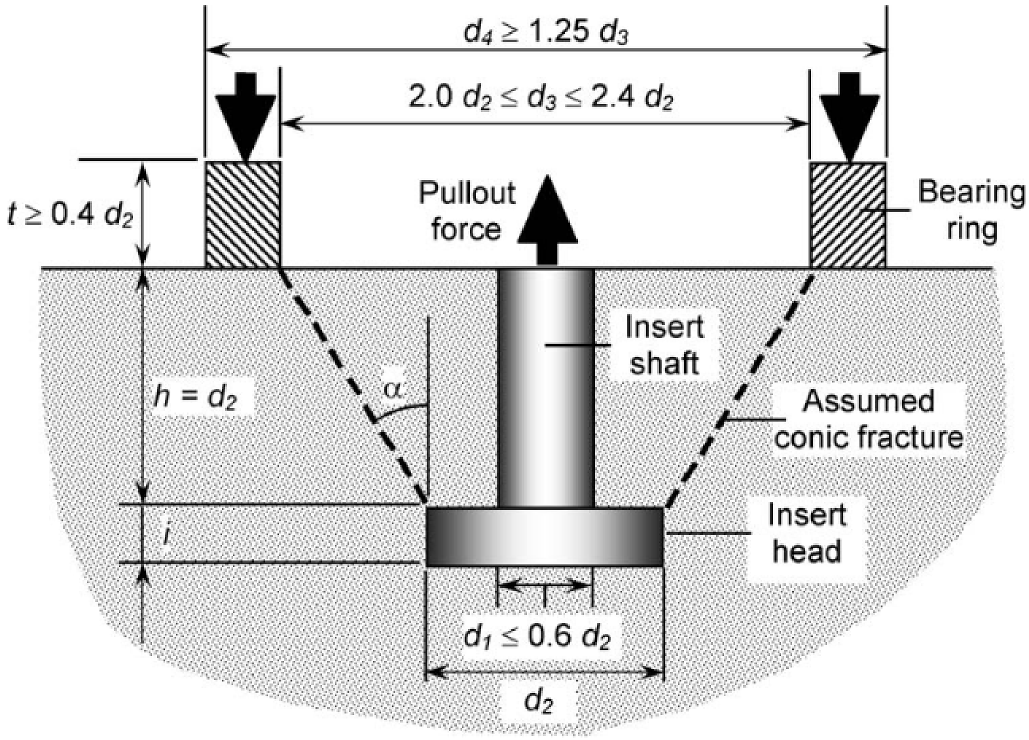

Figure 2-7: Schematic of LOK-Test Pullout Insert (ASTM C900 2007)

2.3.4.1 Summary of ASTM C900

The Standard Test Method for Pullout Strength of Hardened Concrete, ASTM C900 (2007), states that there are three items needed when doing pullout tests: a pullout insert, a loading system, and a load-measuring system. The loading system must be calibrated at least once a year in order to assure that readings collected from the testing are correctly converted to compressive strengths. Bickley (1982) recommends that the relationship between pullout force and compressive strength should be determined for each job site as well as for each type of concrete and aggregate size. Most often times, inserts are cast into fresh concrete and then tested at specified times after the concrete has hardened. Inserts can also be inserted after the concrete has hardened. In the case of cast-in inserts, the length of the stem of the insert must be equal to the diameter of its head. Test locations must be separated by a clear space of at least seven times the diameter of the insert's head. Also, all test locations must have a clear space of at least 3.5 times the diameter of the insert's head away from free edges of the concrete. Whenever pullout tests are used, a minimum of five tests are required. When loading the insert, a load rate of 70 ± 30 kPa/s must be used. Once failure is reached, the failure load is recorded and then converted into an equivalent compressive strength using calibration data.

2.3.4.2 Failure Mechanism

When the pull-out test method was first developed, the exact failure mechanism of the concrete was unknown. Many studies through the years have tried to establish a better understanding of the failure mechanism for the pull-out test method. Carino (1997) states that "the pull-out test subjects the concrete to a non-uniform, three-dimensional state of stress." Carino (1997) also goes on to say that cracking also happens in two circumferential cracking systems: the first being a system which is stable and begins at the head of the insert at approximately 1/3 of the ultimate load which spreads into the surrounding concrete at a large apex angle, and the second being the cracking system which propagates under increasing load and eventually defines the shape of the cone which is extracted. Although there is a consensus on the development cracking systems, the actual failure mechanism is still debated today.

2.3.4.3 Variability

ASTM C900 (2007) states that cast-in inserts which are embedded approximately 1-inch below the concrete surface with a maximum aggregate size of 0.75 inches have a one-operator coefficient of variation of 8 %. Therefore the range of test results should not exceed the values listed in Table

2-10. A list of coeffients of variation from various projects involving the use of pullout tests are summarized in Table 2-10 from ACI 228.1R (2003).

Table 2-10: Acceptable Pull-Out Test Range Based on Number of Tests (ASTM C900 2007)

Number of Tests	Acceptable Range (Percent of Average)
5	31%
7	34%
10	36%

Stone, Carino and Reeve (1986) conducted a test to determine the effect of apex angle and aggregate type on the nature of the relationship between pullout strength and compressive cylinder strength. Three types of aggregate, crushed limestone, river gravel, and lightweight aggregate, were used in the study. It was concluded that the coefficient of variation was much lower for the lightweight aggregate than the two sources of normal-weight aggregate. This can be explained by the different failure mechanism attributed to the particular lightweight aggregates. Since lightweight aggregates most commonly break through the aggregate and not around it, the failure load is governed by the mortar strength. On the other hand, harder aggregates, such as the limestone and river gravel, will cause failure planes which will travel around the coarse aggregate, causing the ultimate pullout force to be based on the amount of aggregate interlock that occurs. Therefore, if a large aggregate is present near a pullout insert, the resulting pullout force will be significantly higher, which would lead to a significantly higher coefficient of variation for a given number of tests. It was also concluded from the study that any apex angle within the 54 to 70 degree range will not have a significant effect on the coefficient of variation.

Table 2-11: Summary of Within-Test Coefficient of Variation of Pullout Test (ACI 228.1R 2003)

Reference	Apex angle, degrees	Embedment depth, in.	Maximum Aggregate Size, in.	Aggregate Type	No. of replicate specimens	Coefficient of variation, %	
						Range	Average
Malhotra and Carette (1980)	67	2	1	Gravel	2	0.9 to 14.3	5.3
Malhotra (1975)	67	2	1/4	Limestone	3	2.3 to 6.3	3.9
Bickley (1982)	62	1	3/8	?	8	3.2 to 5.3	4.1
Khoo (1984)	70	1	3/4	Granite	6	1.9 to 12.3	6.9
Carette and Malhotra (1984)	67	2	3/4	Limestone	4	1.9 to 11.8	7.1
	62	1	3/4	Limestone	10	5.2 to 14.9	8.5
Keiller (1982)	62	1	3/4	Limestone	6	7.4 to 31	14.8
Stone, Carino, and Reeve (1986)	70	1	3/4	Gravel	11	4.6 to 14.4	10.2
	70	1	3/4	Limestone	11	6.3 to 14.6	9.2
	70	1	3/4	Low density	11	1.4 to 8.2	6
	54	1	3/4	Gravel	11	4.3 to 15.9	10
Bocca (1984)	67	1.2	1/2	?	24	2.8 to 6.1	4.3

2.4 Summary of State DOTs Payment Reduction Methods

If cylinder breaks are low for a concrete placement, steps must be taken in order to evaluate the integrity of the in-place concrete. Most of the time, cores are taken from the structure and tested in compression to determine the in-place compressive strength. Based on these results, state DOTs have methods to assess the strength and, if deficient, to reduce the amount that is paid to the contractor for in-place concrete. These methods of price adjustment vary from state to state. The practices of the Alabama Department of Transportation (ALDOT), Tennessee Department of Transportation (TDOT), Florida Department of Transportation (FDOT), and Texas Department of Transportation (TxDOT) were examined and analyzed as part of the review of literature. These states were chosen because their practices for payment correction were explicitly defined within their respective highway construction practice manuals as well as for their location within the southeastern United States.

2.4.1 Alabama Department of Transportation

During the construction of concrete structures, the Alabama Department of Transportation (ALDOT) makes molded cylinders which are tested at 28 days for quality assurance. If the average strength of these cylinders is below f'_c or a single cylinder breaks 500 psi below the design strength for concrete with f'_c less than or equal to 5000 psi or $0.1 \times f'_c$ below the design strength for concrete with f'_c greater than 5000 psi, then concrete cores must be taken. Currently, ALDOT uses the price adjustment equation shown in Equation 2-15. This relationship can be seen in graphical form in Figure 2-8. Under its current practice, the average strength of the cores which have been retrieved from the job site must be equal to or exceed the specified design strength. Although ACI 318 (2011) states that the in-place concrete is structurally adequate if the average of at least 3 cores is greater than 85% of the design load, ALDOT uses a pay scale which pays only 50% of the intended construction cost if the average strength of the cores which have been obtained equal 85% of the design strength after correction factors have been applied.

$$Price\ Adjustment\ (In\ Percent) = 100\ \times\ (1.0 - \left[\frac{f'_c - f_{c,AVG}}{0.30 \times f'_c}\right]) \quad \text{Equation 2-15}$$

Where: f'_c = Required 28-day Compressive Strength

$f'_{c,AVG}$ = Average Compressive Strength of Test Cores

The price adjustment shall be rounded to the nearest tenth of a percent

The price adjustment is valid where: 50% ≥ Price Adjustment < 100%

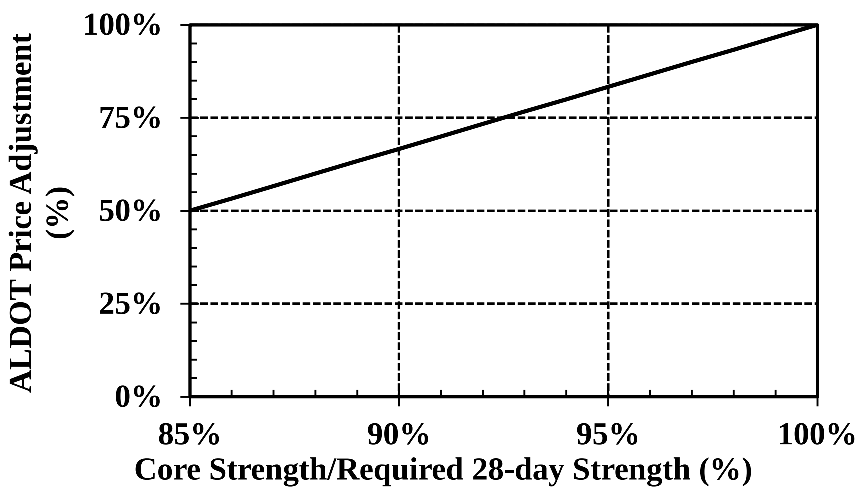

Figure 2-8: Current ALDOT Price Adjustment

(Alabama Department of Transportation 2012)

2.4.2 Tennessee Department of Transportation

The Tennessee Department of Transportation (TDOT) uses specified mixtures which are grouped into classes based on their intended use. These different classes of concrete all have different specified values for minimum 28-day strength, unit weight, water-cement ratio, air content, and slump, which is shown in Table 2-12. TDOT also has a table which defines the acceptable strength of molded concrete cylinders for each class of concrete based on their age, which is shown in Table 2-13. The Tennessee Department of Transportation (2006) states that if cylinder strengths do not meet the specified strength of the concrete, then cores may be taken at the expense of the contractor. If cores are taken, these strengths will become the strength of record and price adjustment will be based off of the strengths obtained from the cored specimens. Two cores are taken and the average strength of these cores becomes the strength of record. Core diameters between 3.75 in. and 4 in. must be used. Also, core lengths should be between 7.5 in. and 8 in. and should have length-to-diameter ratios from 1.9-2.1, but in no case should have a length-to-diameter ratio less than 1.0. Cores which have length-to-diameter ratios less than 1.75 after being capped will have correction factors applied to them according to AASHTO T24 (2009). Table 2-14 outlines the price reduction method used by TDOT.

Table 2-12: Tennessee Department of Transportation Classes of Concrete (Tennessee Department of Transportation 2006)

Concrete Class	Min. 28-Day Comp. Strength (psi)	Min. Cement Content (lb/cy)	Maximum Water-Cement Ratio	Air Content % (Design ± production tolerance)	Slump (in.)
A	3000	564	0.45	6 ± 2	3 ± 1
D	4000	620	0.40	6	8 max.
L	4000	620	0.40	6	8 max.
S (Seal)	3000	682	0.47	6 ± 2	6 ± 1

Table 2-13: Speficied Acceptable Average Strength Concrete Specimens for Given Class of Concrete based on Age of Specimen (Tennessee Department of Transportation 2006)

Class of Concrete	Less than 31 Days	31 to 42 Days	43 Days or More
A, S	3000 psi	3300 psi	3500 psi
D, L	4000 psi	4400 psi	4600 psi

Table 2-14: Price Adjustment for Tennessee Department of Transportation

(Tennessee Department of Transportation 2006)

Percent Below Specified Concrete Strength Specified in Table 2-11	Percent of Bid Price to be Paid
0.1 - 3.3	95
3.4 – 6.7	90
6.8 – 10.0	80
10.1 – 13.3	70
13.4 – 16.7	60
16.8 – 20.0	50
20.1 – 23.3	45
23.4 – 26.7	40
26.8 – 30.0	35
30.1 – 33.3	30
> 33.3	25

It can be seen that if the cylinders or cores fail to meet the specified design strength for the age range which it was tested outlined in Table 2-13, then the price reductions in Table 2-14, which are based on the percentage below the required compressive strength that the test specimens were, will be applied to the concrete which was placed by the contractor. TDOT Division of Materials and Tests (2014) states that cores must be obtained and tested within 56 days of placement. Similarly, the Tennessee Department of Transportation (2006) states that cylinder submitted for testing after 56 days will not be accepted. Once cores are taken, the average core strength becomes the strength of record and payment is based on the average strength of the cores.

2.4.3 Florida Department of Transportation

Similar to many other state DOTs, the Florida Department of Transportation (FDOT) requires that three quality assurance cylinders are made when placing concrete. The Florida Department of Transportation (2010) states that either 6 x 12 in. or 4 x 8 in. cylinders may be used as a method of quality assurance. If the average strength of the quality assurance cylinders falls more than 500 psi or 10 %, whichever is greater, below the specified acceptable minimum compressive strength of the concrete, cores should be taken in order to determine if the in-place concrete is acceptable or if it must be removed and replaced. Core locations must be approved by FDOT and must not induce permanent damage to the structure after the core hole is repaired. The cores are then tested by FDOT in accordance with ASTM C42 in either the wet or dry condition, which is specified by the engineer. If the core strength results are less than 10% or 500 psi, whichever is greater, below the specified acceptable minimum compressive strength of the concrete, the concrete is deemed structurally adequate. FDOT considers concrete from which the average core strength of three specimens is more than 10% or 500 psi, whichever is greater, below the specified acceptable minimum compressive strength of the concrete structurally questionable. If this occurs, a structural analysis of the structure must be performed by the Specialty Engineer. If the analysis indicates that the concrete strength is adequate for the intended purpose of the structure, then the concrete is permitted to be left in place. Otherwise, the concrete must be removed and replaced by the contractor. Cores should not be taken if the average strength of the quality control cylinders is less than 10% or 500 psi, whichever is greater, below the specified allowable minimum strength. If cores are obtained and tested before the concrete has reached an age of 42 days, the average core strength will be taken as the 28-day strength. If cores are tested after 42 days, then the strength will be corrected for age in accordance with Equations 2-1 through 2-10 in Section 2.2.1. The formula for pay reduction can be seen below in Equation 2-16, where f'_c is the specified acceptable minimum strength of the concrete and f_c is the average strength of the core specimens retrieved from the structure.

$$Percent\ Reduction = 100 * \frac{f'_c - f_c}{f'_c} \quad \text{Equation 2-16}$$

2.4.4 Texas Department of Transportation

Similar to other state DOTs, the Texas Department of Transportation (TxDOT) requires that cores be taken in the event that the average strength of quality assurance cylinders does not meet the specified design strength for the project. If the average of the quality assurance cylinders meet the specified design strength and no single cylinder has a strength less than 85% of the design strength, the concrete is paid for at full price. If the average strength of the quality assurance cylinders do not meet the required strength or if one of the cylinders breaks below 85% of the required design strength, then the engineer will perform a structural analysis of the concrete structure to determine its adequacy. If cores must be taken to assess the in-place strength of the concrete, it will be done at the expense of the contractor and the engineer will test the cores. The Texas Department of Transportation (2004) specifies that "if all tested cores meet the required design strength, the concrete will be paid for at full price." If any of the cores do not meet the specified required design strength but the average strength of the cores is determined to be adequate, price reduction is done by using Equation 2-17 with the average strength of the cores being the strength of record.

$$A = [0.10 + 0.75(\frac{S_a}{S_s})^2] \times B_p \quad \text{Equation 2-17}$$

Where:

A = Amount to be paid per unit of measure for the entire placement in question

S_a = Actual strength from cylinders or cores. Use values from cores, if taken

S_s = Minimum required strength (specified)

B_p = Unit bid price

2.4.5 Comparison of State DOT Payment Reduction Methods

From the literature discussed above, it can be seen that each state has varying ways to reduce the price paid for concrete which does not meet specified design strength according to quality assurance cylinder tests but is deemed structurally adequate. Figure 2-9 shows a graphical comparison between the various payment reduction methods of the state DOTs which were discussed in this section.

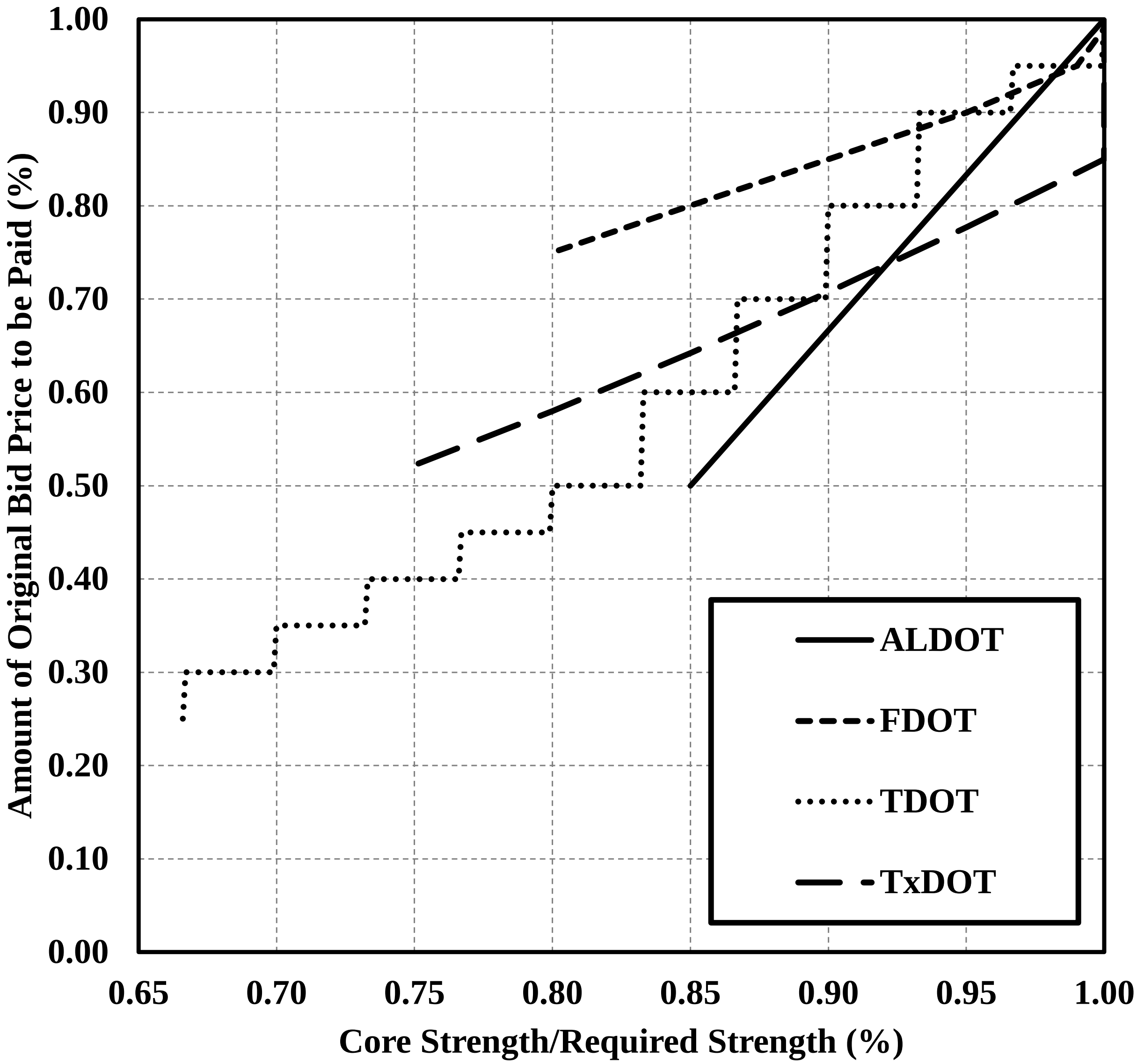

Figure 2-9: Summary of Payment Reduction Methods

From Figure 2-9, it can be seen that each state uses drastically different payment reduction methods. Of the four state DOTs which were examined, TDOT is the only one which uses a stepped function.

2.5 Summary

For anyone conducting in-place strength testing, it should be noted that there are many factors which will affect the apparent strength of the test results. Materials, such as cement type, use of SCMs, and aggregate type, should be considered when making decisions about the acceptability of a given concrete. SCMs in particular will have a significant impact on the set time, durability, and strength of the in-place concrete. Placement temperature and moisture conditions along with curing methods will also have a large impact on the results of in-place strength tests. Testing methods also have a significant effect on the apparent strength of the concrete. Molded cylinders should always be used as a method of quality assurance, but do not provide an accurate estimation of the actual in-place strength of the concrete. If cylinder test results are low, other methods of in-place testing should be used to more accurately determine the in-place strength. If cores are taken, ACI 318 (2011) states that the average strength of three specimens must exceed 85% of the design strength with no single value below 75% of the design strength for the in-place concrete to be considered structurally adequate. When analyzing the data recovered from core testing, a number of factors must be taken into account to produce meaningful estimations of in-place strength. Factors such as core diameter, length-to-diameter ratio, moisture conditioning, core damage, and the presence of steel reinforcement must be taken into account in order to draw valid conclusions from the collected data. Other testing methods are also available to test in-place strength, including cast-in-place cylinders and pullouts testing. The data collected from these tests must also be analyzed carefully to draw valid conclusions about the in-place strength of the concrete.

If the average strength of the quality assurance cylinders is below the specified required strength of the concrete, most states specify that cores can or must be taken to evaluate the adequacy of the in-place concrete. The contractor is most often then paid on a reduced scale depending on the average strength of the core specimens. This reduced pay scale differs from state to state. If cores are taken and the strength of the in-place concrete is not accepted by the state DOT, then the structure must either somehow be strengthened or the concrete must be removed and replaced at the cost of the contractor.

Chapter 3

Experimental Plan

In this chapter, the design and implementation of the experimental plan for the project is presented and discussed. Different types of in-place testing were conducted to determine the relationship between test type and apparent in-place strength as compared to standard molded cylinder strengths. The effect of age, strength level, aggregate type, supplemental cementing materials (SCMs), and degree of microcracking were also evaluated during the course of the project. The objective of this project was to provide ALDOT with data and recommendations for interpreting data collected from core testing as well as means to accept the in-place concrete based on the core strength results.

3.1 Introduction and Problem Definition

After the completion of the literature review, the factors which were to be evaluated during the project were defined. Since multiple procedures have been developed in order to convert core compressive strength at a certain age to a representative 28-day compressive strength, it was determined that the two main procedures outlined in Chapter 2 from Yazadani and McKinnie (2004) and ACI 209.2R (2008) should be evaluated to determine which more accurately predicted the 28-day compressive strength. Since Yazdani and McKinnie (2004) recommended different equations for different types of SCM types and ACI 209.2R did not, it also needed to be known if SCM type plays a role in the strength development of concrete cores.

Secondly, the effect of damage to the core had to be evaluated. Khoury, Aliabdo and Ghazy (2014) found that the type of aggregate contained within concrete had an effect on the apparent strength of a core. Khoury, Aliabdo and Ghazy (2014) also suggested that cores which were taken from high-strength concretes suffered far less damage than those taken from normal-strength concrete. Bartlett and MacGregor (1994c) suggested that microcracking could have an effect on the amount of damage that a core is exposed to. Because of this, aggregate type, strength level, and the effect of microcracking were evaluated.

In today's concrete industry, there are many different ways which in-place strength is evaluated. Because there are a variety of testing methods available, it was necessary to evaluate how differently some of these testing methods predicted in-place strength from coring. One common method for predicting in-place strength throughout Europe and Canada is the pullout test. Previous studies by ALDOT, including Nixon (2006), used pullout testing to evaluate in-place strength. Cast-in-place cylinders had also been used in previous studies, including Nixon (2006). Because of this, pullout testing and cast-in-place cylinders were chosen as the alternative in-place testing methods to compare against core testing.

Most state DOTs have a payment reduction scale which is used to adjust the amount which contractors are paid when the average strength of the quality assurance cylinders do not meet the specified design strength and cores must be taken to evaluate the strength of the in-place concrete. When cores are taken, they are then compared to the required design strength in order to determine the adequacy and strength of the in-place concrete as well as how much the contractor is to be paid for the concrete. The payment reduction practices by ALDOT, FDOT, TDOT, and TxDOT were examined in Chapter 2. In the case of all four of these DOTs, no consideration is given to the amount of damage inflicted upon a core during the drilling process as well as the difference in curing conditions and the presence of microcracking during the payment reduction. Therefore, if cores do not meet the required minimum strength, then payment is reduced for the in-place concrete. In contrast, ACI 318 (2011) states that concrete strength shall be deemed structurally adequate if the average strength of three cores is greater than 85% of the specified compressive-strength (f'_c) for the project as long as no single core strength of three cores is below 75% of the required strength.

For many years, the moist cured, molded 6×12 in. cylinder has been the standard for the quality assurance of concrete. In more recent years, it has been suggested moist cured, molded 4×8 in. cylinders would produce similar results to 6×12 in. cylinders. Day and Haque (1993) propose that this switch would pose numerous advantages, such as easier handling during transportation, smaller required storage spaces, lower required capacity of testing machines, and the reduced costs for molds, capping materials, and concrete.

3.2 Development of Experimental Plan

After all the variables were determined, an experimental testing plan was developed. First, in order to evaluate the effect of strength gain over time, different testing ages were chosen. The first in-place tests were to be conducted at 28-days in order to obtain measured values to compare with both the moist-cured, molded cylinders to be broken at 28 days as well as to compare these values with ones obtained from future testing ages. The second age which was chosen was 42 days. This is because the Alabama Department of Transportation (2012) requires cores to be drilled and tested before 42 days after placement. The long-term strength development needed to be evaluated, but it had to be within the time limitations for the project. Because of this, the testing age of 365 days chosen. Finally, it was determined that testing should take place at an age sometime between 42 and 365 days, so 91 days was chosen as this is three months after placement and is a common testing age. Since strength development is also dependent on SCM type, it was determined that members containing different types of SCMs should be cast. Class C fly ash, Class F fly ash, and slag cement were chosen as the SCMs to be evaluated since these represent the most commonly used SCMs in today's concrete industry.

Next, variables which affected the damage to a core were considered. Since coarse aggregate type could have an effect on the apparent strength of a core due to the difficulty to cut through, three different aggregate types which are local to the state of Alabama were evaluated: uncrushed river gravel, crushed limestone, and crushed granite. Another factor which has an effect on the damage imparted on a core is the amount of microcracking which occurs within the concrete. Since microcracking is heavily impacted by the amount of restraint which the concrete is exposed to, two regions of testing were to be made: one near the exterior edge of the member to represent the low restraint region and one in the middle of the member to represent the highly restrained region. The third variable which could possibly have an impact on the amount of damage is the concrete strength. To evaluate this, it was concluded that both normal-strength and high-strength members needed to be cast in order to evaluate difference in core damage between the two.

There were three different types of in-place testing which were chosen: cores, cast-in-place cylinders, and pullout tests. ALDOT most frequently uses cores when determining the in-place strength of concrete. Cast-in-place cylinders are a type of in-place testing that is not widely used, but produces molded specimens with the same temperature and moisture history as the in-place concrete while still allowing the specimen to expand and contract, therefore reducing the impact of microcracking. The third type of testing used was pullout testing, which is most commonly used in Europe and Canada to evaluate in-place strength, but has been used on past ALDOT projects. Floating pullout inserts were cast into the concrete during casting of the concrete and then tested at the specified ages. Along with the different in-place testing methods, both 6×12 in. and 4×8 in. molded, moist cured cylinders were made and tested at the same age as the in-place testing methods.

It was determined that eight different specimens should be cast to encompass the different aggregate types, strength levels, and SCMs that were tested and that the specimens cast in this study should be slabs as this is the most applicable scenario for which core testing is done in the state of Alabama. A slab size of 15 ft × 15 ft was chosen as this represented a large enough specimen which could be considered representative of a field specimen which would be cored by ALDOT, but also small enough to cast using a single ready-mixed truckload of concrete. This size would also ensure that the interior and exterior testing regions had different degrees of restraint and therefore different degrees of microcracking. A slab thickness of 9 ½ in. was chosen as this could represent the thickness of a bridge girder.

After the testing methods and various materials were chosen, a slab layout was devised in order to satisfy all the testing requirements as well as provide an adequate representation of a typical slab cast by ALDOT. In order to accurately model the effect of axial restraint within a slab, it first needed to be known what factors have an impact on restraint. Rasmussen and Rozycki (2001) showed that it was not just the frictional force caused by the self-weight of the slab which had an effect on the axial restraint of a slab, but also the interlocking and adhesion forces between the slab and the sub-base. An illustration of this can be seen in Figure 3-1. When shrinkage and temperature change occur within a slab, these forces restrain movement. An illustration of the typical stress distribution between a slab and its sub-base can be seen in Figure 3-2.

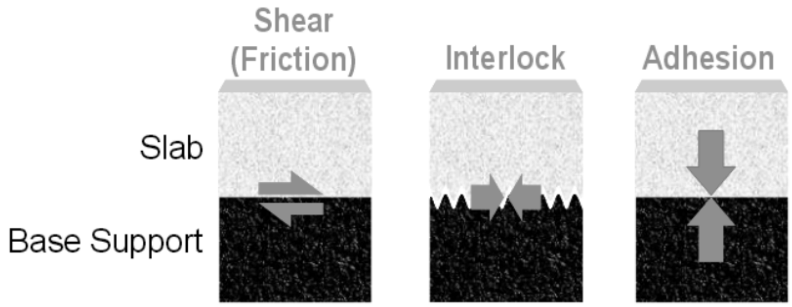

Figure 3-1: Forces Affecting the Axial Restraint of a Slab

(Rasmussen and Rozycki 2001)

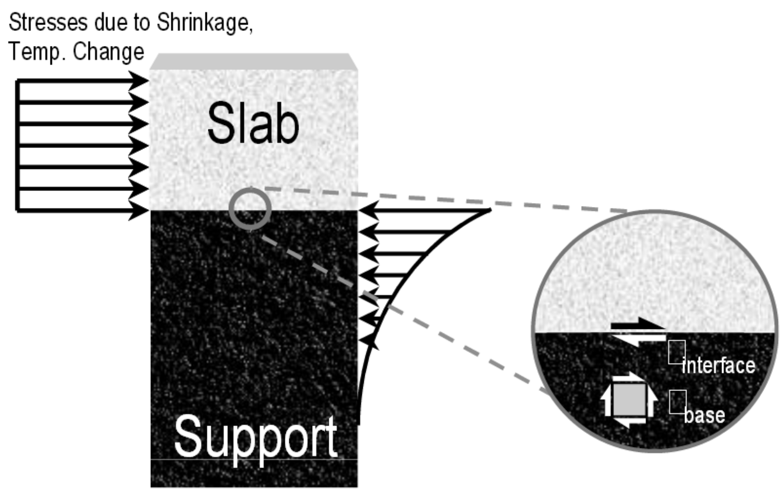

Figure 3-2: Typical Stress Distribution Between Slab and Sub-Base when Exposed to Shrinkage and Temperature Change Forces (Rasmussen and Rozycki 2001)

With these modeling requirements in mind, a square, 15 ft × 15 ft slab with a 9½ in. depth was laid out and divided into 4 square quadrants. Each one of these quadrants would be tested at different ages to determine the effect of age for each testing method. The testing methods within the four quadrants were tested at 28, 42, 91, and 365 days respectively. This was done to establish a strength gain relationship for the in-place tests with respect to testing age. Care was taken to ensure that spacing requirements for the individual testing methods were met so that one testing method did not have an effect on the other surrounding tests. Figure 3-3 shows the anticipated effect that restraint would have on the slab. Testing which was done in the exterior region occurred near the area labeled “Least Restraint to Movement” while interior region testing was conducted near the area labeled “Most Restraint to Movement.” A schematic of the slab layout can be seen below in Figure 3-4. A close up view of the testing layout in a typical quadrant can be seen in Figure 3-5.

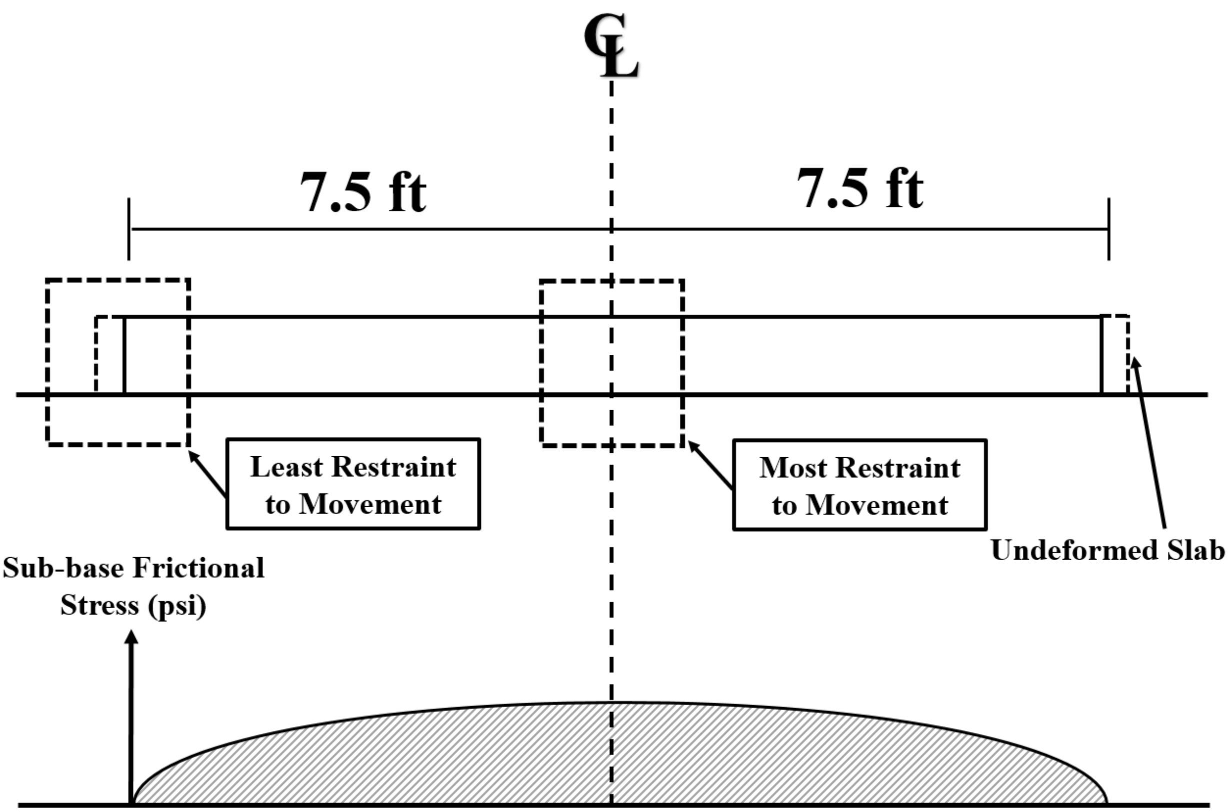

Figure 3-3: Effect of Restraint on Slab Specimens

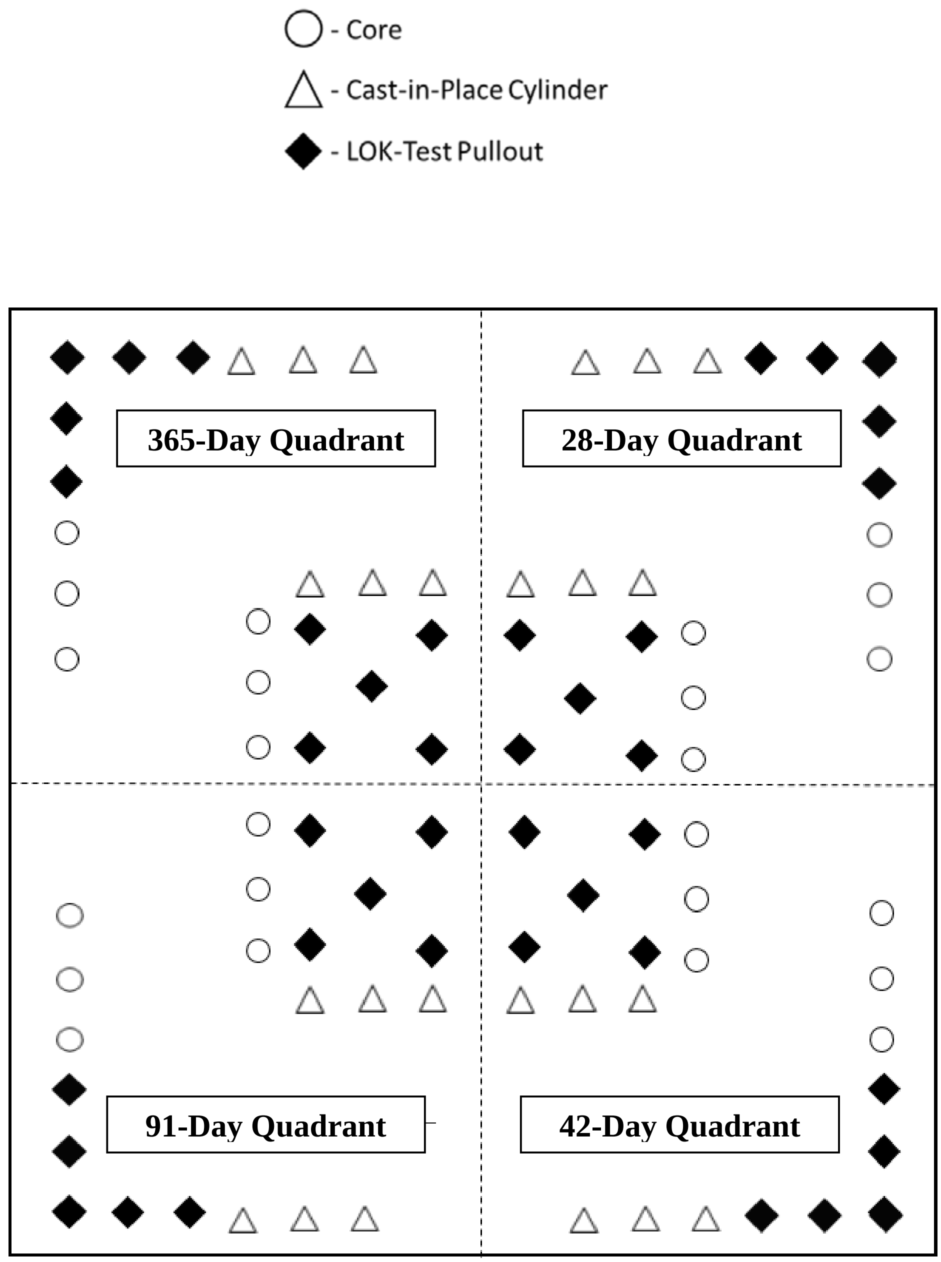

Figure 3-4: Typical Slab Layout (Not to Scale)

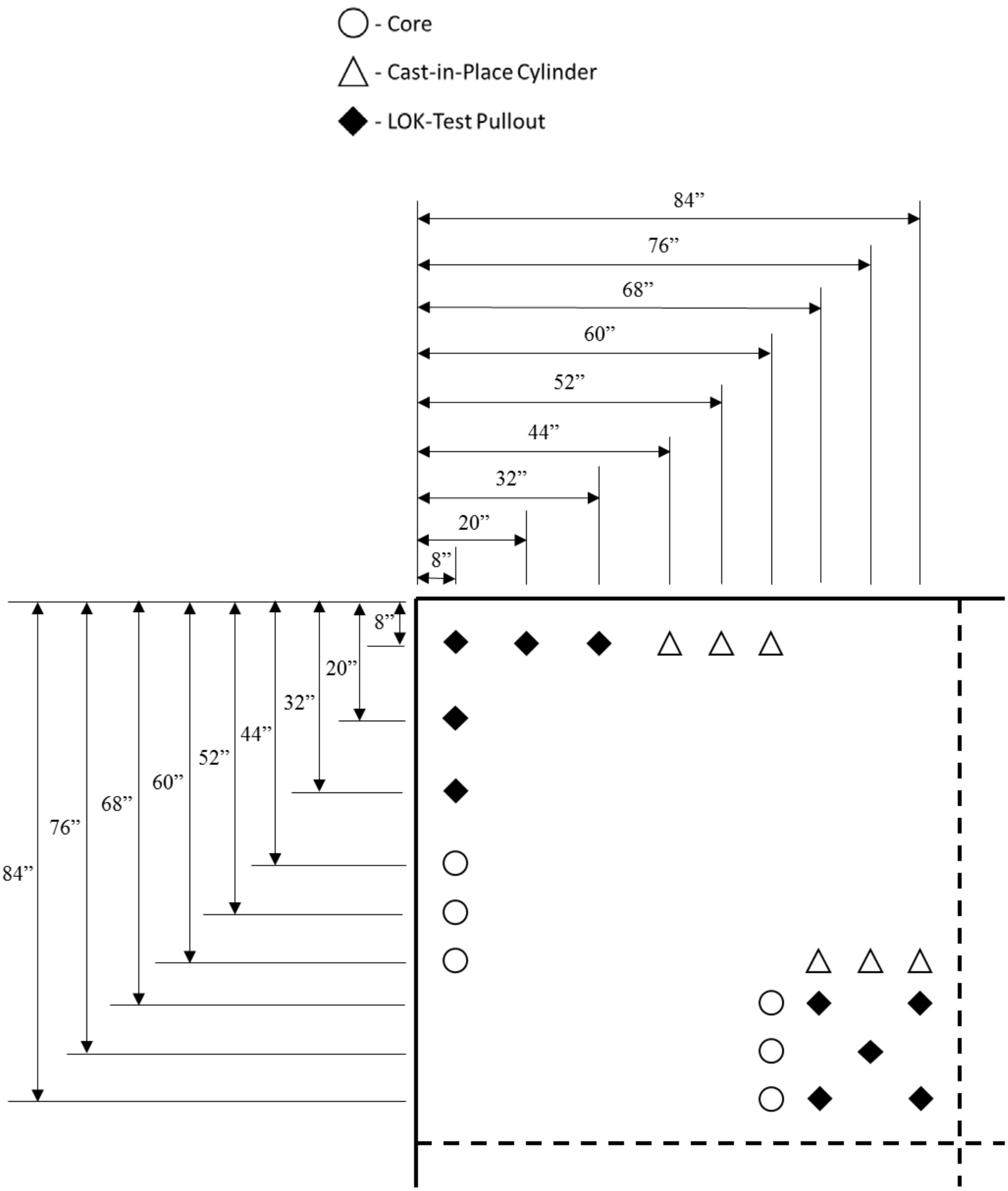

Figure 3-5: Testing Layout within a Typical Quadrant

Once the slab layouts were complete, concrete mixtures were selected. Considering all the variables that had to be evaluated, it was decided that six normal-strength concrete slabs and two high-strength concrete slabs were to be cast for testing purposes. Each mixture contained a different combination of aggregate type, cementing materials, and target strength. Slab identifications were developed using the abbreviations for aggregate types and SCMs found in Table 3-1 and 3-2, respectively. An example of a typical slab identification is shown in Figure 3-6. Where fly ash was used, a 20% replacement by weight of portland cement was used. Where slag cement was used, a 50% replacement by weight of portland cement was used. Where Type I portland cement is specified, the only cementing material used in the concrete was portland cement.

Table 3-1: Abbreviations for Different Aggregate Types

Type	Abbreviation
Granite	GR
Limestone	LS
River Gravel	RG

Table 3-2: Abbreviations for Different Supplemental Cementing Materials

Type	Abbreviation
Type I Portland Cement	PC
Class C Fly Ash	CA
Class F Fly Ash	FA
Slag Cement	SC

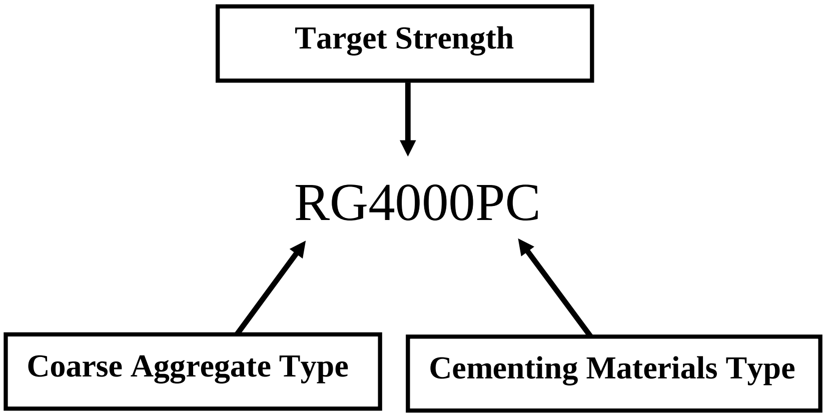

Figure 3-6: Typical Slab Identification

Mixture proportions for the normal-strength slabs were provided by Twin Cities Concrete of Sherman Industries in Auburn, AL. Mixture proportions for high-strength specimens were developed through trial batching. The target strength of all the normal-strength slabs was 4,000 psi while the target strength of the two high-strength slabs was 8,000 psi. SSD batch weights per cubic yard can be seen in Tables 3-3 and 3-4.

Table 3-3: SSD Batch Weights for High-Strength Slabs

	RG8000PC	LS8000PC
Coarse Aggregate SSD (lb/yd^3)	1800	1800
Fine Aggregate SSD (lb/yd^3)	1130	1130
Type I Portland Cement (lb/yd^3)	950	950
Water (lb/yd^3)	266	266
Water-Cement Ratio	0.28	0.28
Glenium 7500 HRWR (oz/cwt)	6	5
Delvo Stabilizer (oz/cwt)	-	6

Table 3-4: **SSD Batch Weights for Normal-Strength Slabs**

	RG4000PC	LS4000PC	GR4000PC	RG4000CA	RG4000FA	RG4000SC
Coarse Aggregate SSD (lb/yd^3)	1880	1880	1880	1900	1900	1900
Fine Aggregate SSD (lb/yd^3)	1225	1225	1225	1105	1105	1105
Type I Portland Cement (lb/yd^3)	560	560	560	461	461	290
Class C Fly Ash (lb/yd^3)	-	-	-	115	-	-
Class F Fly Ash (lb/yd^3)	-	-	-	-	115	-
Grade 100 Slag (lb/yd^3)	-	-	-	-	-	290
Water (lb/yd^3)	280	280	280	268	268	268
Water-Cementitious Materials Ratio	0.50	0.50	0.50	0.46	0.46	0.46
WRA (oz/cwt)	3	3	3	3	3	3
Air (oz/cwt)	0.5	0.5	0.5	0.3	0.3	0.3

From the proportions in Tables 3-3 and 3-4, it can be seen that the water-to-cement ratio for normal-strength slabs containing only portland cement was 0.5 while the water-to-cementing materials ratio for normal-strength slabs containing SCMs was 0.46. The water-to-cement ratio for both of the high-strength slabs was 0.28. The concrete was ready-mixed and then delivered to site.

3.3 Site Preparation

On the selected site, two areas measuring 15 ft.×60 ft. were selected and cleared of all brush, trees, and debris. A skid-steer, which can be seen in Figure 3-7, was used for all site work. Each area was designed to accommodate 4 slabs measuring 15 ft.×15 ft. each. Number 57 crushed limestone aggregate was laid as a sub-base. The second 15 ft.×60 ft. area can be seen before clearing, after clearing, and after the sub-base was placed in Figures 3-8, 3-9, and 3-10, respectively.

Figure 3-7: Skid-Steer Used for Site Work

Figure 3-8: Second 15 ft × 60 ft Area Before Clearing

Figure 3-9: Second 15 ft × 60 ft Area After Clearing

Figure 3-10: Second 15 ft × 60 ft Area After Placement of Sub-Base

Treated pine 2x10s were then used as forms for each casting. Wooden stakes were used on the exterior of the forms to keep them from bowing during casting due to the fluid pressure of the concrete. The cast-in-place cylinder systems were placed in the correct spots within the forms and secured with number 3 standard rebar. Form release agent was applied to the inside of the forms to make form removal easier. Forms were removed from the slabs at 7 days after concrete placement along with all curing materials. A typical setup before casting can be seen in Figure 3-11.

Figure 3-11: Typical Setup before Placement

Since slabs were cast directly adjacent to each other, fiber board was bonded to the shared edge of two slabs so as to prevent mutual restraint.

3.4 iButton Temperature Sensors

iButton temperature sensors were used to record the temperature history of all slabs as well as moist-cured cylinders. For each concrete placement, a total of eight iButton temperature sensors were used. Two sensors were placed near the edge of the slab to collect temperature data to reflect the temperature of the area near the exterior testing locations. The sensors were tied to wooden stakes which were hammered securely into the ground. One sensor was placed at approximately the mid-height of the slab while the second was placed near the surface of the slab. Two more sensors were placed in the interior of the slab in a similar fashion. Slab sensors can been seen in Figure 3-12.

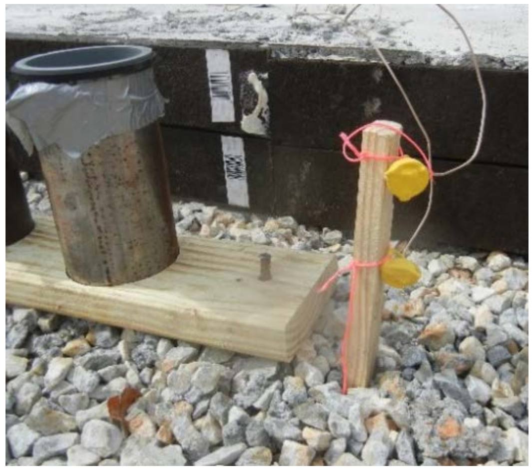

Figure 3-12: Placement of iButton Sensors in a Slab

The other four temperature sensors were placed in cylinders to monitor their temperature development. Two sensors were placed in a 6×12 in. cylinder while the other two sensors were placed in a 4×8 in. cylinder. Placement of sensors in a 4×8 in. cylinder can be seen in Figure 3-13.

Figure 3-13: iButton Temperature Sensors in a 4x8 in. Cylinder

Similar to the slabs, each of the two cylinders had one sensor placed near its mid-height and the other sensor placed near its surface. These cylinders were used strictly to collect temperature data for the molded cylinders from the placement and were not tested in compression. iButtons must be held in place by a clip and then wired in order to collect data. An iButton being held by a clip purchased from Maxim Integrated can be seen in Figure 3-14.

Figure 3-14: iButton Held in a Clip

Two-conductor communication cable with 22-gauge conductors was used to wire the iButtons. One end of the wire had to be soldered to the iButton clip while a typical telephone plug was attached to the other end. A soldered iButton can be seen in Figure 3-15 and the telephone plug end of the wire can be seen in Figure 3-16. Each iButton system was then labeled by attaching duct tape to its wire. The labeling system can be seen below in Table 3-5.

