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Study on The Thermal Properties of Concrete Containing Ground Granulated Blast Furnace Slag, Fly Ash and Steel Reinforcement

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**Study on The Thermal Properties of Concrete Containing Ground
Granulated Blast Furnace Slag, Fly Ash and Steel Reinforcement**

Bekir Erdem Bas

Thesis Submitted

**to the Benjamin M. Statler College of Engineering and Mineral Resources
at West Virginia University**

in partial fulfillment of the requirements for the degree of

**Master of Science
in
Civil and Environmental Engineering**

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Fly Ash; Thermal Conductivity; Semi Adiabatic-Calorimetry**

ABSTRACT

Study on The Thermal Properties of Concrete Containing Ground Granulated Blast Furnace Slag, Fly Ash and Steel Reinforcement

Bekir Erdem Bas

In this study, the thermal and mechanical properties of two types of concrete were measured. The first type contained ground granulated blast furnace slag, and the second type contained Class F fly ash. The compressive and splitting tensile strength, static elastic modulus, thermal conductivity, adiabatic temperature rise, shrinkage, and compressive creep of the two concretes were measured. Then, the experimental shrinkage and compressive creep results were compared to ACI 209.2R-08 using the material properties of each mix design. Additionally, two semi-adiabatic designs were made and compared with the adiabatic calorimetry measurements.

Also, in this study, the thermal conductivity of concrete cylinders embedded with 4% and 8% steel reinforcement was measured. The thermal conductivity of concrete is an essential parameter in predicting the temperature distribution of large structures such as pier stems, and footers. Typically, these structures contain a significant amount of steel reinforcement. Therefore, to accurately predict the temperature gradients, the effect of the steel rebar must be considered. Larger reinforcement ratios were expected to increase the overall heat loss because steel has a higher thermal conductivity than concrete. An overall effective thermal conductivity is proposed to account for the steel reinforcement. The effective thermal conductivity was shown to increase with the reinforcement ratio. Finite-element analysis (FEA) was conducted to model the experimental thermal conductivity tests. Since the production of the experimental specimens is tedious, FEA was used to simulate specimens with different reinforcement ratios. Based on the FEA results, an equation was proposed to estimate the effective thermal conductivity with different reinforcement ratios.

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TABLE OF CONTENTS

ABSTRACT.....	ii
ACKNOWLEDGMENTS	iii
LIST OF TABLES	vii
LIST OF FIGURES	viii
Chapter 1 : Introduction	1
1.1. Overview	1
1.2. Scope & Objectives.....	3
1.3. Thesis Organization	3
Chapter 2 : Literature Review.....	4
2.1. Concrete Properties	4
2.2. Shrinkage	5
2.3. Compressive Creep	6
2.4. Semi-adiabatic Calorimetry	7
2.5. Thermal Conductivity	8
Chapter 3 : Properties of Concrete Used in the Study	10
Chapter 4 : Shrinkage and Compressive Creep Properties	17
4.1. Experimental Procedure.....	17
4.1.1. Shrinkage	17

4.1.2. Compressive Creep	20
4.2. Calculations.....	28
4.2.1. ACI 209R-92 model.....	28
4.2.2. Bažant-Baweja B3 model	32
4.2.3. CEB MC90-99 model	34
4.2.4. GL2000 model	39
4.3. Conclusions.....	47
Chapter 5 : Measurement of Adiabatic Temperature Rise.....	48
5.1. Experimental Setups	48
5.2. Calculations.....	51
5.3. Conclusions.....	56
Chapter 6 : Measurement of Thermal Conductivity	57
6.1. Thermal conductivity of concrete materials	57
6.2. Thermal conductivity of reinforced concrete.....	62
6.3. Analysis of Results	65
6.4. Effective Thermal Conductivity	69
6.5. Conclusions.....	70
Chapter 7 : Conclusion and Future Study	71
7.1. Conclusion	71
7.2. Future Study.....	72

REFERENCES	73
VITA	76

LIST OF TABLES

Table 3-1: Concrete mix design.....	10
Table 3-2: Measured properties of concrete mixes.....	11
Table 3-3: Comparison of 28 days compressive and splitting tensile strength.....	14
Table 4-1: Parameter ranges of each model (ACI Committee 209 2008).	27
Table 4-2: Input values used in the calculation process of 50% slag 1 and 30% fly ash 1 shrinkage and compressive creep predictions	28
Table 4-3: The OPC mix design properties and input parameters.....	44
Table 5-1: Comparison of average λ 's calculated with literature recommendation.	56
Table 5-2: The heat loss characteristic ranges and effecting values taken into consideration of both mixes.	56
Table 6-1: Density and specific heat of 50% slag 1 and 30% fly ash 2 cylinders.....	63
Table 6-2: Thermal conductivities of 50% slag 1 and 30% fly ash 2 cylinders.	63
Table 6-3: Mortar mix design.	64
Table 6-4: Specific heat and thermal conductivity.	67