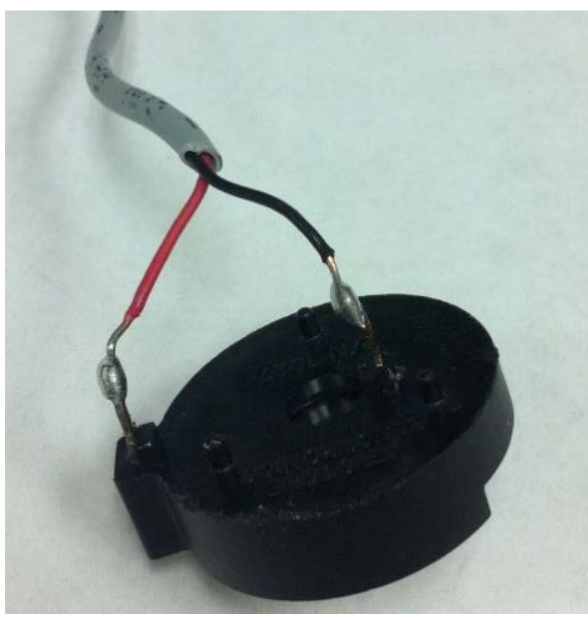

Figure 3-15: Soldered iButton

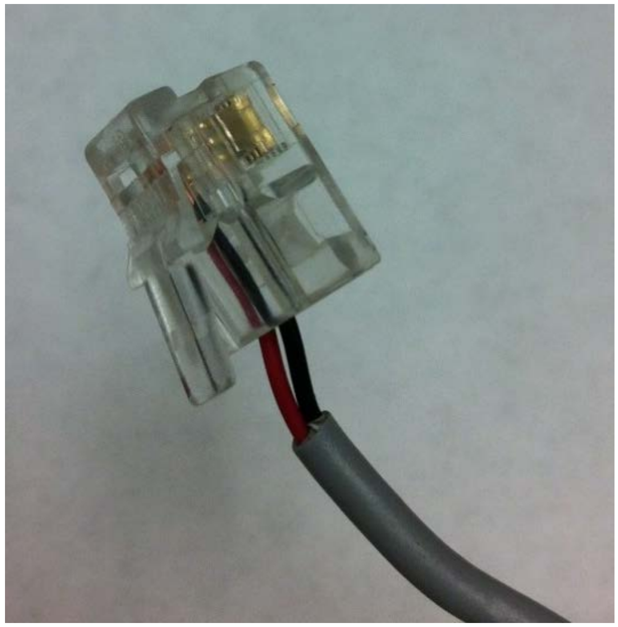

Figure 3-16: Telephone Plug-End of Communication Wire

Table 3-5: Labeling of iButtons

Label	Position
OT	Exterior Top of Slab
OM	Exterior Mid-height of Slab
IT	Interior Top of Slab
IM	Interior Mid-height of Slab
C612T	Top of 6x12 in. Cylinder
C612M	Mid-height of 6x12 in. Cylinder
C48T	Top of 4x8 in. Cylinder
C48M	Mid-height of 4x8 in. Cylinder

After being wired, iButtons were coated in epoxy to protect them from the surrounding concrete after being placed in the slab and molded cylinders. After the epoxy was applied, the iButtons had to be programmed in order to start collecting data. This was done by using One Wire Viewer Software. iButtons were connected to a computer using the telephone plug on the end of the wire. An adapter was used which could be plugged into the USB port of a computer in order to program and retrieve data from the iButtons. When programming an iButton, the user must specify a time interval between when data points are collected between 1 and 255 minutes. For the first seven days after casting, temperature data were recorded every 15 minutes. At seven days, these data were collected using a laptop computer and One Wire Viewer Software. The iButton was then reprogrammed in order to collect more data. Temperature data was collected from 7 to 91 days using a time interval of 90 minutes. At 91 days, these data were collected again, and the iButtons were reprogrammed to collect data every 255 minutes until the slab and cylinders were 365 days old. Temperature data were collected at 365 days. The data were then combined to form a complete temperature history for both the slab and the molded cylinders.

3.5 Casting

Ready-mixed concrete from Twin City Concrete (ALDOT Vendor Code 408) located in Auburn, Alabama was used for all casts. For slabs having targeted strengths of 4,000 psi, mixture proportions supplied by Twin City Concrete were used. For slabs using high-strength concrete, mixture proportions were supplied to Twin City Concrete for batching. Once the truck arrived on site, the slump and total air content were measured. If necessary, admixtures were added to the mixture to achieve slump within a target range of 3 to 5 in. for normal strength concrete and 5 to 9 in. for high strength concrete. Once these values were recorded, casting began. A wheelbarrow was filled with concrete for the making of the molded cylinders and cubes. As concrete was placed in the forms, the cast-in-place cylinders were filled by shovel to ensure uniformity between the surrounding concrete and the cast-in-place specimens. Internal vibration was applied to the slab using a conventional vibrator. Typical vibration of the concrete can be seen in Figure 3-17.

Figure 3-17: Internal Vibration of a Slab

Cast-in-place cylinders were vibrated externally by touching the vibrator to the exterior of the support system which held the molds in place. Once the slab forms were filled, the top of the slab was screeded. After this, the surface was finished with a bull float. After final finishing was applied, the pullout inserts were placed at their appropriate locations within the slab. Blemishes around the inserts were smoothed out with handheld metal trowels. A completed slab prior to the application of curing can be seen in Figure 3-18.

Figure 3-18: A Completed Slab

3.6 Finishing and Curing Methods

After the concrete had reached initial set, wet cotton mats were used to cover the slab. Soaker hoses were connected to a large water tank through a timer which allowed water to flow slowly through the soaker hoses every 12 hours for a 1-hour interval to provide adequate moisture to the cotton mats during curing. The cotton mats with soaker hoses can be seen in Figure 3-19 and the entire curing system can be seen in Figure 3-20. Large sheets of plastic were placed over the cotton mats and soaker hoses to avoid as much evaporation as possible. After seven days, the plastic, soaker hoses, cotton mats, and forms were removed from the slab.

Figure 3-19: Curing Mats and Soaker Hoses

Figure 3-20: Complete Curing System

3.7 Molded Cylinders

For each cast, sixteen 6×12 in. and sixteen 4×8 in. cylinders were made and tested in compression. Both sizes of molded cylinders were cast in order to determine if there was a difference in apparent strength due to cylinder size. Three cylinders of each size were tested at 7, 28, 42, 91, and 365 days. iButton temperature sensors were placed in separate cylinders of each size in order to determine the temperature history of the cylinders. Molded cylinders were made in accordance with AASHTO T23 (2009). An example of the making of cylinders can be seen below in Figure 3-21.

Figure 3-21: Making of 6×12 in. Molded Cylinders

After the molded cylinders had been made, they were placed in temperature controlled coolers filled with water for at least 24 hours, but no more than 48 hours. During this time period, the cylinders were kept within the required temperature range stated in AASHTO T23 (2009). For specimens with anticipated 28-day strength less than 6000 psi, this required temperature range is 60-80°F; for specimens with anticipated 28-day strength greater than 6000 psi, this required range is 68-78°F. An example of the cylinders during their initial-curing state can be seen below in Figure 3-22.

Figure 3-22: Molded Cylinders in Their Initial Curing State

After the initial curing stage the specimens were retrieved from the test site and brought back to the Auburn University Civil Engineering Materials Laboratory. Care was taken so that the specimens were not damaged while being transported back to the laboratory. Special cylinder holders were made in order to ensure that the specimens were transported safely. Cylinders were then stripped from their molds, labeled, and placed in the moist-curing room where they remained until they were sulfur capped and tested. Cylinders being transported back to the Materials Laboratory can be seen in Figure 3-23, while the typical labeling of a cylinder can be seen in Figure 3-24.

The cylinders were sulfur capped the day before being tested for compressive strength. After being capped, the cylinders were again placed in the moist-curing room until tested in compression in accordance with AASHTO T22 (2009).

Figure 3-23: Molded Cylinders and Cubes Being Transported Back to AU Materials Laboratory

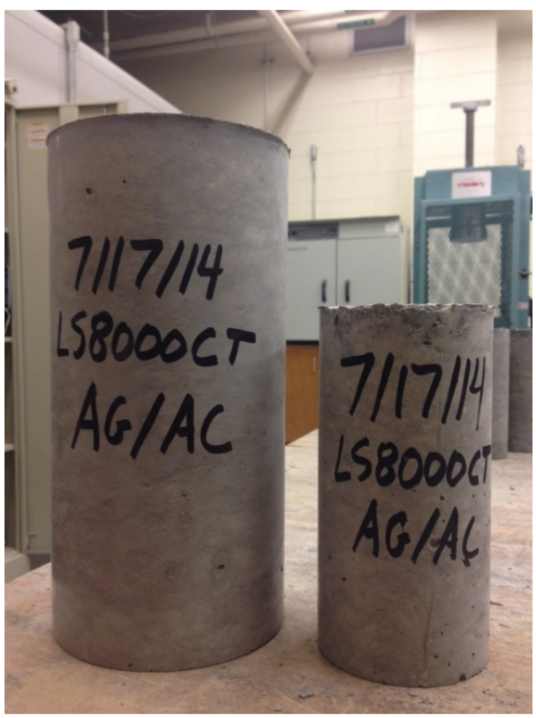

Figure 3-24: Typical Labeling of Molded Cylinders

3.8 Pullout Calibration Cubes

A test method was developed in order to validate the calibration table for the LOK-Test pullout inserts. Cubes (8×8×8 in.) were constructed, and pullout inserts were attached to the four inside vertical walls of the cubes using small bolts which threaded into the pullout inserts. As per the manufacturer's recommendation, Type L-45 inserts were used for concrete with expected 28-day strength of 4000 psi, while Type L-46 inserts were used for high strength concrete. The cubes were formed using the same procedure as molded cylinders. The cubes were made using three equal lifts. The number of rods per lift was determined using the same rod-to-area ratio as molded cylinders. Each lift was rodded 56 times and tapped with a rubber mallet. The making of the calibration cubes can be seen in Figure 3-25.

Figure 3-25: Making of the LOK-Test Pullout Calibration Cubes

After initial set, the cubes were placed inside a shed with a window-mounted air conditioner to control the curing temperature. Wet burlap was placed over the cubes and then covered with plastic sheeting to prevent moisture loss. After a waiting period of 24-48 hours, the cubes were transported back to the Auburn University Materials Laboratory along with the molded 6×12 in. and 4×8 in. cylinders. The screws which held the LOK-Test pullout inserts in place were removed in order to strip the cubes from their wooden molds. After being removed from their respective molds, the cubes were labeled accordingly and placed into the moist-cure room until being tested at their specified ages. During testing, a constant load rate of 70 ± 30 kPa/s was applied to each insert with a pullout testing machine as specified by ASTM C900 (2007). The pullout insert was loaded until failure. Once failure occurred, the maximum gage reading was recorded. The cube calibration pullout tests were conducted at 28, 42, and 365 days. In order to evaluate the accuracy of the tables supplied by the manufacturer, 6×12 in. and 4×8 in. molded, moist-cured cylinders were tested in compression at the same age as when the calibration pullout tests were conducted.

3.9 Cores

For each slab, 4 in. diameter cores were retrieved and tested in compression to establish a relationship between molded, moist-cured cylinder strength and in-place strength as measured by core testing. Six cores were taken from each age-specific quadrant of each slab. Three cores were taken a distance of 8 in. on-center from the edge of the slab which represented the low restraint condition, and three cores were recovered from the interior region of the slab which represented the high restraint condition. A core being extracted from the exterior zone of the slab can be seen in Figure 3-26.

Figure 3-26: Drilling of a Core from the Exterior Region of a Slab

Cores were retrieved from their slabs one week prior to testing. Once cores were drilled, they were removed from the slab using a core snap. After removal, the cores were wiped dry, and all surface water was evaporated. The cores were also measured upon removal from the slab, and their initial lengths recorded. After initial lengths were recorded and the surface water had evaporated, the cores were placed inside two sealable plastic bags, secured with rubber bands, and labeled accordingly as required in AASHTO T24 (2009). The cores were then transported back to the Auburn University Structures Laboratory and trimmed to a length-to-diameter ratio of 2.0 using a wet saw. After being trimmed, the cores were again wiped of all excess surface water. The cores were then re-sealed back in plastic bags and wrapped with rubber bands after all surface water had evaporated as per AASHTO T24 (2009). The cores were kept sealed in their plastic bags for 6 days until sulfur capped. After being sulfur capped, the cores were placed back in sealed plastic bags and wrapped with rubber bands until tested in compression the following day. AASHTO T231 (2009) requires that the sulfur caps of specimens whose expected strength is less than 5,000 psi must harden for at least two hours before the specimen is tested. AASHTO T231 (2009) also requires that the sulfur caps of specimens whose expected strength is greater than or equal to 5,000 psi must harden for at least 16 hours before the specimen is tested. Testing occurred 7 days after removal from the slab, which meets requirements set forth in AASHTO T24 (2009). Before being tested, the average diameter and capped length were recorded. Following the test, the time of the test and peak compressive force were recorded, and compressive strength was calculated based on the cross-sectional area of the core calculated from the average diameter. Cores were tested at concrete ages of 28, 42, 91, and 365 days.

3.10 Cast-In-Place Cylinders

Each slab also contained cast-in-place cylinders, which measured 4 in. in diameter and 8 in. in height, that were retrieved from the slab and tested in compression. The purpose of cast-in-place cylinders is to obtain a compression strength from a molded specimen that has not been damaged due to coring and microcracking yet still has the same temperature and moisture conditioning as the in-place concrete. Cast-in-place cylinders were tested at 28, 42, 91, and 365 days. A relationship was then established relating the strengths measured from the cast-in-place cylinders to the strengths measured from the corresponding molded cylinders. A set-up had to be developed in order to hold the cylinders in place during casting. To do this, 2x6 boards were cut to specified lengths. A hole-saw was used to cut a series of 4 in. diameter holes approximately 1 in. through the thickness of the board. For the first three concrete placements, sheet metal was cut and then wrapped into cylinders which fit into the 2×6 and held the plastic cylinders molds in place. It was determined that sheet metal was not ideal for this purpose because the confining pressure of the surrounding concrete made it extremely difficult to remove the molds from the hardened slab. Because of this, a new system was developed for the remaining five slabs. Steel pipe with an outside diameter of 4½” and an inside diameter of 4 3/8” was used. This provided a more rigid system, which was less deformable under the confining pressure of the surrounding concrete. After being cut to the correct length, the pipe was placed in the same 2×6 system described above. During casting, no internal vibration was used to consolidate the cylinders as regulated in ASTM C873 (2011). The cylinders were filled manually by shovel as the slab was being cast. An example of this can be seen below in Figure 3-27.

Figure 3-27: Filling of Cast-In-Place Cylinders

Consolidation was achieved by touching the vibrator to the outside of the support system. After casting, the cast-in-place cylinders were retrieved from their slabs one week before being tested in compression. The retrieval of a cast-in-place cylinder can be seen in Figure 3-28.

Figure 3-28: Removal of a Cast-In-Place Cylinder From a Slab

To ensure the same moisture conditions, the same procedure was followed for the cast-in-place cylinders as was used for the cored specimens. Once retrieved from their specified locations within the slab, the cast-in-place cylinders were stripped from their plastic molds and placed in two sealed plastic bags and wrapped with rubber bands. At this point, the cast-in-place cylinders were transported back to the Auburn University Materials Laboratory to be trimmed. Only the tops of the cast-in-place cylinders which had been exposed while within the slab were trimmed. Only a minimal amount was trimmed from the top to ensure that the length-to-diameter ratio was as close to 2.0 as possible. After being trimmed, the cast-in-place cylinders were again placed in two sealed plastic bags and wrapped in rubber bands for six days until being sulfur capped. On the day before testing, the cast-in-place cylinders were removed from their bags and sulfur capped. AASHTO

T231 (2009) requires that the sulfur caps of specimens whose expected strength is less than 5,000 psi must harden for at least two hours before the specimen is tested. AASHTO T231 (2009) also requires that the sulfur caps of specimens whose expected strength is greater than or equal to 5,000 psi must harden for at least 16 hours before the specimen is tested. Testing occurred 7 days after removal from the slab, which meets requirements set forth in AASHTO T24 (2009). Once capped, the specimens were again placed back in their bags and wrapped with rubber bands. Before being tested, the average diameter and capped length were recorded. After compression testing, the time at failure as well as the peak compressive failure load were recorded. The strength of the specimen was based on the area calculated using the average measured diameter.

3.11 Pullout Inserts

LOK-Test pullout inserts were placed into each slab just after bull floating of the fresh concrete. As recommended by the manufacturer, L-49 floating inserts were used in slabs with an expected 28-day strength of 4000 psi while L-50 floating inserts were used in high-strength concrete slabs. Typical L-49 and L-50 inserts can be seen in Figure 3-29, and the placement of L-49 inserts into the exterior region of a slab can be seen in Figure 3-30. L-50 inserts are used for testing high-strength concrete because they have a thicker head to withstand a higher pullout force.

Figure 3-29: Typical L-50 (Left) and L-49 (Right) Inserts

Figure 3-30: Placement of L-49 LOK-Test Pullout Inserts Into a Slab

Ten pullout inserts were placed in each quadrant of the slab for a total of 40 inserts per slab. In each zone, five of the ten inserts were placed 8 in. on-center from the edge of the slab in the exterior region, which has the least amount of restraint to movement. The remaining five inserts were placed in the interior region of the slab where the restraint to movement was much higher than the exterior region. During testing, a constant load rate of 70 ± 30 kPa/s was applied by the pullout machine as specified by ASTM C900 (2007). The concrete was loaded until failure. Once failure occurred, the maximum gage reading was recorded. An example of a pullout test being performed can be seen in Figure 3-31.

Figure 3-31: Pullout Test Being Performed

3.12 Raw Materials Summary

3.12.1 Coarse Aggregate and Fine Aggregate

Over the course of the project, three different types of aggregates were used: crushed limestone, uncrushed river gravel, and crushed granite. All aggregates used were a No. 67 gradation. The crushed limestone came from APAC Midsouth in Opelika, Alabama (ALDOT ID Number 1604) while the uncrushed river gravel was from the Foley Materials Company in Shorter, Alabama (ALDOT ID Number 1481), and the crushed granite was from the Columbus Quarry, LLC in

Fortson, Georgia (ALDOT ID Number 0135). All sand used for the project was from the Foley Materials Company in Shorter, AL (ALDOT ID Number 1481). The properties of the coarse and fine aggregates can be seen in Table 3-6. Gradations were performed on all coarse aggregate sources to verify that all were within the requirements for No. 67 gradation specified in AASHTO M 43 (2009). The results from the gradations can be seen in Figure 3-32.

Table 3-6: Coarse Aggregate Properties

Aggregate	Bulk Specific Gravity (SSD)	Absorption Capacity (%)
Crushed Limestone	2.84	0.20
Uncrushed River Gravel	2.63	0.40
Crushed Granite	2.68	0.40
Sand	2.64	0.40

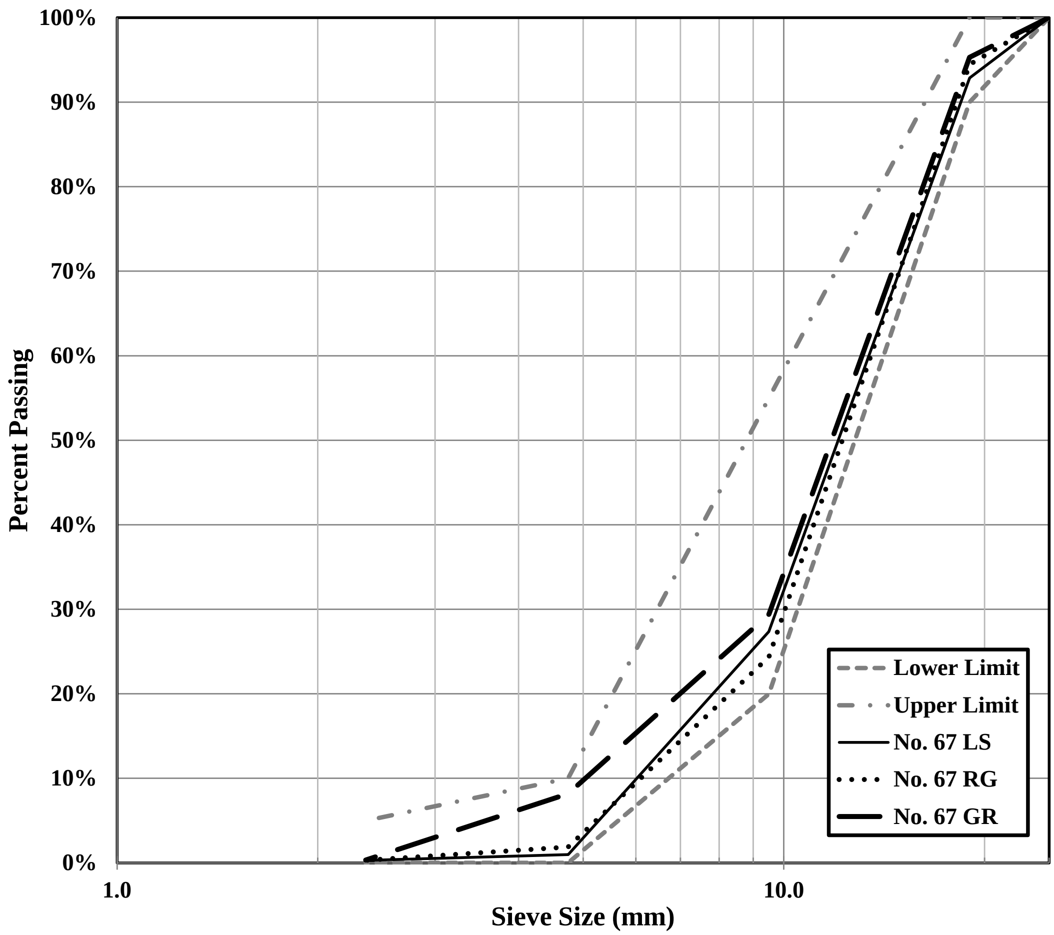

Figure 3-32: Coarse Aggregate Gradations

All coarse aggregates used throughout the duration of the project met the gradation requirements set forth in AASHTO M43 (2009).

3.12.2 Cement and Supplementary Cementing Materials

In today's concrete industry, it is very common that supplemental cementitious materials (SCM) are used in concrete production in order to reduce cost, increase performance, and/or provide a more sustainable option. In this study, several different SCMs were used in casting the experimental slabs in order to determine if a difference in strength gain was present.

3.12.2.1 Class C Fly Ash

Class C fly ash was used in the RG4000CA slab as a 20% replacement for portland cement. The source of the Class C fly ash was Holcim in Quentin, Alabama.

3.12.2.2 Class F Fly Ash

Class F fly ash was used in the RG4000FA slab as a 20% replacement for the portland cement. The source of the Class F fly ash was Boral Industries in Cartersville, Georgia.

3.12.2.3 Slag Cement

Slag cement was used in the RG4000SC slab as a 50% replacement for portland cement. The source of the slag cement was Holcim in Birmingham, Alabama.

3.12.2.4 Type I Cement

Type I cement was used for the duration of the project. The source of the cement was Lehigh Portland Cement Company located in Leeds, Alabama (ALDOT Vendor Code 140).

Chapter 4

Presentation of Results

4.1 Introduction

Throughout the course of the project, many variables were evaluated to determine their effect on the in-place strength of concrete and how these strengths differ between test methods. Compressive strength data were collected on each of the eight test slabs over the course of one year in order to determine these effects. In-place testing was performed at 28, 42, 91, and 365 days in order to develop a relationship between the age of the concrete being tested and in-place strength. The factors which were most closely examined for their effects in this project were the effect of strength level, age, restraint, supplemental cementing materials, aggregate type, and test type. Six of the slabs were normal-strength concrete and the other two slabs were considered high-strength concrete and therefore analyzed separately from the normal-strength concrete data.

Relationships were evaluated between in-place testing methods and 6×12 in. molded, moist-cured cylinders since this is the most commonly recognized form of quality assurance used in the concrete industry. In addition to developing relationships between in-place testing and cylinder strength, a comparison of 4×8 in. and 6×12 in. cylinders was done in order to determine if there was any significant difference between the two types of specimens. The cast-in-place cylinder data from casts LS4000PC, RG4000PC, and RG4000CA were not considered in the analysis due to the amount of damage imparted on the cylinders during removal from the slab. All statistical analyses were performed with a 95% confidence level unless otherwise specified.

4.2 Temperature Data

It has long been known that the temperature of the freshly placed concrete has a significant impact on the hardened properties of the concrete. Increased temperatures during placement can provide numerous challenges to the party placing the concrete, such as decreased workability, decreased set time, and drying and thermal shrinkage cracking. Temperature also has a significant impact on the rate of strength gain within concrete. Mehta and Monteiro (2014) say that the higher the placement temperature, the more rapid the gain of strength which, if placed correctly, typically increases early-age strength, but has a negative effect on long-term strength. During this project, the placement temperature was monitored for each cast. The fresh concrete temperature was taken upon arrival of the ready-mixed concrete truck using a thermometer and recorded, while the ambient temperature data for the day was collected from the Auburn-Opelika Airport from Weather Underground (2014). These temperature data can be seen below in Table 4-1.

Table 4-1: Temperature Data for All Casts

Cast	Date	Maximum Air Temperature (°F)	Mean Air Temperature (°F)	Low Air Temperature (°F)	Fresh Concrete Temperature (°F)
RG4000CA	7/2/2013	84	76	68	93
LS4000PC	7/11/2013	81	76	70	86
RG4000PC	8/1/2013	88	80	72	89
RG4000SC	9/3/2013	88	80	72	88
GR4000PC	9/12/2013	88	77	66	86
RG8000PC	9/24/2013	70	67	64	86
RG4000FA	10/15/2013	73	65	57	85
LS8000PC	7/17/2014	82	72	61	94

4.3 Effect of Cylinder Size

For many decades, the 6×12 in. cylinder has been the standard for testing the quality of concrete that has been delivered to site. These cylinders are often broken at 28 days after concrete has been placed to determine the adequacy of the concrete which was placed. Though the strength of moist-cured molded cylinders does not give the best indication of the actual in-place strength, it does show if the concrete which was delivered to the site is of adequate quality. If these cylinders break below the minimum required strength specified in the design, in-place testing must be done. Though the 6 × 12 in. cylinder has been the standard for many years, Day and Haque (1993) believe that similar results can be obtained using a smaller cylinder size. Day and Haque (1993) also state that there are many advantages to using a smaller cylinder size, including ease of handling, ease of transport, and smaller required compression machines.

In this study, three cylinders of each size were tested at 28, 42, 91, and 365 days for each slab that was placed to determine if cylinder size affected average cylinder strength. The average cylinder strength for each set of molded, moist-cured cylinders was calculated. The relationships between the 6×12 in. and 4×8 in. cylinder strengths from the normal-strength concrete and high-strength concrete were compared against each other to see if there was a statistically significant difference. To do this, the average compressive strength of the 6×12 in. molded cylinders was divided by the average compressive strength of the 4×8 in. molded cylinders to obtain a ratio of the average compressive strengths. A t-test assuming equal variance was then conducted between the ratios of the normal-strength and high-strength cylinders. This resulted in a P-value of 0.084. Since this value is greater than 0.05, this indicates that there was not a statistically significant difference in the relationship between the 6×12 in. and 4×8 in. cylinders for normal-strength and high-strength concrete at a confidence level of 95 %.

Once this was known, the data sets were combined to analyze if there was a statistically significant difference between the average strengths of the 6×12 in. and the 4×8 in. cylinders. A plot for the average compressive strengths of the 6×12 in. versus the 4×8 in. cylinders can be seen in Figure 4-1. A paired t-test was run comparing the average strength of the 6×12 in. cylinders versus the average strength of the 4×8 in. cylinders. This resulted in a P-value of 0.633. Since this value is higher than 0.05, it was concluded that there is no statistical difference between the compressive strengths obtained from 6×12 in. and 4×8 in. molded, moist-cured cylinders at a 95 % confidence level.

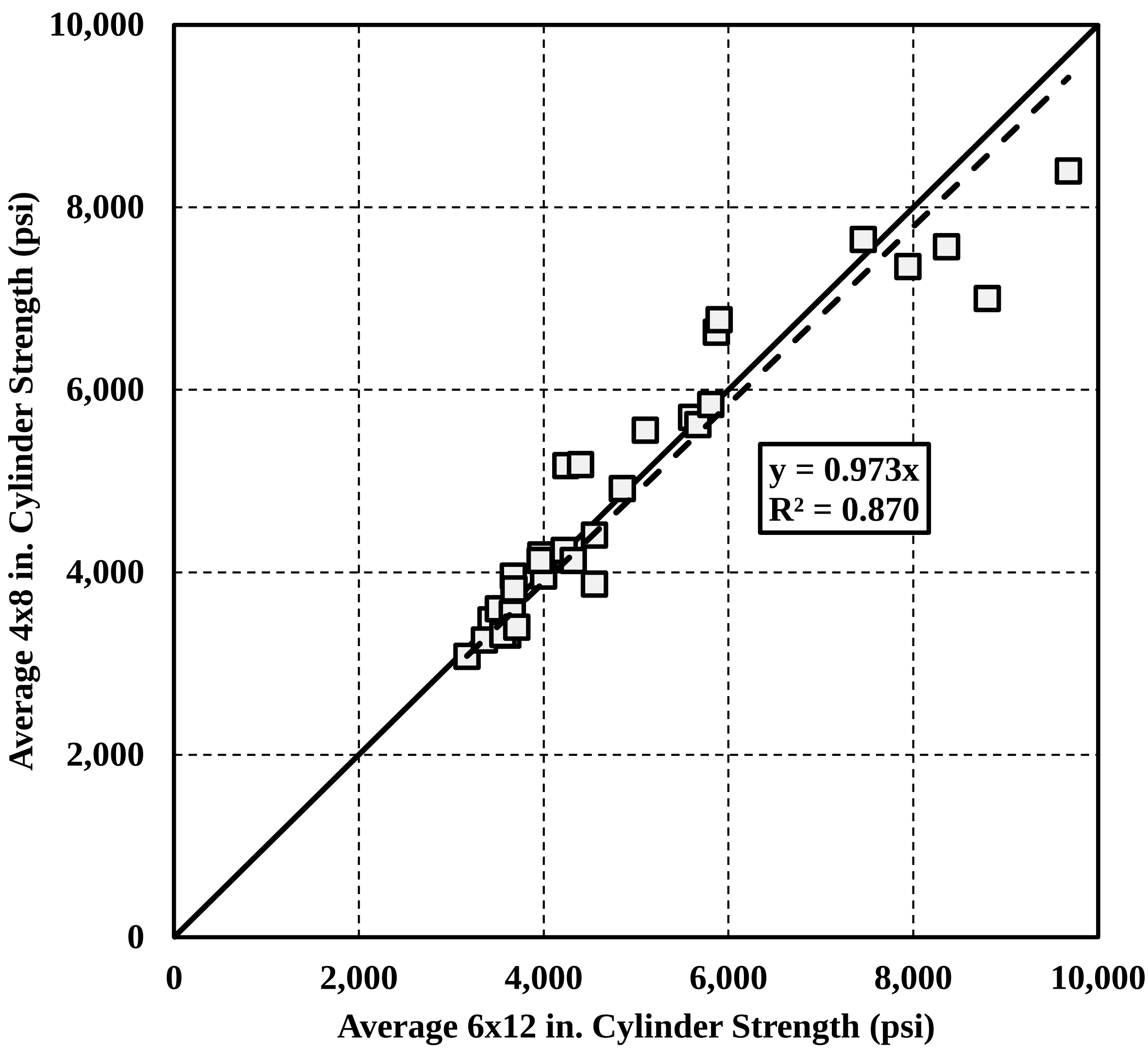

Figure 4-1: Comparison of the Average Strengths of 6×12 in. Cylinders versus 4×8 in. Cylinders

It can be seen in Figure 4-1 that there is a clear trend between the average strengths of the 6×12 in. and 4×8 in. molded, moist cured cylinders. This indicates that cylinder size does not have an effect on the average strength produced by the test. This is in agreement with the findings of Day and Haque (1993). This also indicates that when the strengths of cores and molded, moist cured 6×12 in. cylinders are compared, any difference that is found in strength cannot be attributed to the physical difference in size of the specimens.

4.4 Verification of Pullout Table Provided by Germann Instruments

For each time the LOK-Test pullout machine is calibrated, Germann Instruments provides a table which gives values for compression strength based on the pullout force. This table converts the pullout force in kilonewtons (kN) and equates it to a specified compressive force in pounds per square inch (psi). In order to be assured of the accuracy of the table, testing was done to evaluate if the table provided an accurate conversion from pullout strength to equivalent cylinder strength. In order to do this, six cubes measuring 8×8×8 in. were cast with four pullout tests in each cube as described in Section 3.10. A total of eight calibration pullout tests were performed at each testing age of 28, 42, and 365 days. The eight pullout readings were each converted into an equivalent compressive strength. These equivalent compressive strengths were then averaged to obtain an average pullout compressive strength. Similar to the analysis between the 6×12 in. and 4×8 in. cylinders, it was desired to combine the data into one set containing data from both the normal-strength and high-strength cubes. To do this, the average 6×12 in. molded cylinder strength was divided by the average compressive strength of the cube pullouts. A t-test assuming equal variance was then used to determine if there was a difference in the relationship between the 6×12 in. cylinders and the pullout calibration cubes for the normal-strength and high-strength concretes.

This t-test resulted in a P-value of 0.414 which indicates that there was no difference between the cylinder-cube relationship and that the two data sets could be combined. A scatter plot of average calibration pullout strength versus 6×12 in. cylinder strength was created in order to fit a trend line through the data points to verify the accuracy of the calibration table provided by Germann Instruments. Figure 4-2 shows a plot of average 6×12 in. molded cylinder strength versus average compressive strength obtained from the calibration pullout tests.

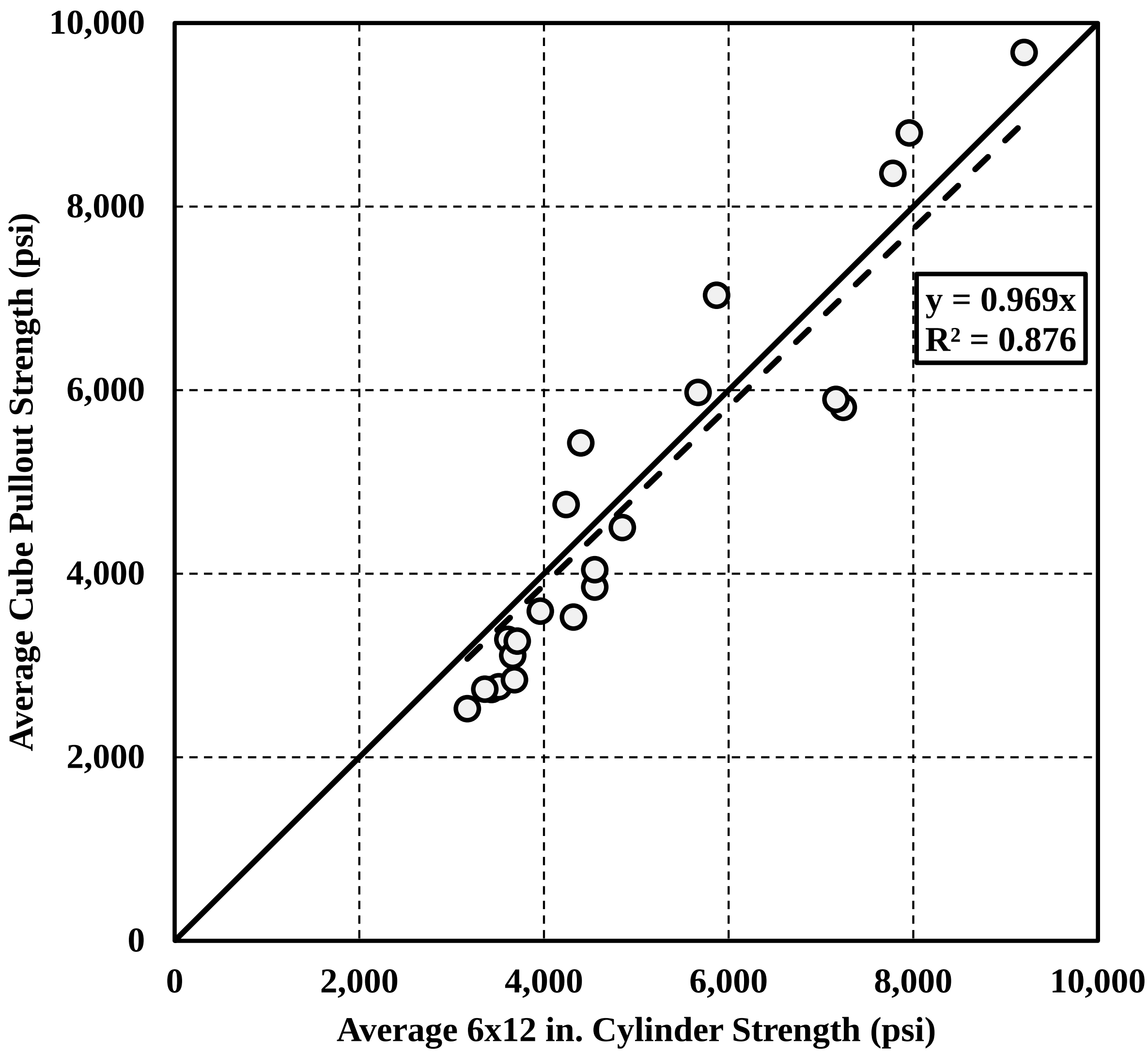

Figure 4-2: Average 6×12 in. Cylinders versus Average Calibration Pullout Cube Strength

Once the data was combined, a paired t-test was done between the average strength of the 6×12 in. cylinders and the pullout calibration cubes. This paired t-test resulted in a P-value of 0.104. This meant that the pullout calibration chart which was supplied by Germann Instruments could be used without any adjustments to the values. As can be seen in Figure 4-2, the average of the cube pullout strengths was approximately 97% of that of the 6×12 in. cylinders. This was determined to be within the range of error for the pullout tests and therefore the strengths provided in the calibration table from Germann Instruments were used.

4.5 Effect of Aggregate Type

In today's concrete industry, many different types of coarse aggregate are used. Most of the time, aggregate selection is most influenced by local availability. In east Alabama, several aggregate types are locally available. In this study, slabs containing uncrushed river gravel, crushed limestone, and crushed granite were used. A No. 67 gradation was used for all aggregates. The purpose of using different aggregate types was to determine if aggregate type impacted the apparent strength of cored specimens relative to the other means used to assess the strength. Specimens from exterior and interior regions were kept separate when performing the statistical analysis to determine if aggregate type influenced the difference in strength between in-place strength and cylinder strength test methods for different restraint conditions.

4.5.1 Effect of Aggregate Type on Strength of 6 × 12 in. versus 4 x 8 in. Molded Cylinders

In order to determine if aggregate type had a significant impact on the strength of 6×12 in. and 4x8 in. cylinders, a statistical analysis was performed. The average strength of 6×12 in. cylinders from a given age and cast was divided by the average strength of the corresponding 4×8 in. cylinders in order to obtain a normalized value which would be comparable without being influenced by strength gain over time. After this was done, values were separated into groups based on coarse aggregate type. An analysis of variance (ANOVA) analysis was then performed on the data to determine if coarse aggregate type had an impact on the relationship between 6×12 in. cylinders versus 4×8 in. cylinders. The ANOVA analysis yielded a P-Value of 0.142. Since this value is less than 0.05, this shows that coarse aggregate type did not have a significant impact on the relationship between the compressive strengths of 6×12 in. versus 4×8 in. molded, moist cured cylinders at a 95% confidence level.

4.5.2 Effect of Aggregate Type on In-Place Strength Test Method

A statistical analysis was completed in order to determine if aggregate type influenced the apparent strength of different in-place testing methods. For this analysis, each in-place specimen strength was divided by the average 6×12 in. molded cylinder strength of the same age so as to normalize the strength gain present in a slab over time. After this, the normalized values were separated by aggregate type and an ANOVA analysis was done for each in-place test type. Specimens from exterior and interior regions were kept separate during this analysis in order to ensure that restraint did not have an effect on the outcome of the ANOVA analysis. A summary of the P-values from the ANOVA analyses are presented in Table 4-2.

Table 4-2: Summary of P-values from ANOVA Analysis for In-Place Testing

Test Method	P-value From ANOVA Analysis
Exterior Cores	9.45×10^{-5}
Interior Cores	0.023
Exterior Pullouts	0.009
Interior Pullouts	1.80×10^{-7}

Since all P-values were less than 0.05, it was found in the normal strength concretes that aggregate type was significant for both the exterior and interior specimens for cores as well as the pullout tests at a 95 % confidence level. An analysis was not performed on the cast-in-place cylinders as there were not data for each type of aggregate used.

After this, the average ratios of in-place strength to molded cylinder strength were calculated for each test method. The results for normal-strength specimens can be seen below in Table 4-3.

Table 4-3: Summary of In-Place Strength to Molded Cylinder Strength Ratios by Coarse Aggregate Type for Normal-Strength Concrete

	RG	**LS**	**GR**
Exterior Cores	0.972	0.852	0.908
Interior Cores	0.919	0.861	0.837
Exterior Pullout	1.071	1.083	0.896
Interior Pullout	0.964	1.093	0.724

From this table, it can be seen that aggregate type had a different effect on each type of testing. It can be seen that the river gravel had the highest core-to-molded cylinder strength ratio while the limestone slab had the highest pullout-to-molded cylinder ratio. It should also be noticed that the in-place specimens from the granite slab had the lowest overall relative strength to their molded cylinder counterparts of the three aggregate types used in the study. Since only one granite and one normal-strength limestone slab were tested during the project, it would be recommended that more testing be done on similar limestone and granite specimens to obtain a more conclusive result.

4.6 Effect of Restraint

The location of an in-place test with respect to the member from which it is being performed has a significant effect on the amount of restraint which the concrete is exposed to. In theory, concrete which is close to the edge of a member is less restrained against movement and therefore less prone to developing microcracks within the microstructure of the concrete. Conversely, concrete which is not near an edge of a member is more restrained and theoretically more likely to develop microcracks. Microcracking increases permeability and lowers concrete strength. Once microcracks are formed, they can develop into larger cracks, which can lead to failure. From this, it can be assumed that microcracking can have an effect on the apparent in-place strength. Therefore, if the restraint that the concrete is subjected to causes significant microcracking, then the amount of restraint present will have an effect on in-place strength.

In order to determine if restraint has an effect on in-place strength, in-place testing was conducted near the edge of each slab as well as in the middle of each slab. The testing near the edge of the slab represented the low restraint condition while testing at the middle of each slab represented the higher level of restraint condition. Since the cast-in-place cylinders were contained within metal sleeves which were used as the support system, it was anticipated that level of restraint would play a less significant role in the apparent strength of these specimens simply because the cast-in-place specimens had room to freely expand and contract.

A statistical analysis was completed on the collected data to determine if restraint had a significant effect on apparent in-place strength. In order to normalize all the data, all in-place test strengths were divided by the average molded 6×12 in. cylinder strength from that cast at the age it was tested. Once these ratios were obtained, equal variance t-tests were performed to determine if there was a difference in strength between the normal-strength and high-strength specimens with respect to the effect of restraint. To do this, separate equal-variance t-tests were conducted on the cylinder-to-in-place test strength ratios for both the exterior and interior specimens. The P-values for the equal-variance t-tests can be seen in Table 4-4.

Table 4-4: P-values for Strength Level t-test

	P-values for t-test Comparing in-place Specimens of Normal-Strength and High-Strength Concrete	
	Exterior	**Interior**
Cores	0.341	0.978
Cast-in-Place Cylinders	0.936	0.980
Pullout Tests	0.278	6.67×10^{-5}

From Table 4-4, it can be seen that all the P-values except one was above 0.05. Because of this, it was determined that the strength level did not have a significant impact on the effect of restraint. Because of this, the data from the normal-strength and high-strength casts were combined within each respective test type.

After this was done, an analysis was performed to determine if restraint impacted the relationship between apparent in-place strength and 6×12 in. molded, moist-cured cylinder strength. This was done by doing an equal-variance t-test between the in-place-to-molded cylinder strength ratios for each in-place testing type. The P-values for the t-tests can be seen in Table 4-5.

Table 4-5: P-values for t-tests Determining the Effect of Restraint

	P-values for t-test
Cores	0.005
Cast-in-Place Cylinders	0.502
Pullout Tests	0.040

From the results of the equal-variance t-tests, it can be seen that both the cores and the pullout tests were effected by restraint while the cast-in-place cylinders were not. This was to be expected as the cast-in-place cylinders have some room to freely expand and contract due to the support system which holds them in-place. Plots containing exterior versus interior strength data for cores and pullouts were also constructed to better show the relationship between the interior and exterior specimens and can be seen in Figures 4-3 through 4-5.

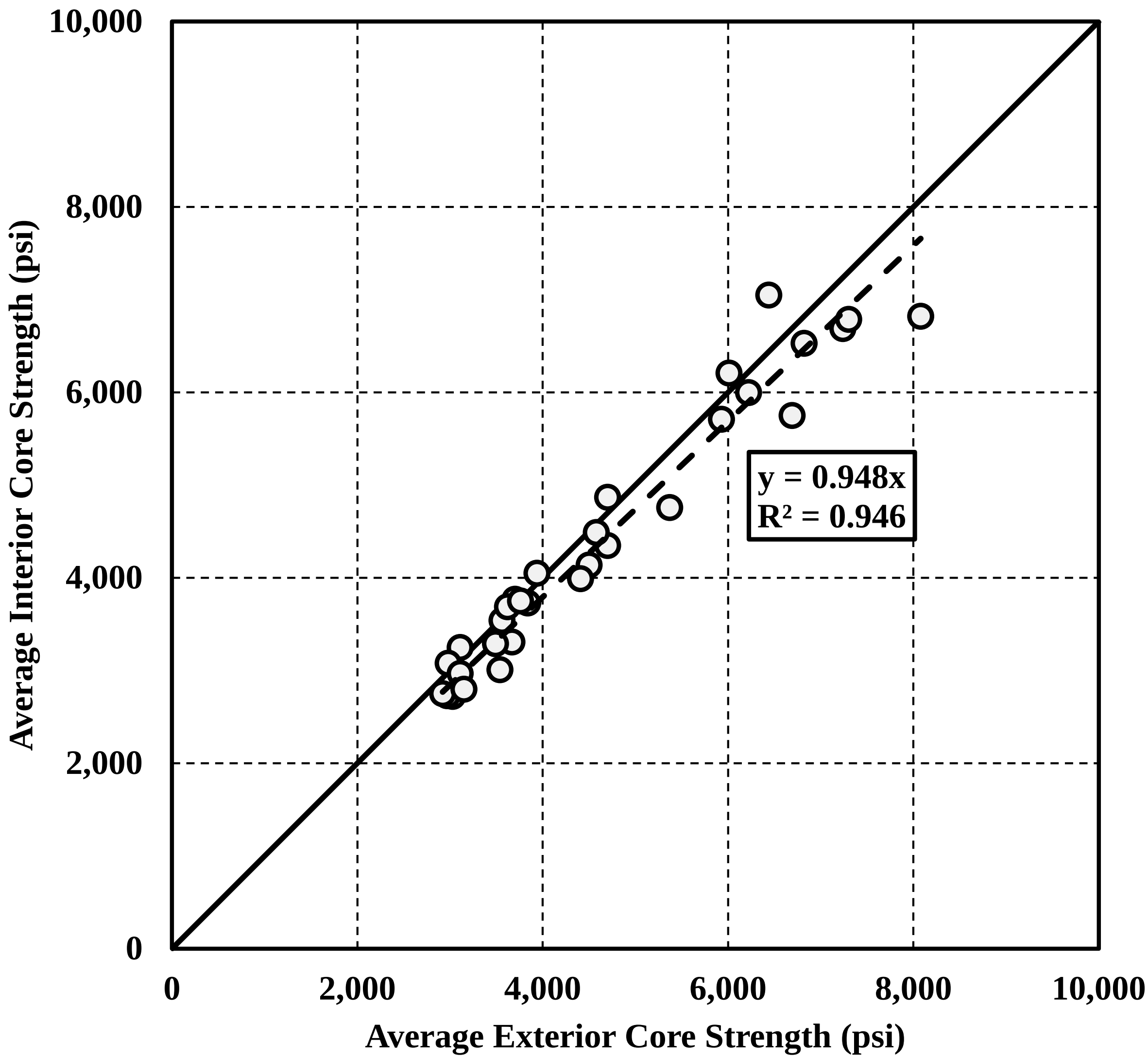

Figure 4-3: Average Exterior Core Strength versus Average Interior Core Strength

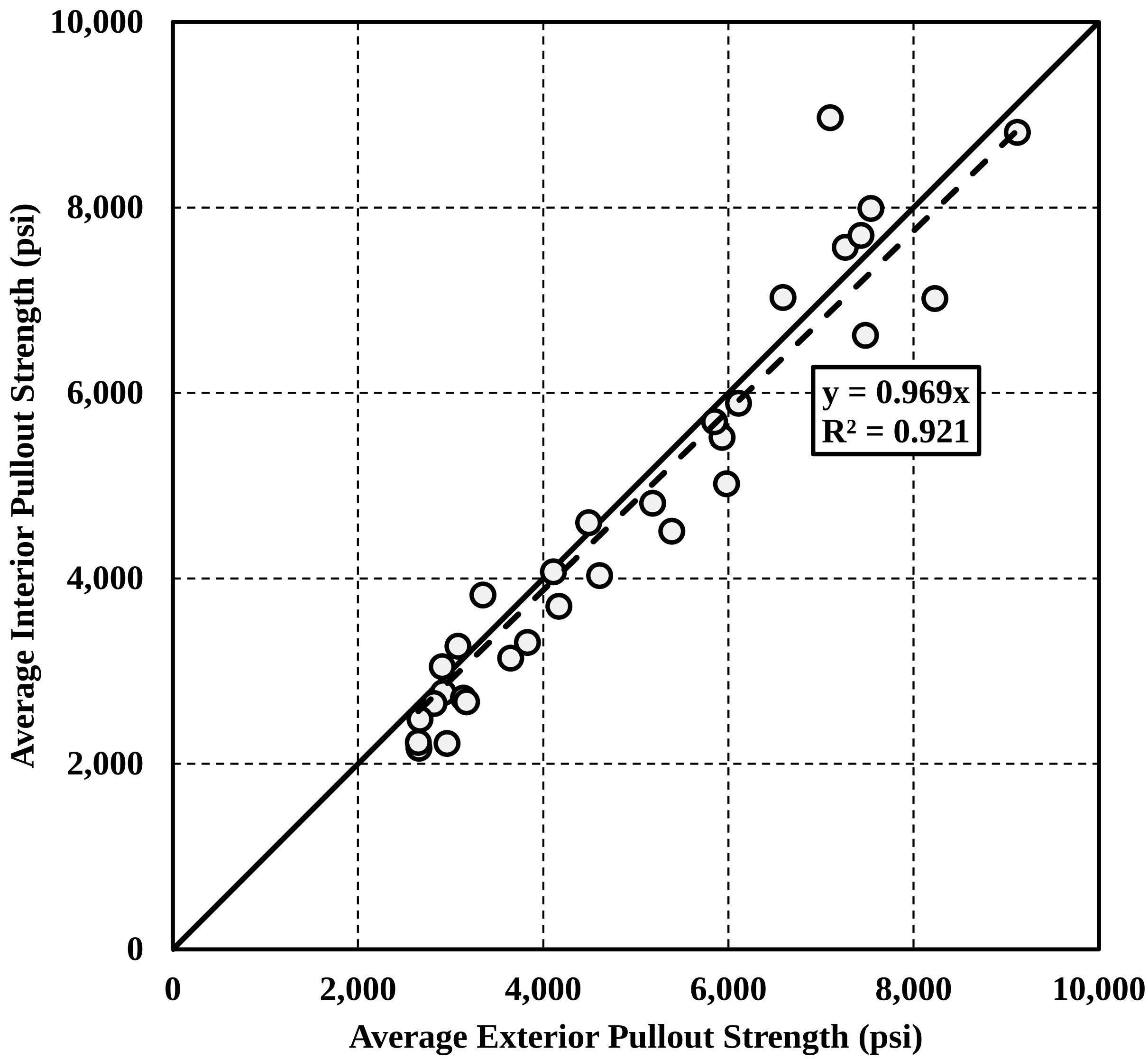

Figure 4-4: Average Strength of Exterior Pullout Tests versus Average Strength of Interior Pullout Tests

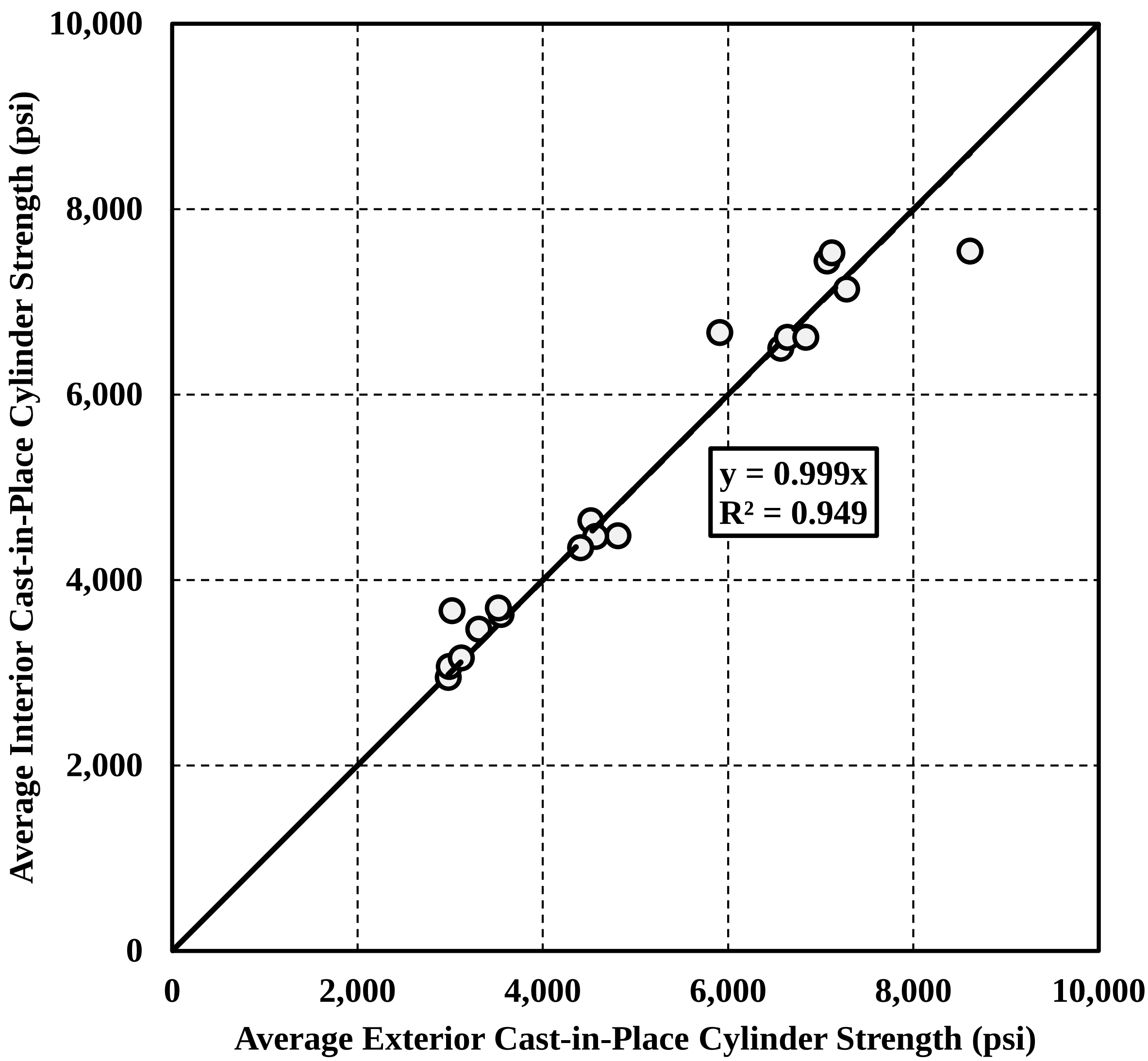

Figure 4-5: Average Strength of Exterior Cast-in-Place Cylinders versus Average Strength of Interior Cast-in-Place Cylinders

From the figures above, it can be seen that restraint has an impact on the apparent strength of the slabs for the cores as well as the pullout tests, but not for the cast-in-place cylinders. The strength of the interior cores averaged to be approximately 94.8 % of that of the exterior cores. Similarly, the strength of the interior pullouts averaged about 96.9 % of the exterior pullout strength. This was to be expected because of the idea that the middle specimens had a higher degree of axial restraint and therefore had a higher degree of microcracking which would, in turn, lower the overall compressive strength results.

4.7 Effect of Supplemental Cementing Materials and Strength Gain Over Time

Partial replacement of cement with SCMs is a common practice in today's concrete industry. Since these SCMs may have an effect on the strength development of in-place concrete, relationships must be developed in order to establish age correction factors which can be applied to in-place specimens test results at any age to convert them to a representative 28-day strength. In order to evaluate the in-place strength from core testing, the core strength must be converted to a 28-day core strength, and then be compared with 6×12 in. molded cylinder strength. Yazdani and McKinnie (2004) developed different equations to correct for the strength gain in in-place specimens based on moist-cured, molded cylinders, which can be seen in Section 2.2.1. These equations, which are now used in practice by the Florida DOT, produce age-correction factors for different concretes that contain different SCMs as well as different cement types. ACI 209.2R (2008) also presents a method for correcting concrete strength results to account for strength gain due to age. This method uses Equation 4-1 with differing constant values for different cement types. The constants for different cement types can be seen in Table 4-6. No consideration is given to partial replacement of cement using SCMs.

$$f'_c(t) = f'_c(28) \times \left(\frac{t}{a+\beta \times t}\right) \quad \text{Equation 4-1}$$

Where:

t = time since casting (days)

a = empirical constant from Table 4-5 (unitless)

β = empirical constant from Table 4-5 (unitless)

Table 4-6: Constants for ACI 209 Age Correction Equation

Cement Type	ACI 209 Empirical Constants for Equation 4-1	
	a	$\boldsymbol{\beta}$
Type I	4	0.85
Type III	2.3	0.92

In this study, the strength gain of concretes containing Class C fly ash, Class F fly ash, and slag cement was compared to the strength gain of the control slab which contained only portland cement. To numerically compare different SCMs, the ratio of the average strength at a certain age to the average 28-day strength of each respective test method was evaluated. The strength development plots can be seen below in Figures 4-6 through 4-9 for Type I portland cement, Class C fly ash, Class F fly ash, and slag cement, respectively. Since it was found that restraint does have an impact on in-place strength, only the interior specimen trend lines are shown on Figures 4-6 through 4-9 as these represent the worst case scenarios for in-place testing with respect to restraint. The strength gain models developed by Yazdani and McKinnie (2004) as well as the strength gain equation found in ACI 209.2R (2008) are also shown on Figures 4-6 through 4-9.

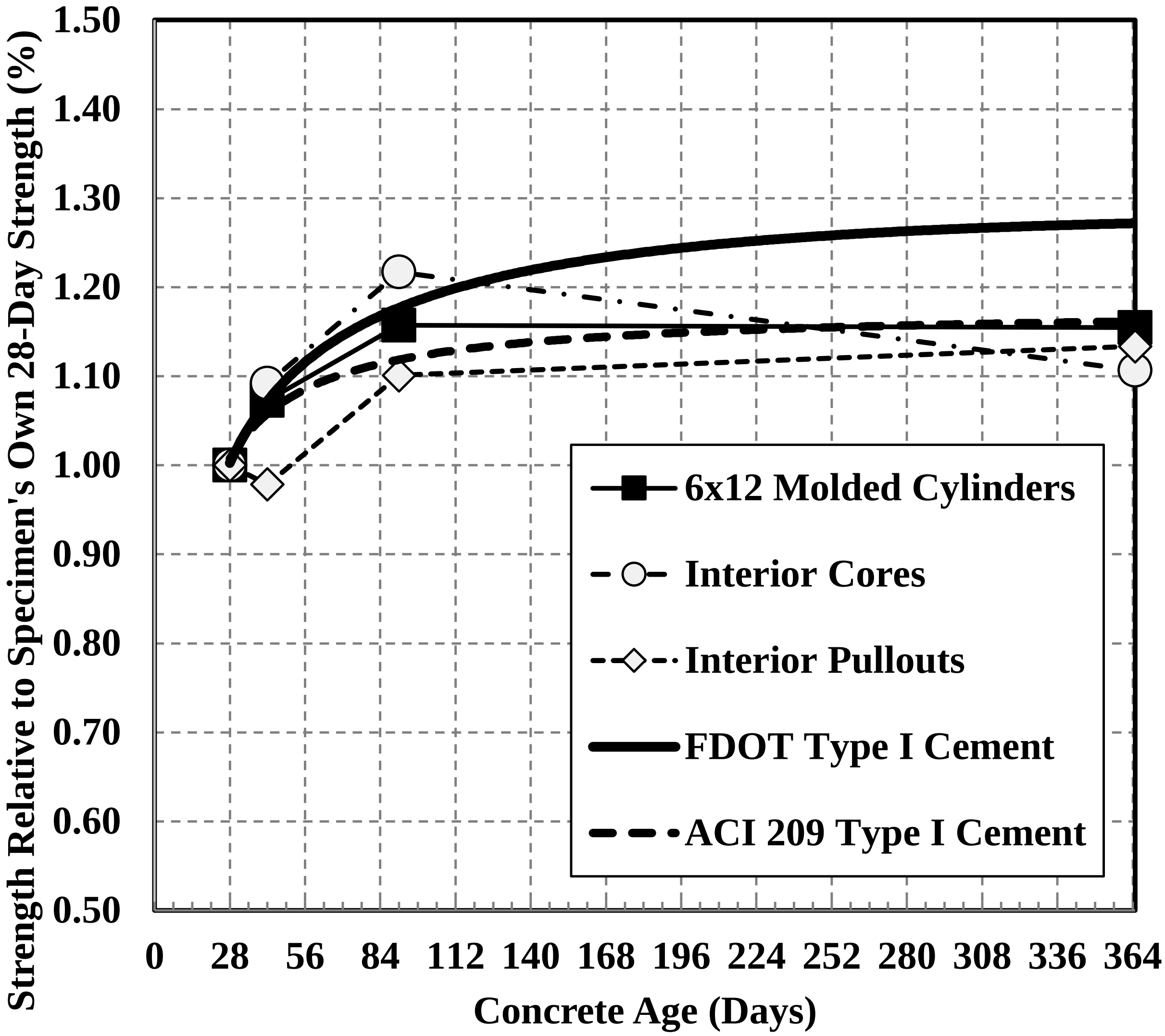

Figure 4-6: Strength Gain of Concrete with Only Type I Portland Cement

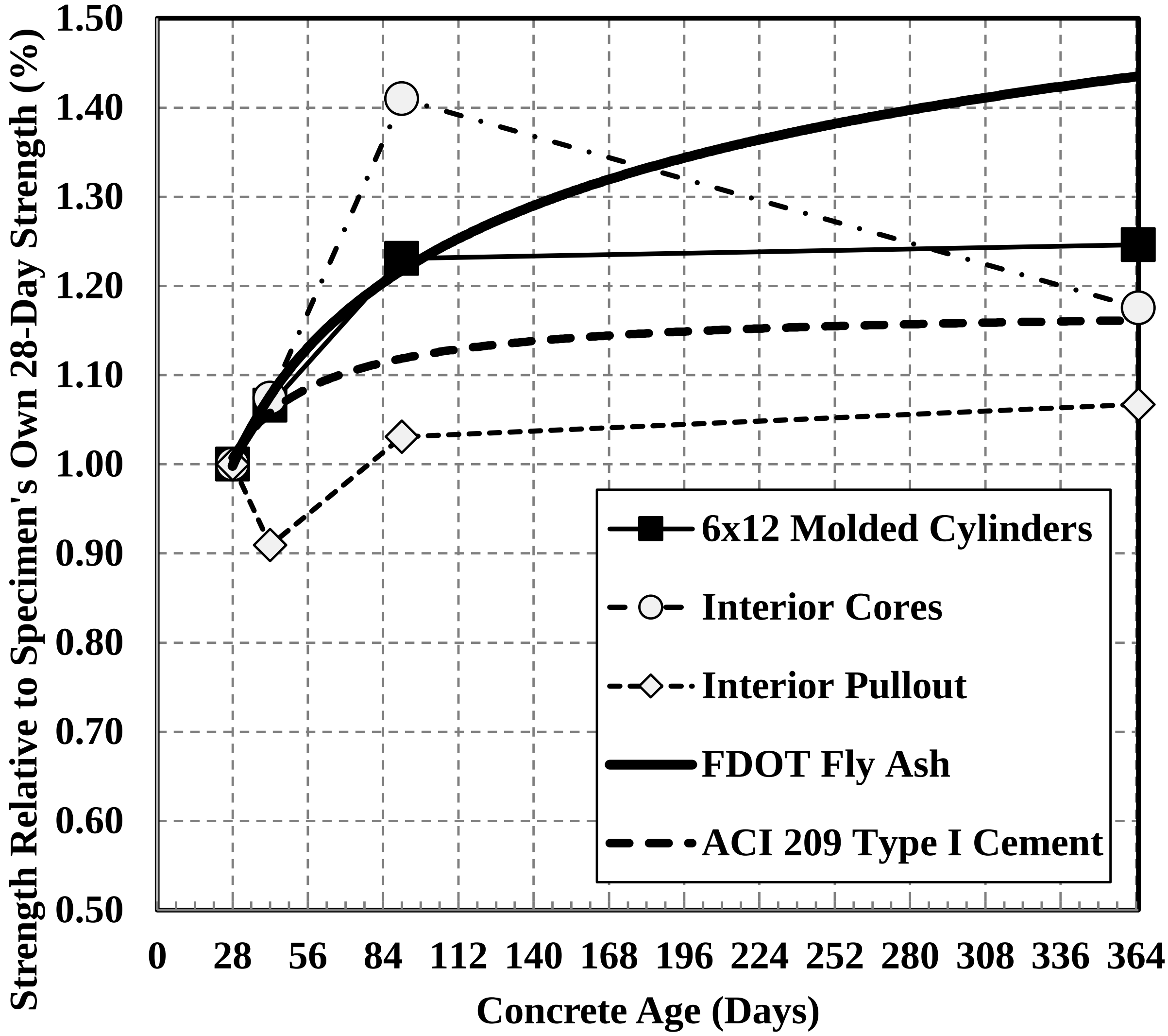

Figure 4-7: Strength Gain of Concrete with 20% Class C Fly Ash

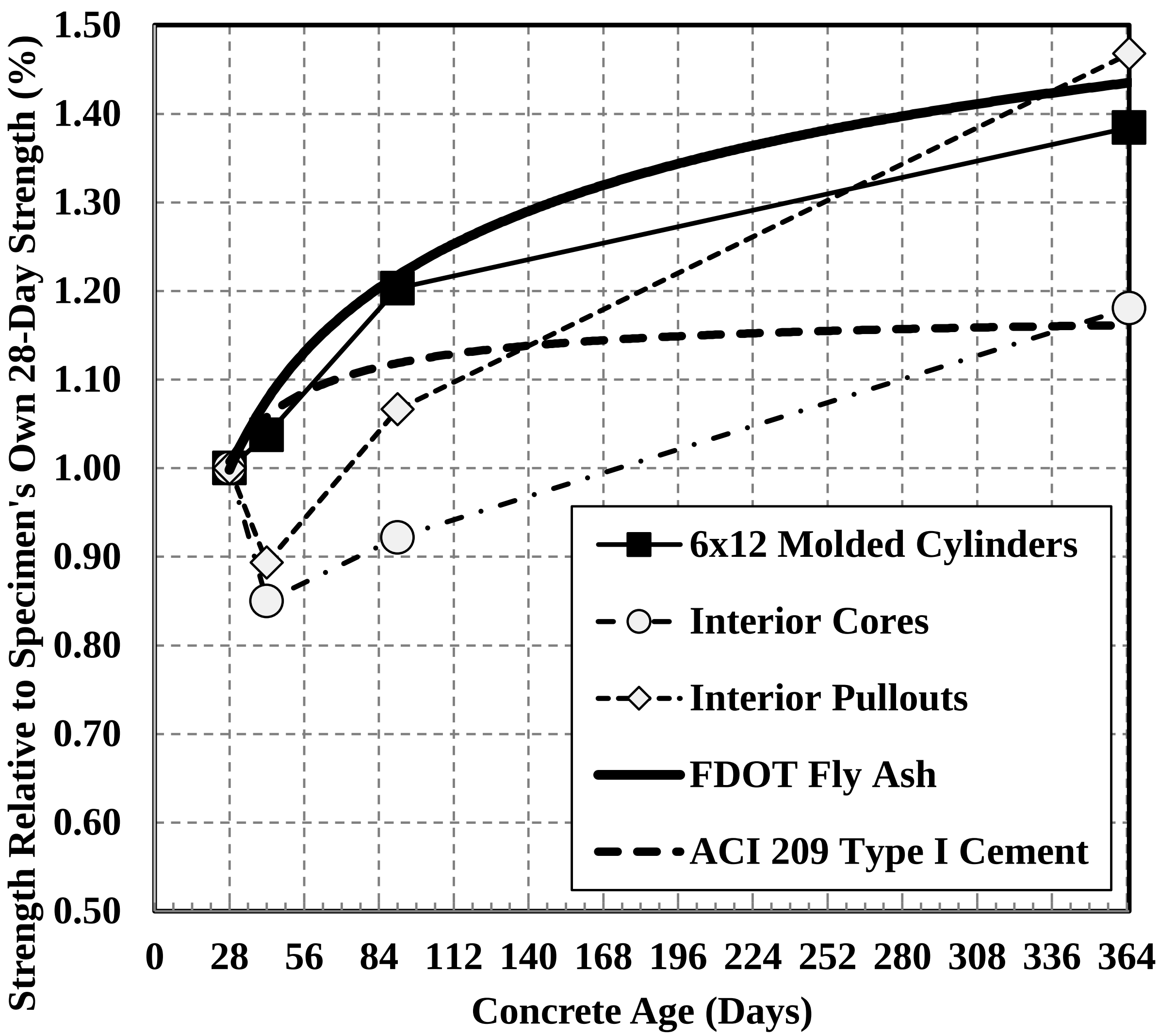

Figure 4-8: Strength Gain of Concrete with 20% Class F Fly Ash

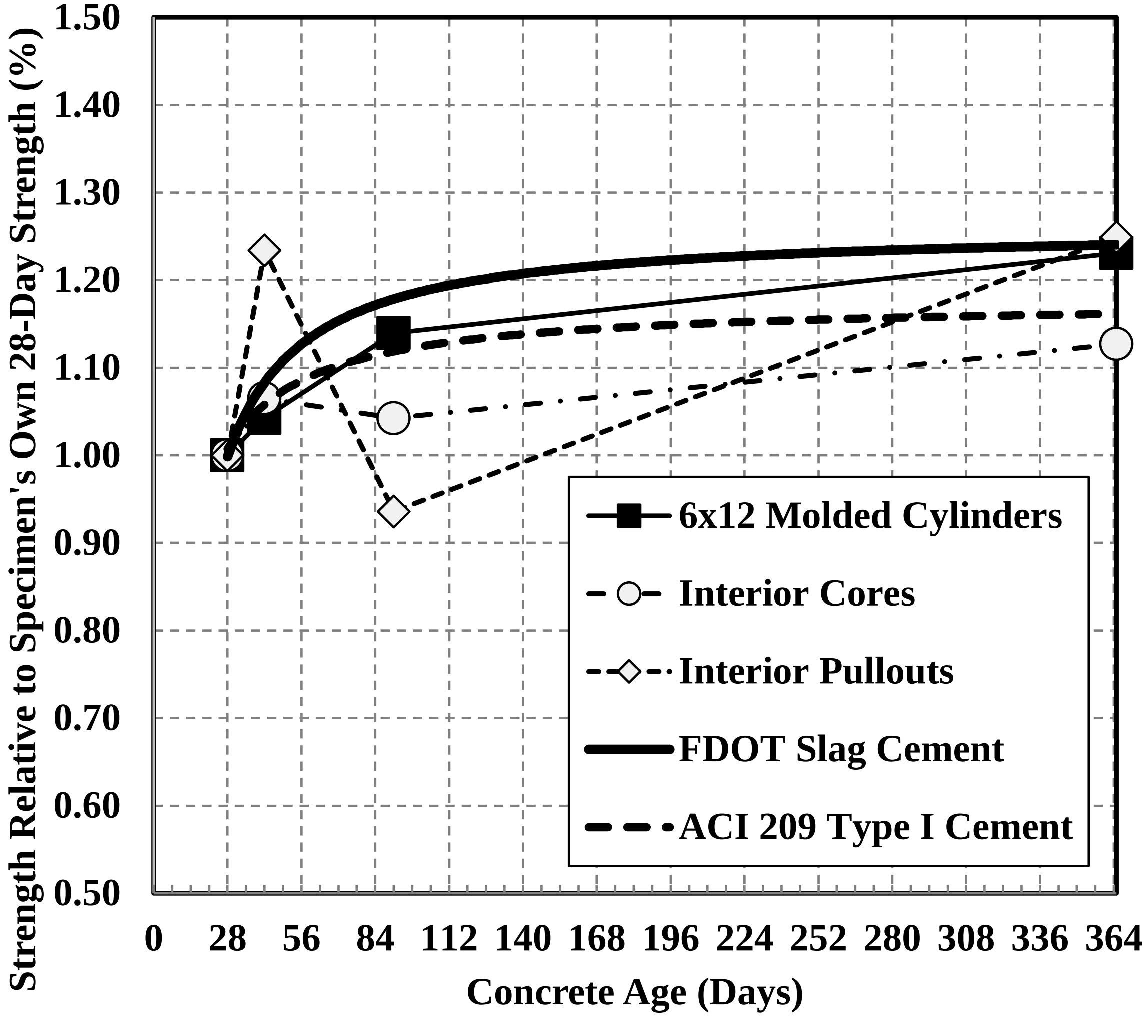

Figure 4-9: Strength Gain of Concrete with 50% Slag Cement

As can be seen from Figures 4-6 through 4-9, the strength development of the pullout tests were far more variable than any other test. This was expected for multiple reasons. First, the pullout test only tests a very small area of concrete, which can be heavily influenced by the aggregate formation and paste strength around the insert. Secondly, the pullout test only assesses surface concrete, which loses moisture the quickest. These two facts make the pullout test a much more variable test than core testing or cast-in-place cylinders.

A statistical analysis was performed on the data to determine the effect that the SCMs had on the overall strength gain over the first year after placement. For both the cores and pullouts as well as the 6×12 in. molded cylinders, an ANOVA analysis at a 95% confidence level was completed to determine if the difference in cementing materials and moist versus field curing caused a different rate of strength gain over time. The results of the ANOVA analyses can be seen in Table 4-7.

Table 4-7: Summary of P-Values from ANOVA Analysis of Strength Gain for Different SCM Types

Specimen Type	P-Value from ANOVA Analysis
6x12 Cylinders	0.804
Interior Cores	0.229
Interior Pullouts	0.755

It can be seen from the ANOVA analysis results that since none of the P-values are less than 0.05, the post 28-day strength gains of all the different types of cementing materials were not significantly different from one another.

After the ANOVA analyses were run comparing all the cementing materials against one another, a three separate analyses were done to see if there was a statistical difference present between the slab which contained only portland cement and each of the slabs which contained different SCMs. This was done in order to determine if there was a statistical difference between portland cement and each individual type of SCM. To do this, a paired t-test was conducted on the ratios of strength at a given time to the strength at 28 days. The results from these paired t-tests can be seen in Table 4-8.

Table 4-8: P-Values of Paired t-tests Conducted on Portland Cement Specimens Versus Specimens Containing Supplemental Cementing Materials

Test Type	Comparison	P-Value from t-test
6x12 Cylinder	Cement vs. C Ash	0.224
	Cement vs. Slag	0.806
	Cement vs. F Ash	0.415
Interior Cores	Cement vs. C Ash	0.315
	Cement vs. Slag	0.412
	Cement vs. F Ash	0.312

Since none of these P-Values were less than 0.05, it was concluded that the data collected for the project show that strength gain was not statistically different between the slab containing only Type I portland cement and the other slabs which contained SCMs. Because of this, one model could be used to represent the strength gain regardless of cementing material type.

The data collected from this study were then compared against two different models for predicting strength gain in hardened concrete: the ACI 209.2R (2008) equation and the Florida DOT equations developed by Yazdani and McKinnie (2004). The strength gain for each in-place testing method as well as each size of molded cylinders were evaluated separately in order to obtain error values for each test method. Since it was proven that the SCM type does not statistically affect strength gain at a 95% confidence level, all specimens of the same test type were grouped together. For example, molded 6×12 in. cylinders from casts RG4000PC, RG4000CA, RG4000FA, and RG4000SC were analyzed together in order to produce a more conclusive result. In order to do an unbiased statistical analysis, ratios of test average at a certain age were divided by the average strength of that test at 28 days in order to produce a value which solely represented strength gain within the testing method for a given cast. In order to determine which function represented the best fit, the collected strength gain ratios were statistically analyzed against strength correction factors obtained from ACI 209 and FDOT. The unbiased estimate of the standard deviation was found for each of the two relationships with respect to the collected data. This was found using Equations 4-2 and 4-3 from McCuen (1985).

$$S_j = \sqrt{\frac{1}{n-1} * \sum_i^n \Delta_i{}^2} \quad \text{Equation 4-2}$$

Where,

S_j = the unbiased estimate of the standard deviation (percent),

n = the number of data points (unitless), and

Δ_i = the absolute error (percent).

$$\Delta_i = \frac{S_{i,e} - S_{i,m}}{S_{i,m}} * 100 \quad \text{Equation 4-3}$$

Where,

Δ_i = the absolute error (percent),

$S_{i,e}$ = the estimated value of the strength gain factor (unitless), and

$S_{i,m}$ = the measured value of the strength gain factor (unitless).

For each type of testing, an individual analysis was completed to determine if different test methods produced different strength gain relationships. The errors for each age were summed and the unbiased estimate of the standard deviation was calculated for each test method. A summary of the results can be seen below in Table 4-9.

Table 4-9: Summary of the Unbiased Estimate of the Standard Deviation for Strength Gain in Normal Strength Concrete

Test Method	FDOT Unbiased Estimate of the Standard Deviation (%)	ACI 209 Unbiased Estimate of the Standard Deviation (%)
Molded 6×12 in. Cylinders	5.99	6.69
Interior Cores	17.56	12.24
Interior Pullouts	18.05	13.20
All Data	14.51	10.77

From these results, it can be seen that the FDOT equations provided a slightly better strength gain estimation for the molded cylinders while the ACI 209 equation provided a much more accurate fit for both the core and pullout tests. The values in Table 4-8 which are listed as "All Data" are the estimates of the standard deviation of the error for the molded 6×12 in. cylinders, interior cores, and interior pullouts combined. Since an updated pay scale for ALDOT is needed for core strengths, it is recommended that ALDOT use the ACI 209 equation to account for the effect of age on the in-place strength. Therefore, recommendations were based on this equation.

After this, measured values obtained during the study were compared solely with the estimates produced by using the ACI 209 equation. For Type I cement, ACI 209 recommends using the values of $a = 4$ and $\beta = 0.85$. Using the measured strengths obtained during the study, a regression analysis was done to determine the best-fit curve to solve for the values for a and β in order to minimize the unbiased estimate of the standard deviation. The results for this analysis can be seen in Table 4-10.

Table 4-10: Adjusted a and β Values for Different Testing Methods

Test Method	a Value	β Value
Molded 6×12 in. Cylinders	6.87	0.781
Interior Cores	6.15	0.853
Interior Pullout	3.18	0.907

It can be seen from Table 4-10 that all the adjusted values of a fall between 0.05 and 9.25 and all the β fall between 0.67 and 0.98 as specified within ACI 209.2R (2008). After the adjusted a and β values were calculated, these values were used to provide an estimate of strength for each testing method. Using these adjusted a and β values, the estimated standard deviations were again calculated. These adjusted standard deviations can be seen in comparison to the unadjusted values in Table 4-11.

Table 4-11: Comparison of Adjusted and Unadjusted ACI 209 Values of the Unbiased Estimate of the Standard Deviation

Test Method	Adjusted ACI 209 Unbiased Estimate of the Standard Deviation (%)	Unadjusted ACI 209 Unbiased Estimate of the Standard Deviation (%)
Molded 6×12 Cylinders	4.43	6.69
Interior Cores	11.58	12.24
Interior Pullout	8.79	13.20

As can be seen from Table 4-10, the use of adjusted ACI 209 values for a and β do not significantly improve the unbiased estimate of the standard deviation. In order to compare the values further, the estimated versus recorded strength values for molded 6×12 in. cylinders and interior cores were plotted against one another for both the unadjusted and adjusted values of a and β and can be seen in Figures 4-10 through 4-13, respectively. Error bars were placed at ±15% due to the variability of the core testing and molded cylinders over multiple batches.

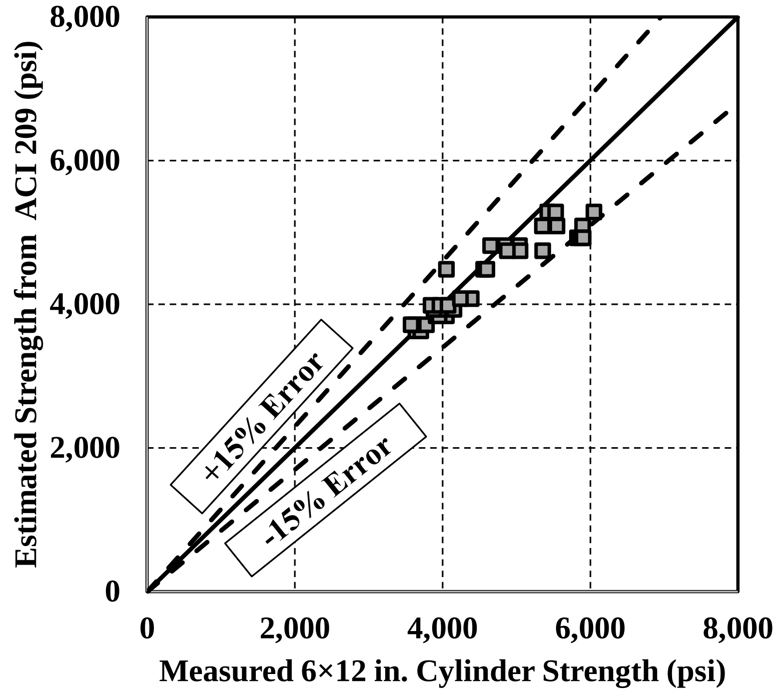

Figure 4-10: Measured Strength Versus Estimated Strength Using ACI 209 for Molded 6×12 in. Cylinders with Unadjusted *a* and *β* Values

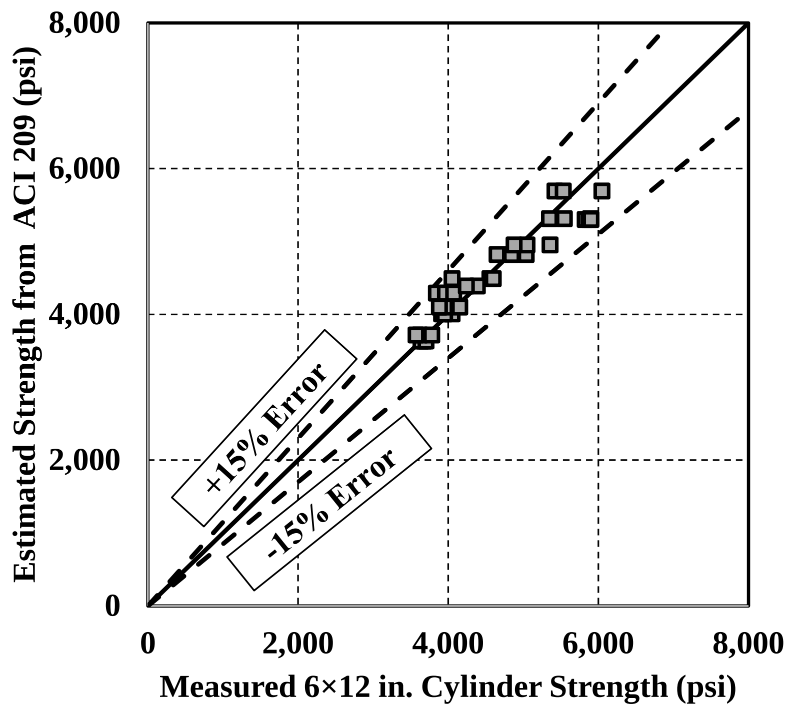

Figure 4-11: Measured Strength Versus Estimated Strength Using ACI 209 for Molded 6×12 in. Cylinders with Adjusted *a* and *β* Values

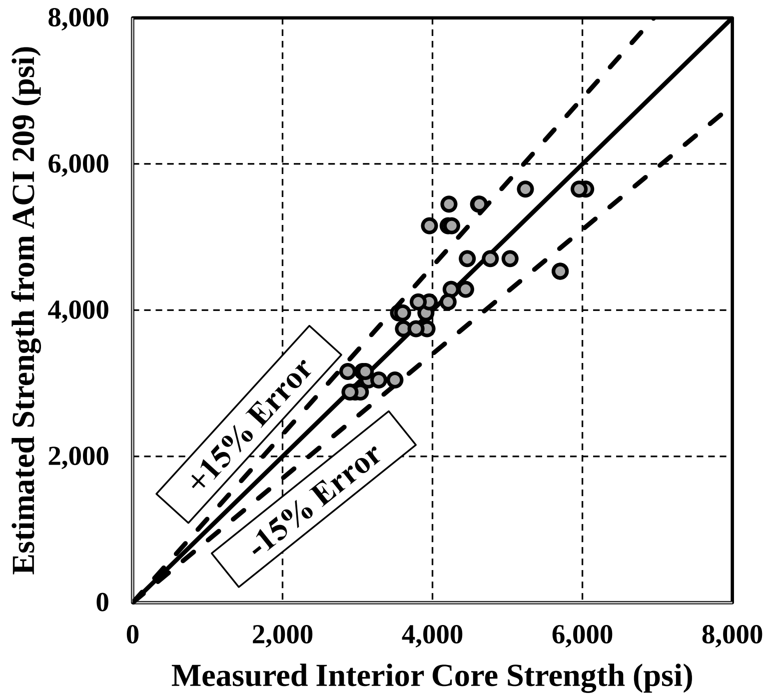

Figure 4-12: **Measured Strength Versus Estimated Strength Using ACI 209 for Interior Cores with Unadjusted *a* and *β* Values**

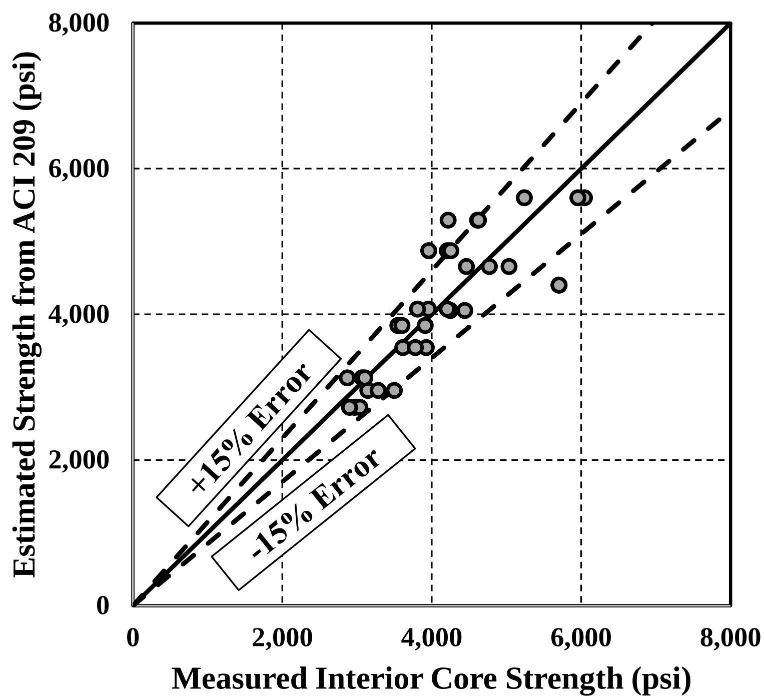

Figure 4-13: Measured Strength Versus Estimated Strength Using ACI 209 for Interior Cores with Adjusted *a* and *β* Values

From the plots shown in Figures 4-10 through 4-13 and the data shown in Table 4-10, it can be seen that there is little improvement using the adjusted values of a and β in the ACI 209 equation. Therefore, the recommended ACI 209 values of $a = 4$ and $\beta = 0.85$ were validated as providing a sufficiently accurate estimate of the effect of concrete age on the strength. The use of the ACI 209 values of $a = 4$ and $\beta = 0.85$ are therefore recommended for ALDOT use.

4.8 Comparison of Core to Molded Cylinder Strengths

In the concrete industry, the standard measure of quality control and assurance has been the average strength of moist-cured, molded 6×12 in. cylinders for many years. In the United States, the most common in-place strength testing method is core testing. When molded 6×12 in. cylinders are tested and do not meet the specified minimum strength for the project, cores are typically extracted and tested to evaluate the in-place strength of the concrete. Most state DOTs use a pay reduction scale based on the strengths obtained from core testing relative to the specified design strength. In most cases, no consideration is given to the amount of damage inflicted upon a core during the drilling process as well as the difference in curing conditions and the presence of microcracking during the payment reduction. Therefore if cores do not meet the required minimum strength, then payment is reduced for the in-place concrete. ACI 318 (2011) states that concrete strength shall be deemed structurally adequate if the average strength of three cores is greater than 85% of the specified compressive-strength (f'_c) for the project as long as no single core strength of three cores is below 75% of the required strength.

Through testing, one of the objectives of the project was to determine if there was a statistical difference between core strength and molded, moist-cured cylinder strength and, if there was, quantify this relationship. In order to do this, a procedure had to be developed in order to convert core strength at a given age back to an equivalent core strength at 28 days. The reason why a 28-day core strength is desired is to compare this value with the specified design strength of the concrete in question. After this, a relationship must be established between core strength and cylinder strength to fairly assess the adequacy of the in-place concrete and therefore create a pay scale which is fair to both the state DOT as well as the contractor. From theory, since the interior cores represented the most restrained condition, this meant that it also represented the worst case scenario in terms of compressive strength for the data collected in this study. In actual structures, the in-place restraint could exceed what was present in this study, especially in very large structural elements. Based on this, comparisons were made between the strength from both exterior and interior core specimens versus molded 6×12 in. cylinders, but conclusions were based on the comparison of interior core strength to cylinder strength to determine if there needed to be adjustments made on how to assess when a comparable level of strength is obtained from the core and cylinder samples.

In order to determine the relationship between core and cylinder strength, it first had to be determined if data from normal-strength and high-strength casts could be analyzed together. To do this, average core strengths from all ages were divided by the average strength of their corresponding 6×12 in. molded cylinders. These ratios were then compared for both interior and exterior cores. A student t-test was performed between the normal-strength ratios and the high-strength ratios for both exterior and interior cores. The results can be seen below in Table 4-12.

Table 4-12: Summary of P-Values Comparing Normal vs. High Strength Cores

	P-Value
Exterior Cores vs. 6x12" Molded Cylinders	0.431
Interior Cores vs. 6x12" Molded Cylinders	0.630
All Cores vs. 6x12" Molded Cylinders	0.375

From the P-values produced from the statistical analysis, it can be seen that there was not a statistical difference for the core-cylinder relationship for normal-strength and high-strength concrete. This means that only one relationship needed to be established for relating core strength to cylinder strength regardless of strength level for the strength range tested. Because of this, both sets of data were combined in order to form one trend from which conclusions could be drawn and recommendations made.

Once it was determined that the normal-strength and the high-strength data could be combined, an analysis was done to determine if there was a statistical difference between the strength results from cores and the results from the 6×12 moist cured, molded cylinders. To do this, two paired t-tests were conducted between the average strengths of the cores and the cylinders for both exterior and interior cores. A third t-test was done to compare all the core strengths combined regardless of restraint type against the cylinders. The results of the t-tests can be seen in Table 4-13.

Table 4-13: P-values from Core versus Molded Cylinder t-test

	P-value from t-test
Exterior Cores vs. 6 × 12 in. Molded Cylinders	0.002
Interior Cores vs. 6 × 12 in. Molded Cylinders	3.65×10^{-5}
All Cores vs. 6 × 12 in. Molded Cylinders	2.68×10^{-5}

From Table 4-13, it can be concluded that there is a statistical difference between core strength and molded, moist cured cylinder strength at a 95 % confidence level at all ages that testing occurred. To graphically show the trend between core strength and cylinder strength, plots were developed, which can be seen in Figures 4-14 through 4-16. These plots include data from all testing ages.

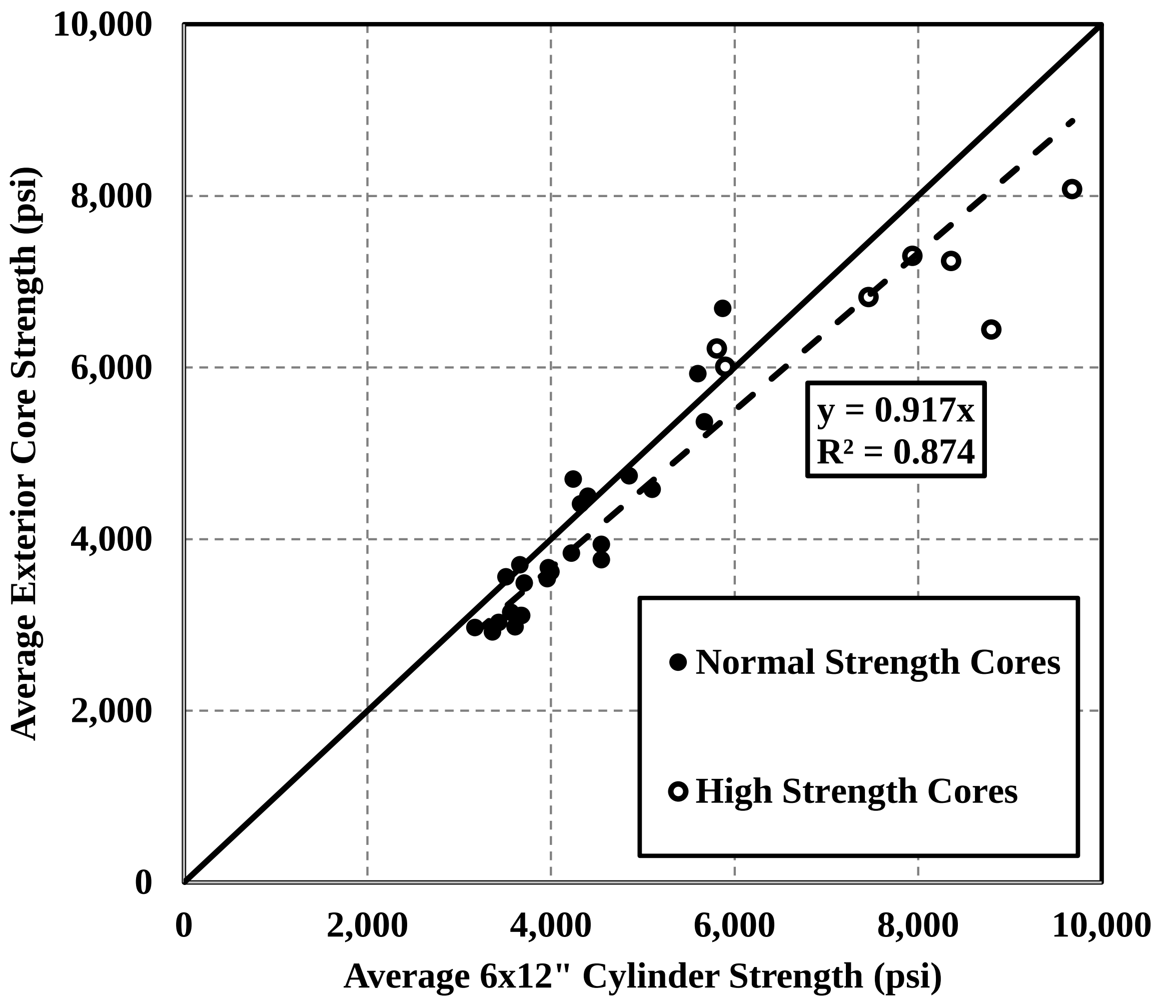

Figure 4-14: Average 6×12 in. Molded Cylinder Strength versus Average Exterior Core Strength

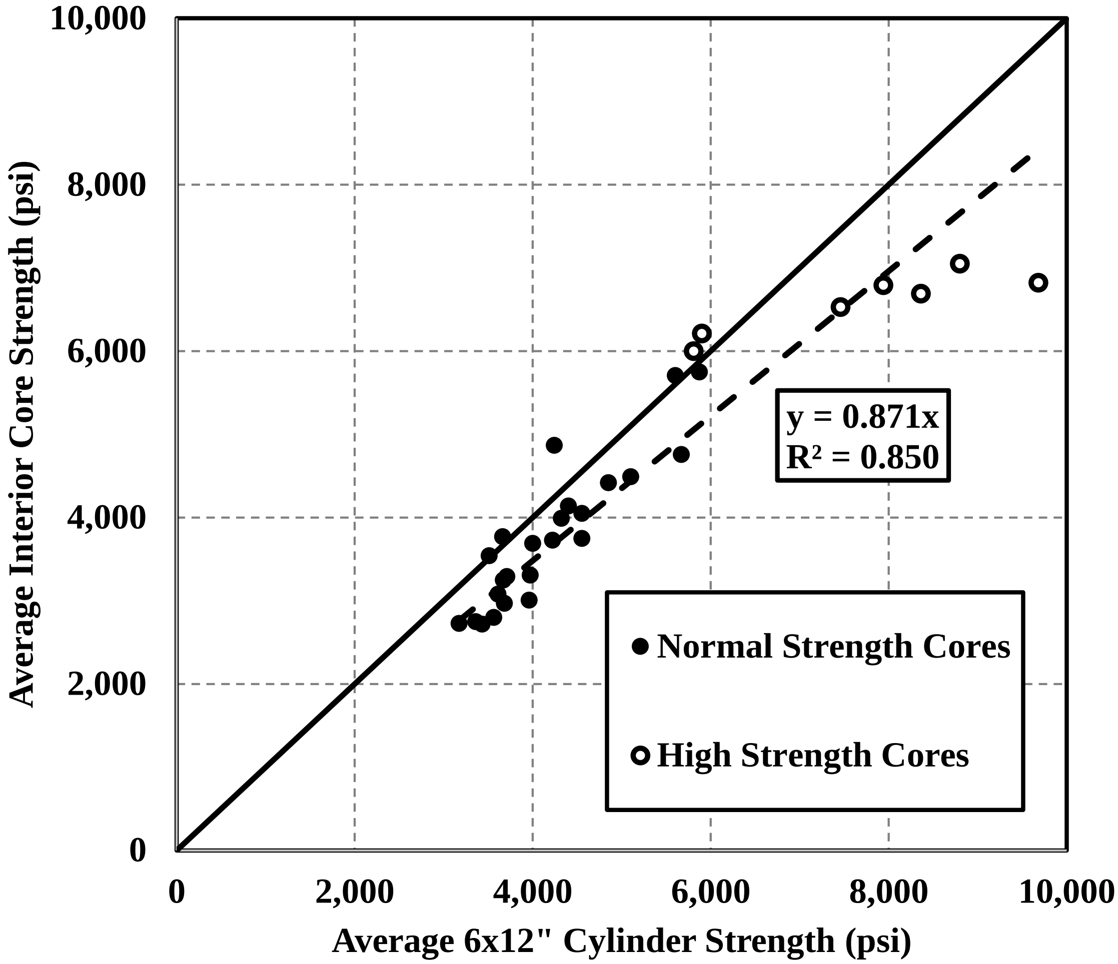

Figure 4-15: Average 6×12 in. Molded Cylinder Strength versus Average Interior Core Strength

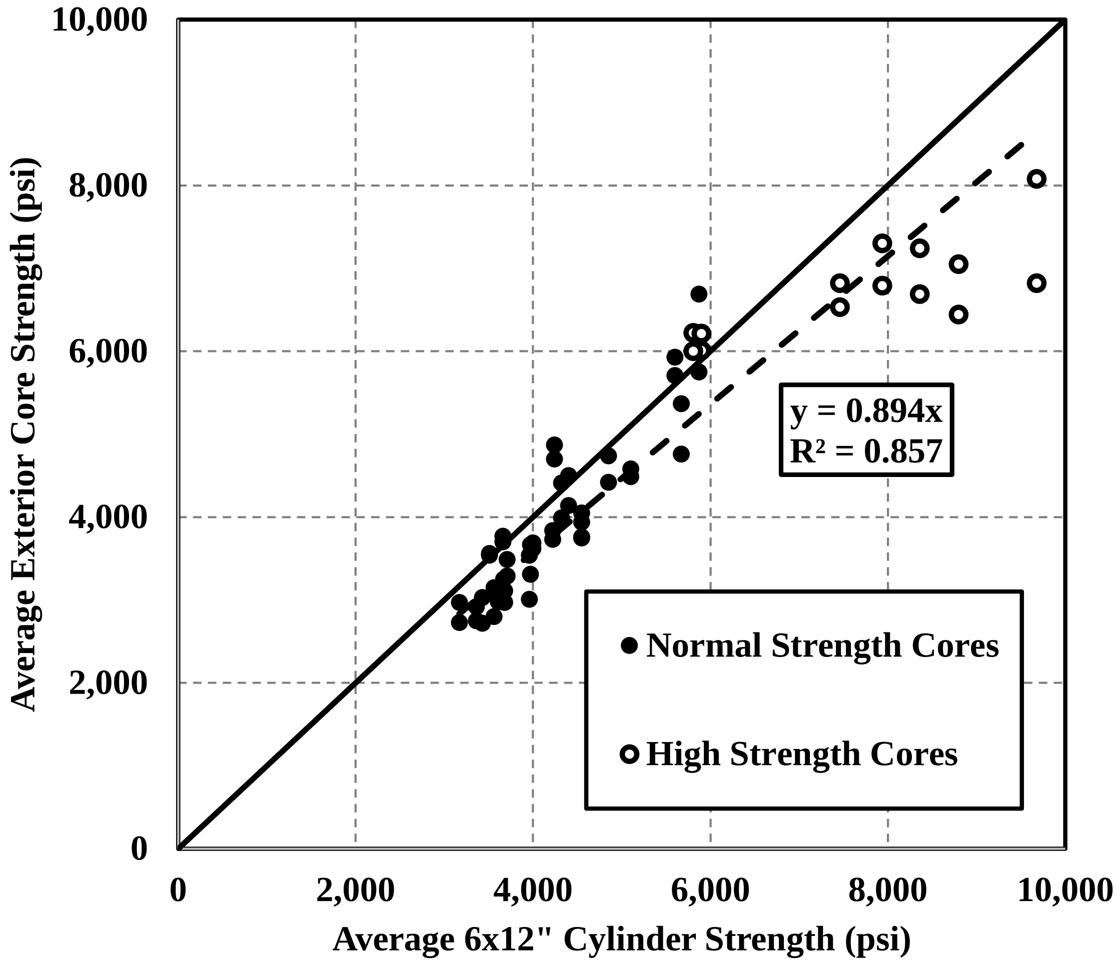

Figure 4-16: Average 6×12 in. Molded Cylinder Strength versus Average Core Strength

From ACI 318 (2011), it is specified that an average core strength of 85% of the specified compressive strength (*f'c*) constitutes satisfactory in-place strength for the concrete in question. From the data collected, it was found that, in terms of restraint, interior cores on average presented a worst case scenario in terms of relationship to molded cylinder strength. Because of this, conclusions and recommendations were made based on the relationship between the interior cores and the molded cylinders. For Figures 4-14 through 4-16, linear trendlines were fitted through the data which run through the origin. The relatively high R^2 values indicate a very well defined trend. It can be seen in the Figure 4-15 that interior core strength averaged approximately 87% of the average molded cylinder strength. It can also be seen that exterior core strength averaged approximately 92% of the average molded cylinder strength, while overall core strength averaged about 89% of the average molded cylinder strength. These differences in core strength and molded, moist cured cylinder strength was expected due in part to the damage imparted on a core from drilling through aggregate, the difference in curing methods, and the presence of microcracking, especially in the interior core specimens. It was expected that the interior cores would have a reduced strength compared to the exterior cores. This trend agrees with the principles presented by Rasmussen and Rozycki (2001) that the friction, interlock, and adhesion forces between the slab and the sub-base have a significant impact on the amount of axial restraint within a slab. Since the slabs only measured 15 ft.×15 ft., it was determined that cores taken from the interior of even larger members could be considered more restrained and therefore could have an even greater reduction in strength than the interior cores examined in the study. Based on these findings and the recommendations of ACI 318 (2011), it is recommended that an average core strength of 85% or greater of the design strength be deemed structurally adequate by ALDOT.

Chapter 5

Development of Implementation Guidelines for Core Testing

5.1 Introduction

The research presented in this thesis was part of a larger research project funded by ALDOT. The main objective of the entire project was to develop a recommended procedure for core testing for future use. Many factors impacting core testing were evaluated during the study including length-to-diameter ratio, core diameter, effect of restraint, aggregate type, aggregate size, presence of steel reinforcement, direction of drilling, SCM type, age, and effect of curing methods. This chapter provides recommendations for a new coring specification for ALDOT.

5.2 Current ALDOT Coring and Evaluation Practice

Currently, when the average compressive strength of standard quality assurance cylinders do not equal or exceed the required design strength, cores are taken from the in-place concrete in question at the expense of the contractor in order to evaluate the concrete which was placed at the site. It is specified in the Alabama Department of Transportation Bureau of Materials and Tests (2013) that cores must be taken, cured, and tested within 42 days of concrete placement. Also, it is stated that "Core specimens will be measured, cured, and tested in accordance with AASHTO T 24. Proper strength correction factor will be applied to cores having length-to-diameter (L/D) ratio less than two." If the average strength of the cores extracted from the in-place concrete do not meet or exceed the specified design strength, the contractor is paid a reduced price for the in-place concrete based on the average strength of the extracted cores. The price of the concrete is adjusted using Equation 2-13. The pay scale can be seen in Figure 2-8. The current pay scale is valid only if the average strength of the cores is greater than or equal to 85 % of the required design strength. The current pay scale does not take into account the difference in core strength due to damage imparted on the core by cutting through coarse aggregate, the effect of different curing methods, and the presence of microcracking within a core.

5.3 Evaluation of Variables Affecting Core Strength

Throughout the project, numerous variables were evaluated in order to determine their impact on core strength. It was at the request of ALDOT that recommendations be given to develop a new coring specification to account for the variables that were examined.

5.3.1 Core Length-to-Diameter Ratio

AASHTO T 24 (2009), ACI 214 (2010), and Bartlett and MacGregor (1995) all list methods to correct core strength due to different length-to-diameter ratios. Research which was conducted during this project and detailed by Carroll (2014) evaluated the effect of length-to-diameter ratio on core specimens. It was recommended by Carroll (2014) that cores that have diameters greater than or equal to 3.75 in., as per AASHTO T 24 (2009), should be corrected for length-to-diameter ratio by using the strength correction functions shown in ACI 214 (2010), which are presented in Table 2-4. These length-to-diameter strength correction factors are applicable for concrete which has a compressive strength between 2,000 and 14,000 psi. Carroll (2014) recommended that these length-to-diameter strength correction factors not be applied to cores with a diameter of 3 in.

5.3.2 Core Diameter

AASHTO T 24 (2009) states that core diameter shall not be less than 3.75 in. Despite this, there are many occasions when it is not possible retrieve a core with a diameter of 3.75 in. due to the presence of embedded steel reinforcement. ACI 214 (2010) recommends strength correction factors for core diameter, which can be seen in Table 2-4. Research conducted for this project and detailed by Carroll (2014) indicated that cores having a diameter of less than 3.75 in. should only be used if a length-to-diameter ratio of 2.0 can be obtained. This was due to the high strength variability of the 3 in. diameter cores which were tested. This variability showed that the strength correction function for length-to-diameter ratio recommended by ACI 214 (2010) is not accurate for 3 in. diameter cores with length-to-diameter ratios less than 2.0. For 3 in. diameter cores having a length-to-diameter ratio of 2.0, Carroll (2014) recommends that a strength correction factor of 1.03 should be applied to account for the effect that core diameter has on strength. This strength correction factor was obtained by interpolating values suggested by ACI 214 (2010), which can be found in Table 2-4, and can be used for concrete having a compressive strength between 1,440 and 13,400 psi.

5.3.3 Coring Direction Relative to Casting Direction

As detailed by Suprenant (1985), the diretion of drilling with respect to the direction in which the concrete was placed creates a difference in the planes of weakness found within a core. Research conducted in this project and detailed by Carroll (2014) investigated the effect of coring direction with respect to casting direction. It was found the specimens which were cored perpendicular to the casting direction had a relative strength of 96 % as compared to those cores which were taken parallel to the casting direction. Because of this, Carroll (2014) recommends that a correction factor of 1.04 be applied to cores which are drilled perpendicular to the casting direction. The correction factor for this shall be denoted as F_{dir}.