LIST OF FIGURES

Figure 3-1: Comparison of 50% slag 4-in (10.2-cm diameter) by 8-in (20.3-cm height) cylinders compressive strength results.	12
Figure 3-2: Comparison of 30% fly ash 4-in (10.2-cm diameter) by 8-in (20.3-cm height) cylinders compressive strength results.	12
Figure 3-3 Comparison 50% slag 4-in (10.2-cm diameter) by 8-in (20.3-cm height) cylinders splitting tensile results.....	13
Figure 3-4: Comparison of 30% fly ash 4-in (10.2-cm diameter) by 8-in (20.3-cm height) cylinders splitting tensile results.....	13
Figure 3-5: The static elastic modulus loading frame and the setup.....	15
Figure 3-6: Static elastic modulus of 50% slag 1 mix using 6-in (15.2-cm diameter) by 12-in (30.5-cm height) cylinders.....	16
Figure 3-7: Static elastic modulus of 30% fly ash 2 mix using 6-in (15.2-cm diameter) by 12-in (30.5-cm height) cylinders.	16
Figure 4-1: Shrinkage apparatus. The leftmost figure demonstrates the Invar calibration bar.....	18
Figure 4-2: Shrinkage test results of 50% slag 1.	19
Figure 4-3: Shrinkage test results of 30% fly ash 1.	19
Figure 4-4: Ambient temperature recordings of two mix designs.	20
Figure 4-5: Ambient humidity recordings of 30% fly ash 1.	20
Figure 4-6: The loading set-up for the compressive creep test.	21
Figure 4-7: The shrinkage test set-up of the sealed specimen.	22
Figure 4-8: Compressive creep test result of 50% slag 1 without sealed shrinkage deducted.	23
Figure 4-9: Compressive creep test result of 30% fly ash 1 without sealed shrinkage deducted.	23

Figure 4-10: Comparison of 50% slag 1 shrinkage results of 3-in (7.62-cm) by 3-in (7.62-cm) by 11.25-in (28.58-cm) fully-wrapped prism and 6-in (15.2-cm) by 12-in (30.5-cm) cylinder.	24
Figure 4-11: Comparison of 30% fly ash 1 shrinkage results of 3-in (7.62-cm) by 3-in (7.62-cm) by 11.25-in (28.58-cm) fully-wrapped prism and 6-in (15.24-cm) by 12-in (30.5-cm) cylinder.	24
Figure 4-12: Compressive creep test result of 50% slag 1 with sealed shrinkage deducted.	25
Figure 4-13: Compressive creep test result of 30% fly ash 1 with sealed shrinkage deducted.	25
Figure 4-14: The compressive creep loading over time of 50% slag 1.	26
Figure 4-15: The compressive creep loading over time of 30% fly ash 1.	26
Figure 4-16: Authorized reprint of shrinkage strain predictions of numeric examples from ACI 209 (ACI Committee 209 2008).	41
Figure 4-17: Authorized reprint of compliance predictions of numeric examples from ACI 209 (ACI Committee 209 2008).	42
Figure 4-18: Shrinkage strain predictions of numeric examples of ACI 209 document (ACI Committee 209 2008) were built in the Excel.	42
Figure 4-19: Compliance predictions of numeric examples of ACI 209 document (ACI Committee 209 2008) were built in the Excel.	43
Figure 4-20: Comparison of the OPC shrinkage test results and predictions.	44
Figure 4-21: Comparison of the OPC compressive creep test results and predictions.	44
Figure 4-22: Comparison of the shrinkage test results of 50% slag 1, 30% fly ash 1 and the OPC mixes.	45
Figure 4-23: Comparison of 50% slag 1 shrinkage test results and predictions.	45
Figure 4-24: Comparison of 50% slag 1 compressive creep test result and predictions.	46
Figure 4-25: Comparison of 30% fly ash 1 shrinkage test results and predictions.	46