5.3.4 Aggregate Size

AASHTO T 24 (2009) specifies that cores which are obtained from concrete with a nominal maximum aggregate size (NMAS) greater than 1 ½ in. should preferably have a diameter of at least three times the NMAS and must have a diameter of at least two times the NMAS. Due to this recommendation, research was conducted during this project on the relationship between NMAS and core diameter and is detailed by Carroll (2014). Based on the findings of the study, it was concluded that the length-to-diameter ratio strength correction factors in ACI 214 (2010) were not valid for 3-in. cores with No. 67 aggregate and larger. Carroll (2014) also concluded that coarse aggregate sizes of No. 67 and 57 do not have a significant impact on the effects of core length-to-diameter ratio.

5.3.5 Moisture Conditioning

AASHTO T 24 (2009) recommends a standard treatment for cores which prevents the formation of moisture gradients within a core after removal from a slab. It is highly recommended that this procedure be followed so that moisture gradients have no effect on the apparent strength of a core. If different moisture conditioning is applied to cores once removed from a specimen, it is recommended that the moisture conditioning correction factors recommended by ACI 214 (2010), which can be seen in Table 2-4, be applied to the test results.

5.3.6 Damage

ACI 214 (2010) recommends a strength correction factor of 1.06 which compensates for excessive damage imparted on a core during drilling. Bartlett and MacGregor (1994c) says that this factor accounts for microcracking caused by drilling as well as undulation of the core barrel during drilling and aggregate pop outs which may occur during testing. No advice is given on to what degree the damage must be for the factor to be applied. ACI 214 (2010) states that this factor is valid for normal weight concretes which have a compressive strength between 2,000 and 13,400 psi. If a core is extracted that would require the use of this excessive damage factor, it is recommended that another core specimen be drilled and the damaged core be discarded instead of testing the damaged core and applying the strength correction factor of 1.06.

5.3.7 Aggregate Type

Research conducted by Khoury, Aliabdo and Ghazy (2014) concluded that concrete containing natural aggregate types, such as river gravel, are more difficult to core than concrete which contains aggregates such as limestone and that because of this, a higher amount of damage was inflicted on concretes which contain natural aggregates. Since there are many coarse aggregate types available to the concrete industry in the state of Alabama, slabs were cast in this study which contained different types of coarse aggregate to evaluate the effect that it had on core strength. A statistical analysis was completed on the data which was collected to see if coarse aggregate type had an effect on the relationship between in-place strength and moist cured, molded cylinder strength. This analysis produced mixed results for each type of test type. It is recommended that no strength correction factor due to aggregate type should be used. This is because the difference in apparent strength due to aggregate type is most likely caused by damage imparted on the core due to aggregate type, which is accounted for by deeming that cores that have an average strength of greater than or equal to 85 % of the required design strength be considered structurally adequate.

5.3.8 Effect of Restraint

Rasmussen and Rozycki (2001) outlined the principles of axial slab restraint and the effect it has on the strength of concrete specimens. These principles were tested during this project to evaluate their effect on core strength. It was found that there was a statistical difference between cores which were taken from the low restrained exterior region of the slab and cores taken from the highly restrained interior region of the slab. Although this difference was statistically significant in the study, the damage factor recommended by ACI 214 (2010) accounts for damage due to microcracking. Therefore there is no specific strength correction factor recommended the amount of axial restraint.

5.3.9 Age

Several different age correction models have been developed in order to convert a concrete compressive strength at any age to an equivalent 28-day strength. One of these models was developed by Yazdani and McKinnie (2004) using moist cured, molded cylinders. Different strength equations were developed based on different cement types and SCM types. These strength correction equations can be seen in Equations 2-1 through 2-10. ACI 209 (2008) provides Equation 2-11 as a way to convert concrete compressive strength at any age to an equivalent 28-day strength. Both of these methods were evaluated, and it was determined that the ACI 209 equation provided the best estimation of equivalent 28-day strength with the recommended empirical constants of a=4.0 and β=0.85. It is recommended that this equation be used with the given constants to convert the average strength of three cores back to an equivalent 28-day strength which can then be compared with the specified design strength. This strength conversion factor shall now be referred to as F_{age}. This factor shall be calculated based on the ACI 209 equation with the recommended empricial values which are specified in Table 2-1 where the time since placement, t, is greater than 28 days. F_{age} is calculated by using Equation 5-1. A statistical analysis was done on the presence of SCM and the effect they had on strength development. The statistical analysis concluded that this strength conversion factor can be accurately used for concretes containing any type of SCM.

$$F_{age} = \left(\frac{a+\beta\times t}{t}\right) \quad \text{Equation 5-1}$$

Where: t = time since casting (days)

a = empirical constant from Table 2-1 (days)

β = empirical constant from Table 2-1 (unitless)

5.3.10 Presence of Steel Reinforcement

AASHTO T 24 (2009) recommends that no core which contains steel reinforcement should be tested unless necessary. Though no data were collected during this study on the effect of steel reinforcement contained with cores, a review of literature was done in order to recommend a strength correction factor for cores containing steel reinforcement. The most practical recommendation that can be made is that no core containing embedded reinforcement should be tested. If it is impossible to attain a core without the presence of steel reinforcement, then the strength correction factors recommended by Bartlett and MacGregor (1995) in Table 2-7 should be applied to the core. These strength correction factors are specified only for bars which are perpendicular to the axis of drilling. Examples of how these correction factors must be applied are shown in Figures 5-1 and 5-2.

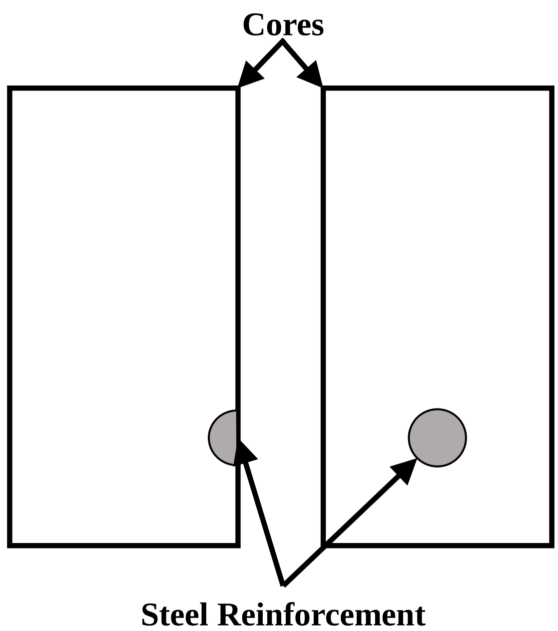

Figure 5-1: Example of When Strength Correction Factor for One Reinforcing Bar Should Be Applied

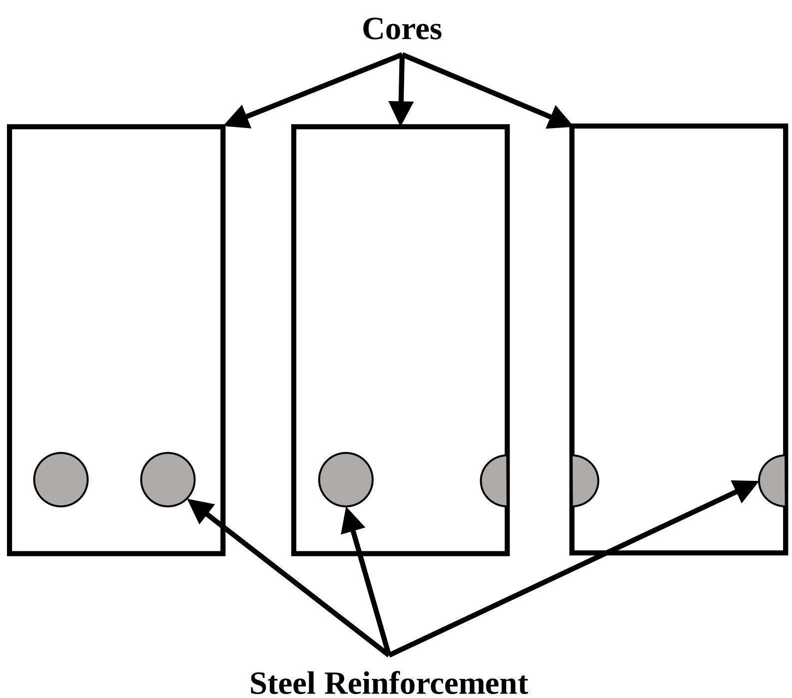

Figure 5-2: Example of When Strength Correction Factor for Two Reinforcing Bars Should Be Applied

5.3.11 Relationship Between Core and Quality Assurance Cylinder Strengths

The current pay reduction method used by ALDOT does not reflect differences in core strength and molded, moist cured cylinder strength due to damage, differences in curing methods, and the presence of microcracking within core. Because of these factors, ACI 318 (2011) recommends that if the average strength of a set of three cores equals or exceeds 85 % of the specified design strength and no single core has a strength below 75 % of the specified design strength, the in-place concrete be deemed structurally adequate. This study showed that there is a statistically significant difference between core strength and molded, moist-cured cylinder strength. The cores that were obtained from the interior region of the slabs averaged approximately 87 % of the strength of the molded, moist cured cylinder counterparts. Since cores are taken from structures which have more restraint than the 15 ft.×15 ft. slabs which were cast in this study, it is recommended that the in-place concrete be deemed structurally adequate if the average of three cores equals or exceeds 85 % of the specified design strength so long as no single core has a strength below 75 % of the specified design strength.

5.4 Recommended Procedure for Correcting Core Strength

It is recommended that at least four cores be taken in the event that in-place strength has to be evaluated due to the low strength of quality assurance cylinders. This is so that if one core is damaged or found to be an outlier, then an average of the remaining three cores is still useful to determine the structural adequacy of the in-place concrete.

From the findings in this study, it is recommended that strength correction factors be applied to cores for the following: length-to-diameter ratio, core diameter, steel reinforcement, moisture conditioning, and age. This shall be done using Equation 5-2.

$$f_{c,es28} = F_{l/d}F_{dia}F_rF_{mc}F_{age}F_{dir}f_c \quad \text{Equation 5-2}$$

Where:

$F_{l/d}$ is the length-to-diamater strength correction factor

F_{dia} is the diameter strength correction factor

F_r is the steel reinforcement strength correction factor

F_{mc} is the moisture conditioning strength correction factor

F_{age} is the age strength correction factor

F_{dir} is the strength correction factor for drilling direction relative to casting direction

$f_c(t)$ is the core compressive strength at any time, t

$f_{c,es28}$ is the estimated core compressive strength at 28 days

Once equivalent 28-day core strengths are obtained by using Equation 5-2, the strengths should be checked for outliers. This can be done using the method from ASTM E178 (2008). This is done by using Equation 5-3 and comparing the output to the corresponding $T_{critical}$ value which can be looked up in Table 5-1.

$$\frac{\mu - X_{min}}{\sigma} \leq T_{critical} \quad \text{Equation 5-3}$$

Where: μ is the average compressive strength of the cores

X_{min} is the lowest core compressive strength

σ is the standard deviation of the core strengths

If one of the strength values from the data set is determined to be an outlier, then the strength value should be discarded and not used in the calculation of the average compressive strength that is used to determine if the in-place concrete is structurally adequate. After an outlier is discarded, the remaining data set should again be checked for outliers using the same procedure.

Once all outliers have been removed, the average compressive strength should be computed again without the outlying strength values. This average compressive strength should then be used to determine the structural adequacy of the in-place concrete.

Table 5-1: $T_{critical}$ Values for Given Number of Cores in a Sample (ASTM E178 2008)

n	$T_{critical}$ Value for Upper 5% Significance Level	n	$T_{critical}$ Value for Upper 5% Significance Level	n	$T_{critical}$ Value for Upper 5% Significance Level
3	1.153	19	2.532	35	2.811
4	1.463	20	2.557	36	2.823
5	1.672	21	2.58	37	2.835
6	1.822	22	2.603	38	2.846
7	1.938	23	2.624	39	2.857
8	2.032	24	2.644	40	2.866
9	2.11	25	2.663	41	2.877
10	2.176	26	2.681	42	2.887
11	2.234	27	2.698	43	2.896
12	2.285	28	2.714	44	2.905
13	2.331	29	2.73	45	2.914
14	2.371	30	2.745	46	2.923
15	2.409	31	2.759	47	2.931
16	2.443	32	2.773	48	2.94
17	2.475	33	2.786	49	2.948
18	2.504	34	2.799	50	2.956

To be considered structurally adequate, two criteria must be met. First, the average compressive strength of the cores must be equal to or exceed 85 % of the required design strength. Secondly, no single core compressive strength is permitted to fall below the specified percentage, k, of f'_c shown in Table 5-2. The value of k is dependent on the number of cores extracted from the concrete in question. If the sample size is three cores, then the value of k is 0.750. These values were calculated in accordance with the findings of Bartlett and Lawler (2011) using a P_1 of 10%

as well as an allowable value of V_{ws} of 7.8 %. This value comes from Table 2-9 which is from ACI 214 (2010). This value of V_{ws} was chosen because it lies between the specified values of 7 % for a single batch of concrete and 12 % for multiple batches of concrete for a single member while also producing a *k* value of 0.75 for three cores.

Table 5-2: Values of *k* based on the Number of Cores Extracted

n	k	n	k	n	k	n	k
3	0.75	**15**	0.69	**27**	0.67	**39**	0.66
4	0.74	**16**	0.69	**28**	0.67	**40**	0.66
5	0.73	**17**	0.69	**29**	0.67	**41**	0.66
6	0.72	**18**	0.69	**30**	0.67	**42**	0.66
7	0.72	**19**	0.68	**31**	0.67	**43**	0.66
8	0.71	**20**	0.68	**32**	0.67	**44**	0.66
9	0.71	**21**	0.68	**33**	0.67	**45**	0.66
10	0.70	**22**	0.68	**34**	0.67	**46**	0.66
11	0.70	**23**	0.68	**35**	0.67	**47**	0.66
12	0.70	**24**	0.68	**36**	0.66	**48**	0.66
13	0.70	**25**	0.68	**37**	0.66	**49**	0.66
14	0.69	**26**	0.67	**38**	0.66	**50**	0.66

If the average core strength is greater than or equal to 85 % of the required design strength and no single core strength falls below $k \times f'_c$, then the concrete shall be deemed structurally adequate. If the average core strength is lower than 85 % of the specified design strength or the data set contains a core strength lower than $k \times f'_c$, then a structural analysis must be done to determine if the in-place concrete is satisfactory, needs to be strengthened, or must be removed and replaced.

Chapter 6

Summary, Conclusions, and Recommendations

6.1 Summary

In order to determine the adequacy of in-place concrete, different methods of in-place testing can be used. Common methods which can be classified as non-destructive or only slightly destructive testing include core testing, cast-in-place cylinders, and pullout testing. Of these, the most commonly used testing method in today's concrete industry is the core test. Although cores are easy to obtain, their apparent strengths are difficult to analyze due to the number of factors which can affect their apparent compressive strength. These factors must be evaluated and proper measures must be taken in order to obtain valid conclusions from core testing. When cores are taken, it is often times because the average 28-day strength of the 6×12 in. molded cylinders did not meet the minimum specified design strengths of the concrete set forth at the onset of the project. Therefore cores are typically taken after 28 days. Since there are many factors which impact the apparent strength of cores, strength correction factors must be applied to core strengths in order to convert them to a standard which can be compared with specified design strength. In order to fairly evaluate core strength with molded, moist cured cylinder strength, a relationship between the two strengths must be established. Once this relationship is established, then and only then can core strength and cylinder strengths be fairly compared against one another. Currently, ALDOT uses a pay scale which pays contractors based on the in-place strength of the concrete which was placed. If the average 6×12 in. molded cylinder strength at 28 days does not meet the specified minimum design strength, then cores must be taken in order to determine the amount which will be paid to the contractor for the in-place concrete. Currently, ALDOT uses the pay scale seen below in Figure 6-1.

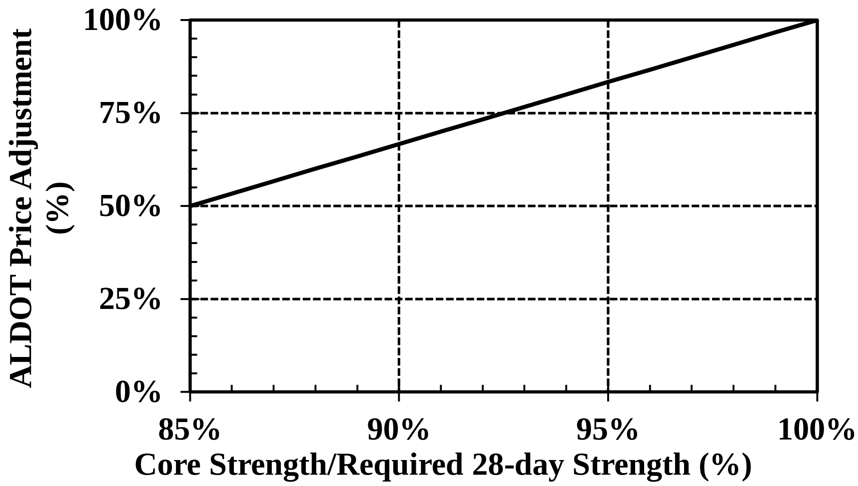

Figure 6-1: Current ALDOT Price Adjustment

(Alabama Department of Transportation, 2012)

As can be seen from the Figure 6-1, if the equivalent 28-day core strength is 85% of the required design strength, then the contractor only gets paid 50% of the original price agreed to in the contract. If the average core strength is lower than 85% of the specified design strength, then a structural analysis must be done to determine if the in-place concrete is satisfactory, needs to be strengthened, or if it must be removed and replaced. However, ACI 318 (2011) currently states that if the average strength of the cores obtained from the member in question is 85 % of the required design strength, then the concrete is structurally adequate.

As stated above, there are also alternative methods to evaluate the in-place strength of concrete. One of these ways is to make cast-in-place cylinders and test them in compression. Cast-in-place cylinders are an ideal alternative to coring because the specimens are exposed to the same curing, temperature, and moisture conditions as the member, much the same as cores, but are not susceptible to damage from cutting through aggregate as is the case with core testing. Pullout testing can be performed on in-place concrete to determine its compressive strength. Pullout inserts are cast into the fresh concrete shortly after placement and finishing. This method of testing is advantageous because of the short amount of time it takes to perform a test. It is recommended by ASTM C900 (2007) that at least 5 tests be performed when doing pullout testing due to the heightened variability of the test compared to that of core testing and cast-in-place cylinders.

Some important factors which were considered during testing were age, strength level, coarse aggregate type, restraint, and difference in strength gain due to SCM type. Age greatly impacts the measured strength of concrete and therefore conclusions and recommendations were drawn in order to accurately convert a strength at any age back to an equivalent 28-day comprssive strength. Part of this analysis focused on how different types of SCMs effected the rate of strength gain with the specimens.

In order to evaluate the amount of impact that strength level had on the relationship between in-place testing and required design strength, two high strength concrete slabs were cast in addition to six normal strength slabs for comparison.

Since a variety of coarse aggregates are currently used to produce concrete in the state of Alabama, an analysis was also completed to determine the impact that different aggregate types had on apparent strength.

Restraint was another factor which was evaluated during the study. In theory, concrete that is more restrained is more likely to develop microcracks due to not being able to freely expand and contract due to the principles presented by Rasmussen and Rozycki (2001). Conversely, specimens that are taken close to free edges of members are less likely to develop microcracks due to less restraint and the ability to expand and contract more freely with temperature change.

6.2 Conclusions

- There was no statistical difference between 6×12 in. and 4×8 in. molded cylinders for both normal-strength and high-strength concrete.
- Core strength was statistically less than molded cylinder strength for all concrete strength levels containing uncrushed river gravel, crushed limestone, and crushed granite. Average interior core strengths were approximately 87% of the average molded cylinder strength. The findings of this study agreed with ACI 318 (2011) in that concrete should be deemed structurally adequate if the average compressive strength of three cores exceeds 85 % of the required design strength with no single core having a compressive strength of 75 % of the required design strength.
- The type of SCM did not have a statistically significant impact on the post-28-day rate of strength gain on in-place specimens as well as molded cylinders. The rate of strength gain was not statistically different at a 95 % confidence level for concretes containing SCMs compared to concretes which contained only portland cement for the data collected during the project. Therefore, a single strength gain function is warranted for all concretes whether or not they contain SCMs. Equation 2-11 from ACI 209.2R (2008) in Chapter 4 was determined to provide the most accurate estimation of strength gain over time when the recommended β value of 0.85 and a value of 4.0 for concretes containing Type I portland cement were used.
- The cast-in-place cylinders which were analyzed were not affected by the amount of restraint to which they were exposed. This was expected as the cylinder molds have room to expand and contract within the support system which holds them in place. Restraint level did have an impact on the apparent compressive strength of both the cores and the pullout

tests. This is most likely due to the principles regarding axial restraint due to the friction, adhesion, and interlock forces between the slab and the sub-base, which are outlined by Rasmussen and Rozycki (2001).

- Aggregate type did have an impact on the relationship between in-place strength and molded cylinder strength. It was found that the river gravel cores had the highest relative strength as compared to the molded, moist cured cylinders while the limestone pullouts produced the highest relative strength with respect to the molded, moist cured cylinders. The specimens and tests containing granite aggregates produced the weakest relative strengths for both cores and pullout tests with respect to molded, moist cured cylinder strength.
- No core containing embedded reinforcement should be tested unless absolutely necessary. If cores containing steel reinforcement must be tested, it is recommended that the correction factors proposed by Bartlett and MacGregor (1995) be used to correct the obtained strengths.

6.3 Recommendations

- The size of the molded cylinders used for quality control should be decided upon at the beginning of each project. This decision should be made based on which size is more practical for the project
- When taking cores in the field, the compressive strength of each core should be converted back to an equivalent core strength at 28 days and then compared with the specified design compressive strength using Equation 2-11, to produce an accurate estimation of 28-day core strength by using the recommended values of a = 4.0 and β = 0.85. A strength

correction factor, F_{age} should be computed using Equation 5-1. This factor can then be multiplied by a core strength at any age and be converted to an equivalent 28-day core strength. When comparing equivalent 28-day core strengths to the specified design compressive strength, the data analysis showed that an average equivalent 28-day core strength of 85% of the specified design compressive strength with no single value being lower than 75% of the specified design compressive strength for a set of three cores is satisfactory which is in agreement with ACI 318 (2011).

- No correction factor is recommended for aggregate type or restraint. Further research is recommended to study the impact that coarse aggregate has on the relationship between in-place testing and moist-cured, molded cylinders.
- It is recommended that further research be done on high-strength concrete and how different factors relate to in-place testing results.
- It is recommended that no core containing steel reinforcement be tested, as per AASHTO T24, except for in circumstances where it is not possible to obtain a core without hitting steel reinforcement. In such a case, it is recommended that the correction factors proposed by Bartlett and MacGregor (1995) be used to correct the compressive strength of the core.

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Appendix A

Temperature Data

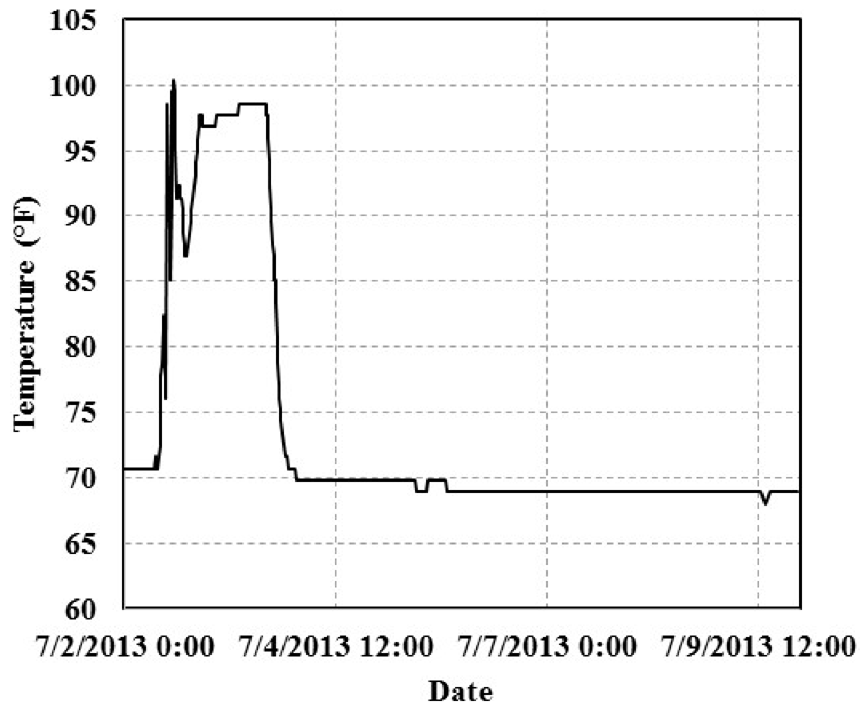


Figure A-1: RG4000CA C612M 7-Day Temperature Data

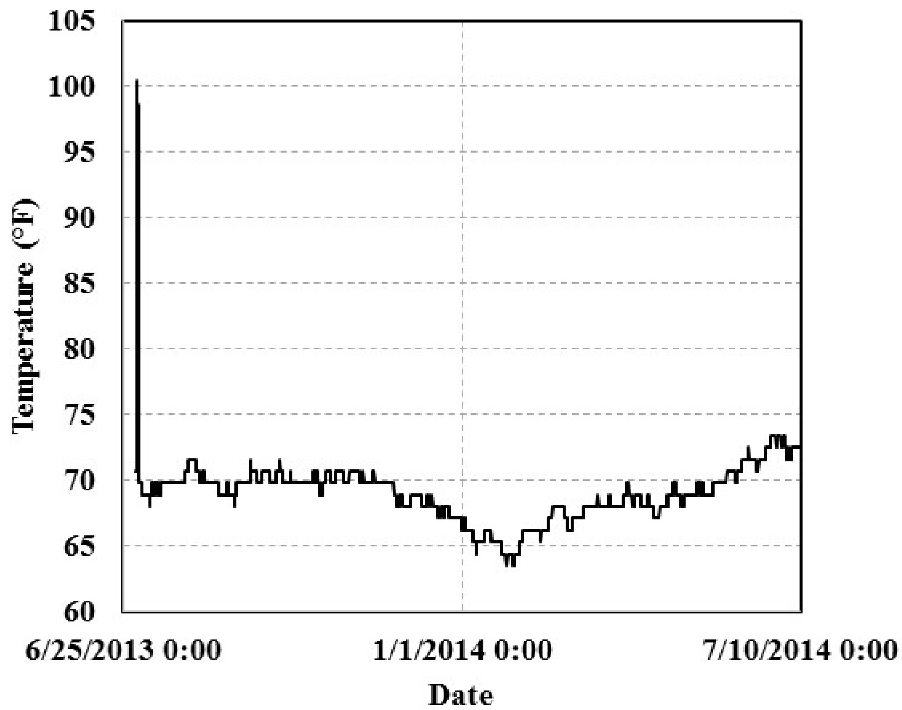


Figure A-2: RG4000CA C612M 365-Day Temperature Data

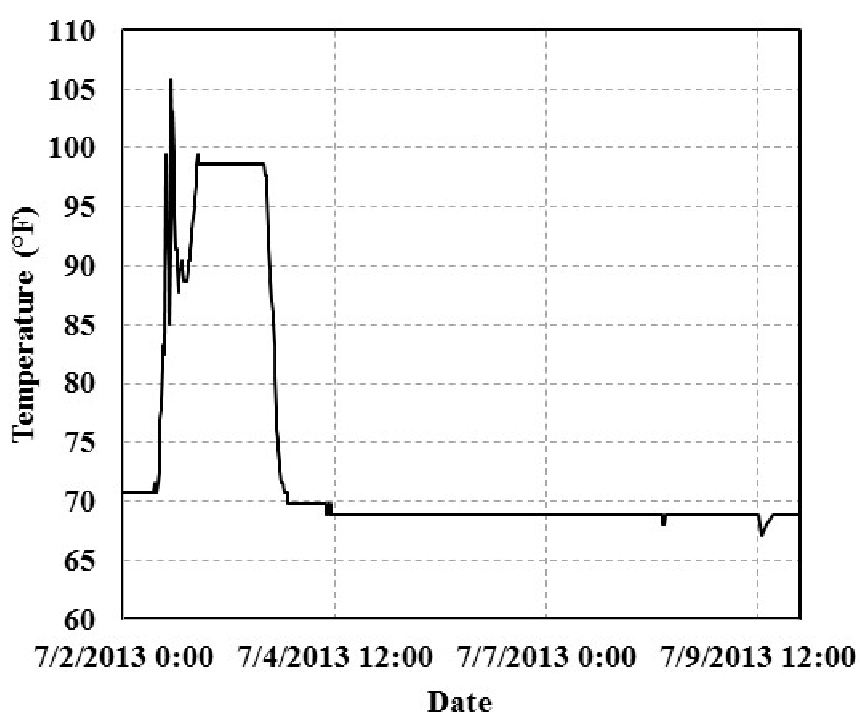


Figure A-3: RG4000CA C612T 7-Day Temperature Data

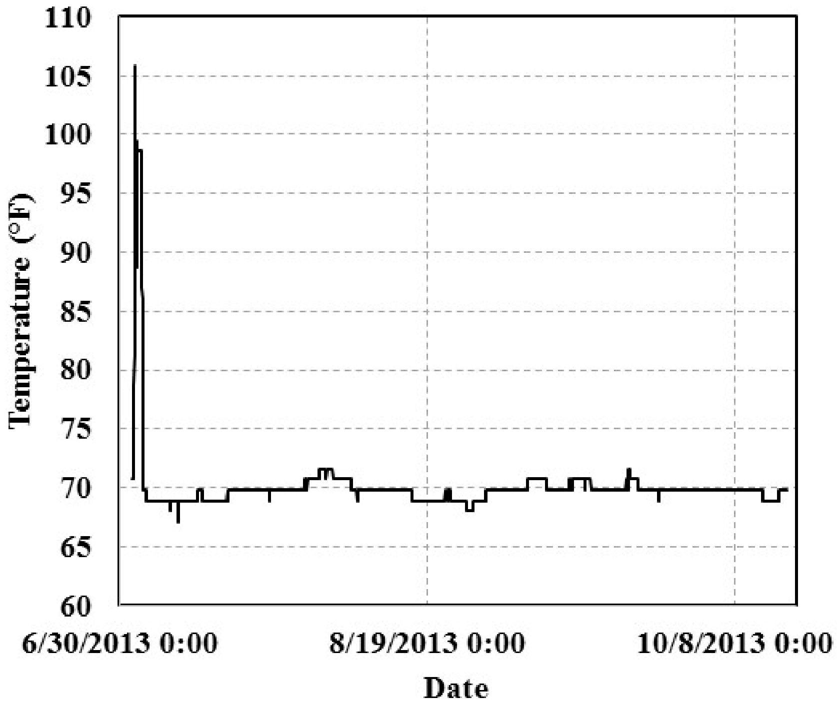
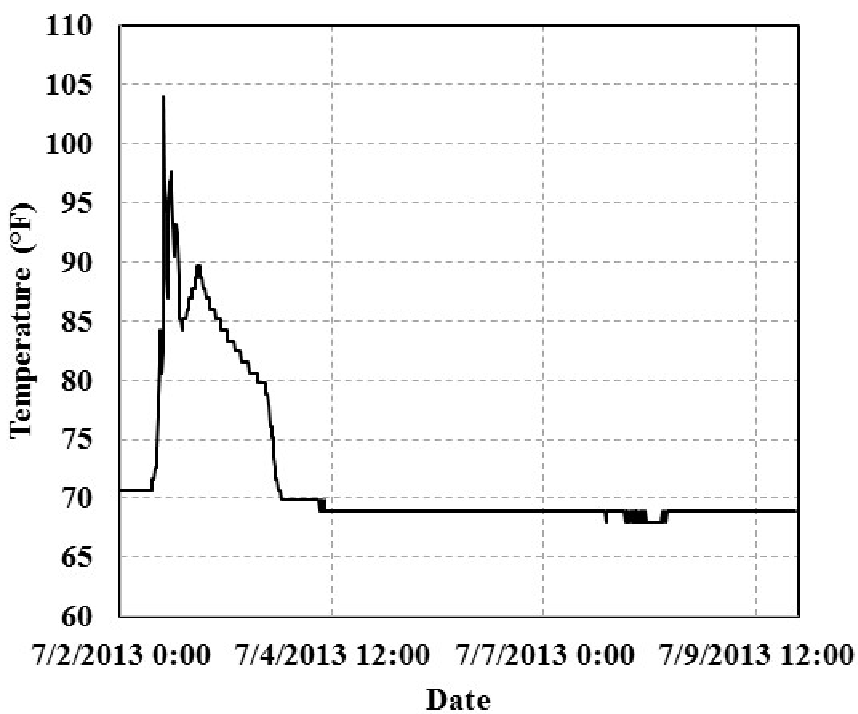


Figure A-4: RG4000CA C612T 91-Day Temperature Data



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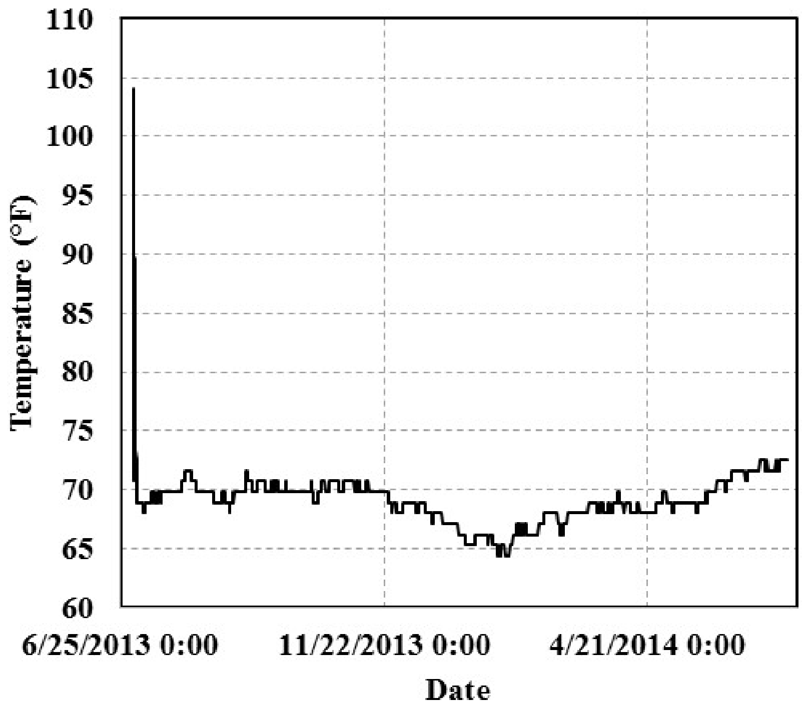


Figure A-6: RG4000CA C48M 365-Day Temperature Data

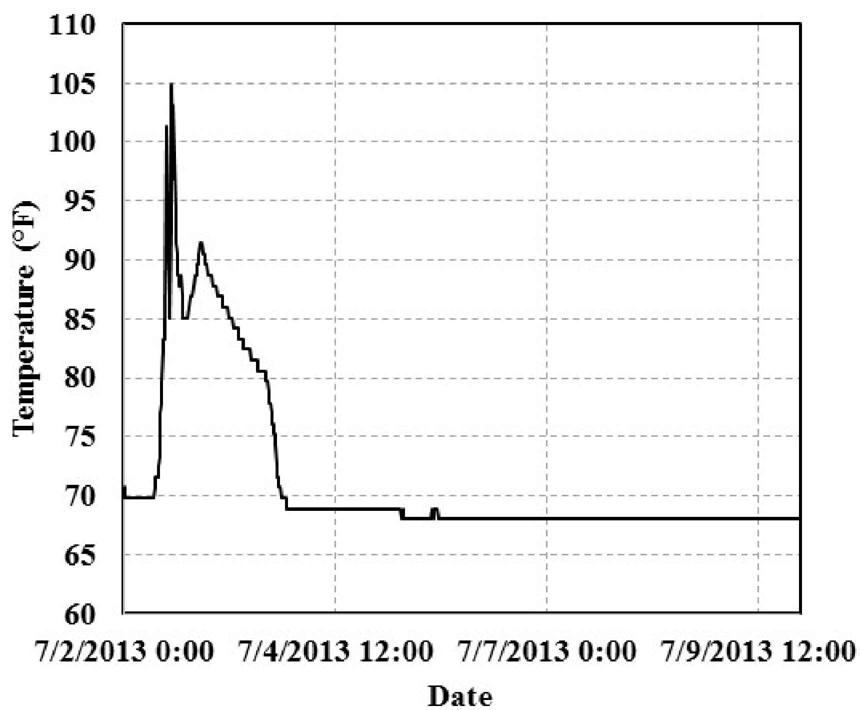


Figure A-7: RG4000CA C48T 7-Day Temperature Data

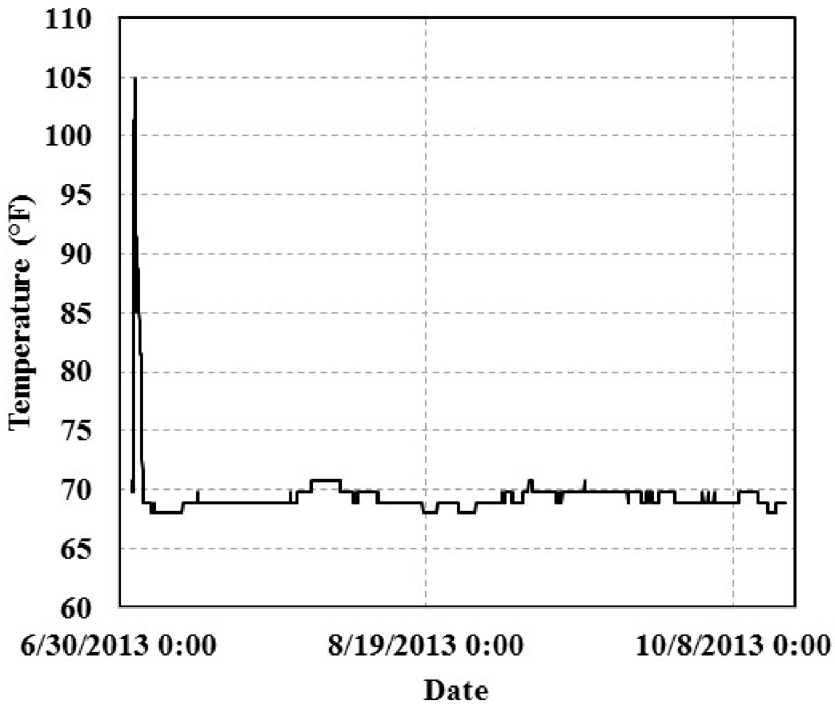


Figure A-8: RG4000CA C48T 91-Day Temperature Data

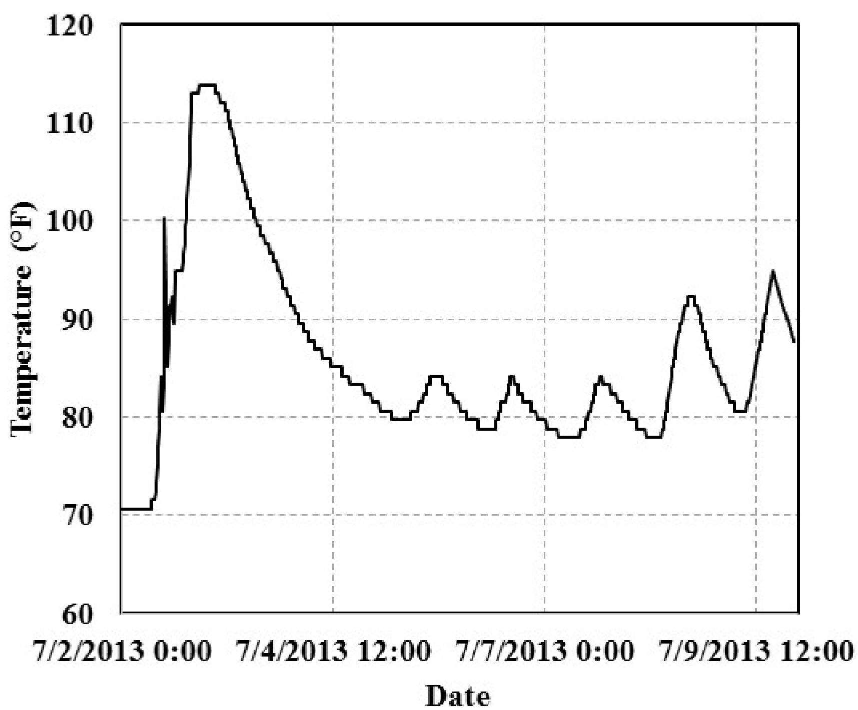


Figure A-9: RG4000CA IM 7-Day Temperature Data

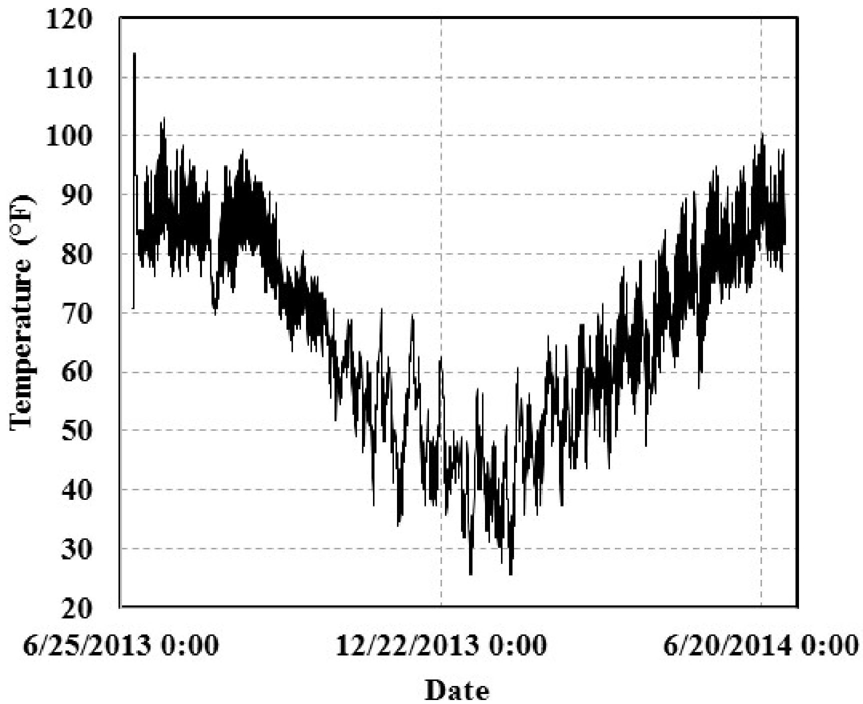


Figure A-10: RG4000CA IM 365-Day Temperature Data

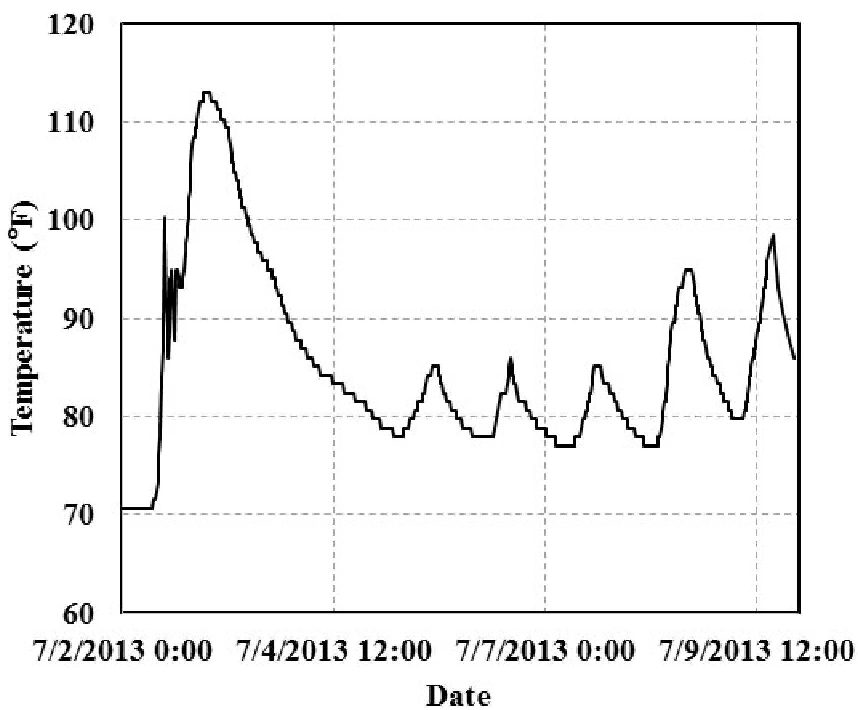


Figure A-11: RG4000CA IT 7-Day Temperature Data

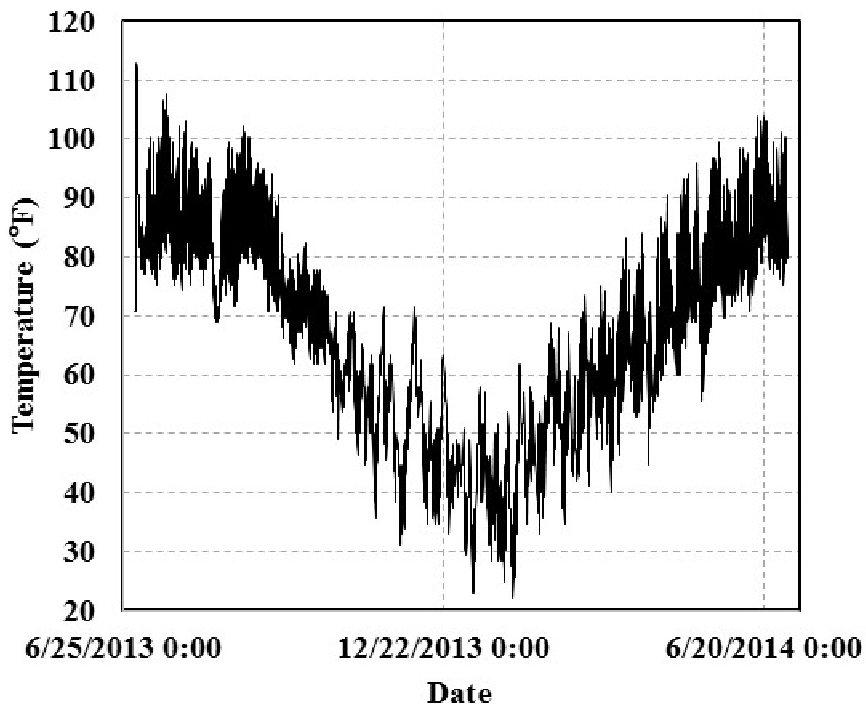


Figure A-12: RG4000CA IT 365-Day Temperature Data

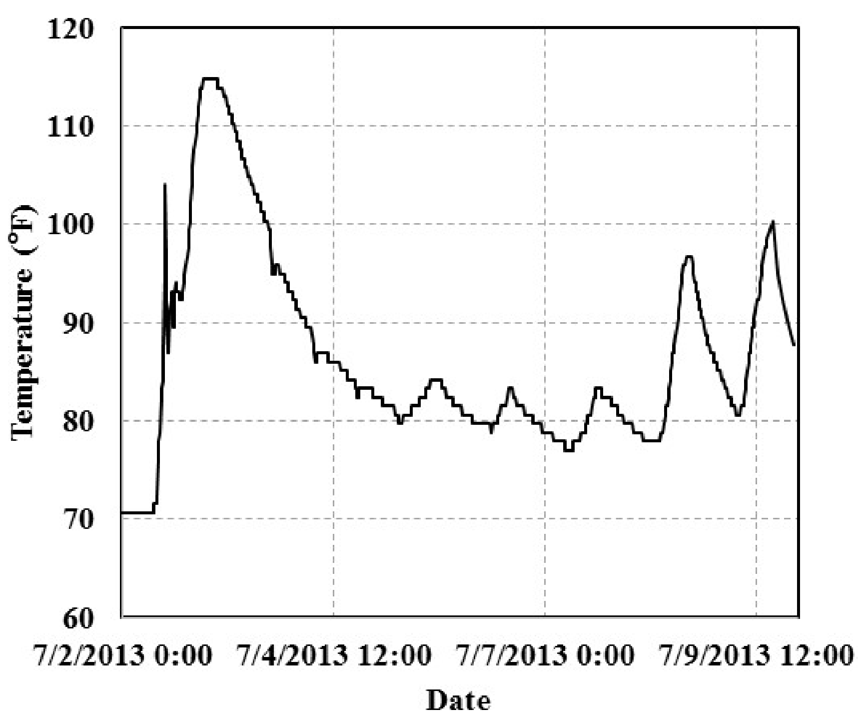
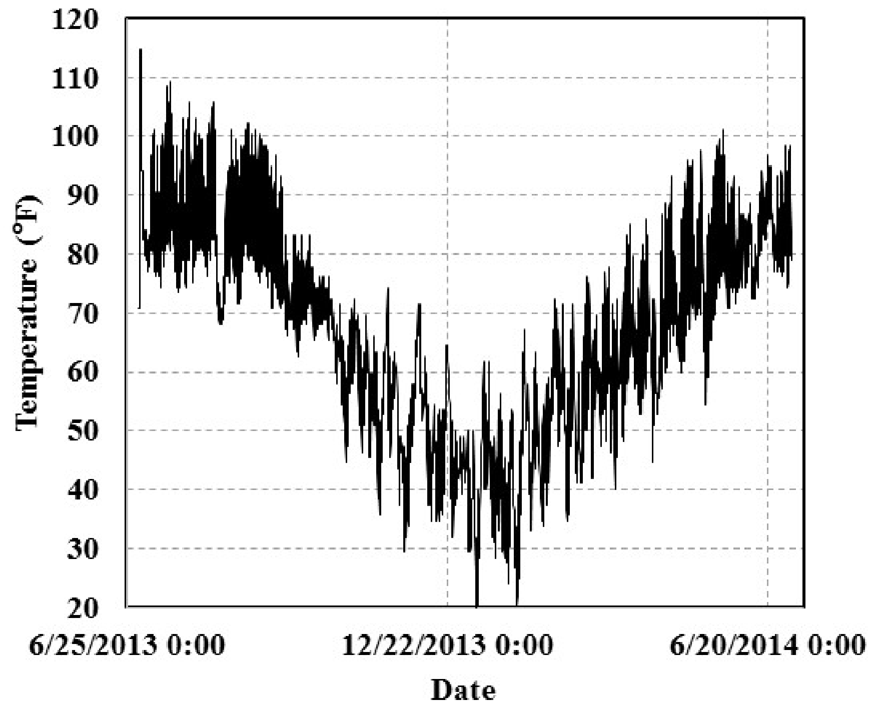


Figure A-13: RG4000CA OT 7-Day Temperature Data



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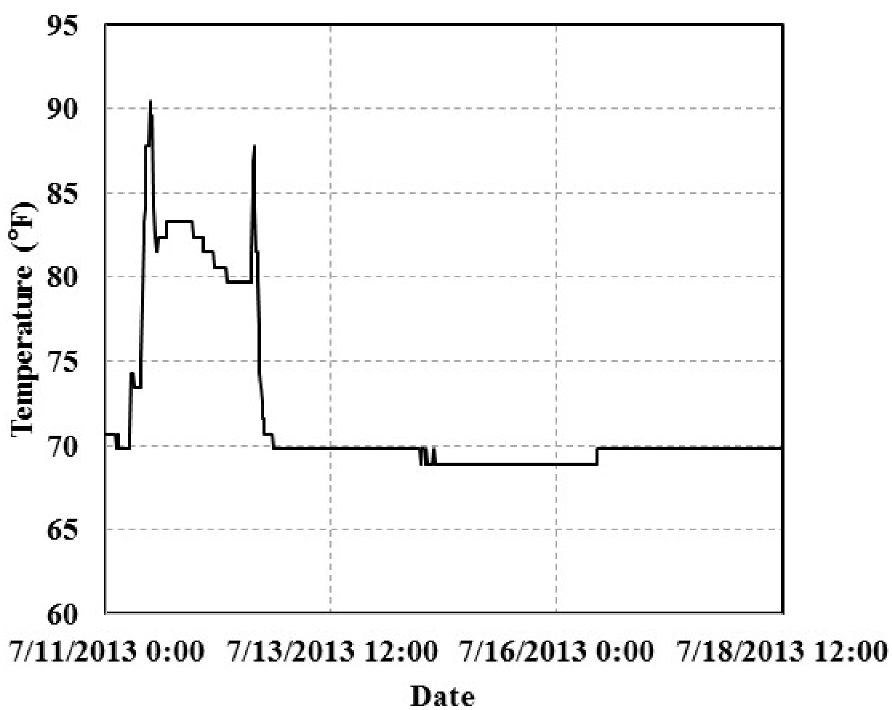
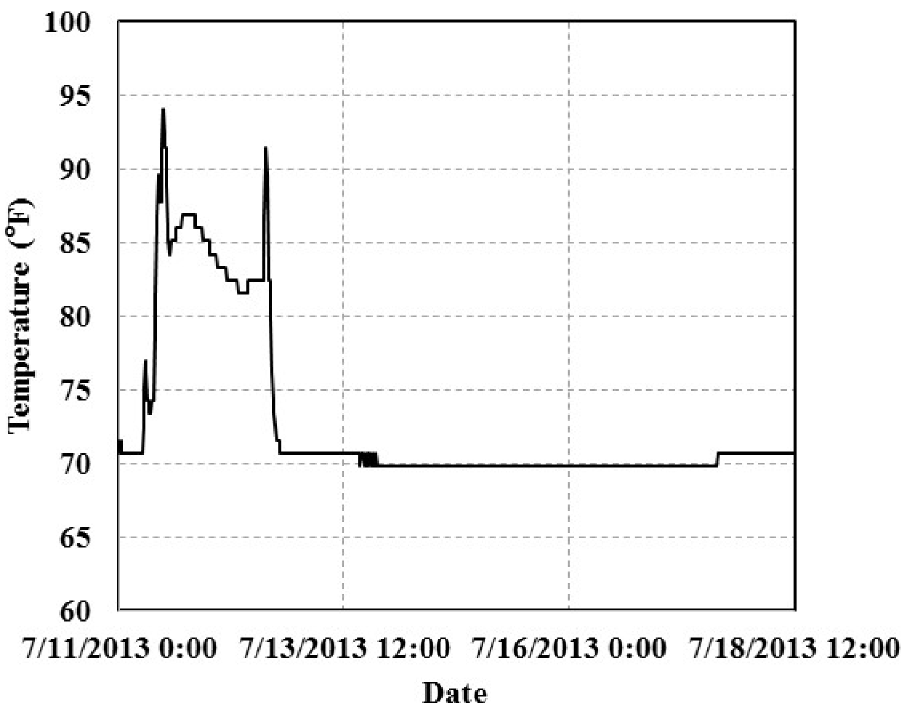


Figure A-15: LS4000CT C48M 7-Day Temperature Data

Figure A-16: LS4000CT C48M 365-Day Temperature Data



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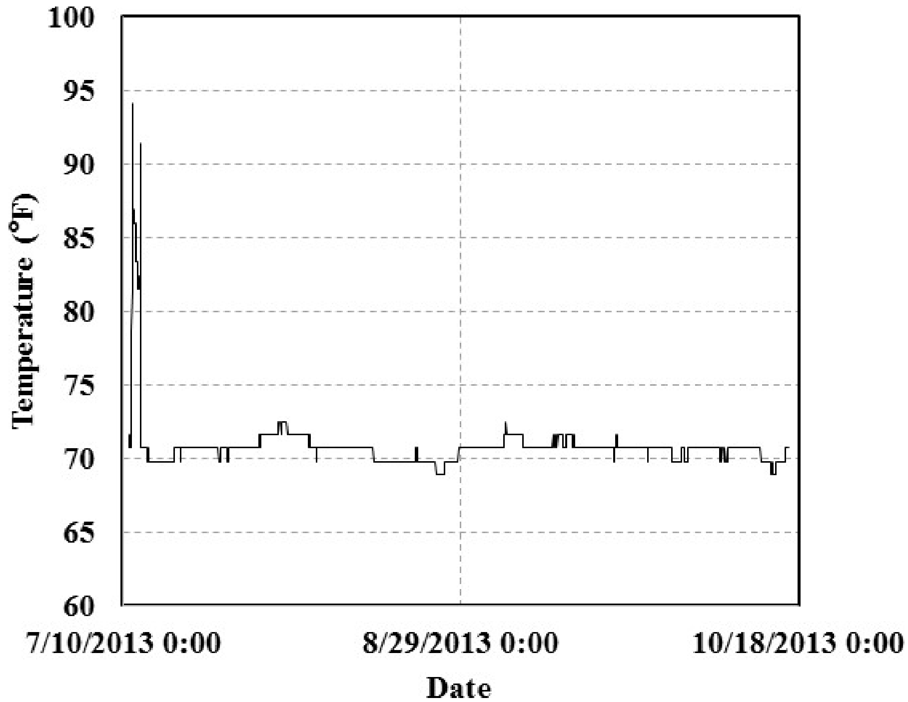


Figure A-18: LS4000CT C48T 91-Day Temperature Data

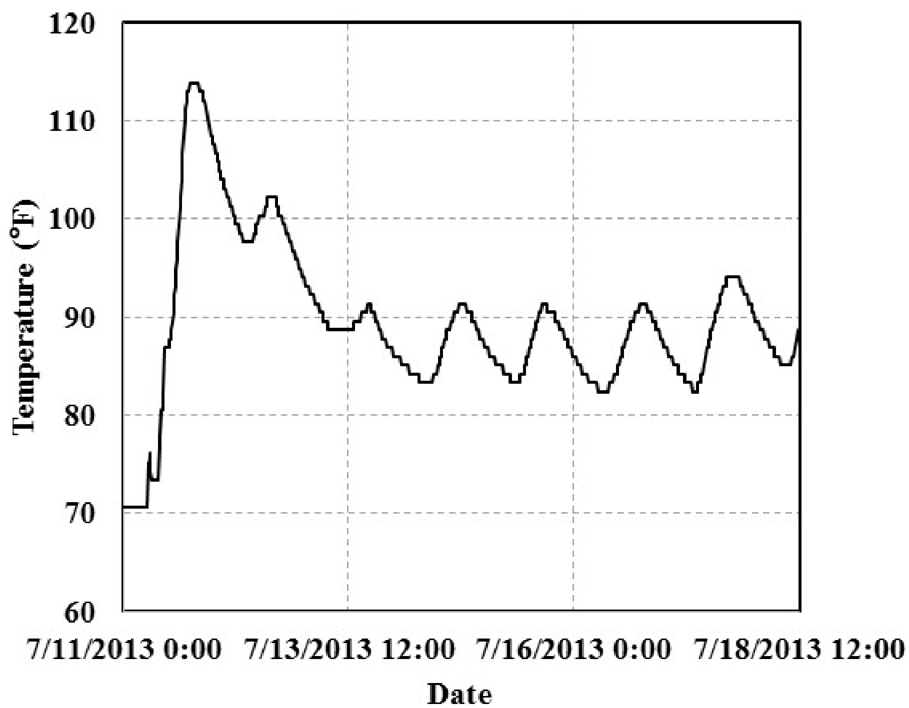


Figure A-19: LS4000CT IM 7-Day Temperature Data

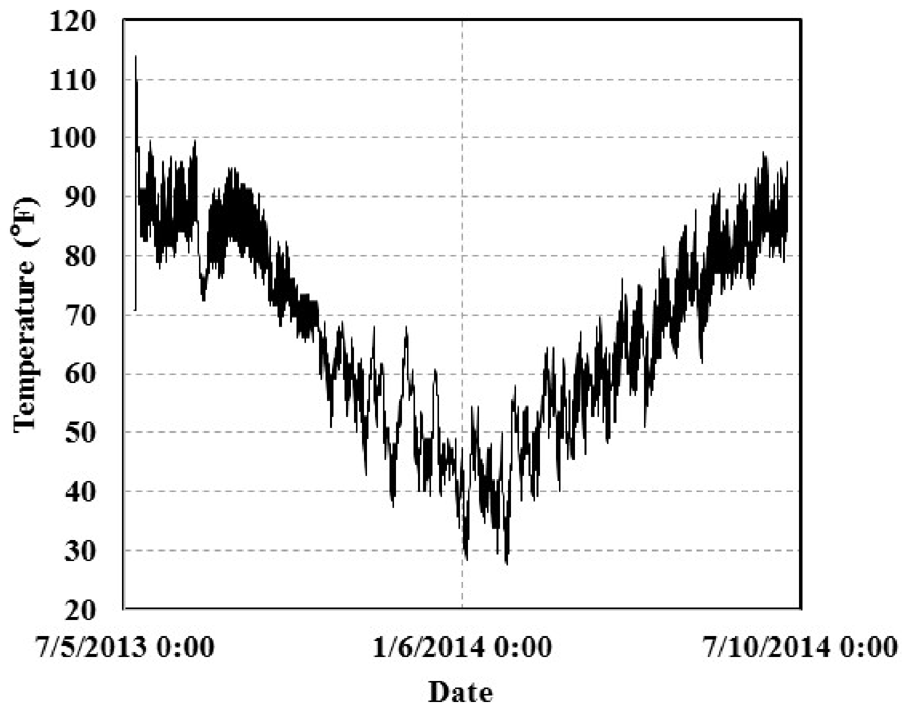


Figure A-20: LS4000CT IM 365-Day Temperature Data

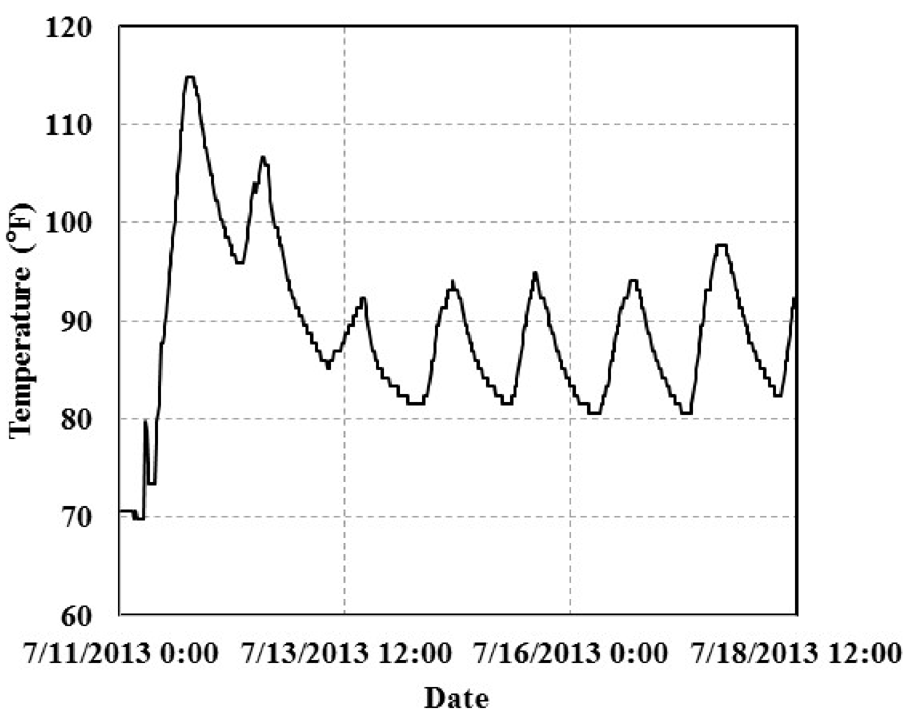


Figure A-21: LS4000CT IT 7-Day Temperature Data

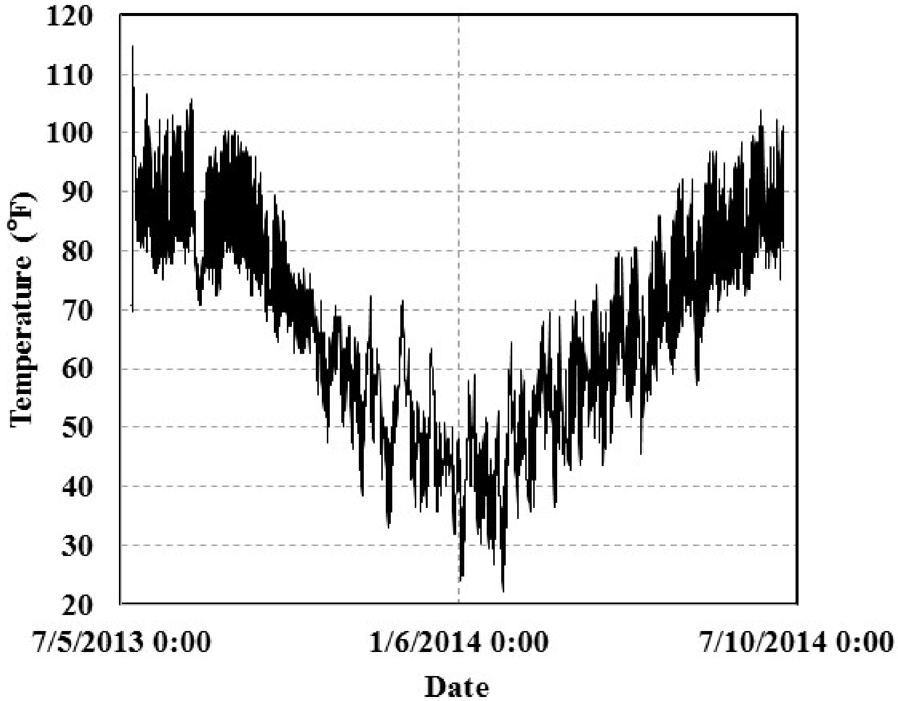
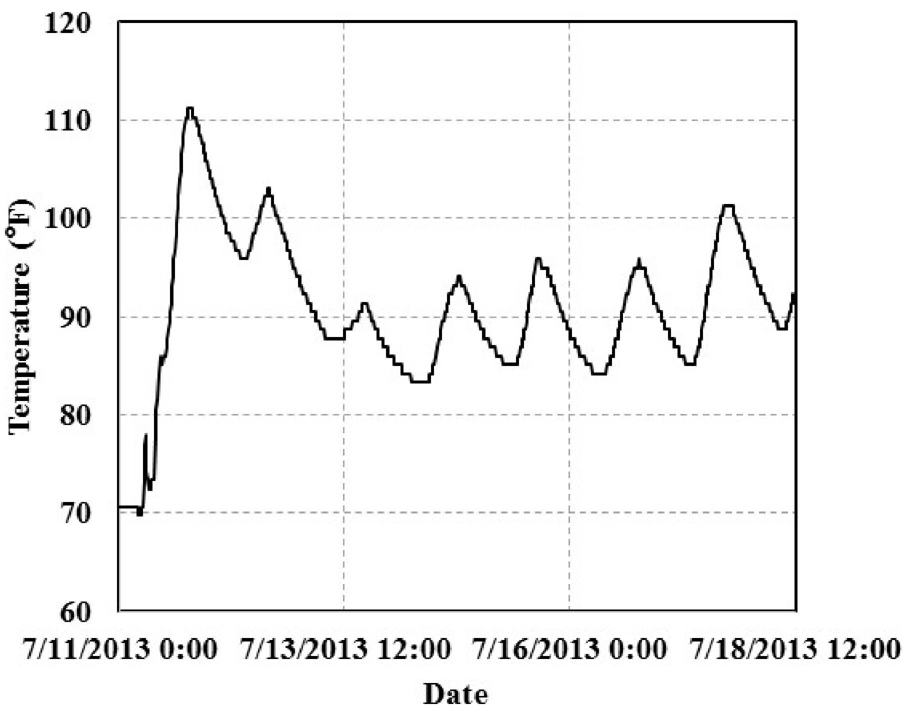


Figure A-22: LS4000CT IT 365-Day Temperature Data



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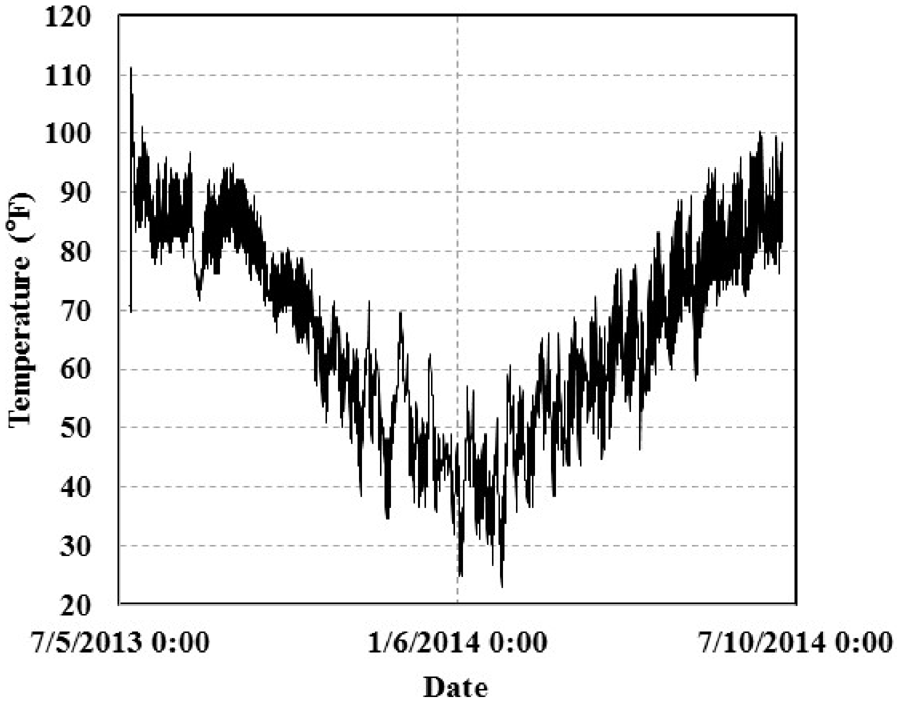