Figure 4-26: Comparison of 30% fly ash 1 compressive creep test result and predictions.	47
Figure 5-1: Omega RDXL4SD 4-channel datalogger thermometer.	49
Figure 5-2: The first design of the semi-adiabatic device.....	49
Figure 5-3: The second semi-adiabatic device with the lid opened in the first two pictures and one side opened in the right picture.	50
Figure 5-4: Insulated mold for 20-in (50.8-cm) cube specimen.	51
Figure 5-5: Temperature-time history of 10-in (25.4-cm) cylinder specimen cast from 50% slag 1.	53
Figure 5-6: Temperature-time history of 20-in (50.8-cm) cube specimen cast from 30% fly ash 2.	54
Figure 5-7: Comparison of the semi-adiabatic device and ATR calorimetry of 50% slag 1.....	55
Figure 5-8: Comparison of the semi-adiabatic device and ATR calorimetry of 30% fly ash 2. ..	56
Figure 6-1: 50% slag 1 cylinder 1 temperature decay of center location and water temperature. 59	
Figure 6-2: 50% slag 1 cylinder 2 temperature decay of center location and water temperature. 59	
Figure 6-3: Comparison of the temperature differences between two 50% slag 1 cylinders.	60
Figure 6-4: 30% fly ash 2 cylinder 1 temperature decay of center location and ambient water. .	60
Figure 6-5: 30% fly ash 2 cylinder 2 temperature decay of center location and ambient water. .	61
Figure 6-6: Comparison of the temperature differences between two 30% fly ash 2 cylinders. ..	61
Figure 6-7: Specimens layout of (a) 4.24% reinforcement ratio (mortar + 6 steel rebars) (b) 7.78% reinforcement ratio (mortar + 11 steel rebars) (dimensions are in inch).	63
Figure 6-8: 4.24% reinforcement ratio (mortar + 6 steel rebars) and 7.78% reinforcement ratio (mortar + 11 steel rebars).	64
Figure 6-9: Location of thermocouples (dimensions are in inch).	65

Figure 6-10: Temperature distribution at 10 min for (a) mortar (b) 4.24% reinforcement ratio (mortar + 6 steel rebars) (c) 7.78% reinforcement ratio (mortar + 11 steel rebars) (temperatures are in °F).	67
Figure 6-11: Temperature time history comparisons of experiment and FEA at different locations: (a) mortar (b) 4.24% reinforcement ratio (mortar + 6 steel rebars) (c) 7.78% reinforcement ratio (mortar + 11 steel rebars).	68
Figure 6-12: (a) Natural logarithm of temperature difference vs. time, (b) normalized effective thermal conductivity vs. volumetric reinforcement ratio.	70

Chapter 1 : Introduction

1.1. Overview

Every passing year, manufacturers and consumers start to require higher quality concrete due to durability and performance demands. This trend sometimes prompts an increase in the strength of the concrete produced. To enhance the strength of the concrete, companies began adding more cement in the mix designs. However, the additional cement and higher fineness of cement lead to further heat accumulation in the concrete members, due to the hydration reaction. Lin (2015) stated that the increase of heat generation is generally not negligible in concrete members with large dimensions, and in substantial concrete members, so-called mass concrete structures, the buildup of heat prompts problems such as thermal cracking. The definition of mass concrete per the American Concrete Institute (ACI) is “any volume of concrete with dimensions large enough to require that measures be taken to cope with generation of heat from hydration of the cement and attendant volume change to minimize cracking” (ACI Committee 2000; 2016; ACI Committee 207 2005).

Thermal cracking generally develops because the temperature difference between the inside and surface of the concrete is immensely high. Rising heat causes the interior of the concrete to enlarge, but due to heat loss, the exterior of the concrete contracts, thus resulting in extreme tensile forces on the surface, especially for early age concretes. The need to decrease this temperature difference compels manufacturers and designers to come up with a variety of solutions such as arranging internal cooling pipes, using pre-cooled materials, or lowering the heat of hydration with supplementary cementitious materials (SCM).

The most common SCMs are ground granulated blast furnace slag (GGBFS) and fly ash. Kosmatka et al. (2016) defined GGBFS as “a nonmetallic hydraulic cement consisting essentially of silicates and aluminosilicates of calcium developed in a molten condition simultaneously with iron in a blast furnace.” They also defined fly ash “a byproduct of the combustion of pulverized coal in electric power generating plants” (2016). They are common because of their availability and ability to reduce the total heat of hydration. Determining the thermal properties of concrete containing slag or fly ash is significant for mass concrete structures such as arch dams due to their cost efficiency and thermal cracking preventing characteristics.