Figure A-24: LS4000CT OM 365-Day Temperature Data

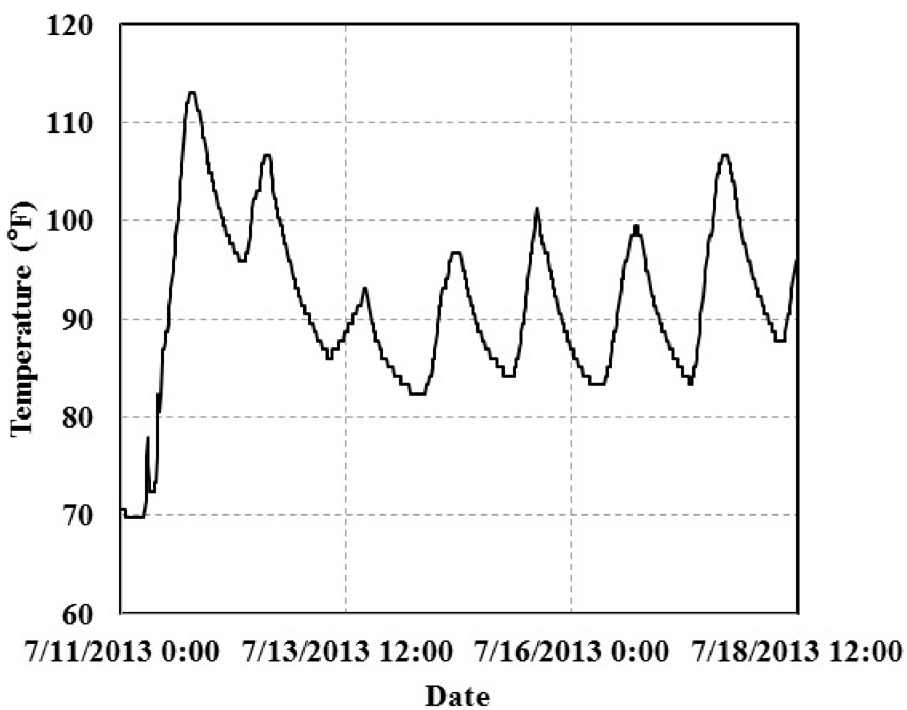

Figure A-25: LS4000CT OT 7-Day Temperature Data

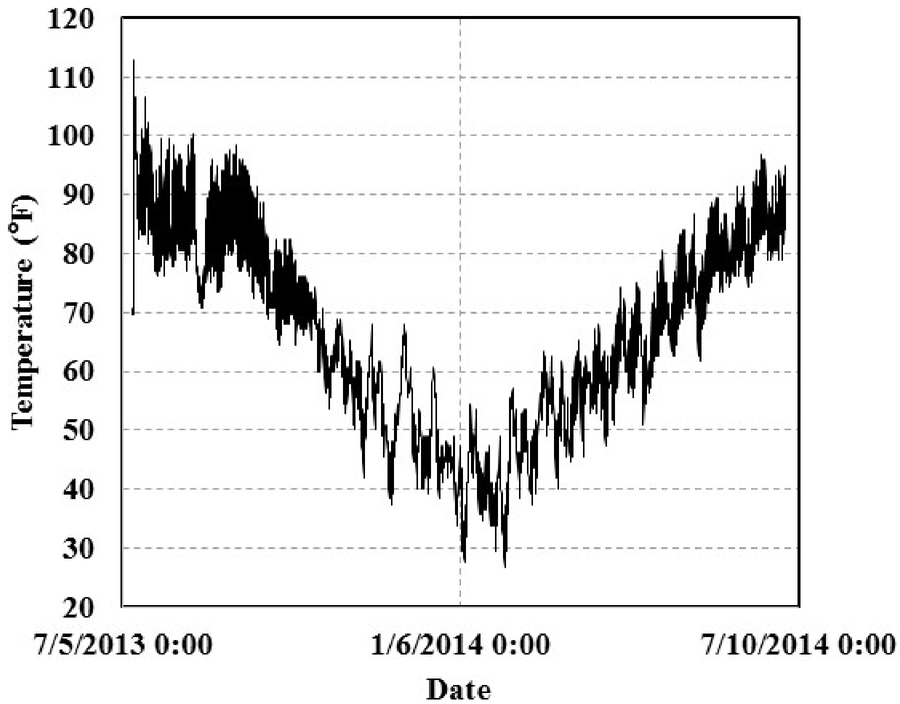

Figure A-26: LS4000CT OT 365-Day Temperature Data

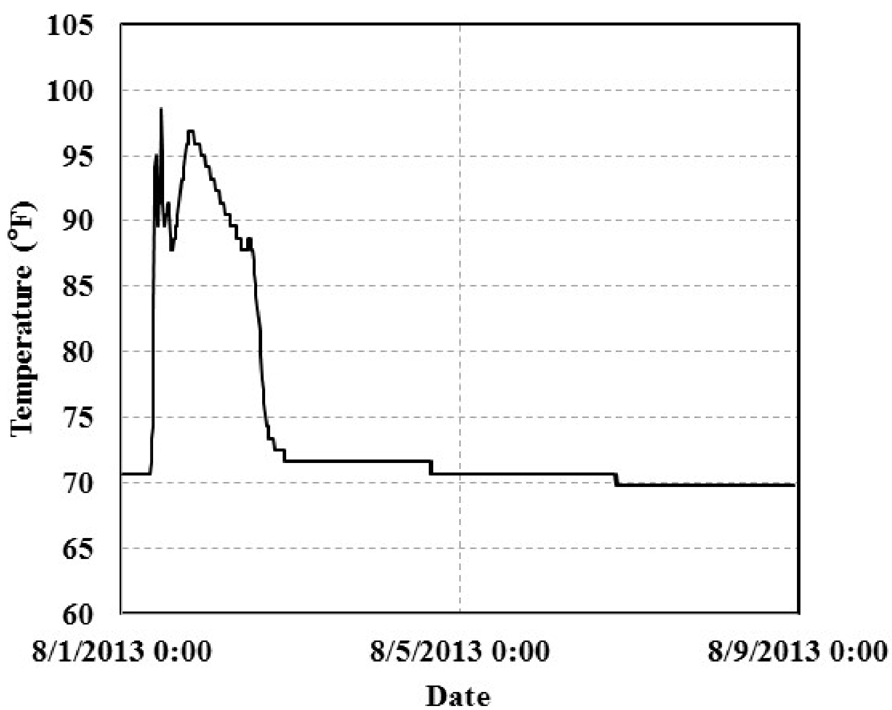

Figure A-27: RG4000CT 7-Day Temperature Data

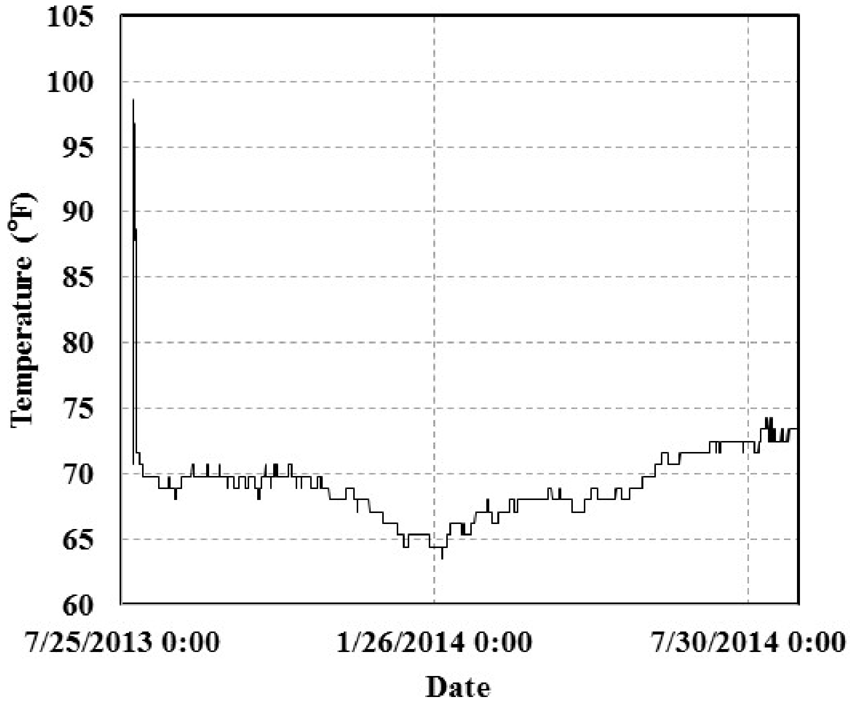

Figure A-28: RG4000CT C612M 365-Day Temperature Data

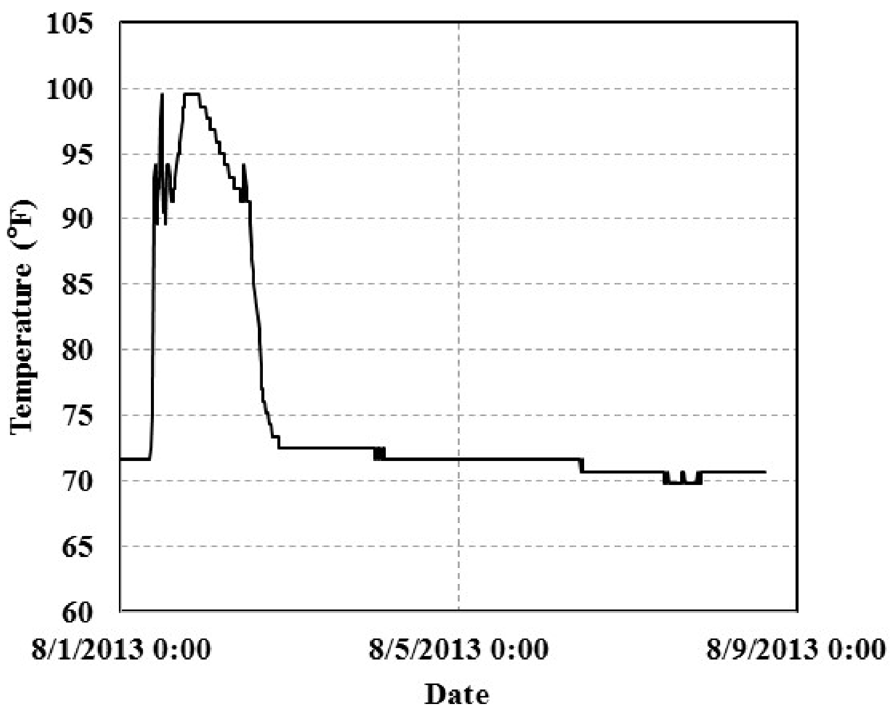

Figure A-29: RG4000CT C612T 7-Day Temperature Data

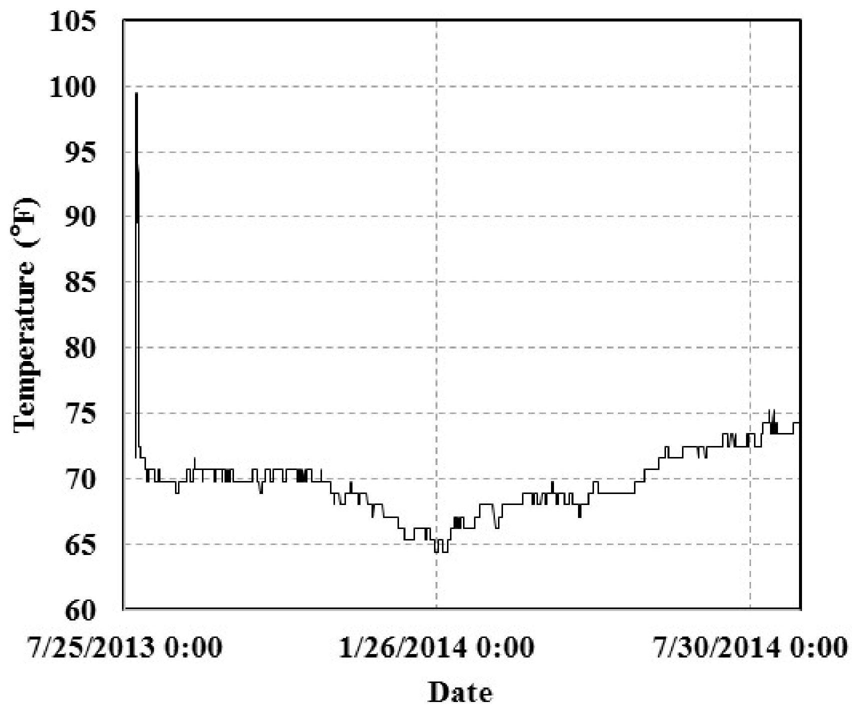

Figure A-30: RG4000CT C612T 365-Day Temperature Data

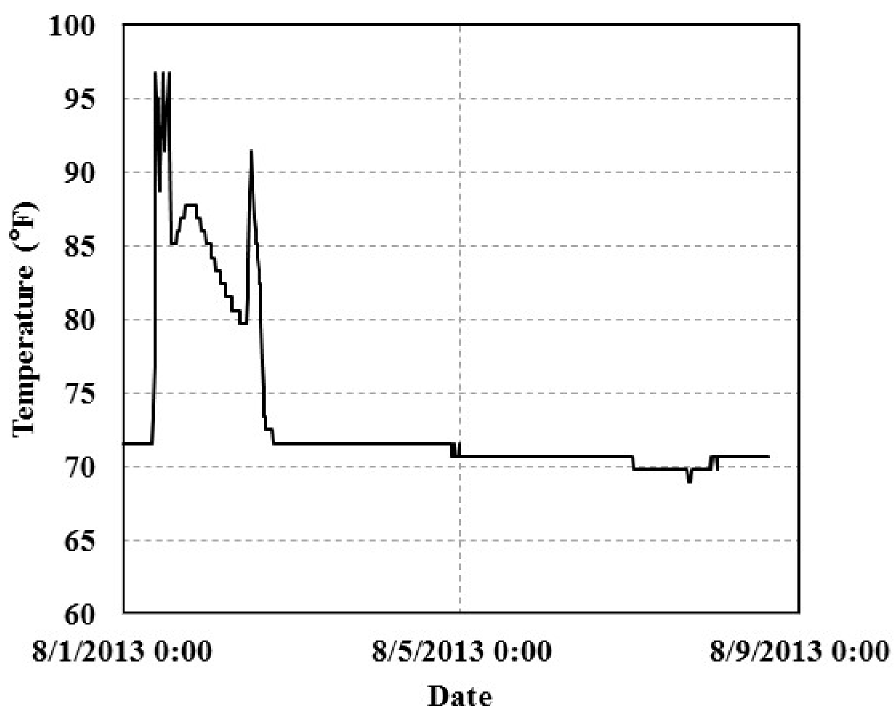

Figure A-31: RG4000CT C48M 7-Day Temperature Data

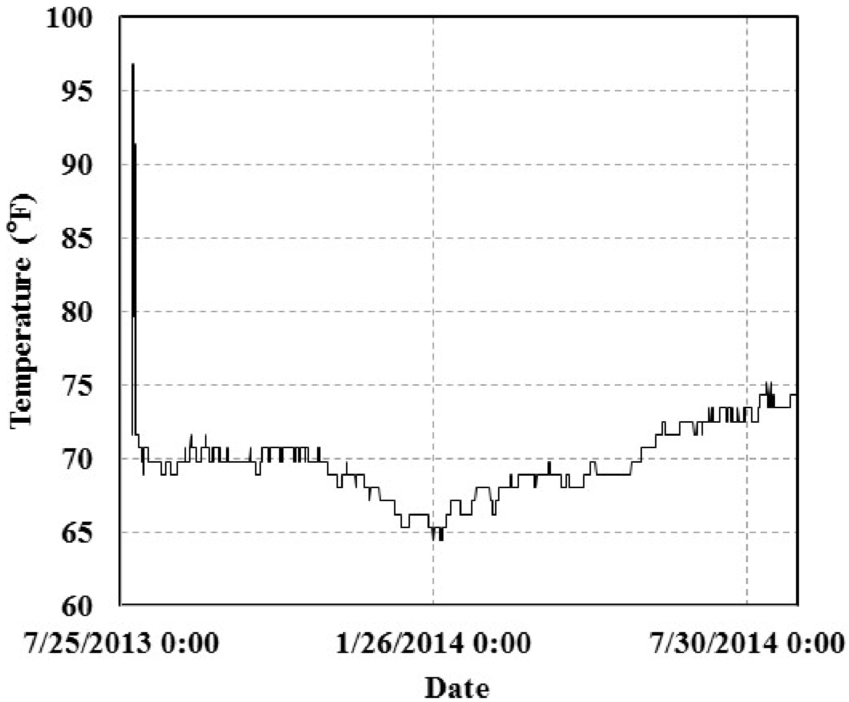

Figure A-32: RG4000CT C48M 365-Day Temperature Data

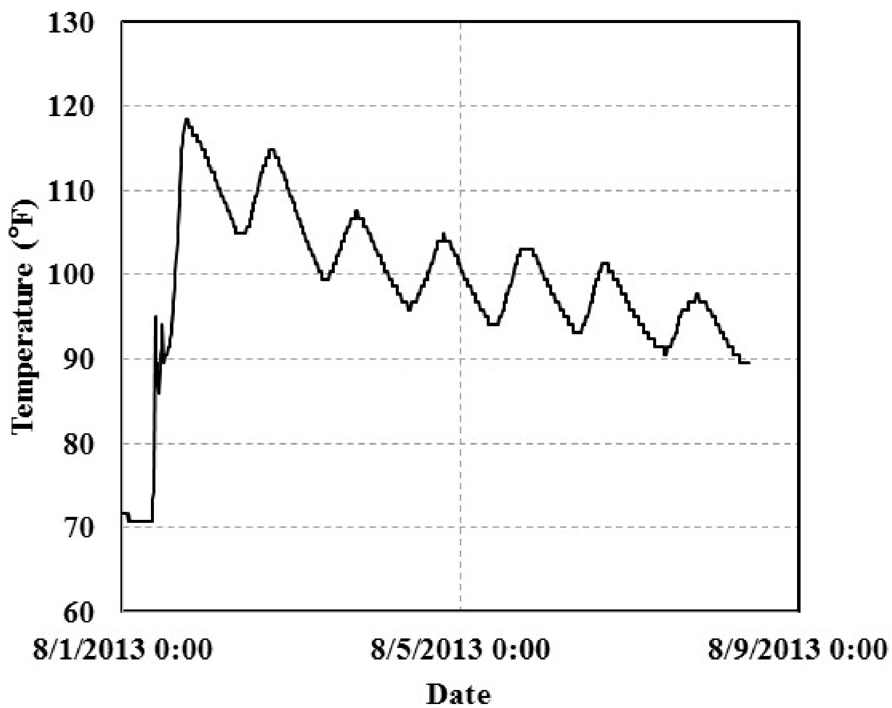

Figure A-33: RG4000CT IM 7-Day Temperature Data

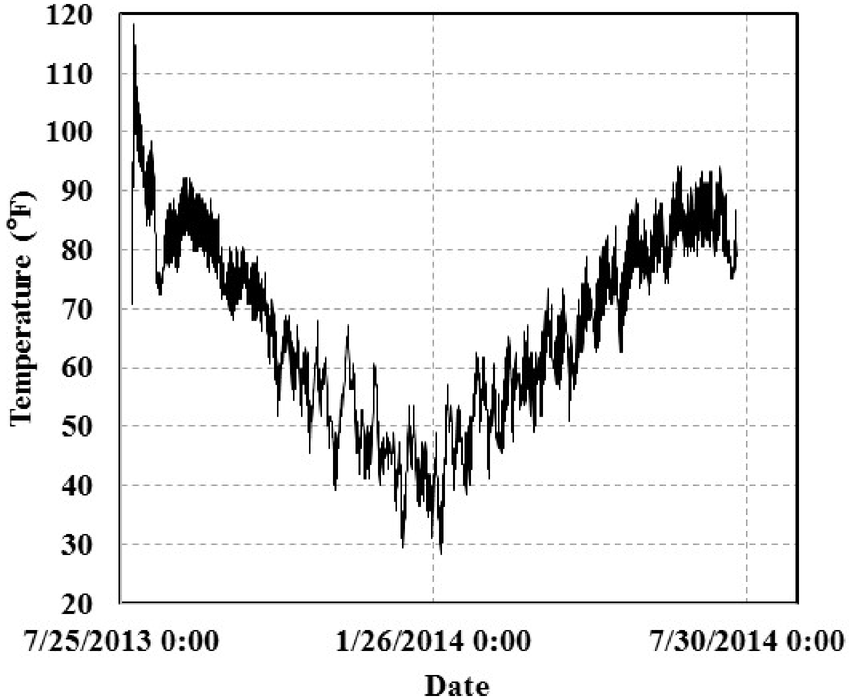

Figure A-34: RG4000CT IM 365-Day Temperature Data

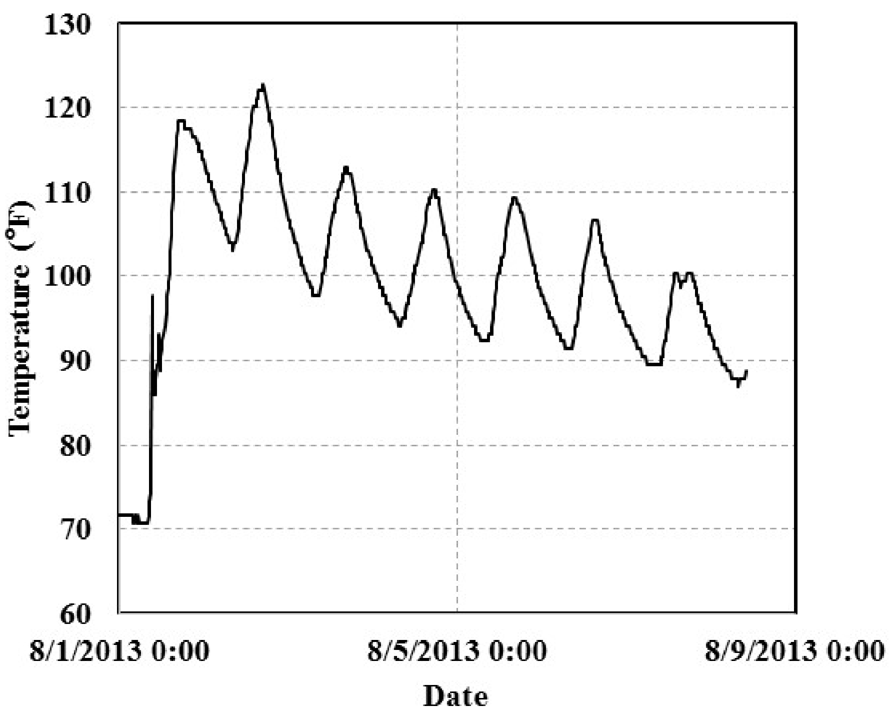

Figure A-35: RG4000CT IT 7-Day Temperature Data

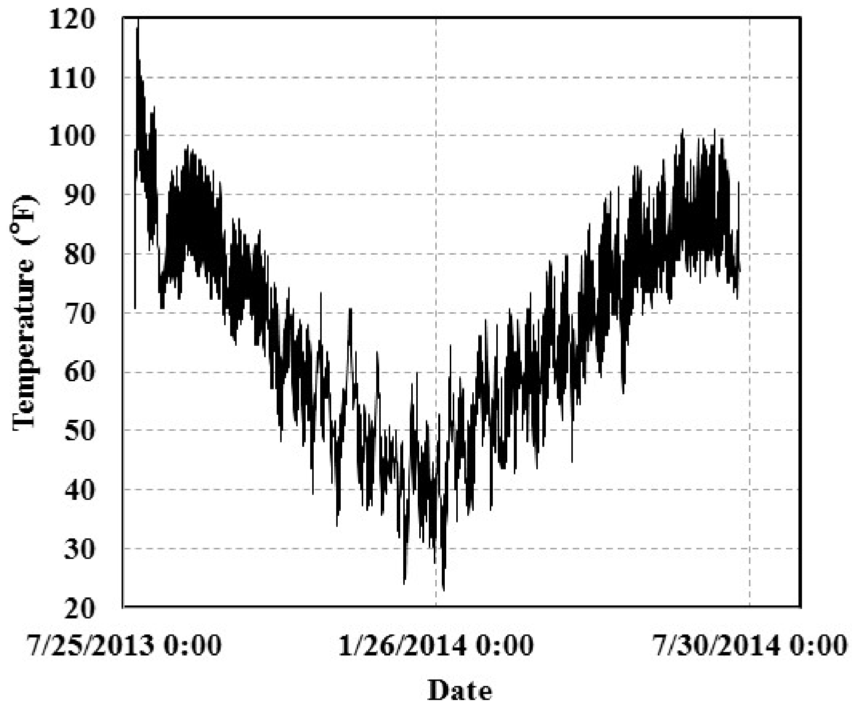

Figure A-36: RG4000CT IT 365-Day Temperature Data

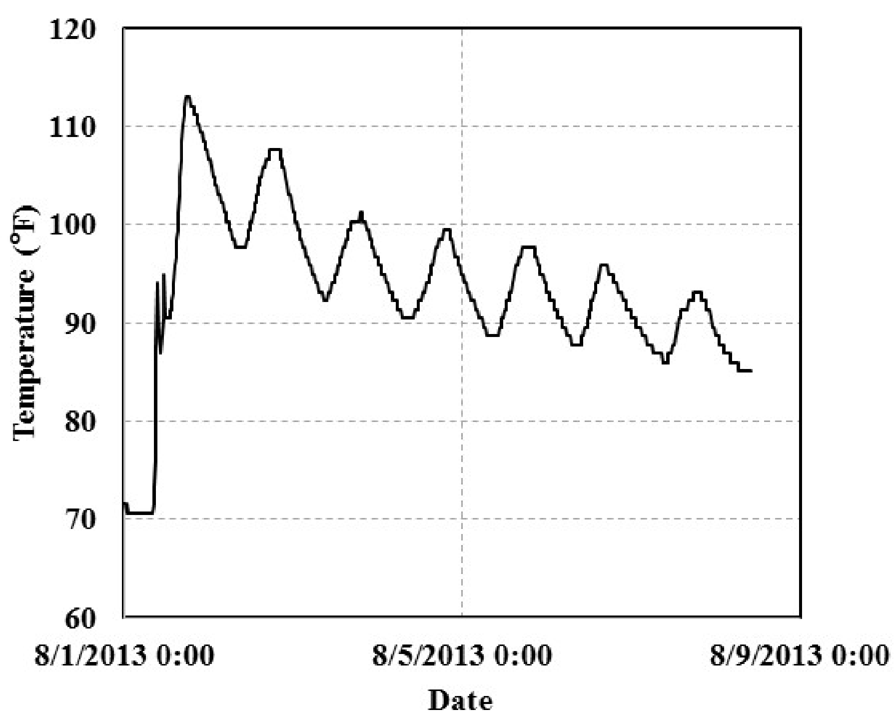

Figure A-37: RG4000CT OM 7-Day Temperature Data

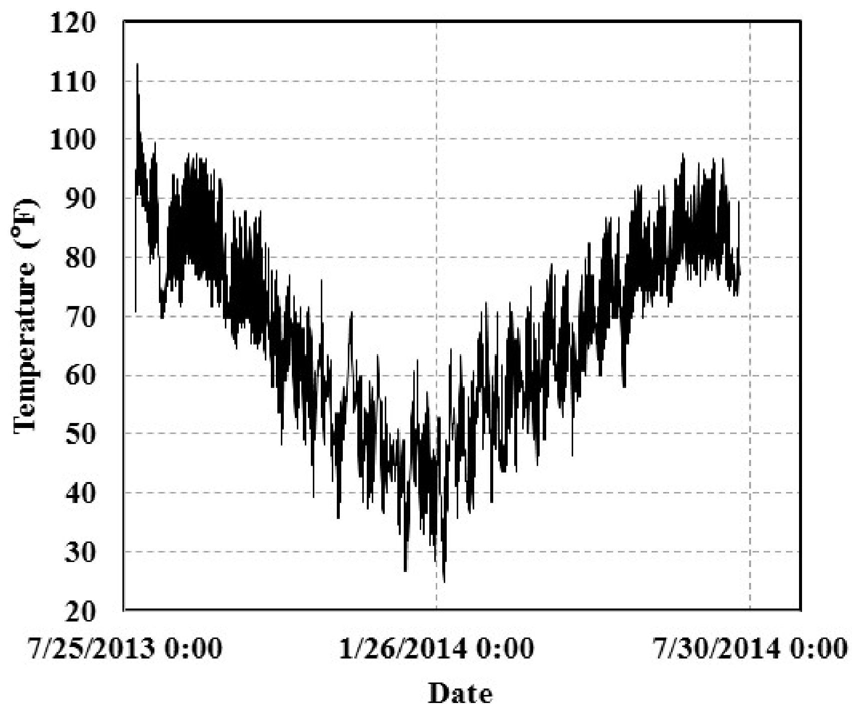

Figure A-38: RG4000CT OM 365-Day Temperature Data

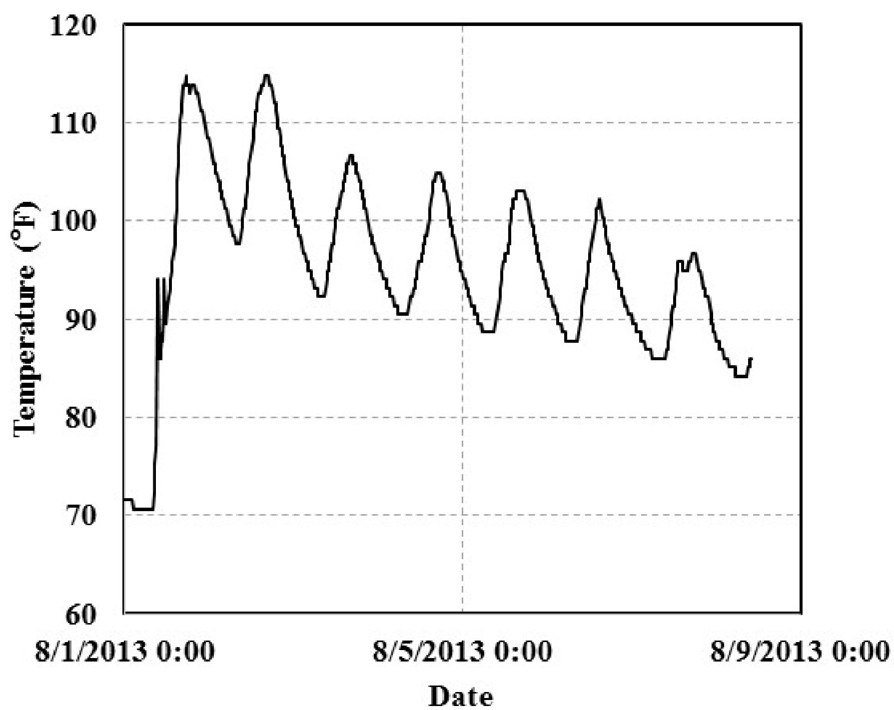

Figure A-39: RG4000CT OT 7-Day Temperature Data

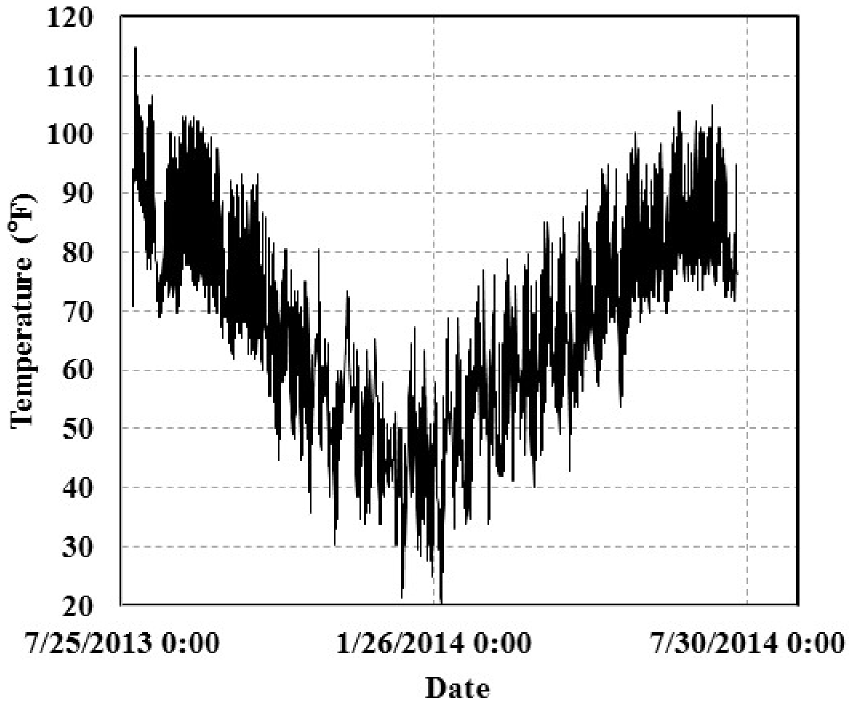

Figure A-40: RG4000CT OT 365-Day Temperature Data

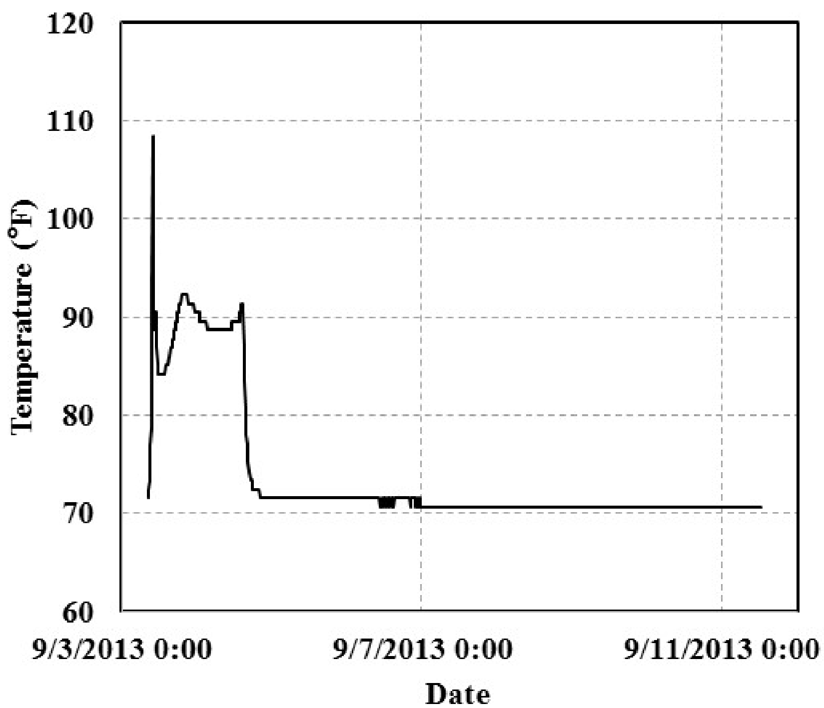

Figure A-41: RG4000SC C612M 7-Day Temperature Data

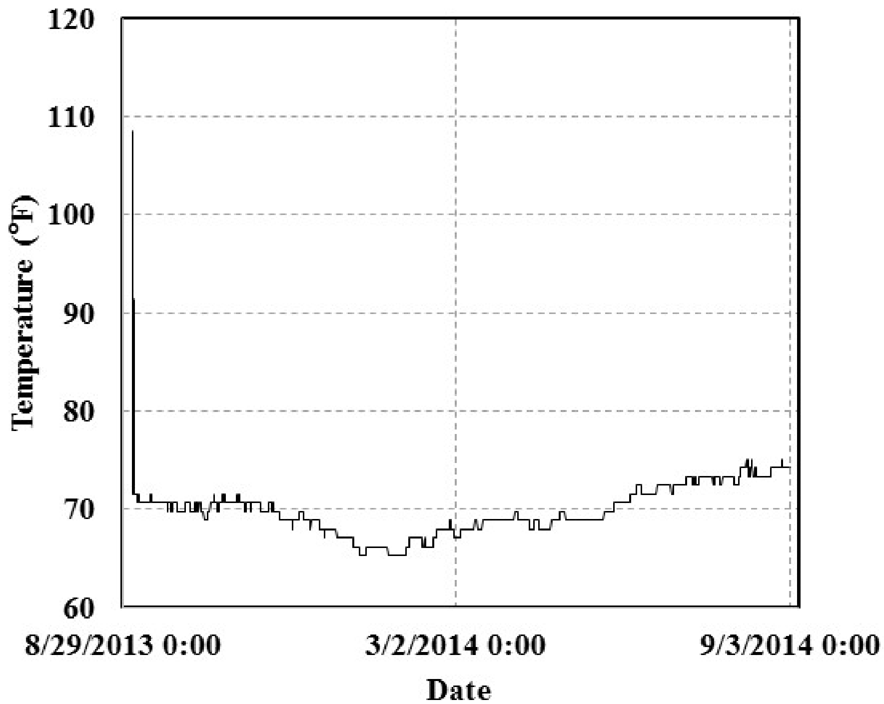

Figure A-42: RG4000SC C612M 365-Day Temperature Data

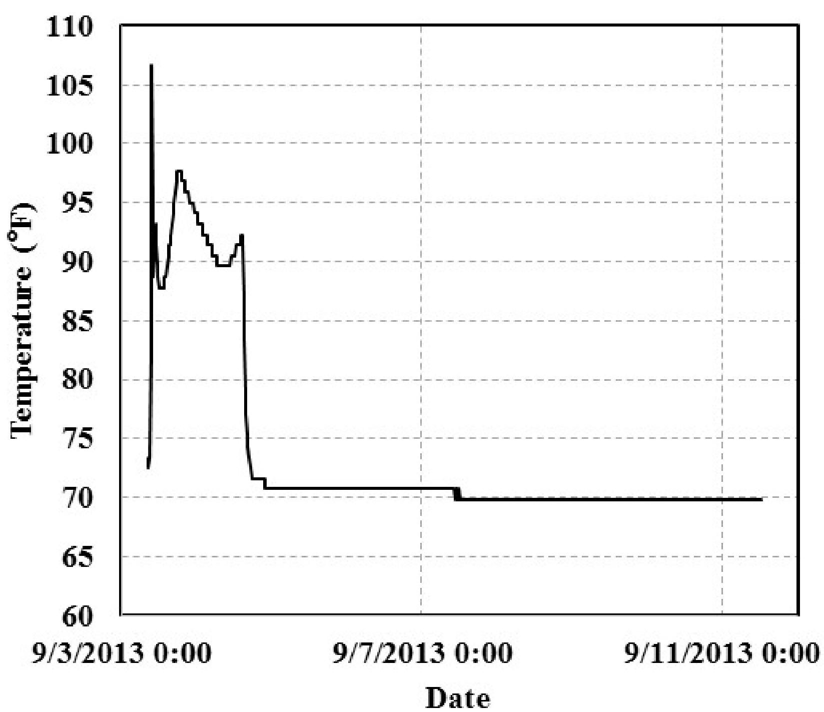

Figure A-43: RG4000SC C612T 7-Day Temperature Data

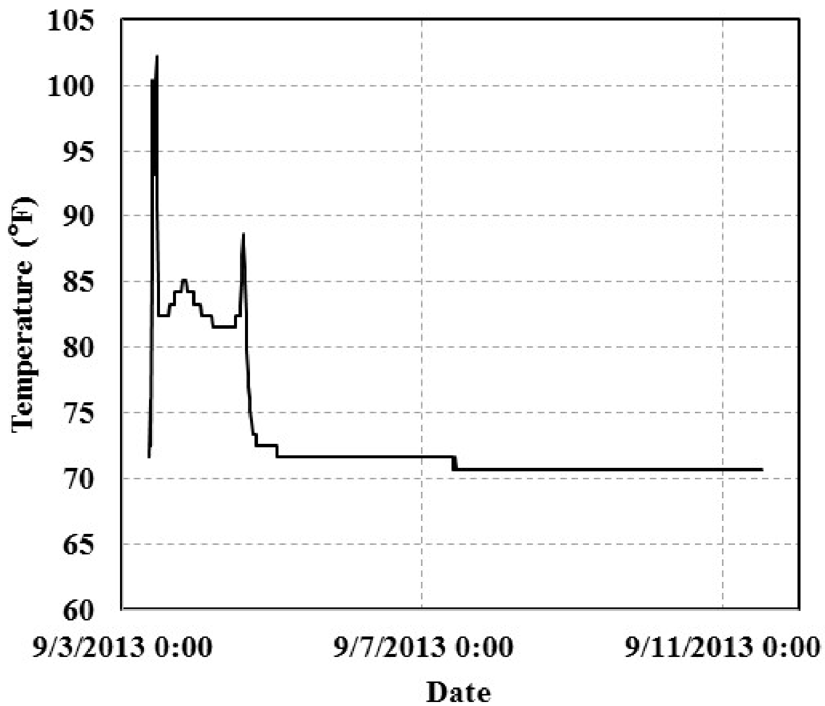

Figure A-44: RG4000SC C48M 7-Day Temperature Data

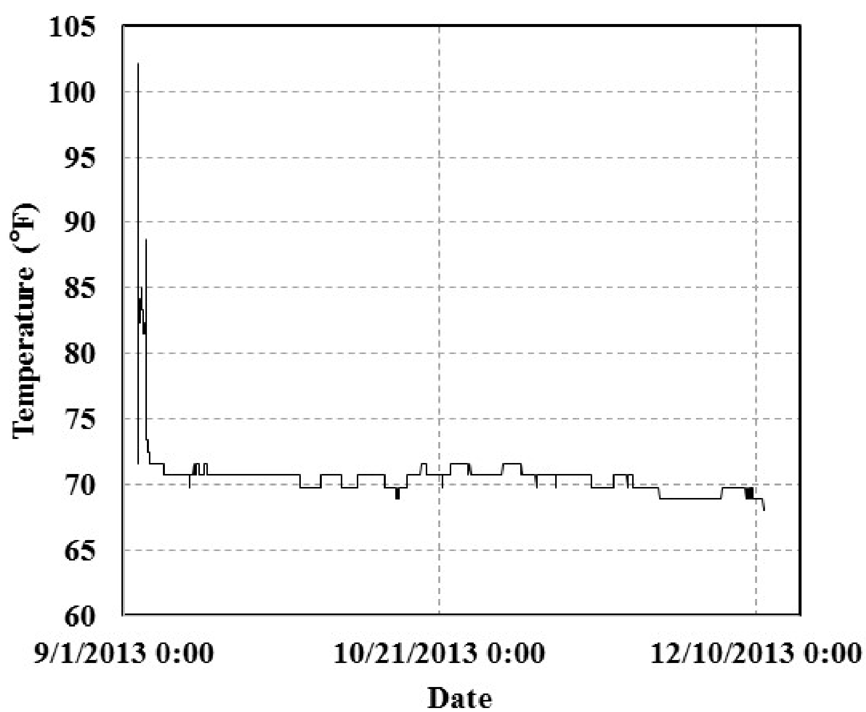

Figure A-45: RG4000SC C48M 91-Day Temperature Data

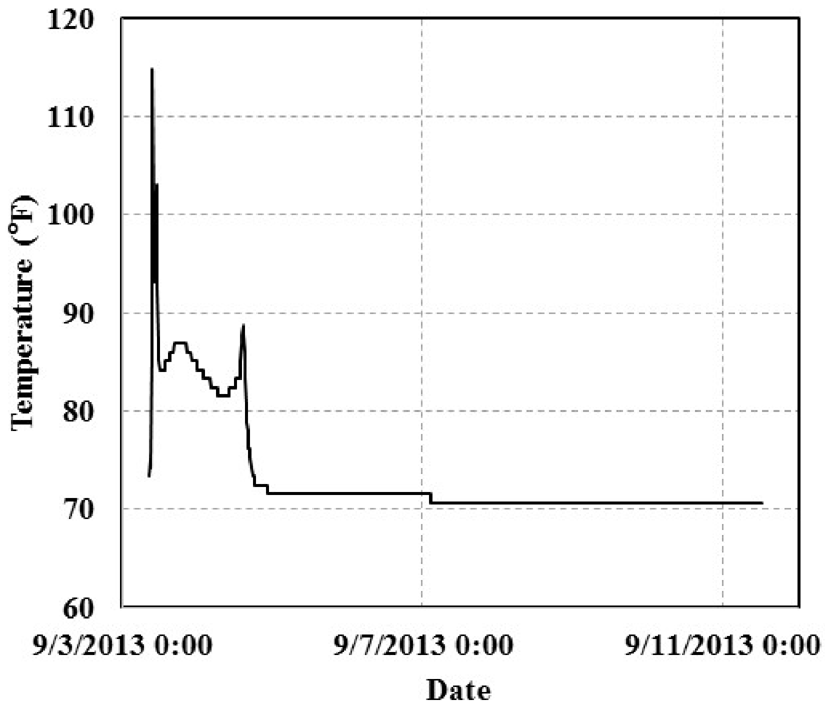

Figure A-46: RG4000SC C48T 7-Day Temperature Data

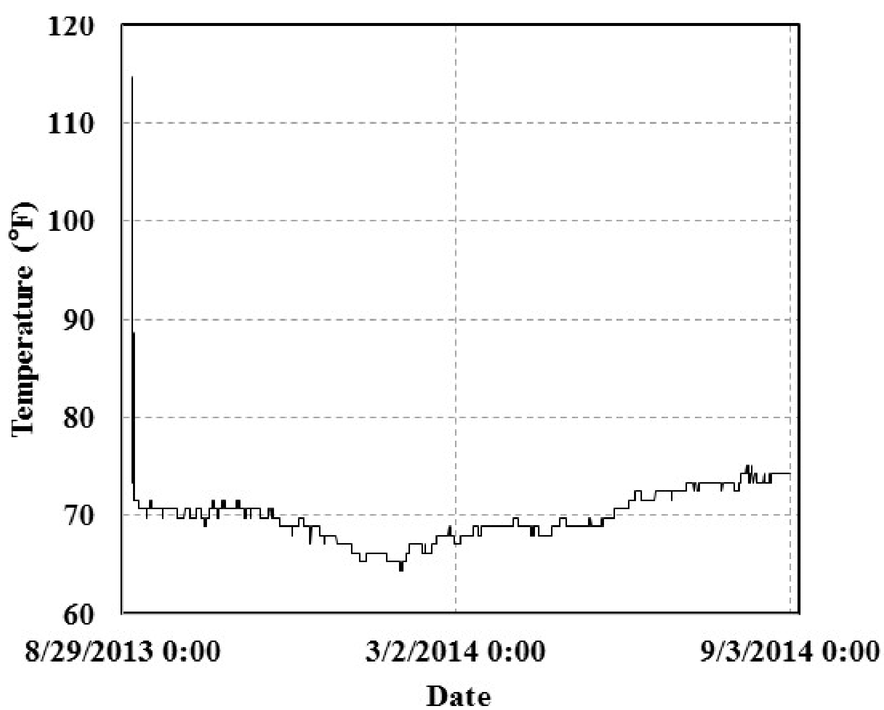

Figure A-47: RG4000SC C48T 365-Day Temperature Data

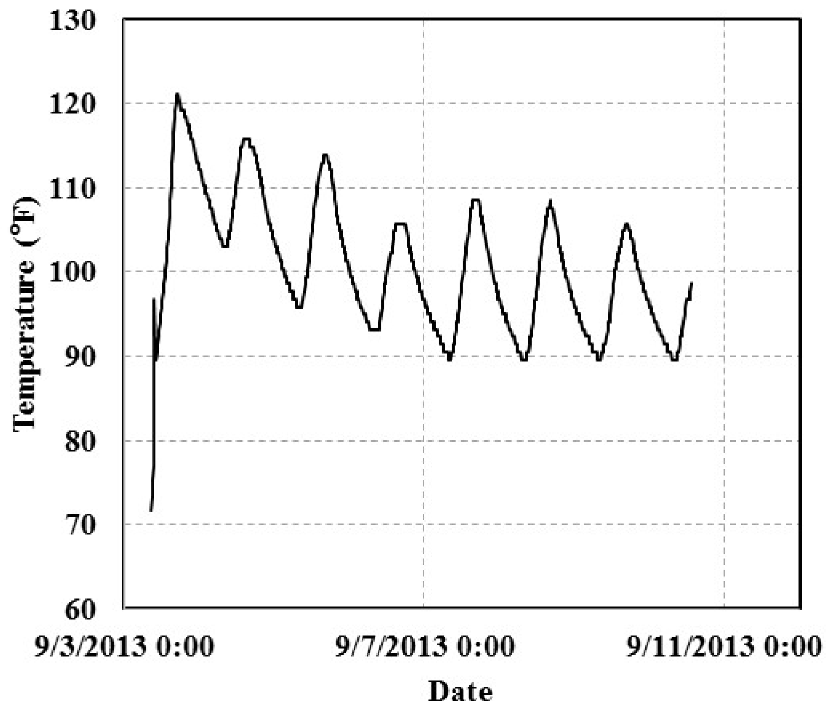

Figure A-48: RG4000SC IM 7-Day Temperature Data

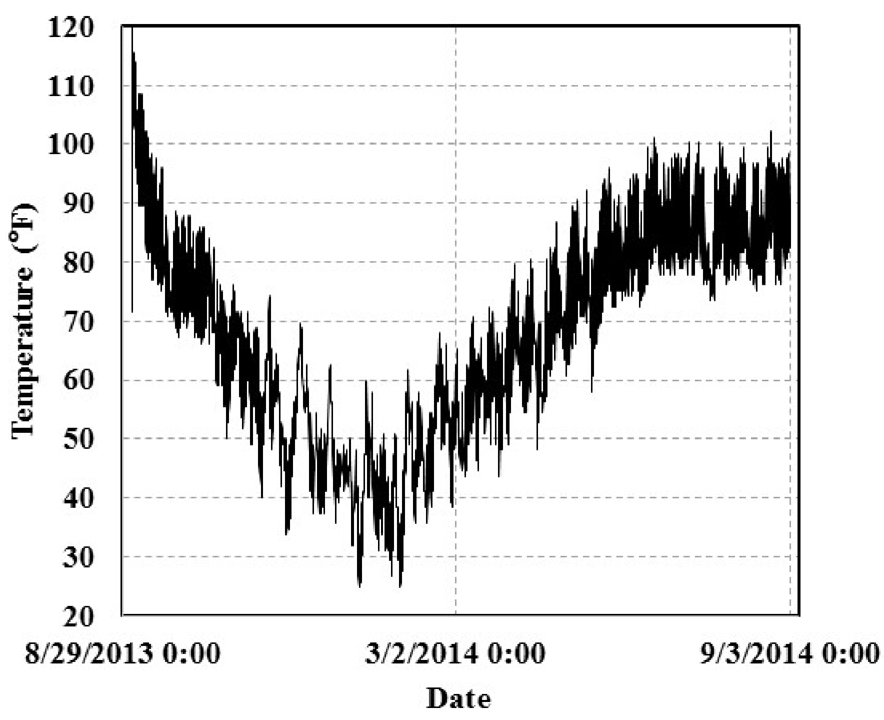

Figure A-49: RG4000SC IM 365-Day Temperature Data

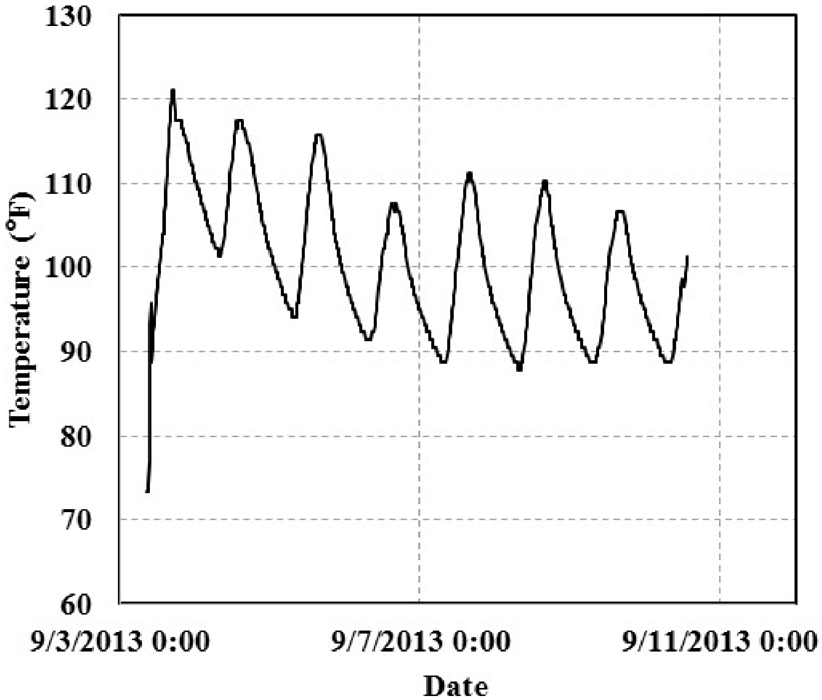

Figure A-50: RG4000SC IT 7-Day Temperature Data

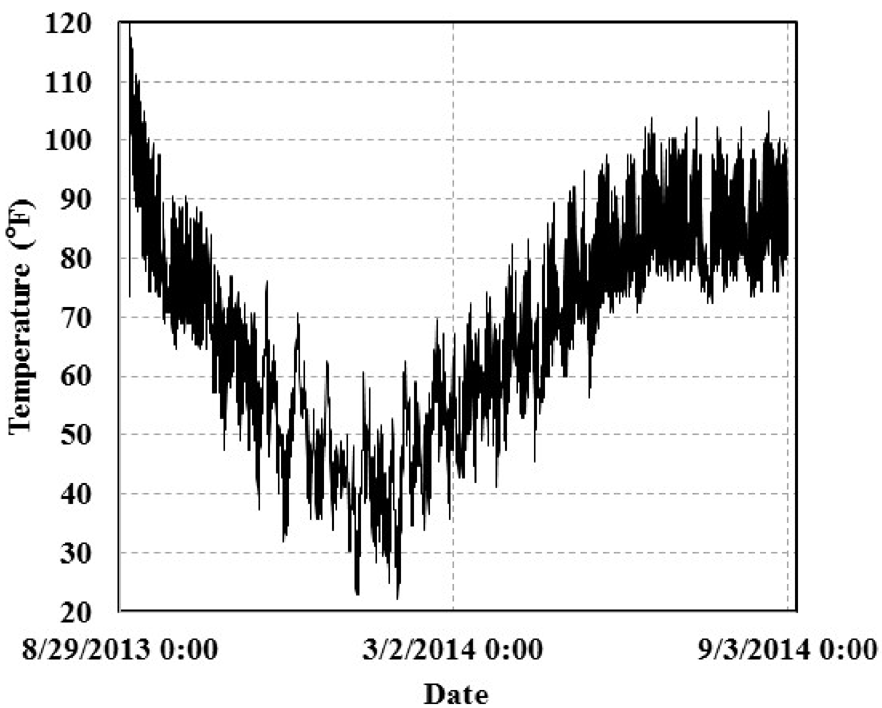

Figure A-51: RG4000SC IT 365-Day Temperature Data

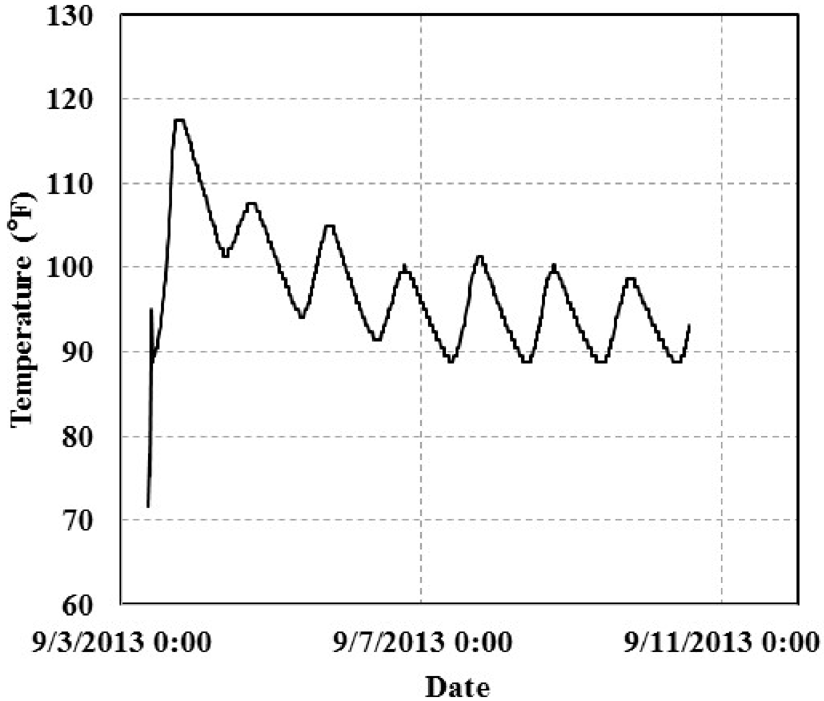

Figure A-52: RG4000SC OM 7-Day Temperature Data

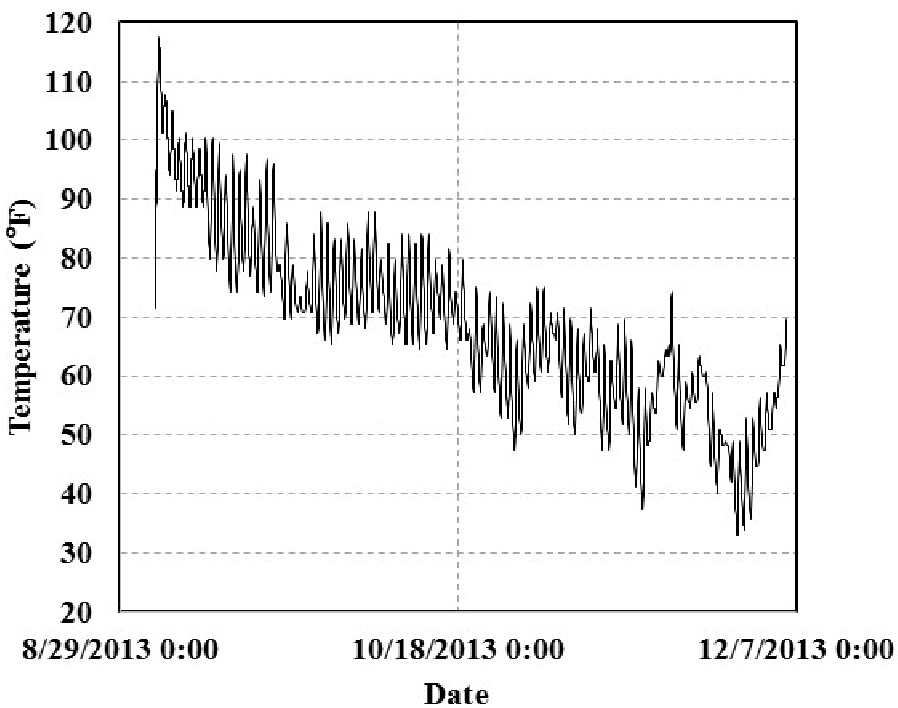

Figure A-53: RG4000SC OM 91-Day Temperature Data

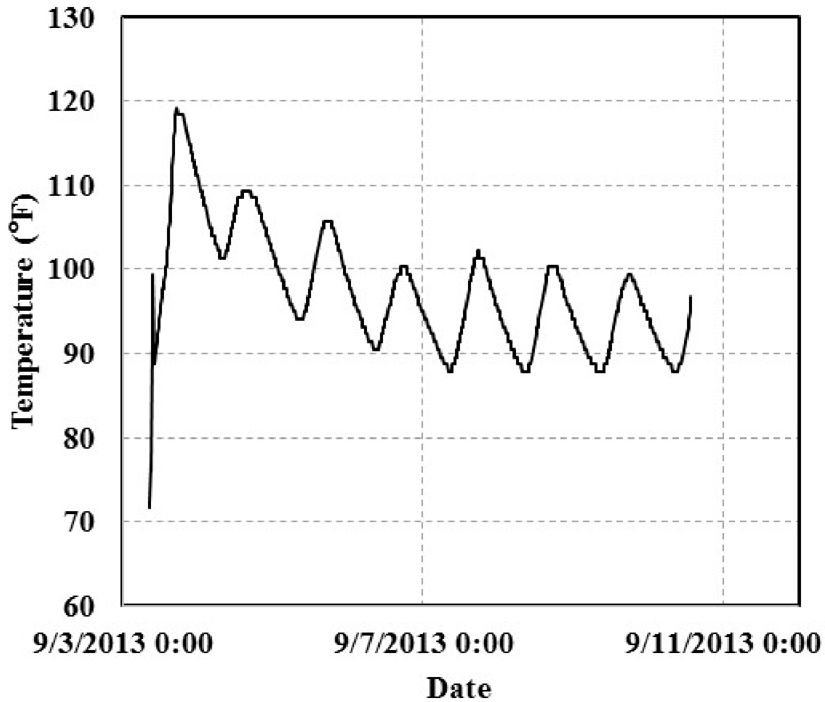

Figure A-54: RG4000SC OT 7-Day Temperature Data

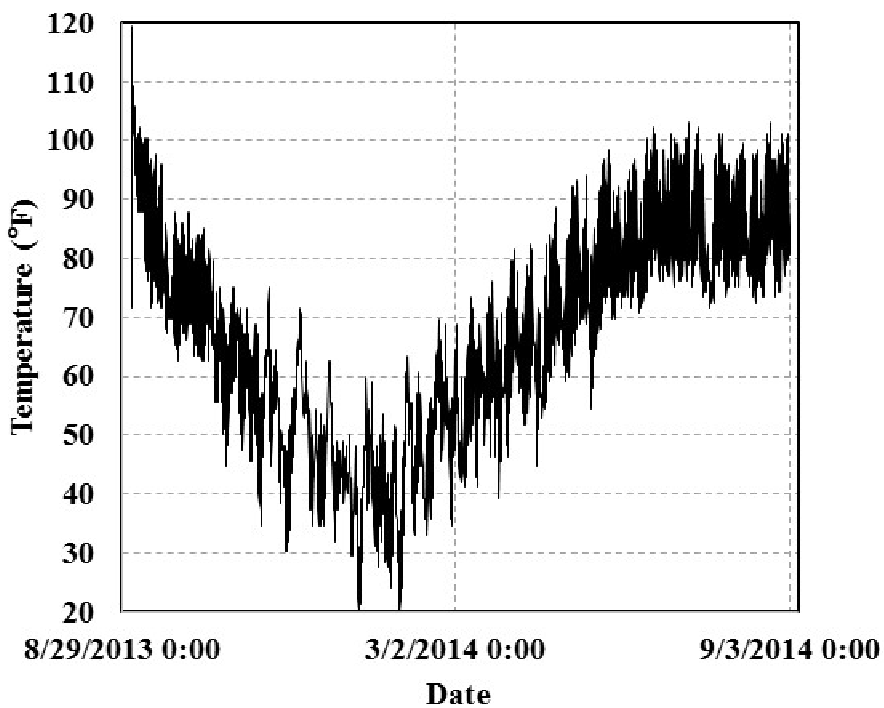

Figure A-55: RG4000SC OT 365-Day Temperature Data

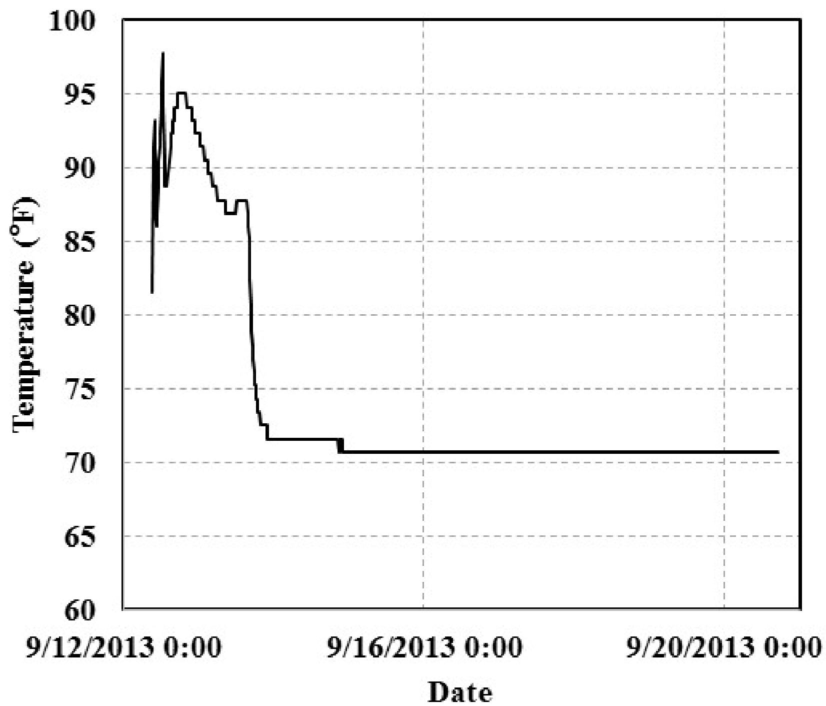

Figure A-56: GR4000CT C612M 7-Day Temperature Data

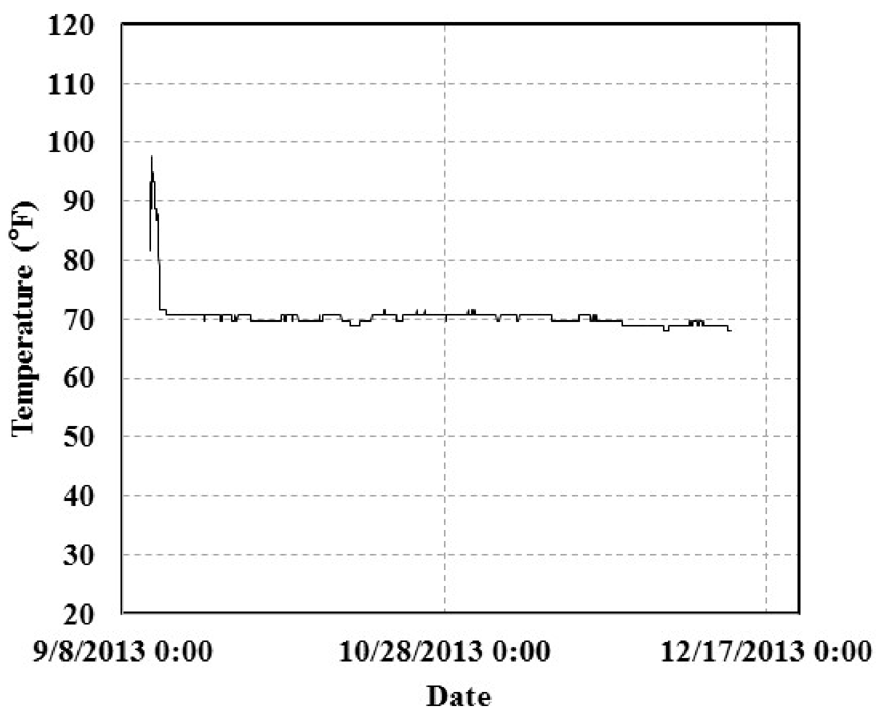

Figure A-57: GR4000CT C612M 91-Day Temperature Data

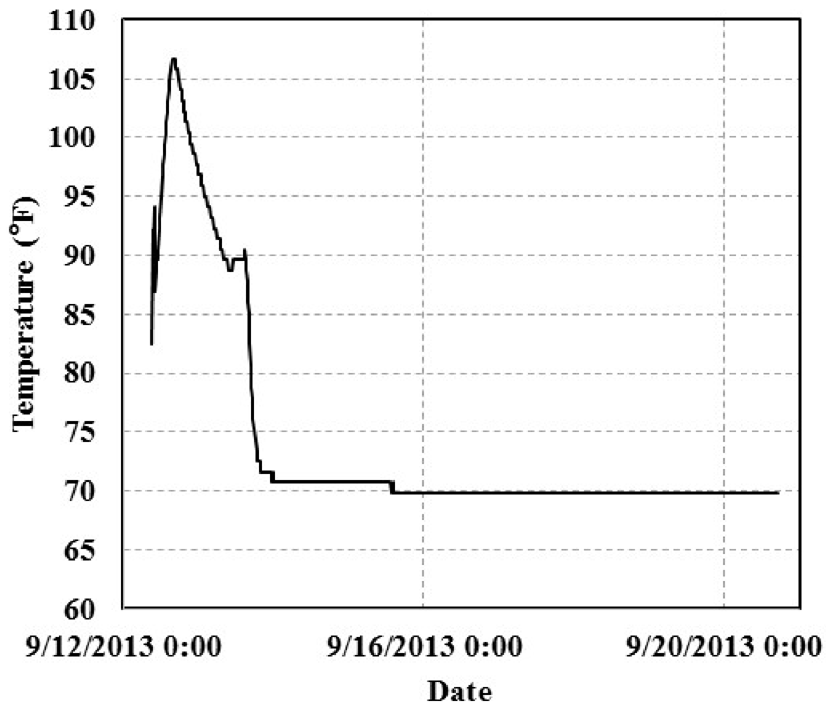

Figure A-58: GR4000CT C612T 7-Day Temperature Data

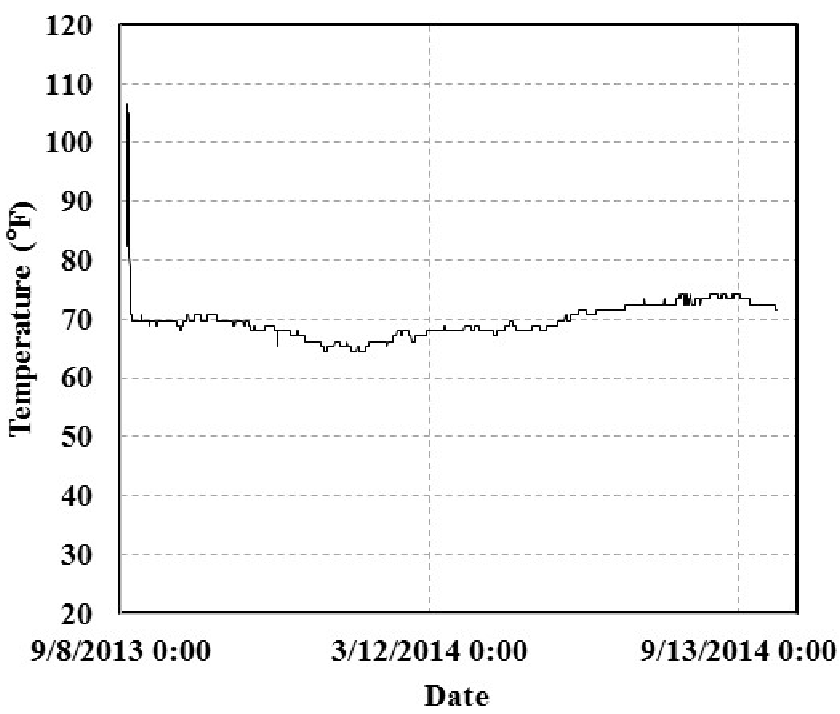

Figure A-59: GR4000CT C612T 365-Day Temperature Data

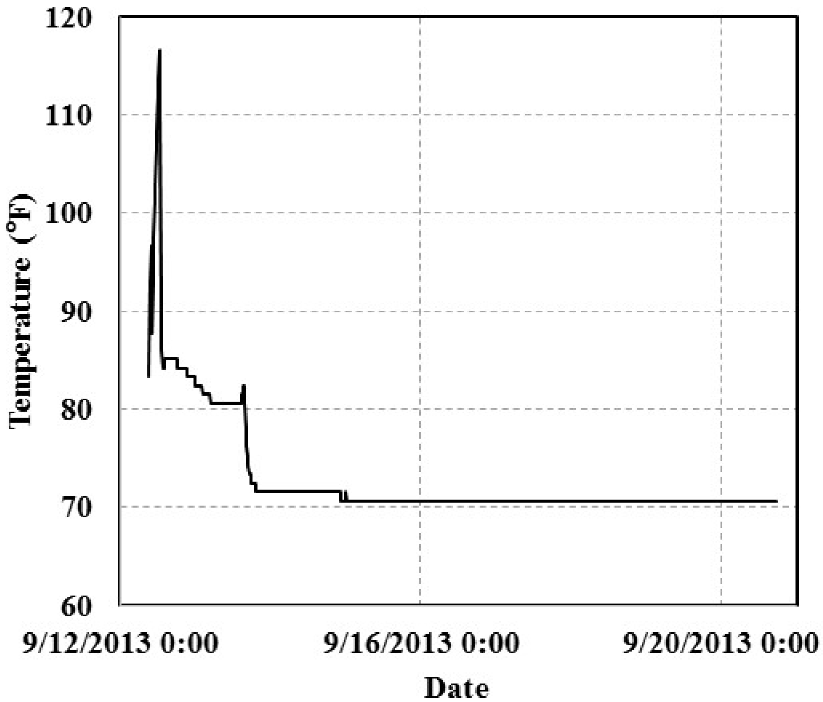

Figure A-60: GR4000CT C48M 7-Day Temperature Data

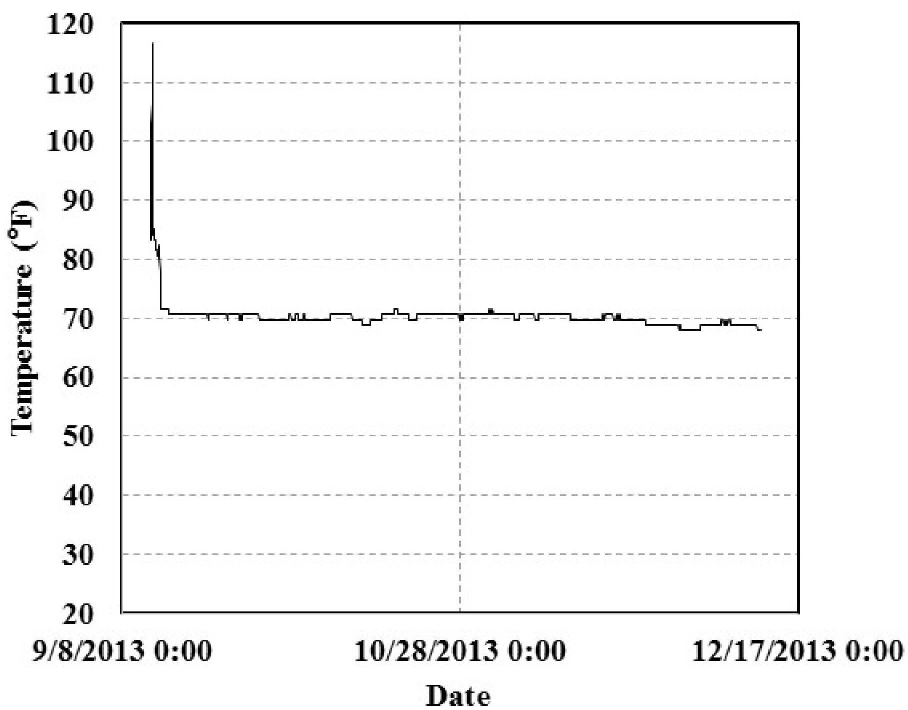

Figure A-61: GR4000CT C48M 91-Day Temperature Data

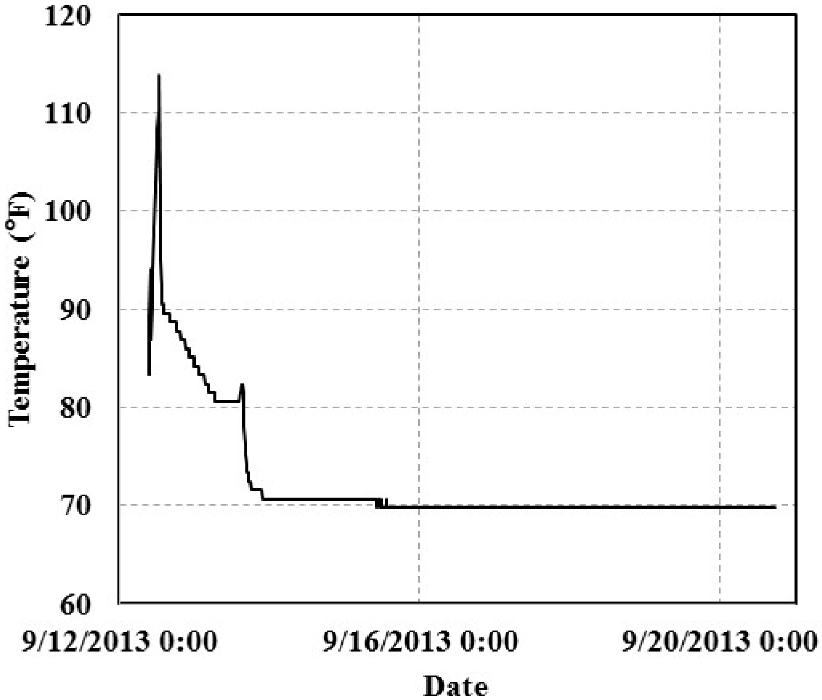