Part of the objectives of this study comprises of casting and testing concretes containing slag and fly ash. The experimental shrinkage and compressive creep were compared to ACI-209 (ACI Committee 2008). The adiabatic temperature rise (ATR) was back-calculated from a semi-adiabatic test. Also, material properties such as compressive and tensile strength, static modulus of elasticity, and thermal conductivity were tested.

The thermal analysis is a vital task needed to define the thermal stresses and cracking potential of large reinforced concrete structures. One of the most critical parameters, in the thermal analysis, is the thermal conductivity of the materials. They should be defined precisely to predict the correct temperature distribution within the structure. Often, the steel reinforcement inside concrete is neglected, in the thermal analysis of a reinforced concrete structure, because of its layout complexity. However, the steel rebars can significantly affect the overall thermal conductivity and, therefore, the temperature and stress distribution.

1.2. Scope & Objectives

The scope and objectives of this thesis were investigating the comparability of the shrinkage and compressive creep prediction methods containing slag or fly ash. Afterward, designing an efficient semi-adiabatic calorimetry to estimate the adiabatic temperature rise of those mix designs. Then, proposing an equation to estimate the effective thermal conductivity in concrete with different reinforcement ratios.

1.3. Thesis Organization

The organization of this thesis consists of,

- Chapter 1 gives an overview and introduction.
- Chapter 2 gives a short review of studies about shrinkage, compressive creep, semi-adiabatic calorimetry, and thermal conductivity.
- Chapter 3 shows the tested properties of the investigated concrete mix designs, such as the static modulus of elasticity, compressive, and splitting tensile strengths.
- Chapter 4 explains the experimental setups of shrinkage and compressive creep and their results; afterward, it compares with empirical predictions.
- Chapter 5 explains designs for the semi-adiabatic calorimetry and compare their back-calculated results with actual adiabatic temperature rise test results.
- Chapter 6 illustrates the test procedures for the thermal conductivity of both plain and reinforced concretes and the comparison with a finite element analysis (FEA) predictions. Afterward, an equation to estimate the effective thermal conductivity of reinforced concretes is proposed.
- Chapter 7 presents conclusions and recommendations.

Chapter 2 : Literature Review

2.1. Concrete Properties

ASTM C39/C39M (ASTM Committee C09 2018) describes the testing of compressive strength property. The limitations of the test set-up, specimen properties, testing technique, and calculation stages were described.

ASTM C496/C496M (ASTM C09 Committee 2011) describes the testing of splitting tensile strength property. The limitations of the test set-up, specimen properties, testing technique, and calculation stages were described.

ASTM C469/C469M (ASTM C09 Committee 2010) describes the testing method of static modulus of elasticity and Poisson's ratio properties. The limitations of the test set-up, specimen properties, testing technique, and calculation stages were described.

LaBarca et al. (2007) stated that 30% and 50% slag replacement does not have any performance reduction in compressive and tensile strength after 14 days. American Coal Ash Association (2003) stated that fly ash replacement to some levels increases slump, the strength and durability of hydrated concrete. It also decreases water requirement in the mix design and hydration heat.

There are some studies about mass concrete and arc dam properties. Engineer Manual 1110-2-2201 (U.S. Army Corps of Engineers 1994) describes the temperature control of the mass concrete structures such as arch dams due to the generation of the heat of hydration. Subsequently, various design solutions are proposed to address this issue.

2.2. Shrinkage

There are many tests and empirical models developed to measure and predict the shrinkage of concrete. ASTM C157–17 (Committee C09 2017) describes the testing of shrinkage properties. The limitations of the test set-up, specimen properties, testing technique, and calculation stages were described. The shrinkage prediction method developed by the ACI committee (ACI Committee 2008) has a limitation indicated by high and low predictions for early and late desiccation.

The estimated shrinkage properties by Bažant and Baweja (2000) are limited to ordinary Portland cement (OPC) and are limited by the water to cementitious ratio (0.35 to 0.85), the aggregate to cementitious material ratio (2.5 to 13.5), 28-days compressive strength of cylinder [17 MPa (2465.6 psi) to 69 MPa (10008 psi)], cementitious content [160 kg/m³ (269.7 lb./yd³) to 720 kg/m³ (1214 lb./yd³)]. Its calculation was very responsive to the water amount and permitted for extrapolation from early test data through early test data and early humidity deficit.