Figure A-62: GR4000CT C48T 7-Day Temperature Data

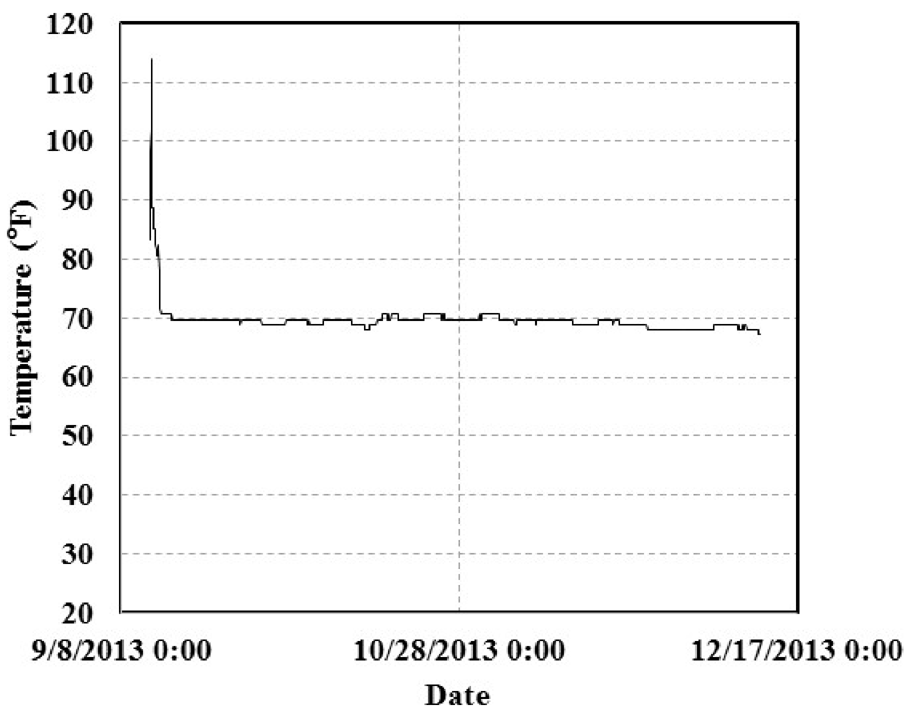

Figure A-63: GR4000CT C48T 91-Day Temperature Data

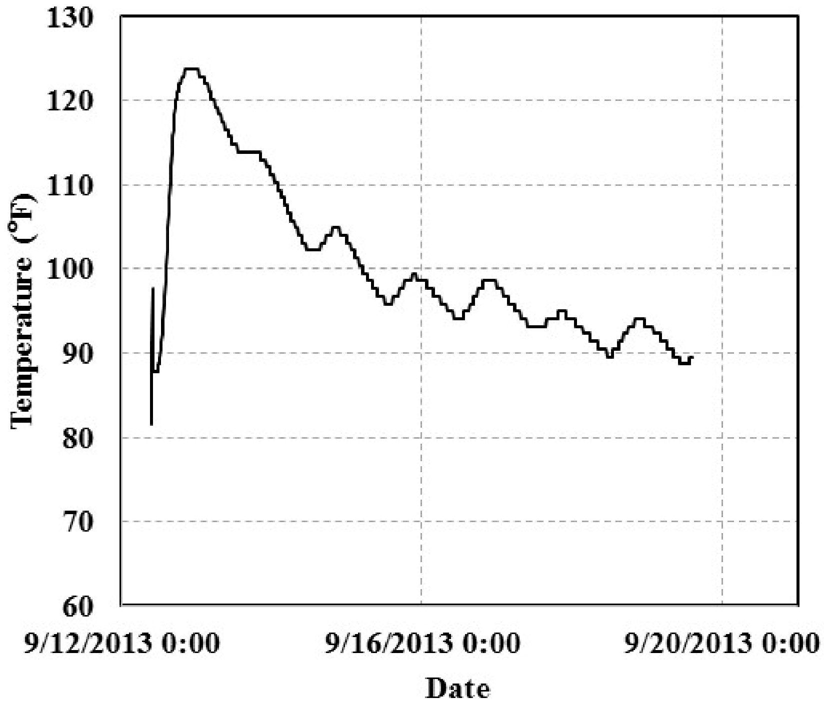

Figure A-64: GR4000CT IM 7-Day Temperature Data

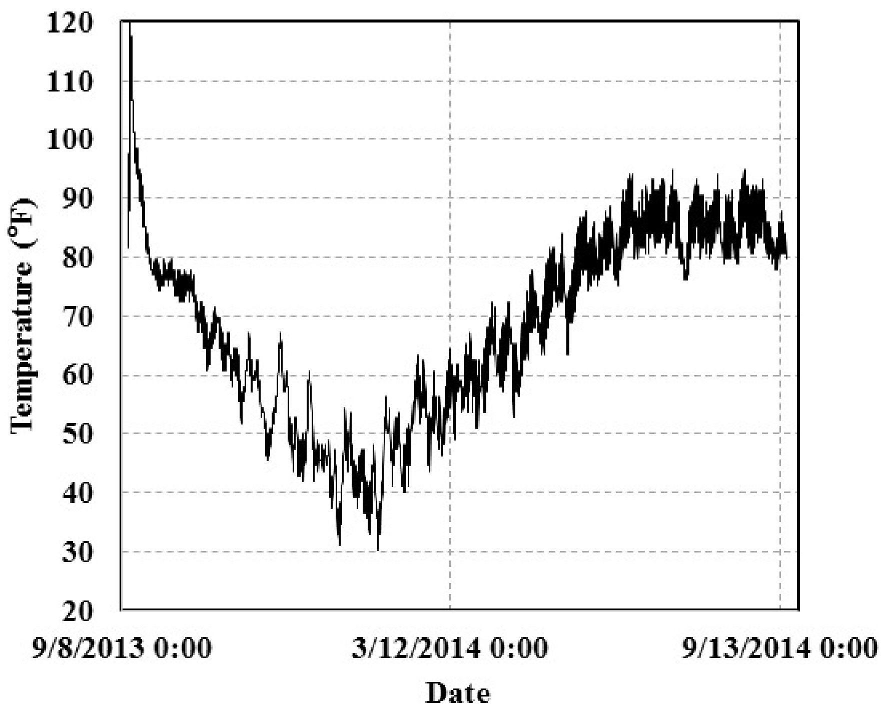

Figure A-65: GR4000CT IM 365-Day Temperature Data

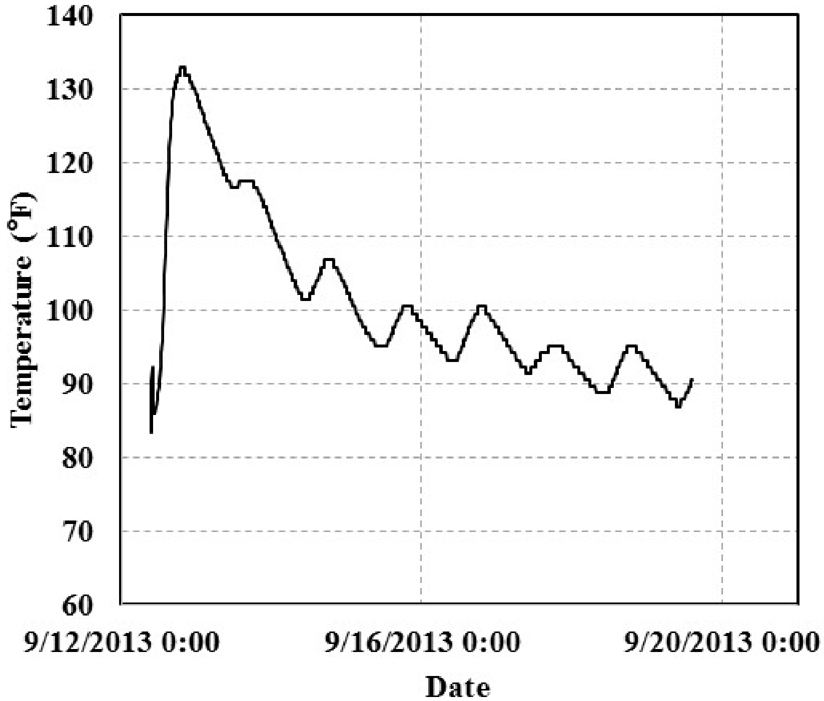

Figure A-66: GR4000CT IT 7-Day Temperature Data

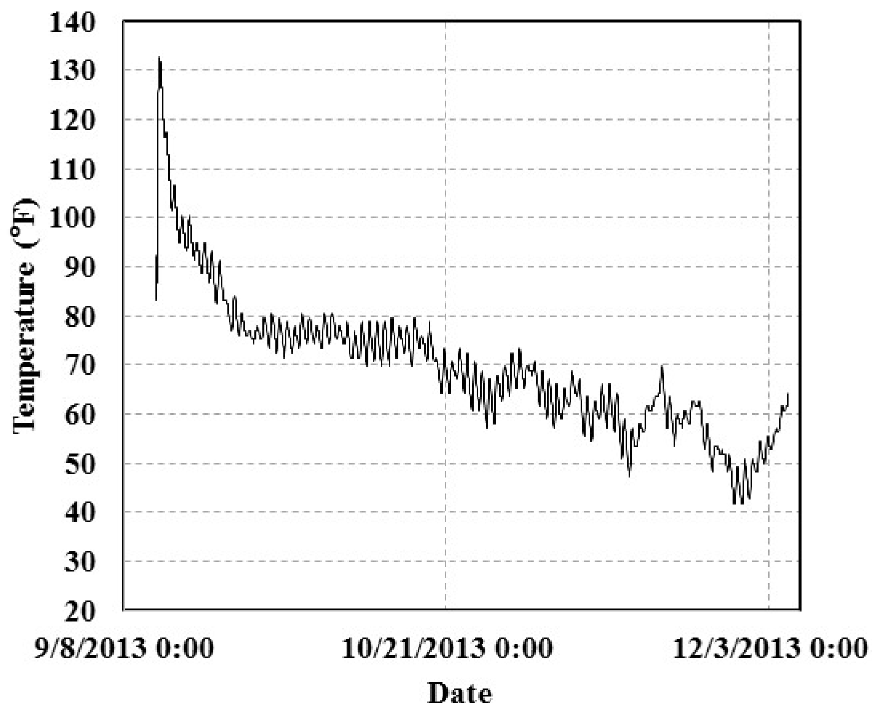

Figure A-67: GR4000CT IT 91-Day Temperature Data

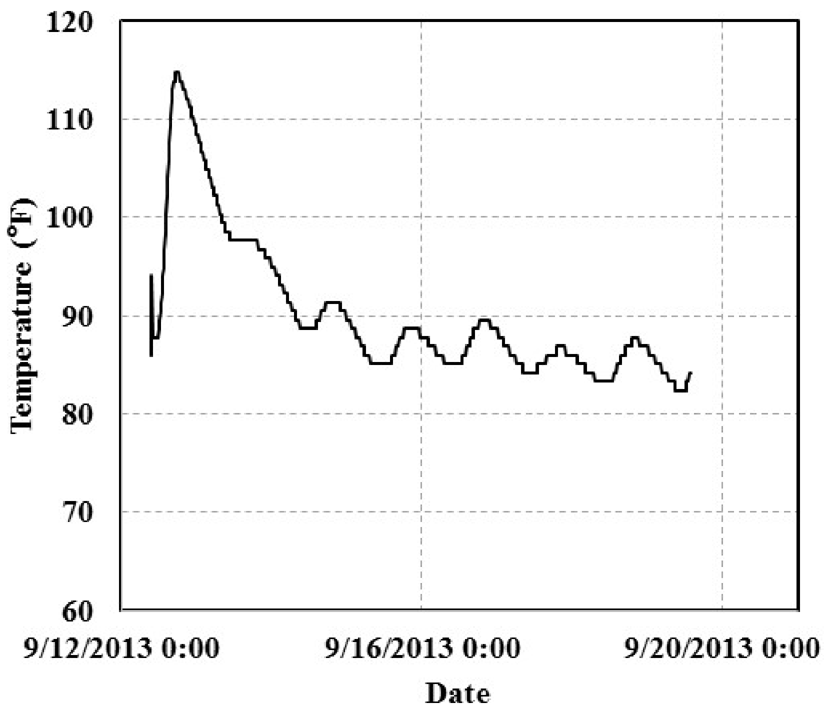

Figure A-68: GR4000CT OM 7-Day Temperature Data

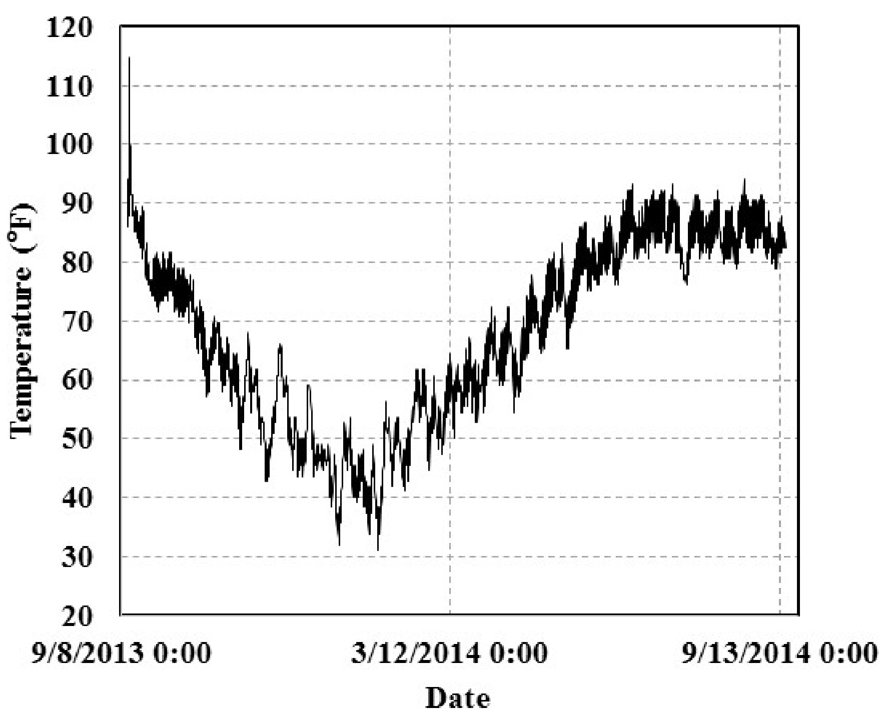

Figure A-69: GR4000CT OM 365-Day Temperature Data

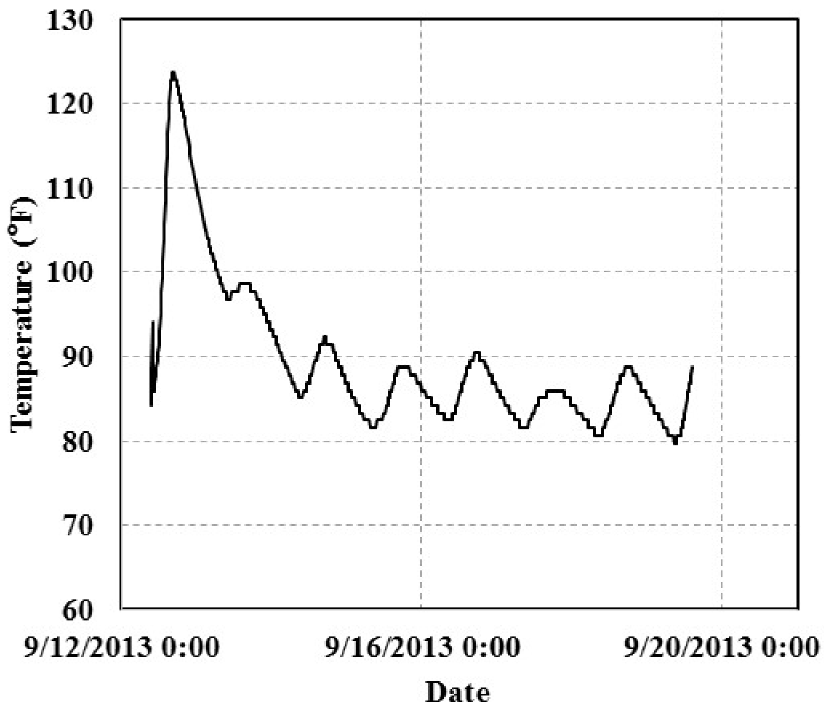

Figure A-70: GR4000CT OT 7-Day Temperature Data

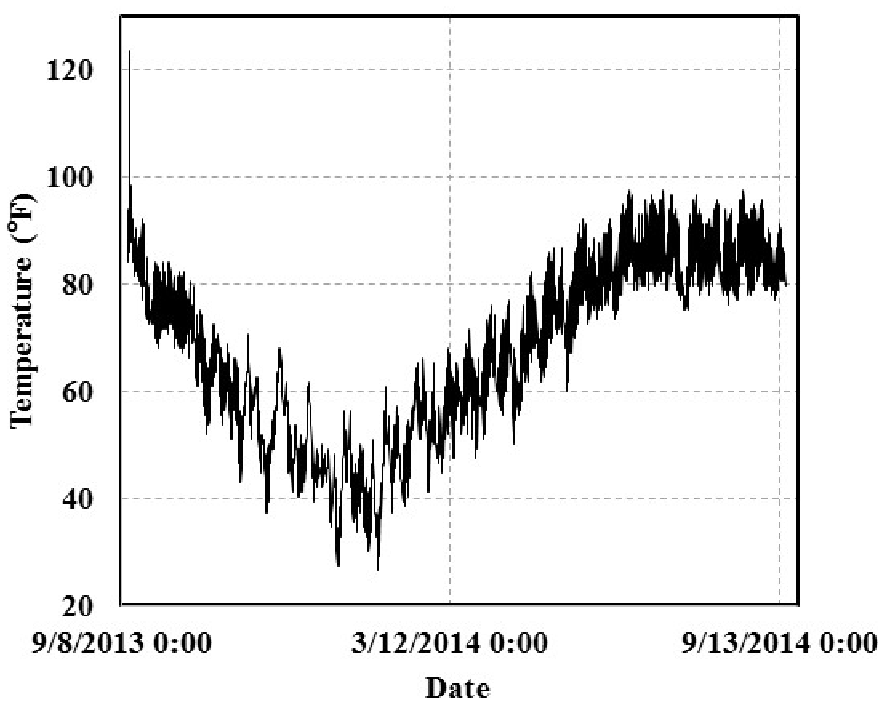

Figure A-71: GR4000CT OT 365-Day Temperature Data

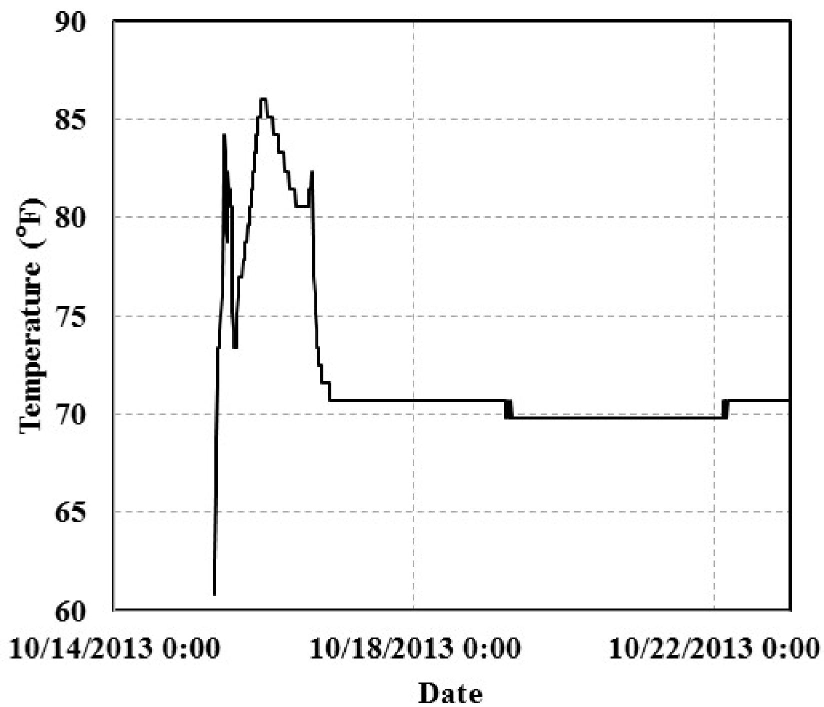

Figure A-72: RG4000FA C612M 7-Day Temperature Data

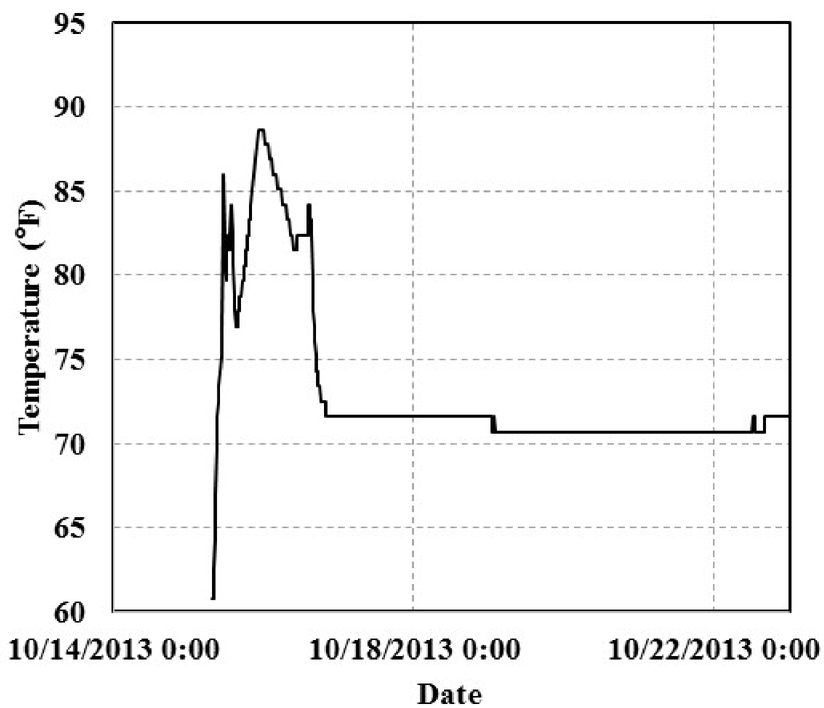

Figure A-73: RG4000FA C612T 7-Day Temperature Data

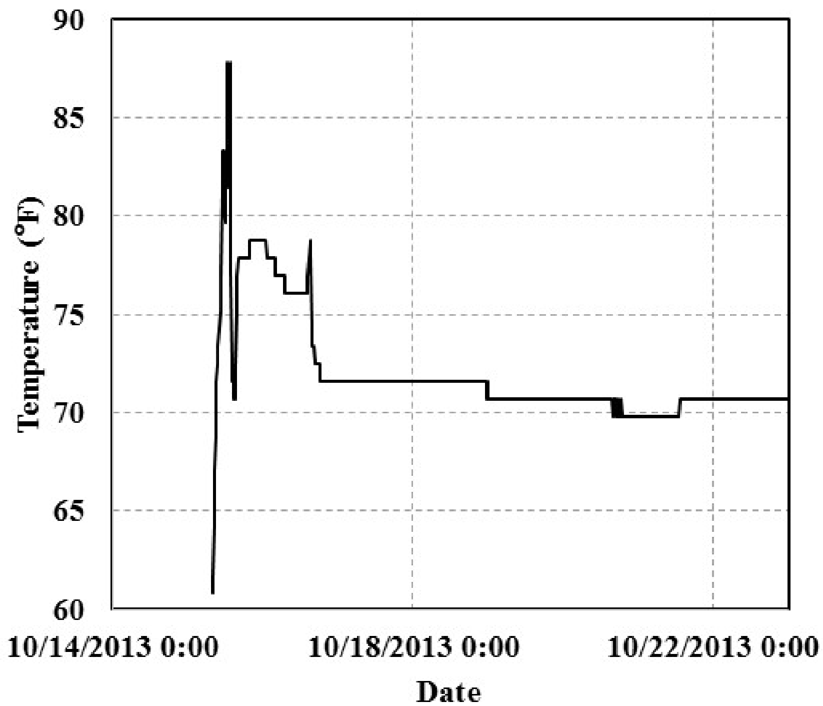

Figure A-74: RG4000FA C48M 7-Day Temperature Data

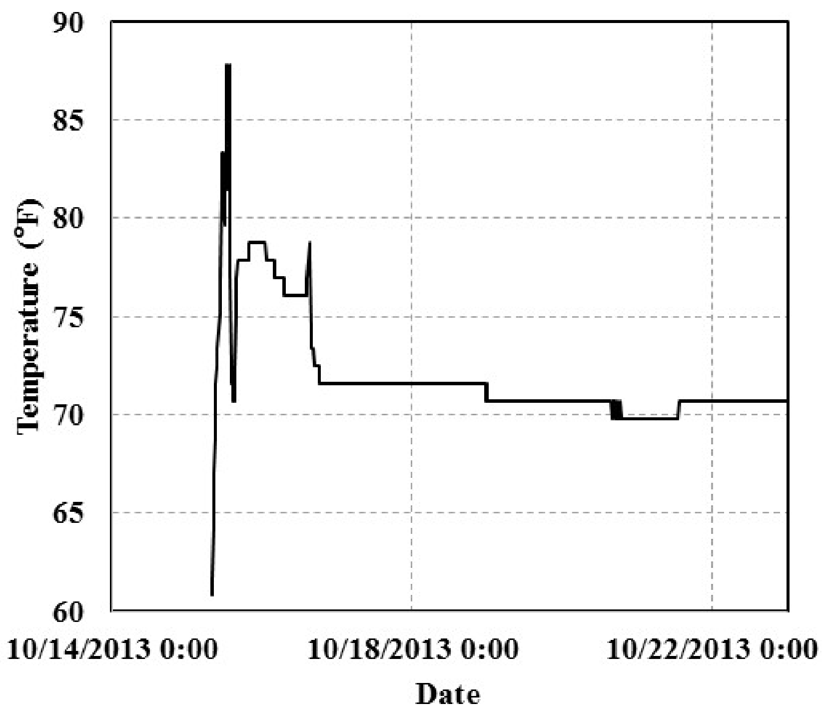

Figure A-75: RG4000FA C48T 7-Day Temperature Data

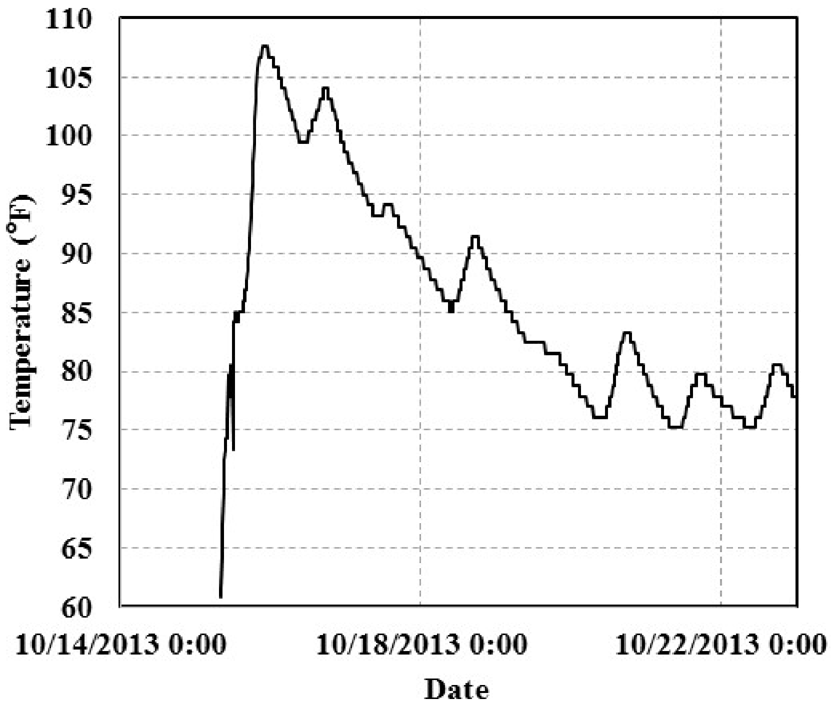

Figure A-76: RG4000FA IM 7-Day Temperature Data

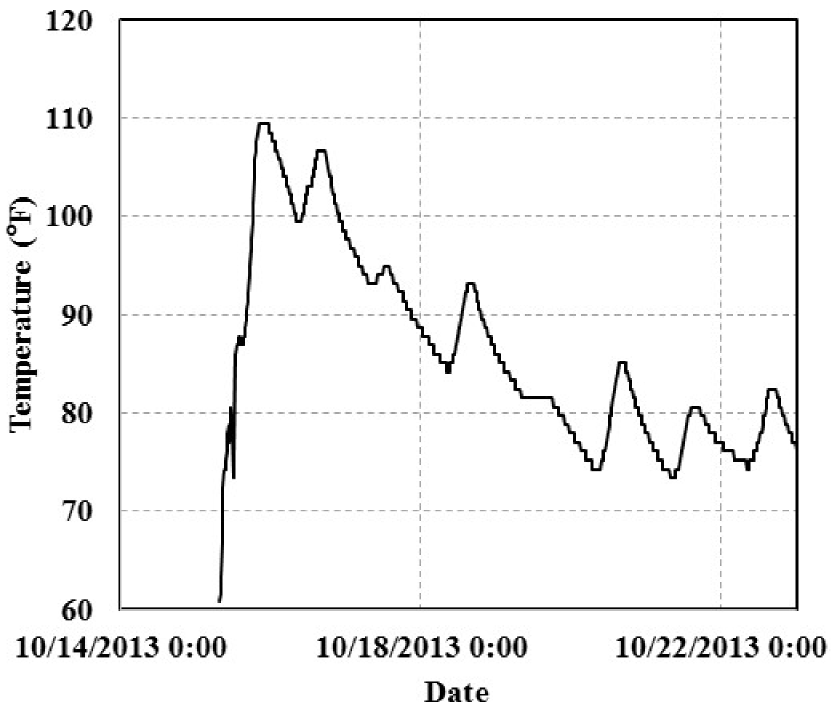

Figure A-77: RG4000FA IT 7-Day Temperature Data

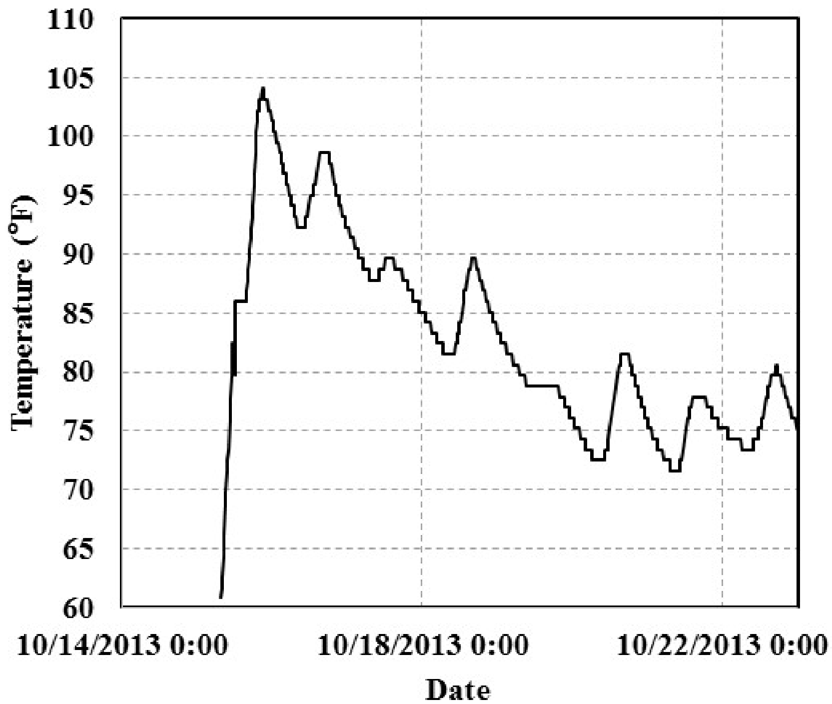

Figure A-78: RG4000FA OM 7-Day Temperature Data

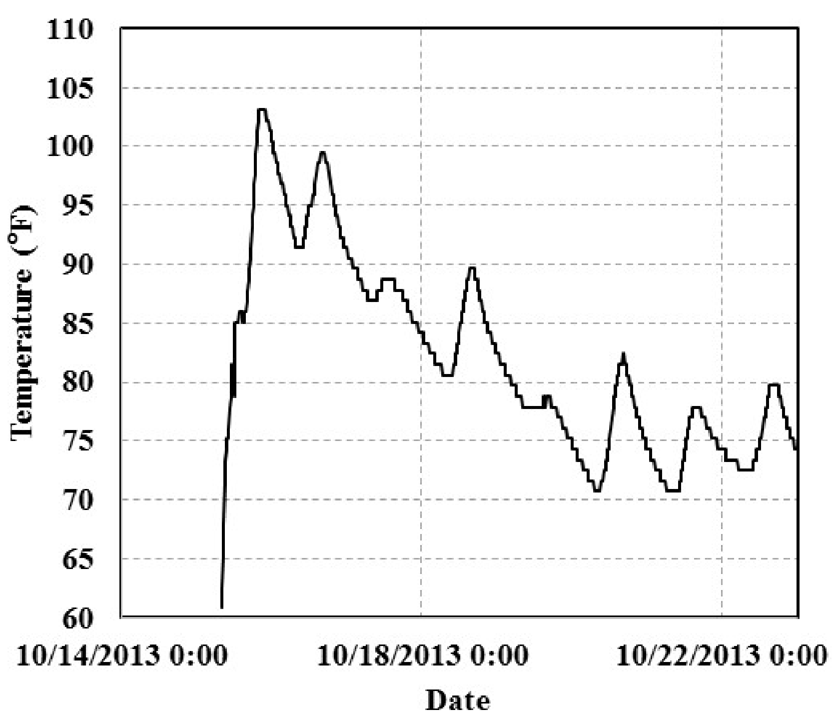

Figure A-79: RG4000FA OT 7-Day Temperature Data

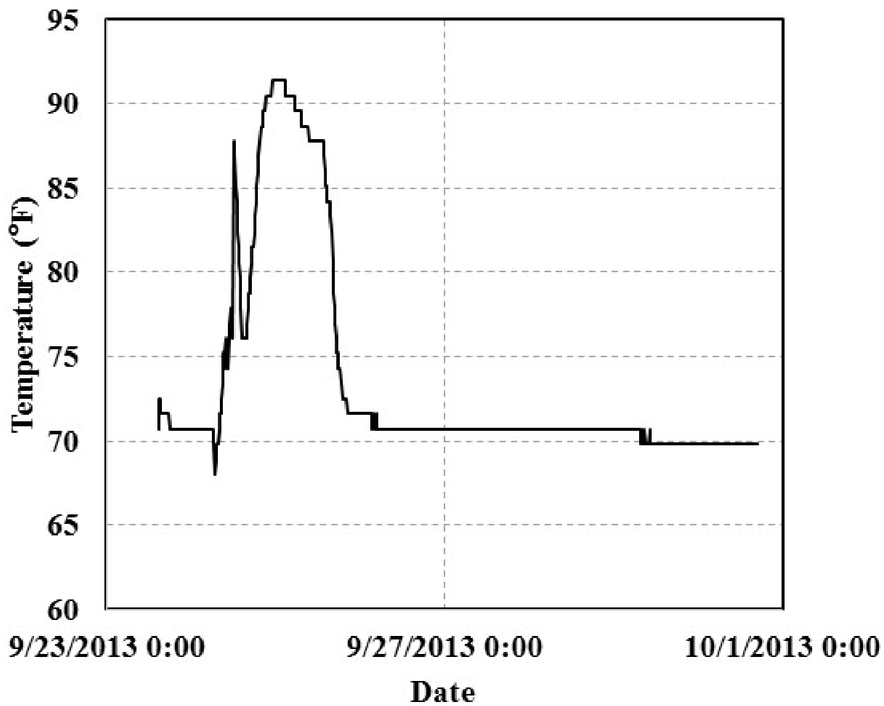

Figure A-80: RG8000CT C612M 7-Day Temperature Data

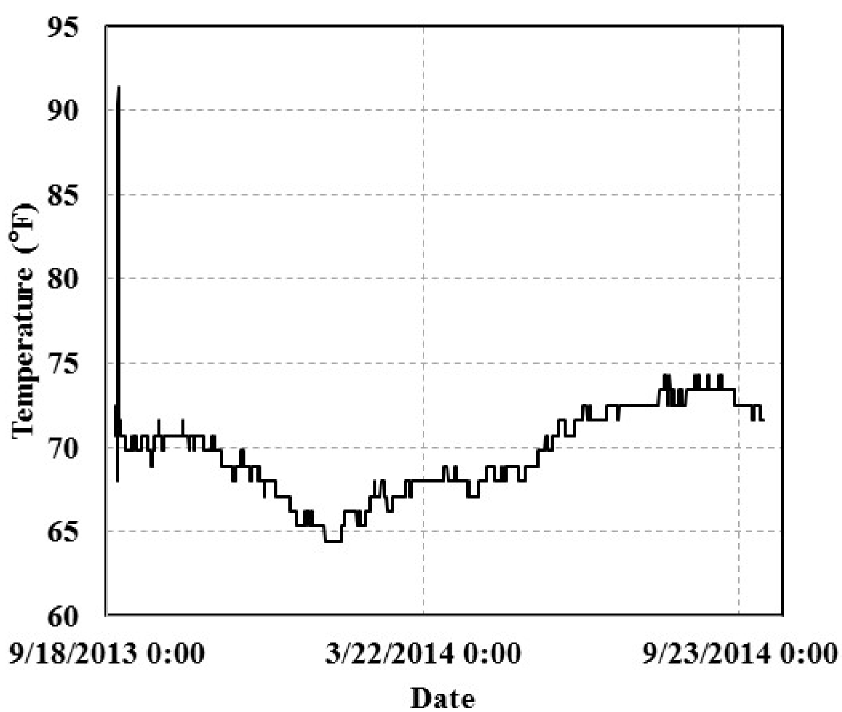

Figure A-81: RG8000CT C612M 365-Day Temperature Data

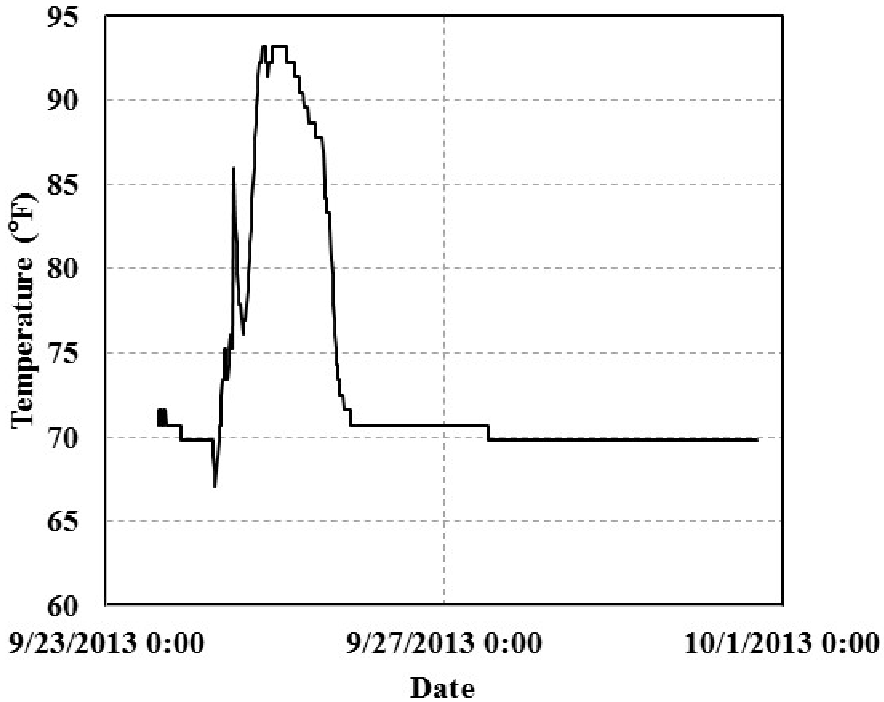

Figure A-82: RG8000CT C612T 7-Day Temperature Data

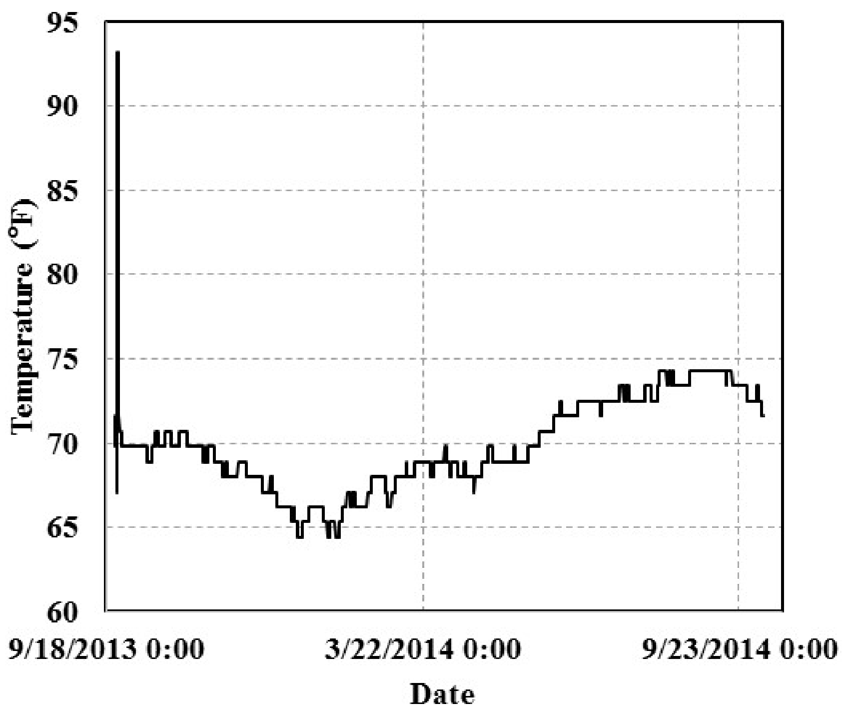

Figure A-83: RG8000CT C612T 365-Day Temperature Data

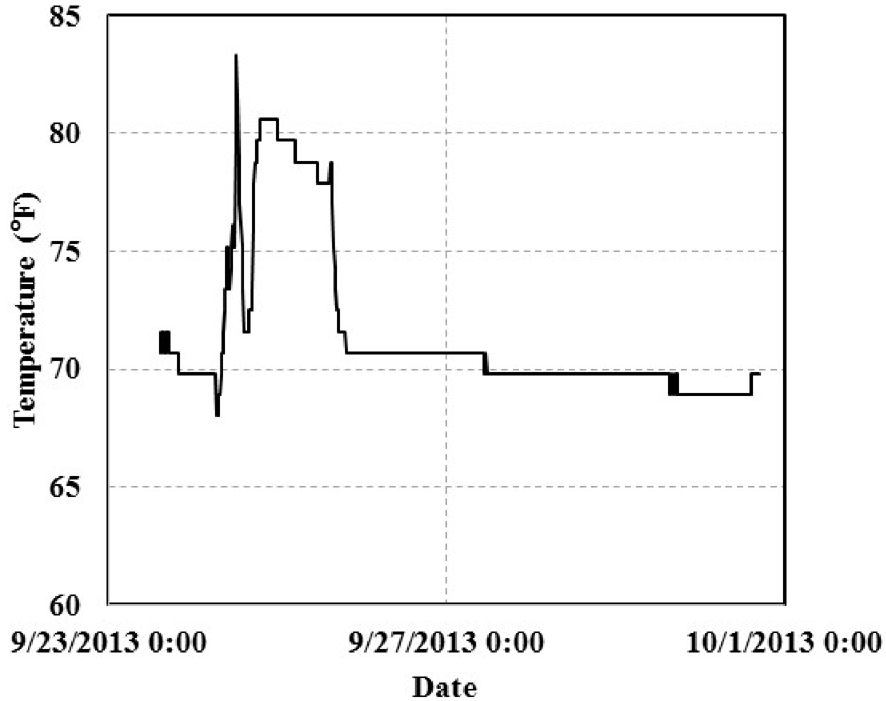

Figure A-84: RG8000CT C48M 7-Day Temperature Data

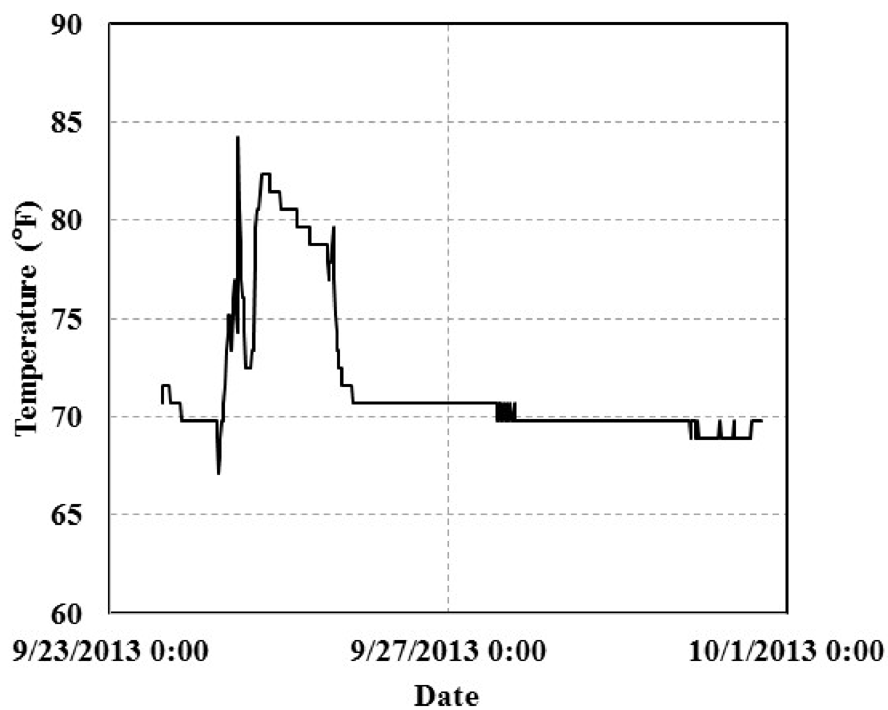

Figure A-85: RG8000CT C48T 7-Day Temperature Data

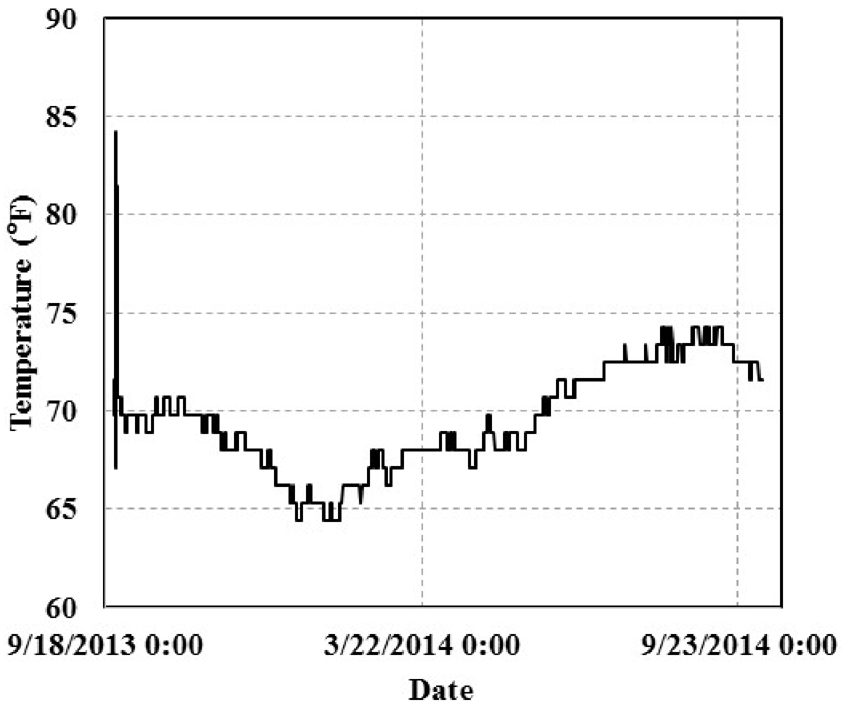

Figure A-86: RG8000CT C48T 365-Day Temperature Data

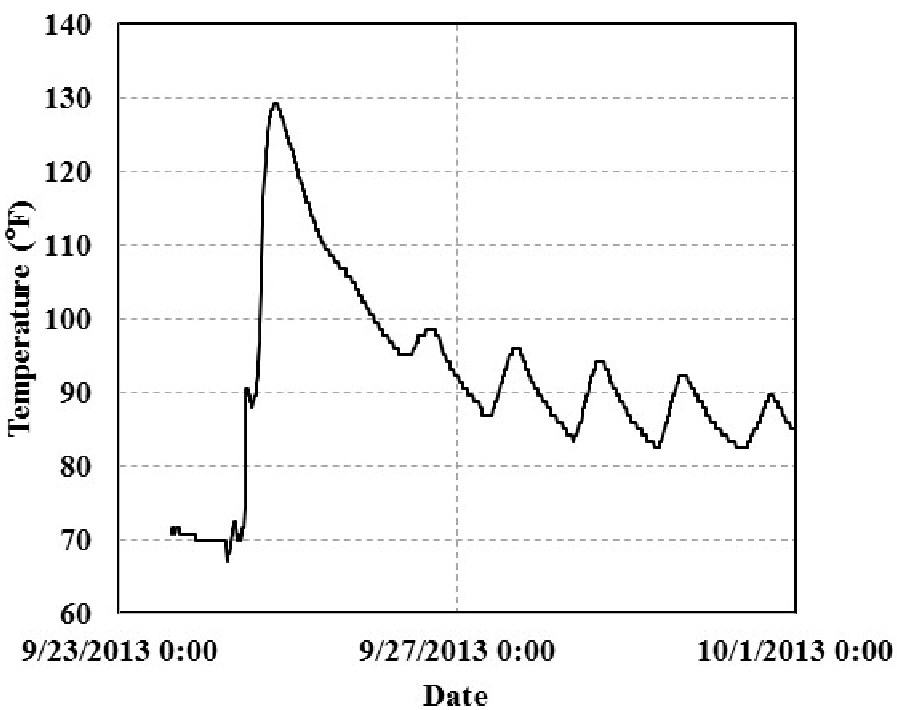

Figure A-87: RG8000CT IM 7-Day Temperature Data

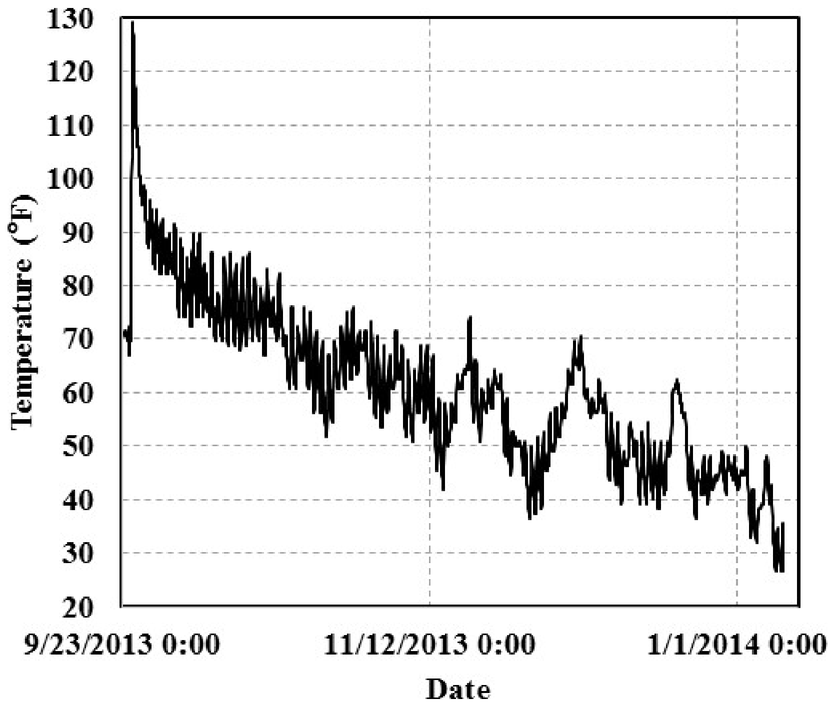

Figure A-88: RG8000CT IM 91-Day Temperature Data

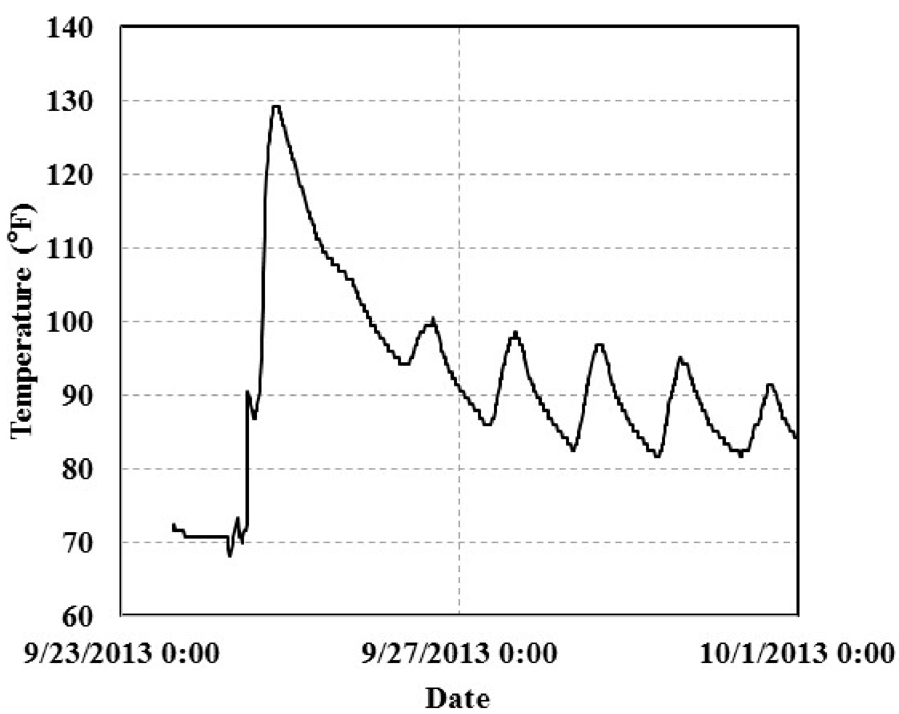

Figure A-89: RG8000CT IT 7-Day Temperature Data

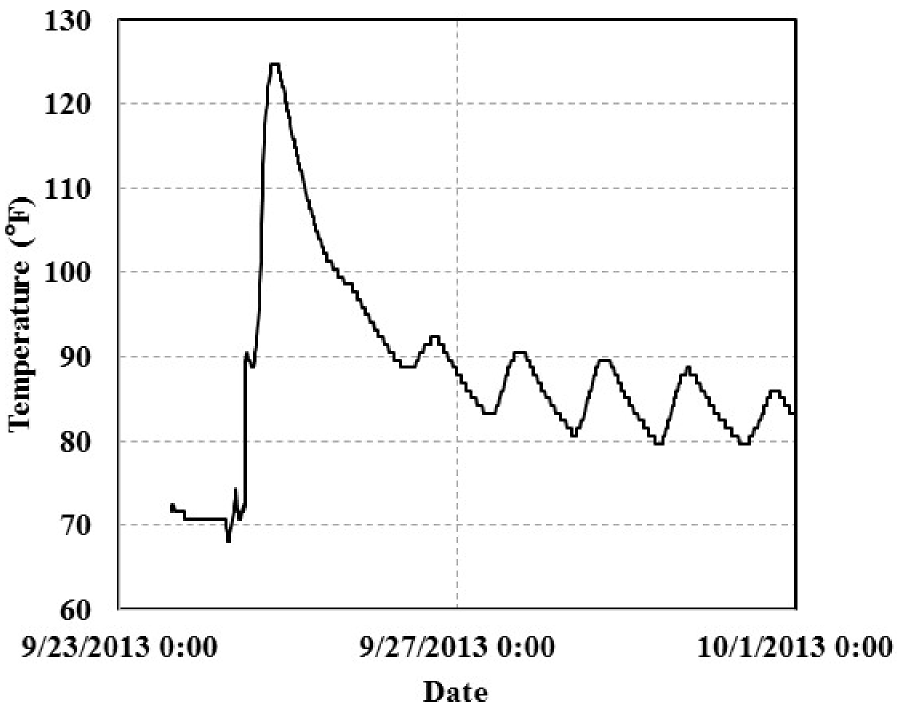

Figure A-90: RG8000CT OM 7-Day Temperature Data

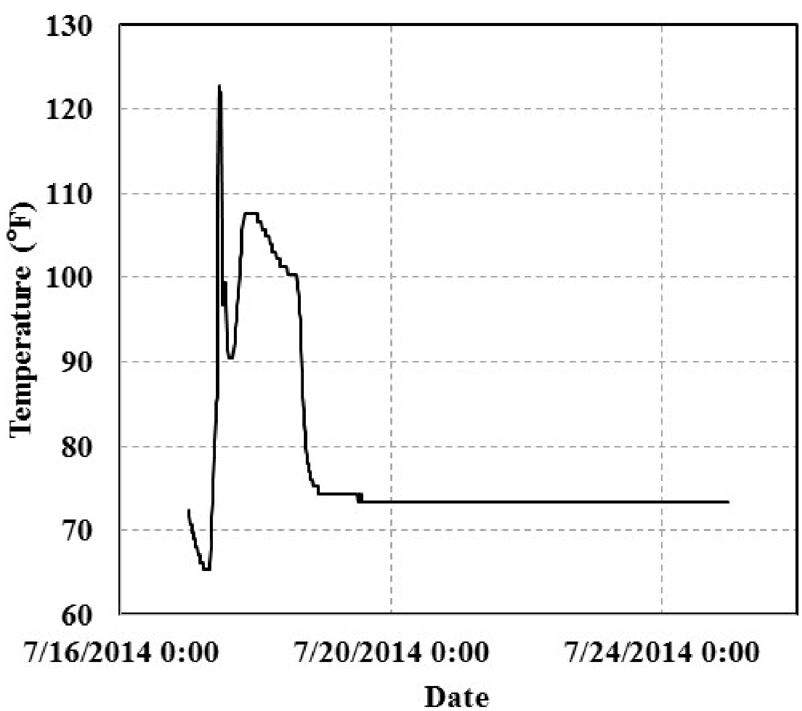

Figure A-91: LS8000CT C612M 7-Day Temperature Data

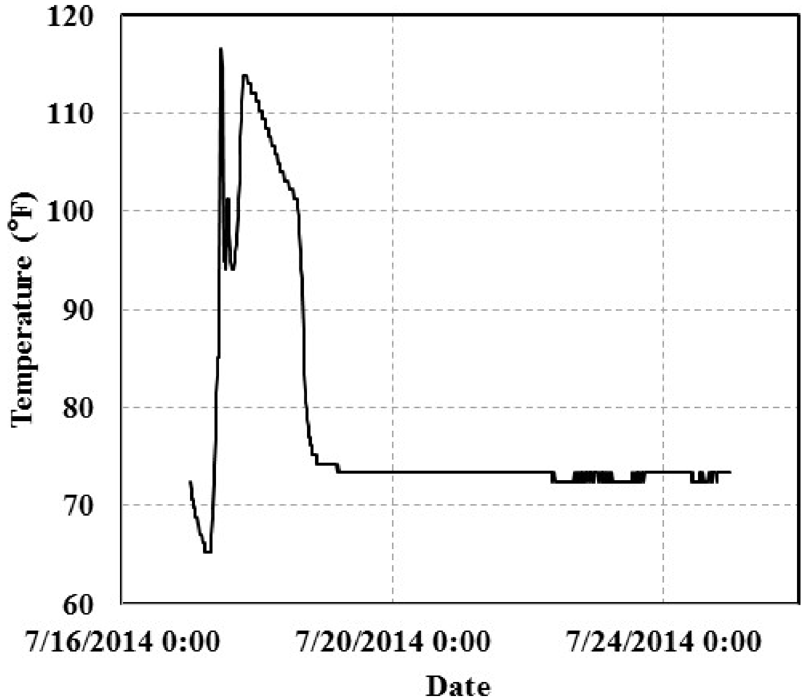

Figure A-92: LS8000CT C612T 7-Day Temperature Data

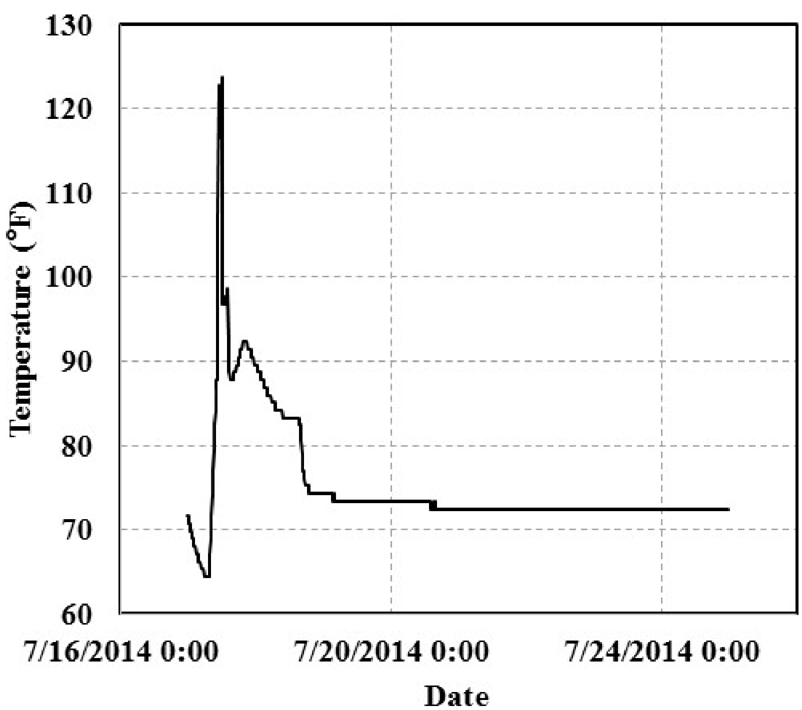

Figure A-93: LS8000CT C48M 7-Day Temperature Data

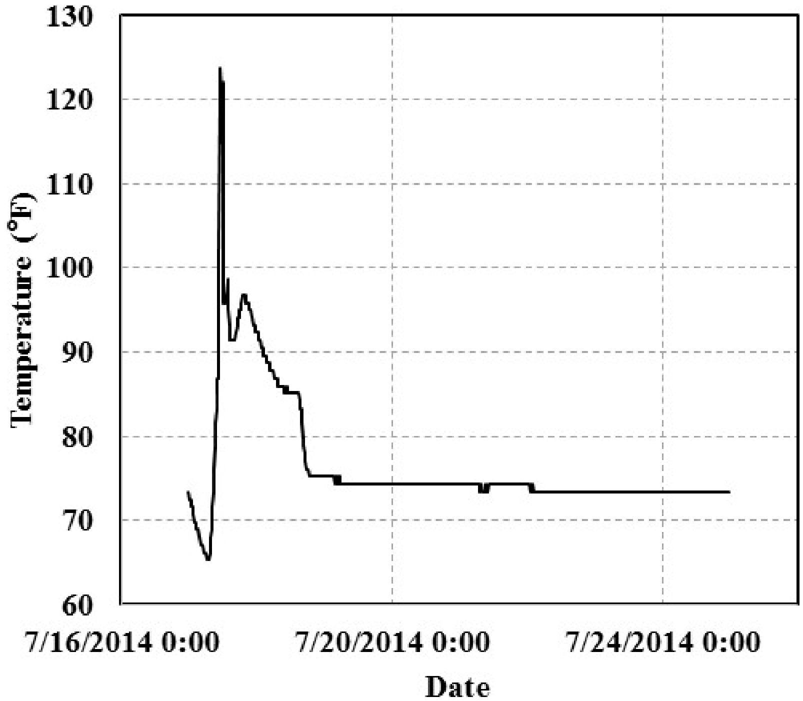

Figure A-94: LS8000CT C48T 7-Day Temperature Data

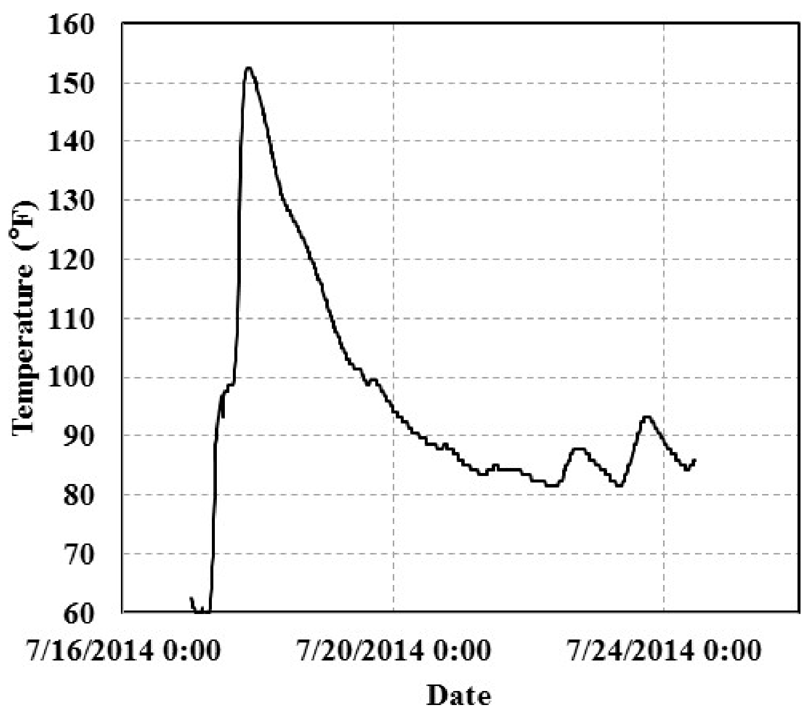

Figure A-95: LS8000CT IM 7-Day Temperature Data

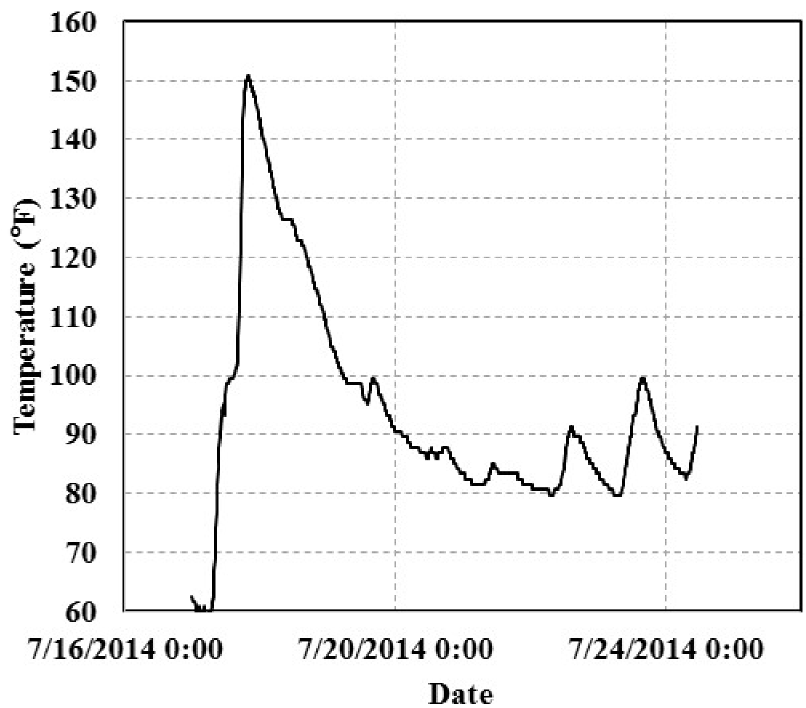

Figure A-96: LS8000CT IT 7-Day Temperature Data

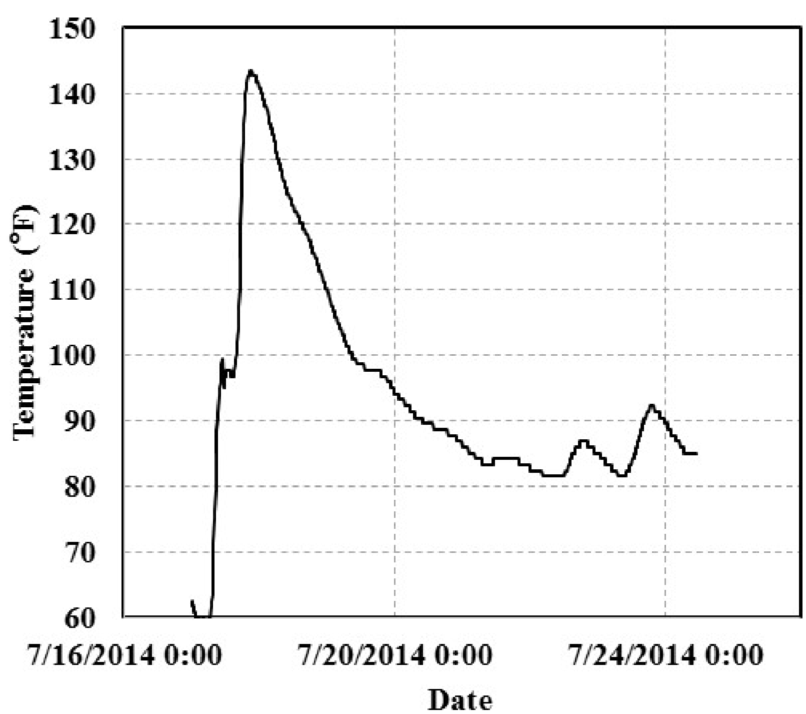

Figure A-97: LS8000CT OM 7-Day Temperature Data

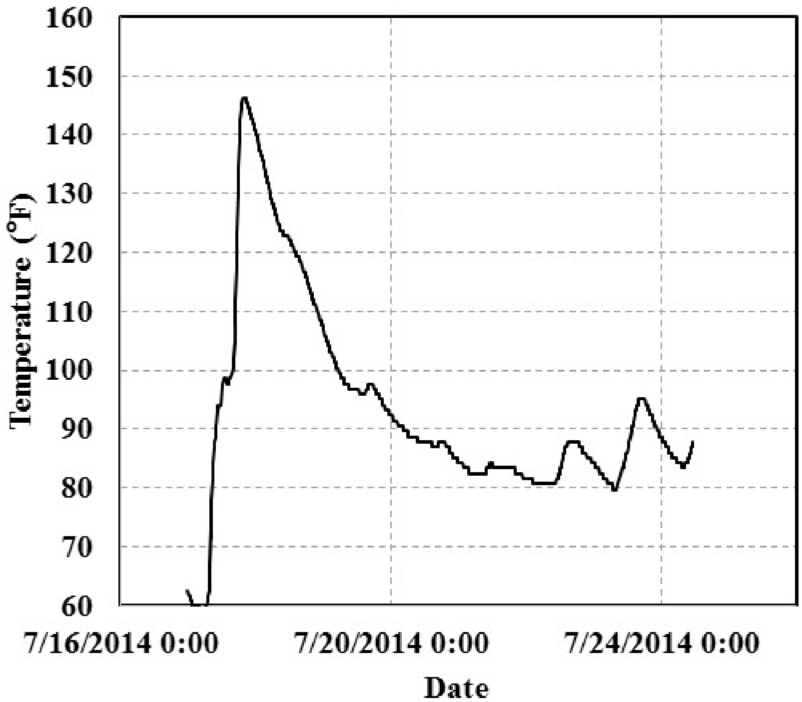

Figure A-98: LS8000CT OT 7-Day Temperature Data

Appendix B

Pullout Calibration Table from Germann Instruments

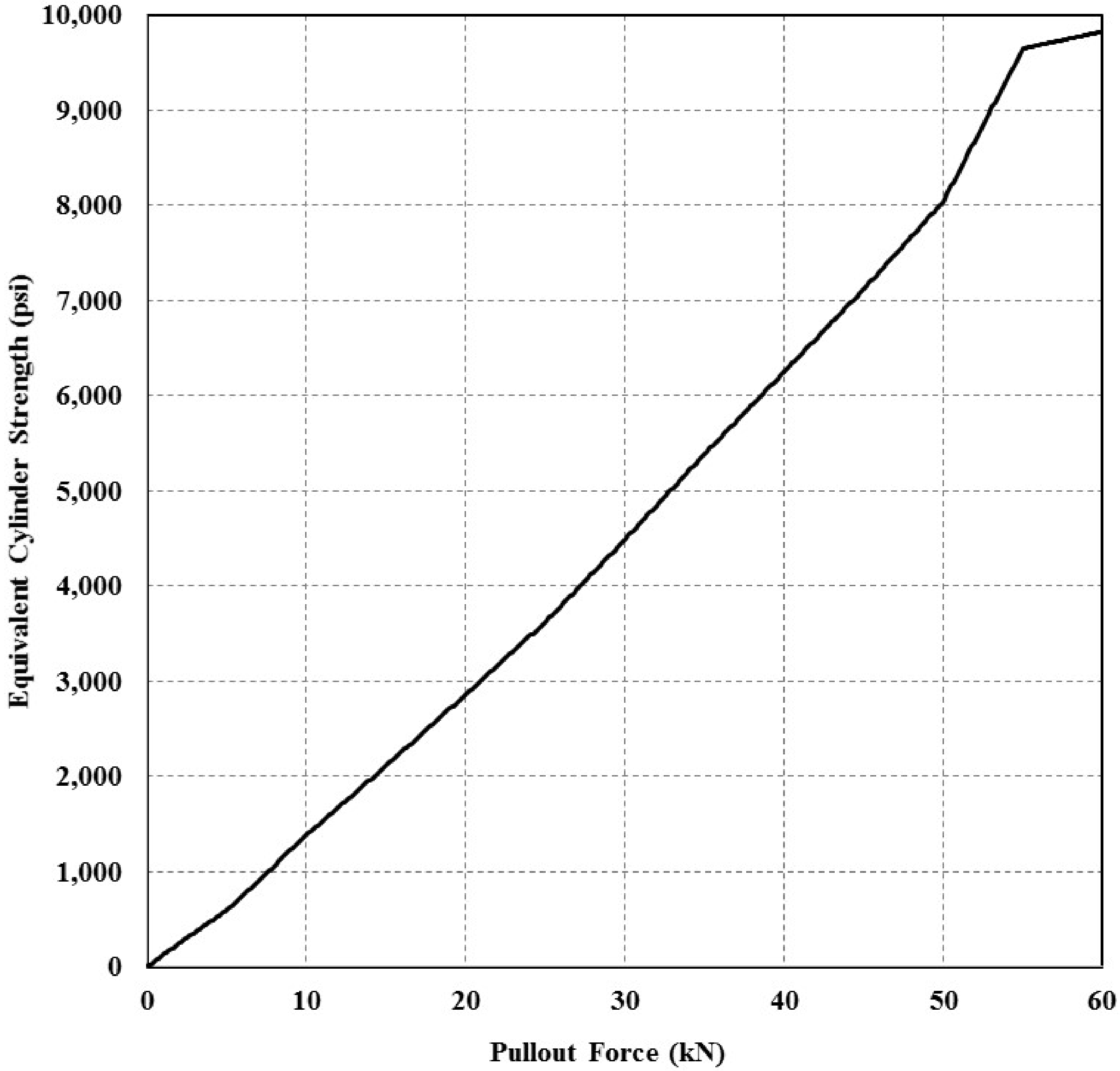

Figure B-1: Pullout Force vs. Predicted Molded Cylinder Strength for 6/4/2013-6/4/2014

Table B-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
0	0	0	3.8	3.7	453
0.1	0.1	12	3.9	3.8	464
0.2	0.2	24	4	3.9	476
0.3	0.3	37	4.1	4	487
0.4	0.4	49	4.2	4.1	499
0.5	0.5	61	4.3	4.2	510
0.6	0.6	73	4.4	4.3	522
0.7	0.7	85	4.5	4.4	534
0.8	0.8	98	4.6	4.5	545
0.9	0.9	110	4.7	4.6	557
1	1	122	4.8	4.7	568
1.1	1.1	134	4.9	4.8	580
1.2	1.2	147	5	4.9	592
1.3	1.3	159	5.1	5	604
1.4	1.4	171	5.2	5.1	620
1.5	1.5	183	5.3	5.2	636
1.6	1.6	195	5.4	5.3	652
1.7	1.7	208	5.5	5.4	667
1.8	1.8	220	5.6	5.5	683
1.9	1.9	232	5.7	5.6	699
2	2	244	5.8	5.7	715
2.1	2.1	256	5.9	5.8	731
2.2	2.2	267	6	5.9	747
2.3	2.3	279	6.1	6	762
2.4	2.4	290	6.2	6.2	778
2.5	2.5	302	6.3	6.3	794
2.6	2.6	314	6.4	6.4	810
2.7	2.7	325	6.5	6.5	826
2.8	2.8	337	6.6	6.6	842
2.9	2.9	348	6.7	6.7	857
3	3	360	6.8	6.8	873
3.1	3.1	372	6.9	6.9	889
3.2	3.2	383	7	7	905
3.3	3.3	395	7.1	7.1	921
3.4	3.4	406	7.2	7.2	936
3.5	3.5	418	7.3	7.3	952
3.6	3.6	429	7.4	7.4	968
3.7	3.6	441	7.5	7.5	984

Table B-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
7.6	7.6	1000	11.4	11.5	1584
7.7	7.7	1016	11.5	11.6	1598
7.8	7.8	1031	11.6	11.7	1613
7.9	7.9	1047	11.7	11.8	1627
8	8	1063	11.8	11.9	1642
8.1	8.1	1079	11.9	12	1657
8.2	8.2	1095	12	12.1	1671
8.3	8.4	1111	12.1	12.2	1686
8.4	8.5	1126	12.2	12.3	1700
8.5	8.6	1142	12.3	12.4	1715
8.6	8.7	1158	12.4	12.5	1729
8.7	8.8	1174	12.5	12.5	1744
8.8	8.9	1190	12.6	12.6	1759
8.9	9	1206	12.7	12.7	1773
9	9.1	1221	12.8	12.8	1788
9.1	9.2	1237	12.9	12.9	1802
9.2	9.3	1253	13	13	1817
9.3	9.4	1269	13.1	13.1	1832
9.4	9.5	1285	13.2	13.2	1846
9.5	9.6	1300	13.3	13.3	1861
9.6	9.7	1316	13.4	13.4	1875
9.7	9.8	1332	13.5	13.5	1890
9.8	9.9	1348	13.6	13.6	1904
9.9	10	1364	13.7	13.7	1919
10	10.1	1380	13.8	13.8	1934
10.1	10.2	1394	13.9	13.9	1948
10.2	10.3	1409	14	14	1963
10.3	10.4	1423	14.1	14.1	1977
10.4	10.5	1438	14.2	14.2	1992
10.5	10.6	1452	14.3	14.3	2007
10.6	10.7	1467	14.4	14.4	2021
10.7	10.8	1482	14.5	14.5	2036
10.8	10.9	1496	14.6	14.6	2050
10.9	11	1511	14.7	14.7	2065
11	11.1	1525	14.8	14.8	2079
11.1	11.2	1540	14.9	14.9	2094
11.2	11.3	1555	15	15	2109
11.3	11.4	1569	15.1	15.1	2124

Table B-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
15.2	15.2	2139
15.3	15.3	2154
15.4	15.4	2169
15.5	15.5	2184
15.6	15.6	2199
15.7	15.7	2214
15.8	15.8	2229
15.9	15.9	2244
16	16	2259
16.1	16.1	2274
16.2	16.2	2289
16.3	16.3	2304
16.4	16.4	2319
16.5	16.4	2334
16.6	16.5	2348
16.7	16.6	2363
16.8	16.7	2378
16.9	16.8	2393
17	16.9	2408
17.1	17	2423
17.2	17.1	2438
17.3	17.2	2453
17.4	17.3	2468
17.5	17.4	2483
17.6	17.5	2498
17.7	17.6	2513
17.8	17.7	2528
17.9	17.8	2543
18	17.9	2558
18.1	18	2573
18.2	18.1	2588
18.3	18.2	2603
18.4	18.3	2618
18.5	18.4	2633
18.6	18.5	2648
18.7	18.6	2663
18.8	18.7	2678
18.9	18.8	2693

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
19	18.9	2708
19.1	19	2723
19.2	19.1	2738
19.3	19.2	2753
19.4	19.3	2768
19.5	19.4	2783
19.6	19.5	2798
19.7	19.6	2813
19.8	19.7	2828
19.9	19.8	2843
20	19.9	2858
20.1	20	2874
20.2	20.1	2889
20.3	20.2	2904
20.4	20.3	2919
20.5	20.4	2934
20.6	20.5	2950
20.7	20.6	2965
20.8	20.7	2980
20.9	20.8	2995
21	20.9	3011
21.1	21	3026
21.2	21.1	3041
21.3	21.2	3056
21.4	21.3	3072
21.5	21.4	3087
21.6	21.5	3102
21.7	21.6	3117
21.8	21.7	3133
21.9	21.8	3148
22	21.9	3163
22.1	22	3178
22.2	22.1	3193
22.3	22.2	3209
22.4	22.3	3224
22.5	22.4	3239
22.6	22.5	3254
22.7	22.6	3270

Table B-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
22.8	22.7	3285	26.6	26.5	3897
22.9	22.8	3300	26.7	26.6	3914
23	22.9	3315	26.8	26.7	3931
23.1	23.1	3331	26.9	26.8	3949
23.2	23.2	3346	27	26.9	3966
23.3	23.3	3361	27.1	27	3983
23.4	23.4	3376	27.2	27.1	4001
23.5	23.5	3391	27.3	27.2	4018
23.6	23.6	3407	27.4	27.3	4035
23.7	23.7	3422	27.5	27.4	4053
23.8	23.8	3437	27.6	27.5	4070
23.9	23.9	3452	27.7	27.6	4087
24	24	3468	27.8	27.6	4105
24.1	24.1	3483	27.9	27.7	4122
24.2	24.2	3498	28	27.8	4140
24.3	24.3	3513	28.1	27.9	4157
24.4	24.4	3529	28.2	28	4174
24.5	24.5	3544	28.3	28.1	4192
24.6	24.6	3559	28.4	28.2	4209
24.7	24.7	3574	28.5	28.3	4226
24.8	24.8	3589	28.6	28.4	4244
24.9	24.9	3605	28.7	28.5	4261
25	25	3620	28.8	28.6	4278
25.1	25.1	3636	28.9	28.7	4296
25.2	25.2	3654	29	28.8	4313
25.3	25.3	3671	29.1	28.9	4330
25.4	25.3	3688	29.2	29	4348
25.5	25.4	3706	29.3	29.1	4365
25.6	25.5	3723	29.4	29.2	4382
25.7	25.6	3740	29.5	29.3	4400
25.8	25.7	3758	29.6	29.4	4417
25.9	25.8	3775	29.7	29.5	4435
26	25.9	3792	29.8	29.6	4452
26.1	26	3810	29.9	29.7	4469
26.2	26.1	3827	30	29.8	4487
26.3	26.2	3845	30.1	29.9	4505
26.4	26.3	3862	30.2	30	4523
26.5	26.4	3879	30.3	30.1	4541

Table B-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
30.4	30.2	4559
30.5	30.3	4577
30.6	30.3	4595
30.7	30.4	4613
30.8	30.5	4631
30.9	30.6	4649
31	30.7	4667
31.1	30.8	4685
31.2	30.9	4703
31.3	31	4721
31.4	31.1	4739
31.5	31.2	4757
31.6	31.3	4775
31.7	31.4	4793
31.8	31.5	4811
31.9	31.6	4828
32	31.7	4846
32.1	31.8	4864
32.2	31.9	4882
32.3	32	4900
32.4	32.1	4918
32.5	32.2	4936
32.6	32.3	4954
32.7	32.4	4972
32.8	32.5	4990
32.9	32.6	5008
33	32.7	5026
33.1	32.8	5044
33.2	32.9	5062
33.3	33	5080
33.4	33.1	5098
33.5	33.2	5116
33.6	33.3	5134
33.7	33.4	5152
33.8	33.5	5170
33.9	33.6	5188
34	33.7	5206
34.1	33.8	5224

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
34.2	33.9	5242
34.3	34	5260
34.4	34.1	5278
34.5	34.2	5296
34.6	34.3	5314
34.7	34.4	5332
34.8	34.5	5350
34.9	34.6	5368
35	34.7	5386
35.1	34.8	5404
35.2	34.9	5421
35.3	35	5438
35.4	35.1	5455
35.5	35.2	5473
35.6	35.3	5490
35.7	35.4	5507
35.8	35.5	5525
35.9	35.6	5542
36	35.7	5559
36.1	35.8	5576
36.2	35.9	5594
36.3	36	5611
36.4	36.1	5628
36.5	36.1	5646
36.6	36.2	5663
36.7	36.3	5680
36.8	36.4	5697
36.9	36.5	5715
37	36.6	5732
37.1	36.7	5749
37.2	36.8	5767
37.3	36.9	5784
37.4	37	5801
37.5	37.1	5818
37.6	37.2	5836
37.7	37.3	5853
37.8	37.4	5870
37.9	37.5	5888

Table B-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
38	37.6	5905	41.8	41.2	6563
38.1	37.7	5922	41.9	41.3	6580
38.2	37.8	5939	42	41.4	6598
38.3	37.9	5957	42.1	41.5	6615
38.4	38	5974	42.2	41.6	6632
38.5	38.1	5991	42.3	41.7	6650
38.6	38.2	6008	42.4	41.8	6667
38.7	38.2	6026	42.5	41.9	6684
38.8	38.3	6043	42.6	42	6702
38.9	38.4	6060	42.7	42.1	6719
39	38.5	6078	42.8	42.2	6736
39.1	38.6	6095	42.9	42.3	6754
39.2	38.7	6112	43	42.4	6771
39.3	38.8	6129	43.1	42.5	6788
39.4	38.9	6147	43.2	42.5	6806
39.5	39	6164	43.3	42.6	6823
39.6	39.1	6181	43.4	42.7	6840
39.7	39.2	6199	43.5	42.8	6858
39.8	39.3	6216	43.6	42.9	6875
39.9	39.4	6233	43.7	43	6893
40	39.5	6250	43.8	43.1	6910
40.1	39.6	6268	43.9	43.2	6927
40.2	39.7	6285	44	43.3	6945
40.3	39.8	6303	44.1	43.4	6962
40.4	39.9	6320	44.2	43.5	6979
40.5	40	6337	44.3	43.6	6997
40.6	40.1	6355	44.4	43.7	7014
40.7	40.2	6372	44.5	43.8	7031
40.8	40.3	6389	44.6	43.9	7049
40.9	40.3	6407	44.7	44	7066
41	40.4	6424	44.8	44.1	7083
41.1	40.5	6441	44.9	44.2	7101
41.2	40.6	6459	45	44.3	7118
41.3	40.7	6476	45.1	44.4	7137
41.4	40.8	6493	45.2	44.5	7155
41.5	40.9	6511	45.3	44.6	7174
41.6	41	6528	45.4	44.7	7192
41.7	41.1	6545	45.5	44.8	7210

Table B-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
45.6	44.9	7229
45.7	45	7247
45.8	45.1	7266
45.9	45.2	7284
46	45.3	7303
46.1	45.4	7321
46.2	45.5	7340
46.3	45.6	7358
46.4	45.7	7377
46.5	45.8	7395
46.6	45.9	7413
46.7	46	7432
46.8	46.1	7450
46.9	46.2	7469
47	46.3	7487
47.1	46.4	7506
47.2	46.5	7524
47.3	46.6	7543
47.4	46.7	7561
47.5	46.8	7580
47.6	46.9	7598
47.7	47	7616
47.8	47.1	7635
47.9	47.2	7653
48	47.3	7672
48.1	47.4	7690
48.2	47.5	7709
48.3	47.6	7727
48.4	47.7	7746
48.5	47.8	7764
48.6	47.9	7783
48.7	48	7801
48.8	48.1	7819
48.9	48.2	7838
49	48.3	7856
49.1	48.4	7875
49.2	48.5	7893
49.3	48.7	7912

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
49.4	48.8	7930
49.5	48.9	7949
49.6	49	7967
49.7	49.1	7986
49.8	49.2	8004
49.9	49.3	8023
50	49.4	8041
50.1	49.5	8073
50.2	49.7	8105
50.3	49.9	8137
50.4	50.1	8169
50.5	50.2	8202
50.6	50.4	8234
50.7	50.6	8266
50.8	50.8	8298
50.9	51	8330
51	51.1	8362
51.1	51.3	8394
51.2	51.5	8426
51.3	51.7	8458
51.4	51.8	8491
51.5	52	8523
51.6	52.2	8555
51.7	52.4	8587
51.8	52.6	8619
51.9	52.7	8651
52	52.9	8683
52.1	53.1	8715
52.2	53.3	8747
52.3	53.4	8780
52.4	53.6	8812
52.5	53.8	8844
52.6	54	8876
52.7	54.1	8908
52.8	54.3	8940
52.9	54.5	8972
53	54.7	9004
53.1	54.9	9036

Table B-1: Pullout Table from Germann Instruments for 6/4/2013-6/4/2014 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
53.2	55	9069	57	58.6	9718
53.3	55.2	9101	57.1	58.6	9721
53.4	55.4	9133	57.2	58.7	9725
53.5	55.6	9165	57.3	58.7	9728
53.6	55.7	9197	57.4	58.7	9732
53.7	55.9	9229	57.5	58.7	9735
53.8	56.1	9261	57.6	58.7	9739
53.9	56.3	9293	57.7	58.8	9743
54	56.5	9325	57.8	58.8	9746
54.1	56.6	9358	57.9	58.8	9750
54.2	56.8	9390	58	58.8	9753
54.3	57	9422	58.1	58.8	9757
54.4	57.2	9454	58.2	58.9	9760
54.5	57.3	9486	58.3	58.9	9764
54.6	57.5	9518	58.4	58.9	9767
54.7	57.7	9550	58.5	58.9	9771
54.8	57.9	9582	58.6	58.9	9775
54.9	58	9614	58.7	58.9	9778
55	58.2	9647	58.8	59	9782
55.1	58.2	9650	58.9	59	9785
55.2	58.3	9654	59	59	9789
55.3	58.3	9657	59.1	59	9792
55.4	58.3	9661	59.2	59	9796
55.5	58.3	9664	59.3	59.1	9799
55.6	58.3	9668	59.4	59.1	9803
55.7	58.4	9671	59.5	59.1	9807
55.8	58.4	9675	59.6	59.1	9810
55.9	58.4	9679	59.7	59.1	9814
56	58.4	9682	59.8	59.2	9817
56.1	58.4	9686	59.9	59.2	9821
56.2	58.5	9689	60	59.2	9824
56.3	58.5	9693			
56.4	58.5	9696			
56.5	58.5	9700			
56.6	58.5	9703			
56.7	58.6	9707			
56.8	58.6	9711			
56.9	58.6	9714			

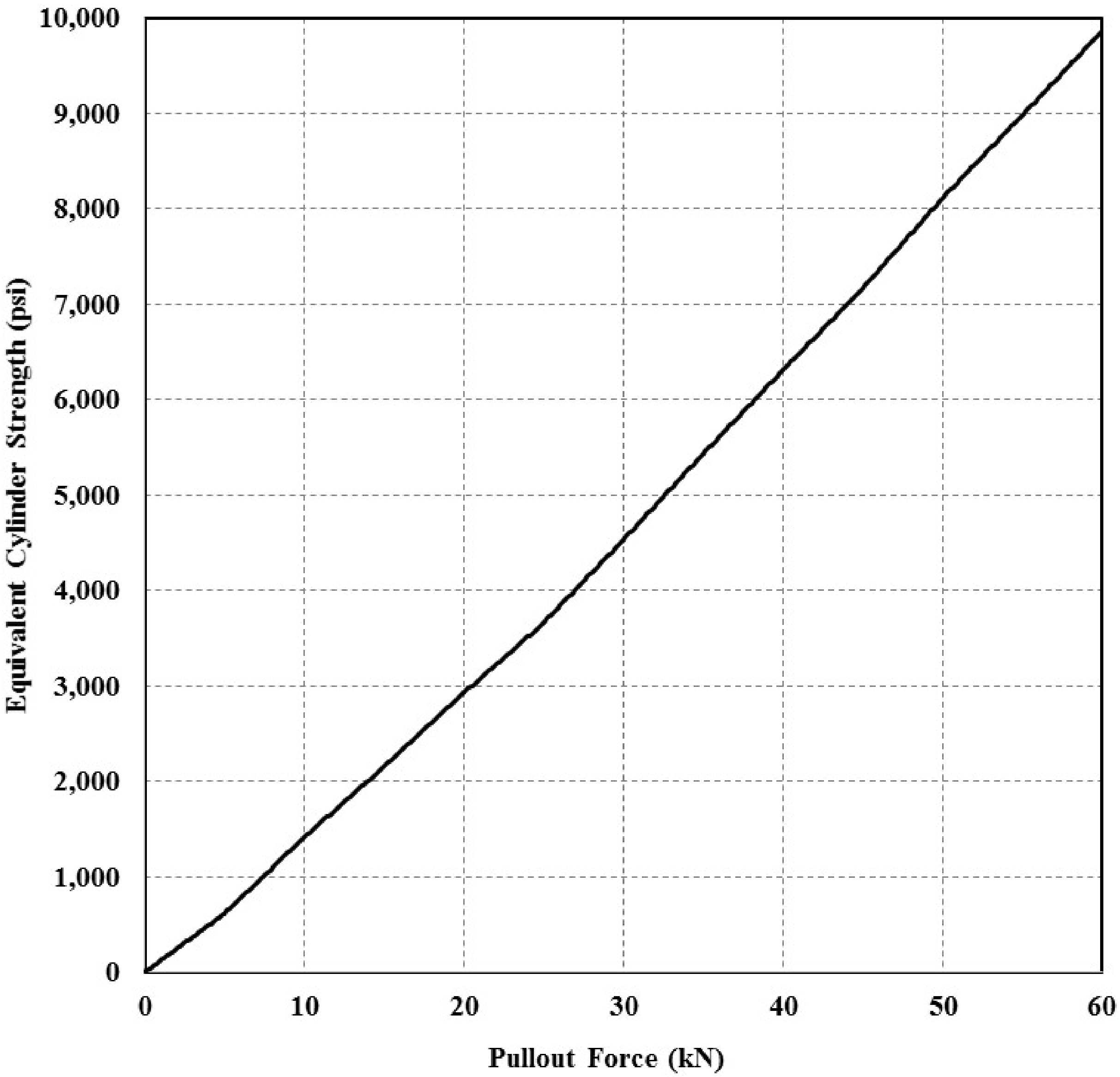

Figure B-2: Pullout Force vs. Predicted Molded Cylinder Strength for 6/5/2014-6/5/2015

Table B-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
0	0	0	3.8	3.9	466
0.1	0.1	12	3.9	4	479
0.2	0.2	25	4	4.1	491
0.3	0.3	37	4.1	4.2	503
0.4	0.4	49	4.2	4.3	515
0.5	0.5	62	4.3	4.4	527
0.6	0.6	74	4.4	4.5	540
0.7	0.7	86	4.5	4.6	552
0.8	0.8	99	4.6	4.7	564
0.9	0.9	111	4.7	4.8	576
1	1	123	4.8	4.9	588
1.1	1.1	136	4.9	5	601
1.2	1.2	148	5	5.1	613
1.3	1.3	160	5.1	5.2	631
1.4	1.4	173	5.2	5.3	647
1.5	1.5	185	5.3	5.4	663
1.6	1.6	197	5.4	5.5	679
1.7	1.7	210	5.5	5.6	695
1.8	1.8	222	5.6	5.7	711
1.9	1.9	234	5.7	5.8	727
2	2	247	5.8	5.9	743
2.1	2.1	259	5.9	6	759
2.2	2.2	271	6	6.1	775
2.3	2.3	283	6.1	6.2	791
2.4	2.4	295	6.2	6.3	807
2.5	2.5	308	6.3	6.4	823
2.6	2.6	320	6.4	6.6	839
2.7	2.7	332	6.5	6.7	855
2.8	2.8	344	6.6	6.8	871
2.9	2.9	356	6.7	6.9	887
3	3.1	369	6.8	7	903
3.1	3.2	381	6.9	7.1	919
3.2	3.3	393	7	7.2	935
3.3	3.4	405	7.1	7.3	951
3.4	3.5	417	7.2	7.4	967
3.5	3.6	430	7.3	7.5	983
3.6	3.7	442	7.4	7.6	999
3.7	3.8	454	7.5	7.7	1015

Table B-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
7.6	7.8	1031
7.7	7.9	1047
7.8	8	1063
7.9	8.1	1079
8	8.3	1095
8.1	8.4	1111
8.2	8.5	1127
8.3	8.6	1143
8.4	8.7	1159
8.5	8.8	1175
8.6	8.9	1191
8.7	9	1207
8.8	9.1	1223
8.9	9.2	1239
9	9.3	1255
9.1	9.4	1271
9.2	9.5	1287
9.3	9.6	1303
9.4	9.7	1319
9.5	9.8	1335
9.6	9.9	1351
9.7	10.1	1367
9.8	10.2	1383
9.9	10.3	1399
10	10.4	1415
10.1	10.5	1430
10.2	10.6	1445
10.3	10.7	1460
10.4	10.8	1474
10.5	10.9	1489
10.6	11	1504
10.7	11.1	1519
10.8	11.2	1534
10.9	11.3	1548
11	11.4	1563
11.1	11.4	1578
11.2	11.5	1593
11.3	11.6	1608

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
11.4	11.7	1623
11.5	11.8	1637
11.6	11.9	1652
11.7	12	1667
11.8	12.1	1682
11.9	12.2	1697
12	12.3	1711
12.1	12.4	1726
12.2	12.5	1741
12.3	12.6	1756
12.4	12.7	1771
12.5	12.8	1786
12.6	12.9	1800
12.7	13	1815
12.8	13.1	1830
12.9	13.2	1845
13	13.3	1860
13.1	13.4	1874
13.2	13.5	1889
13.3	13.6	1904
13.4	13.7	1919
13.5	13.8	1934
13.6	13.9	1949
13.7	14	1963
13.8	14.1	1978
13.9	14.2	1993
14	14.3	2008
14.1	14.4	2023
14.2	14.5	2037
14.3	14.6	2052
14.4	14.7	2067
14.5	14.8	2082
14.6	14.9	2097
14.7	15	2112
14.8	15.1	2126
14.9	15.2	2141
15	15.3	2156
15.1	15.4	2171

Table B-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
15.2	15.5	2187
15.3	15.6	2202
15.4	15.7	2218
15.5	15.8	2233
15.6	15.9	2249
15.7	16	2264
15.8	16.1	2280
15.9	16.2	2295
16	16.3	2310
16.1	16.4	2326
16.2	16.5	2341
16.3	16.6	2357
16.4	16.7	2372
16.5	16.8	2388
16.6	16.9	2403
16.7	17	2418
16.8	17.1	2434
16.9	17.2	2449
17	17.3	2465
17.1	17.4	2480
17.2	17.5	2496
17.3	17.6	2511
17.4	17.7	2527
17.5	17.8	2542
17.6	17.9	2557
17.7	18	2573
17.8	18.1	2588
17.9	18.2	2604
18	18.3	2619
18.1	18.4	2635
18.2	18.5	2650
18.3	18.6	2665
18.4	18.7	2681
18.5	18.9	2696
18.6	19	2712
18.7	19.1	2727
18.8	19.2	2743
18.9	19.3	2758

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
19	19.4	2774
19.1	19.5	2789
19.2	19.6	2804
19.3	19.7	2820
19.4	19.8	2835
19.5	19.9	2851
19.6	20	2866
19.7	20.1	2882
19.8	20.2	2897
19.9	20.3	2913
20	20.4	2928
20.1	20.5	2943
20.2	20.6	2957
20.3	20.7	2972
20.4	20.8	2987
20.5	20.9	3001
20.6	21	3016
20.7	21.1	3030
20.8	21.2	3045
20.9	21.3	3060
21	21.4	3074
21.1	21.5	3089
21.2	21.5	3104
21.3	21.6	3118
21.4	21.7	3133
21.5	21.8	3148
21.6	21.9	3162
21.7	22	3177
21.8	22.1	3191
21.9	22.2	3206
22	22.3	3221
22.1	22.4	3235
22.2	22.5	3250
22.3	22.6	3265
22.4	22.7	3279
22.5	22.8	3294
22.6	22.9	3309
22.7	23	3323

Table B-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
22.8	23.1	3338	26.6	26.8	3945
22.9	23.2	3353	26.7	26.9	3962
23	23.3	3367	26.8	27	3979
23.1	23.4	3382	26.9	27.1	3997
23.2	23.5	3396	27	27.1	4014
23.3	23.6	3411	27.1	27.2	4031
23.4	23.7	3426	27.2	27.3	4049
23.5	23.8	3440	27.3	27.4	4066
23.6	23.9	3455	27.4	27.5	4083
23.7	24	3470	27.5	27.6	4101
23.8	24.1	3484	27.6	27.7	4118
23.9	24.2	3499	27.7	27.8	4135
24	24.3	3514	27.8	27.9	4153
24.1	24.4	3528	27.9	28	4170
24.2	24.5	3543	28	28.1	4188
24.3	24.6	3557	28.1	28.2	4205
24.4	24.6	3572	28.2	28.3	4222
24.5	24.7	3587	28.3	28.4	4240
24.6	24.8	3601	28.4	28.5	4257
24.7	24.9	3616	28.5	28.6	4274
24.8	25	3632	28.6	28.7	4292
24.9	25.1	3649	28.7	28.8	4309
25	25.2	3667	28.8	28.9	4326
25.1	25.3	3684	28.9	29	4344
25.2	25.4	3702	29	29.1	4361
25.3	25.5	3719	29.1	29.2	4378
25.4	25.6	3736	29.2	29.3	4396
25.5	25.7	3754	29.3	29.3	4413
25.6	25.8	3771	29.4	29.4	4430
25.7	25.9	3788	29.5	29.5	4448
25.8	26	3806	29.6	29.6	4465
25.9	26.1	3823	29.7	29.7	4483
26	26.2	3840	29.8	29.8	4500
26.1	26.3	3858	29.9	29.9	4517
26.2	26.4	3875	30	30	4535
26.3	26.5	3893	30.1	30.1	4553
26.4	26.6	3910	30.2	30.2	4571
26.5	26.7	3927	30.3	30.3	4589

Table B-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
30.4	30.4	4607	34.2	34.2	5292
30.5	30.5	4625	34.3	34.3	5310
30.6	30.6	4643	34.4	34.4	5328
30.7	30.7	4661	34.5	34.5	5346
30.8	30.8	4679	34.6	34.6	5364
30.9	30.9	4697	34.7	34.7	5382
31	31	4715	34.8	34.8	5400
31.1	31.1	4733	34.9	34.9	5418
31.2	31.2	4751	35	35	5436
31.3	31.3	4769	35.1	35.1	5454
31.4	31.4	4787	35.2	35.2	5471
31.5	31.5	4805	35.3	35.3	5489
31.6	31.6	4823	35.4	35.4	5506
31.7	31.7	4841	35.5	35.5	5524
31.8	31.8	4859	35.6	35.6	5541
31.9	31.9	4877	35.7	35.7	5559
32	32	4895	35.8	35.8	5576
32.1	32.1	4913	35.9	35.9	5594
32.2	32.2	1931	36	36	5611
32.3	32.3	4949	36.1	36.1	5629
32.4	32.4	4967	36.2	36.2	5646
32.5	32.5	4985	36.3	36.2	5664
32.6	32.6	5003	36.4	36.3	5682
32.7	32.7	5021	36.5	36.4	5699
32.8	32.8	5039	36.6	36.5	5717
32.9	32.9	5057	36.7	36.6	5734
33	33	5076	36.8	36.7	5752
33.1	33.1	5094	36.9	36.8	5769
33.2	33.2	5112	37	36.9	5787
33.3	33.3	5130	37.1	37	5804
33.4	33.4	5148	37.2	37.1	5822
33.5	33.5	5166	37.3	37.2	5839
33.6	33.6	5184	37.4	37.3	5857
33.7	33.7	5202	37.5	37.4	5874
33.8	33.8	5220	37.6	37.5	5892
33.9	33.9	5238	37.7	37.6	5909
34	34	5256	37.8	37.7	5927
34.1	34.1	5274	37.9	37.8	5945

Table B-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)	Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
38	37.9	5962	41.8	41.5	6622
38.1	38	5980	41.9	41.6	6639
38.2	38.1	5997	42	41.7	6656
38.3	38.2	6015	42.1	41.8	6673
38.4	38.3	6032	42.2	41.9	6691
38.5	38.4	6050	42.3	42	6708
38.6	38.5	6067	42.4	42.1	6725
38.7	38.6	6085	42.5	42.2	6742
38.8	38.7	6102	42.6	42.3	6759
38.9	38.8	6120	42.7	42.4	6776
39	38.9	6137	42.8	42.5	6794
39.1	39	6155	42.9	42.6	6811
39.2	39.1	6172	43	42.7	6828
39.3	39.2	6190	43.1	42.8	6845
39.4	39.2	6207	43.2	42.9	6862
39.5	39.3	6225	43.3	43	6879
39.6	39.4	6243	43.4	43.1	6897
39.7	39.5	6260	43.5	43.1	6914
39.8	39.6	6278	43.6	43.2	6931
39.9	39.7	6295	43.7	443.3	6948
40	39.8	6313	43.8	43.4	6965
40.1	39.9	6330	43.9	43.5	6983
40.2	40	6347	44	43.6	7000
40.3	40.1	6364	44.1	43.7	7017
40.4	40.2	6381	44.2	43.8	7034
40.5	40.3	6399	44.3	43.9	7051
40.6	40.4	6416	44.4	44	7068
40.7	40.5	6433	44.5	44.1	7086
40.8	40.6	6450	44.6	44.2	7103
40.9	40.7	6467	44.7	44.3	7120
41	40.8	6484	44.8	44.4	7137
41.1	40.9	6502	44.9	44.5	7154
41.2	41	6519	45	44.6	7171
41.3	41.1	6536	45.1	44.7	7190
41.4	41.2	6553	45.2	44.8	7209
41.5	41.3	6570	45.3	44.9	7228
41.6	41.3	6588	45.4	45	7247
41.7	41.4	6605	45.5	45.1	7266

Table B-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
45.6	45.2	7285
45.7	45.3	7304
45.8	45.4	7323
45.9	45.5	7342
46	45.6	7361
46.1	45.7	7380
46.2	45.8	7399
46.3	45.9	7417
46.4	46	7436
46.5	46.1	7455
46.6	46.2	7474
46.7	46.3	7493
46.8	46.4	7512
46.9	46.6	7531
47	46.7	7550
47.1	46.8	7569
47.2	46.9	7588
47.3	47	7607
47.4	47.1	7626
47.5	47.2	7644
47.6	47.3	7663
47.7	47.4	7682
47.8	47.5	7701
47.9	47.6	7720
48	47.7	7739
48.1	47.8	7758
48.2	47.9	7777
48.3	48	7796
48.4	48.1	7815
48.5	48.2	7834
48.6	48.3	7853
48.7	48.4	7871
48.8	48.5	7890
48.9	48.6	7909
49	48.7	7928
49.1	48.8	7947
49.2	49	7966
49.3	49.1	7985

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
49.4	49.2	8004
49.5	49.3	8023
49.6	49.4	8042
49.7	49.5	8061
49.8	49.6	8080
49.9	49.7	8098
50	49.8	8117
50.1	49.9	8135
50.2	50	8152
50.3	50.1	8169
50.4	50.2	8187
50.5	50.3	8204
50.6	50.4	8222
50.7	50.5	8239
50.8	50.6	8256
50.9	50.6	8274
51	50.7	8291
51.1	50.8	8308
51.2	50.9	8326
51.3	51	8343
51.4	51.1	8360
51.5	51.2	8378
51.6	51.3	8395
51.7	51.4	8412
51.8	51.5	8430
51.9	51.6	8447
52	51.7	8464
52.1	51.8	8482
52.2	51.9	8499
52.3	52	8517
52.4	52.1	8534
52.5	52.2	8551
52.6	52.3	8569
52.7	52.4	8586
52.8	52.5	8603
52.9	52.6	8621
53	52.7	8638
53.1	52.8	8655

Table B-2: Pullout Table from Germann Instruments for 6/5/2014-6/5/2015 (continued)

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
53.2	52.8	8673
53.3	52.9	8690
53.4	53	8707
53.5	53.1	8725
53.6	53.2	8742
53.7	53.3	8760
53.8	53.4	8777
53.9	53.5	8794
54	53.6	8812
54.1	53.7	8829
54.2	53.8	8846
54.3	53.9	8864
54.4	54	8881
54.5	54.1	8898
54.6	54.2	8916
54.7	54.3	8933
54.8	54.4	8950
54.9	54.5	8968
55	54.6	8985
55.1	54.7	9003
55.2	54.8	9020
55.3	54.9	9038
55.4	55	9056
55.5	55.1	9073
55.6	55.2	9091
55.7	55.3	9109
55.8	55.4	9126
55.9	55.4	9144
56	55.5	9161
56.1	55.6	9179
56.2	55.7	9197
56.3	55.8	9214
56.4	55.9	9232
56.5	56	9250
56.6	56.1	9267
56.7	56.2	9285
56.8	56.3	9303
56.9	56.4	9320

Reading (kN)	Actual Pullforce (kN)	Cylinder Strength (psi)
57	56.5	9338
57.1	56.6	9356
57.2	56.7	9373
57.3	56.8	9391
57.4	56.9	9408
57.5	57	9426
57.6	57.1	9444
57.7	57.2	9461
57.8	57.3	9479
57.9	57.4	9497
58	57.5	9514
58.1	57.6	9532
58.2	57.7	9550
58.3	57.8	9567
58.4	57.9	9585
58.5	58	9602
58.6	58.1	9620
58.7	58.2	9638
58.8	58.3	9655
58.9	58.4	9673
59	58.5	9691
59.1	58.6	9708
59.2	58.7	9726
59.3	58.8	9744
59.4	589.9	9761
59.5	59	9779
59.6	59	9796
59.7	59.1	9814
59.8	59.2	9832
59.9	59.3	9849
60	59.4	9867

Appendix C

Collected Strength Data

Table C-1: Molded Cylinder Strengths from Cast RG4000CA

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	3497	3719
	3820	3828
	Outlier	3551
28-Day	4706	3623
	4483	3864
	4453	4119
42-Day	4848	4771
	5042	5127
	4654	4871
91-Day	5352	5855
	5551	5332
	5898	5902
365-Day	6051	5963
	5426	5355
	5533	5544

Table C-2: Molded Cylinder Strengths from Cast LS4000CT

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	2923	2694
	2881	3037
	2728	2760
28-Day	3746	4004
	3521	3890
	3758	4000
42-Day	3546	3364
	3574	3192
	3713	3375
91-Day	4279	4335
	4235	4203
	4134	4178
365-Day	4499	4182
	4699	4480
	4444	4564

Table C-3: Molded Cylinder Strengths from Cast RG4000CT

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	2690	2742
	2477	2778
	2486	NC
28-Day	3455	3483
	3468	3559
	3379	3401
42-Day	3682	3904
	3642	3822
	3708	3747
91-Day	3914	4137
	4053	4171
	3953	4266
365-Day	3845	4166
	3965	4180
	4076	4034

Table C-4: Molded Cylinder Strengths from Cast RG4000SC

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	2447	2617
	2545	2667
	2606	2713
28-Day	3397	3424
	3551	3640
	3579	3736
42-Day	3624	3602
	3574	3484
	3781	3565
91-Day	3955	3998
	4155	3666
	3884	4221
365-Day	4388	3919
	4244	4337
	NC	4128

Table C-5: Molded Cylinder Strengths from Cast GR4000CT

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	2303	2155
	2296	1972
	2262	2041
28-Day	3185	3055
	3180	3031
	3156	3141
42-Day	3448	3250
	3301	3289
	3333	3250
91-Day	3702	3231
	3577	3411
	3406	3327
365-Day	3690	3412
	3713	3471
	3713	3328

Table C-6: Molded Cylinder Strengths from Cast RG4000FA

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	2955	3705
	3168	3795
	3189	NC
28-Day	4105	5125
	4297	5245
	4330	5151
42-Day	4054	5160
	4556	4941
	4602	5436
91-Day	5051	5441
	4879	5854
	5357	5380
365-Day	5826	7208
	5877	6907
	5904	5774

Table C-7: Molded Cylinder Strengths from Cast RG8000CT

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	7095	6895
	8027	7185
	7977	6109
28-Day	8379	6878
	9099	6846
	8925	7271
42-Day	8647	7793
	7957	7558
	8468	7370
91-Day	8097	7333
	7626	7368
	8092	NC
365-Day	9311	8496
	9611	8305
	10120	8404

Table C-8: Molded Cylinder Strengths from Cast LS8000CT

Age (days)	6x12 Cylinder Strength (psi)	4x8 Cylinder Strength (psi)
7-Day	4960	5270
	4828	5344
	4276	4958
28-Day	5744	5797
	5652	6034
	6028	5699
42-Day	5663	7088
	5692	6694
	6351	6520
91-Day	7437	7413
	7628	7803
	7327	7738

Table C-9: Cast-In-Place Cylinder Strengths from Cast RG4000SC

Age (days)	Exterior CIP Strength (psi)	Interior CIP Strength (psi)
28-Day	3832	3327
	3323	3810
	3488	3758
42-Day	3240	3412
	3116	3910
	2714	3692
91-Day	3260	3540
	3432	3371
	3225	3493
365-Day	4494	4562
	4329	4132
	NC	NC

Table C-10: Cast-In-Place Cylinder Strengths from Cast GR4000CT

Age (days)	Exterior CIP Strength (psi)	Interior CIP Strength (psi)
28-Day	2835	2911
	2941	2902
	3159	3024
42-Day	3110	3259
	2920	3071
	2951	2869
91-Day	3070	3065
	2949	3314
	3354	3113
365-Day	3505	3694
	3542	3659
	NC	3735

Table C-11: Cast-In-Place Cylinder Strengths from Cast RG4000FA

Age (days)	Exterior CIP Strength (psi)	Interior CIP Strength (psi)
28-Day	4790	4452
	4431	4805
	4342	4658
42-Day	4651	4650
	5078	4645
	4693	4137
91-Day	4808	4467
	4550	4728
	4364	4213
365-Day	7076	6617
	6676	6589
	5958	6304

Table C-12: Cast-In-Place Cylinder Strengths from Cast RG8000CT

Age (days)	Exterior CIP Strength (psi)	Interior CIP Strength (psi)
28-Day	7127	7278
	Outlier	Outlier
	7018	7598
42-Day	5752	6323
	6077	7015
	Outlier	Outlier
91-Day	6211	7272
	7021	5961
	7294	Outlier
365-Day	8600	7047
	8092	8056
	9124	Outlier

Table C-13: Cast-In-Place Cylinder Strengths from Cast LS8000CT

Age (days)	Exterior CIP Strength (psi)	Interior CIP Strength (psi)
28-Day	6693	6601
	6398	6573
	6823	6684
42-Day	7199	7784
	7189	6751
	7453	6900
91-Day	6829	6999
	6675	7647
	7860	7949

Table C-14: Core Strengths from Cast RG4000CA

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	3688	4501
	4152	3835
	3995	3814
42-Day	5063	4443
	4634	4252
	4415	NC
91-Day	6001	5706
	6076	5706
	5714	NC
365-Day	5317	5036
	5389	4775
	5406	4465

Table C-15: Core Strengths from Cast LS4000CT

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	3077	3041
	3119	3348
	3130	3352
42-Day	3111	3252
	2846	3026
	2972	2955
91-Day	3742	3672
	4118	3736
	3654	3785
365-Day	3723	4042
	3762	3656
	3782	3538

Table C-16: Core Strengths from Cast RG4000CT

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	3293	2580
	2863	2981
	2937	2605
42-Day	2957	2972
	3272	3041
	3101	2902
91-Day	3748	3145
	3649	3502
	3606	3285
365-Day	3271	2872
	3719	3068
	3623	3104

Table C-17: Core Strengths from Cast RG4000SC

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	3812	3647
	3510	3506
	3364	3478
42-Day	3578	3614
	3822	3928
	3714	3782
91-Day	3595	3914
	3748	3549
	3531	3602
365-Day	4361	4210
	4427	3959
	4435	3810

Table C-18: Core Strengths from Cast GR4000CT

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	2916	2799
	2953	2712
	3051	2664
42-Day	2984	2711
	2869	2800
	2905	2739
91-Day	3260	2779
	2974	2852
	3213	2760
365-Day	3421	3608
	3518	3072
	3532	3180

Table C-19: Core Strengths from Cast RG4000FA

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	4669	4664
	4719	4940
	4707	4993
42-Day	4585	3960
	4438	4209
	4485	4256
91-Day	4728	4221
	4414	4616
	4611	4626
365-Day	6469	6046
	6800	5239
	6806	5957

Table C-20: Core Strengths from Cast RG8000CT

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	6567	7196
	6316	6901
	Outlier	NC
42-Day	7271	6333
	7350	Outlier
	7108	7042
91-Day	7175	6970
	7299	6836
	7423	6574
365-Day	8291	7152
	8279	6493
	7658	NC

Table C-21: Core Strengths from Cast LS8000CT

Age (days)	Exterior Core Strength (psi)	Interior Core Strength (psi)
28-Day	6485	6066
	6236	6142
	5943	5788
42-Day	5773	6242
	5978	6210
	6286	6169
91-Day	6904	6311
	7046	6716
	6499	6553

Table C-22: Pullout Strengths from Cast RG4000CA

	Exterior Pullouts		Interior Pullouts	
	Pullout Force (kN)	**Equivalent Compressive Strength (psi)**	**Pullout Force (kN)**	**Equivalent Compressive Strength (psi)**
28-Day	37.4	5801	47.1	7506
	44.4	7014	31.1	4685
	36.9	5715	37.4	5801
	34.6	5314	33.4	5098
	37.4	5801	30	4487
42-Day	37.1	5749	NC	NC
	36.8	5697	33.5	5116
	37.1	5749	33.4	5098
	38.7	6026	32.5	4936
	42.5	6684	32.5	4936
91-Day	40.4	6320	41	6424
	42.6	6702	30.7	4613
	28.2	4174	41.7	6545
	36.4	5628	42.9	6754
	40.9	6407	27.9	4122
365-Day	43.6	6931	NC	NC
	34	5256	40.6	6416
	38.8	6102	37.2	5822
	44.1	7017	37.5	5874
	34	5256	35	5436

Table C-23: Pullout Strengths from Cast LS4000CT

	Exterior Pullouts		Interior Pullouts	
	Pullout Force (kN)	**Equivalent Compressive Strength (psi)**	**Pullout Force (kN)**	**Equivalent Compressive Strength (psi)**
28-Day	24	3468	24.6	3559
	25.6	3723	24.7	3574
	20.4	2919	24.2	3498
	21.6	3102	28.5	4226
	24.3	3513	28.6	4244
42-Day	33.2	5062	27.2	4001
	20.5	2934	26.9	3949
	30.6	4595	21.5	3087
	27.7	4087	25.9	3775
	NC	NC	NC	NC
91-Day	25	3620	27.4	4035
	31	4667	30.5	4577
	27.3	4018	27.5	4053
	31.5	4757	25.7	3740
	24.1	3483	26.8	3931
365-Day	28.1	4205	26.8	3979
	31.2	4751	32.3	4949
	30.3	4589	29.6	4465
	29.2	4396	34.4	5328
	NC	NC	28.5	4274

Table C-24: Pullout Strengths from Cast RG4000CT

	Exterior Pullouts		Interior Pullouts	
	Pullout Force (kN)	**Equivalent Compressive Strength (psi)**	**Pullout Force (kN)**	**Equivalent Compressive Strength (psi)**
28-Day	19.1	2723	19.4	2768
	19.4	2768	19.4	2768
	22	3163	19.6	2798
	18.5	2633	18.3	2603
	22.9	3300	20.5	2934
42-Day	24.2	3498	18.1	2573
	21.8	3133	16.1	2274
	22	3163	22.2	3193
	20.8	2980	19.7	2813
	20.3	2904	NC	NC
91-Day	21.5	3087	16.9	2393
	18.1	2573	20	2858
	23.9	3452	24.5	3544
	18.8	2678	20.6	2950
	19.3	2753	24.2	3498
365-Day	23.6	3455	16	2310
	26.5	3927	22.6	3309
	26.7	3962	21.3	3118
	23.9	3499	21.4	3133
	23.3	3411	26	3840

Table C-25: Pullout Strengths from Cast RG4000SC

	Exterior		Interior	
	Pullout Force (kN)	**Equivalent Compressive Strength (psi)**	**Pullout Force (kN)**	**Equivalent Compressive Strength (psi)**
28-Day	18.8	2678	23	3315
	21.6	3102	13.9	1948
	19.7	2813	20.8	2980
	22.1	3178	15.3	2154
	16.3	2304	20	2858
42-Day	22	3163	22.2	3193
	21.2	3041	20.6	2950
	19.9	2843	24.7	3574
	23	3315	20.5	2934
	21.1	3026	25.5	3706
91-Day	15.1	2124	17	2408
	18.3	2603	16.9	2393
	21.5	3087	17.6	2498
	18.2	2588	17.5	2483
	20.5	2934	18.4	2618
365-Day	30.2	4571	20.7	3030
	26.9	3997	22.6	3309
	27.6	4118	21.8	3191
	21.7	3177	19.3	2820
	22.4	3279	28	4188

Table C-26: Pullout Strengths from Cast GR4000CT

	Exterior Pullouts		Interior Pullouts	
	Pullout Force (kN)	**Equivalent Compressive Strength (psi)**	**Pullout Force (kN)**	**Equivalent Compressive Strength (psi)**
28-Day	16.6	2348	15.6	2199
	18.4	2618	12.7	1773
	18.2	2588	13.2	1846
	20	2858	17.8	2528
	20.3	2904	17.6	2498
42-Day	22.2	3193	16.3	2304
	19.3	2753	14.7	2065
	21.2	3041	13.5	1890
	19.4	2768	16.6	2348
	21.3	3056	17.4	2468
91-Day	24	3468	18.3	2603
	20.5	2934	16	2259
	20.3	2904	20.4	2919
	16.5	2334	12.3	1715
	11.6	1613	12	1671
365-Day	20.6	3016	16.7	2418
	22.1	3235	17.9	2604
	23.3	3411	NC	NC
	NC	NC	16.4	2372
	20.6	3016	22.4	3279

Table C-27: Pullout Strengths from Cast RG4000FA

	Exterior		Interior	
	Pullout Force (kN)	**Equivalent Compressive Strength (psi)**	**Pullout Force (kN)**	**Equivalent Compressive Strength (psi)**
28-Day	35.4	5455	26.5	3879
	35.3	5438	27.2	4001
	35	5386	NC	NC
	38.5	5991	35.3	5438
	31.2	4703	31.2	4703
42-Day	34.3	5260	22.8	3285
	31.5	4757	27.8	4105
	29.9	4469	31.6	4775
	32.2	4882	32.3	4900
	25.3	3671	21.6	3102
91-Day	NC	NC	41	6424
	28.9	4296	25.6	3723
	28.6	4244	NC	NC
	34.8	5350	25.6	3723
	43.3	6823	34.9	5368
365-Day	50	8117	41.8	6622
	47	7550	45	7171
	57	9338	NC	NC
	42.9	6811	40.4	6381
	35.7	5559	39.9	6295