CEB MC90-99 (Gardner and Lockman 2001) model does not need input concerning the curing time or curing condition. GL2000 (Gardner and Lockman 2001) used the specified 28-day strength, loading strength, specimen size, relative humidity for concrete with a mean compressive strength lower than 82 MPa (11893 psi), and no self-desiccation assumed.

He (2013) stated that AASHTO LRFD (2010) model is better than ACI 209R-90 (92), ACI 209R-Modified by Huo (2001), CEB-FIP 90 (1990) and the B3 (1995) models to predict the shrinkage of slag or fly ash replaced concrete. Kar (2010) stated that ACI 209R-Modified by Huo (2001) and ACI 209R-90 (92) models overestimate the shrinkage of slag or fly ash replaced concrete at a low water-cementitious material ratio by weight (w/cem) and underestimate it at

high w/cem. Kar (2010) stated that the B3 (1995) and the CEB MC 90-99 (2008) models underestimate the shrinkage of slag or fly ash replaced concrete, but the GL 2000 (Gardner and Lockman 2001) model estimated the shrinkage of slag or fly ash replaced concrete closer than the other models.

2.3. Compressive Creep

There are many tests and empirical models developed to measure and predict the compressive creep of concrete, such as the ACI committee (ACI Committee 2008) defined primary creep as a sealed concrete, experiencing stable continuous load and having time-dependent growth in its strain. The pros of the model include ease of use, and it is straightforward to adjust to fit short-term test-data. However, the disadvantages of the model are that it does not have enough precision, especially when using different member sizes and being empirical. Elastic modulus calculation might have problems because it evaluated creep coefficient, not creep compliances.

ASTM C512/C512M–15 (C09 Committee 2015) described the testing of compressive creep. It proposes the limitations of the test set-up and specimen properties as well as testing technique and calculation stages.

Bazant and Baweja (2000) described a creep depiction in the concrete designing model (B3); however, it contains disadvantages from concrete design effect and design strength on the parameters while providing a way of compensation through revising some parameters based on short-time creep tests, and this parameter revising was significant for pozzolanic components. CEB MC90-99 and GL2000 (Gardner and Lockman 2001) described their models requiring the parameters such as curing and loading time, 28 days average compressive strength, relative humidity, volume/surface, and the type of the cement. He (2013) stated that AASHTO LRFD

(2010) model is better than ACI 209R-90 (92), ACI 209R-Modified by Huo (2001), CEB-FIP 90 (1990) and the B3 (1995) models to predict the creep of slag or fly ash replaced concretes.

2.4. Semi-adiabatic Calorimetry

Predicting and testing the adiabatic temperature rise of the mix design is important for the mass concrete structures. Having and maintaining a proper adiabatic calorimetry at construction sites is not always feasible. However, semi-adiabatic calorimetry does not require any heating and does not consume electricity. It is much more economical and relatively easy to construct and maintain semi-adiabatic calorimetry. There are many studies that design a semi-adiabatic calorimetry. Morabito (Morabito 1998) expressed problems about the estimation of the adiabatic curve from semi-adiabatic calorimetry because of temperature variations and reaction rates affecting each other.

TC 119-TCE (1997) described a semi-adiabatic calorimeter consisting of a commercial vacuum bottle, insulating foam rubber, and a protecting exterior shell, respectively. Furthermore, it stated that the semi-adiabatic calorimeter's ultimate heat loss should be smaller than 100 J/h/K (0.05 BTU/h/F).

Bai and Wild (Bai and Wild 2002) used semi-adiabatic calorimetry to study fly ash, and metakaolin added mortar. They found that fly ash is more effective than metakaolin to reduce the adiabatic temperature rise.

Poole et al. (L. Poole et al. 2007) examined cementitious substances' hydration with semi-adiabatic calorimetry by casting twenty different concrete mixes. They studied numerous mix properties such as type and content of cementitious materials, w/cem ratio, coarse aggregate type,

and placing temperature. They found that fly ash replacement, using low specific heat aggregate and low heat cement, and lowering cement dosage necessary to reduce ATR.

Ng et al. (P. L. Ng 2008) developed a mathematical model to compensate heat loss in semi-adiabatic calorimetry by testing ten in-lab and two ready mixed batches to validate the proposed model and to lower the minimum limits of concrete size and insulation perfection. The method attained to stay below of error occurrence $\pm 1.3\text{ }^{\circ}\text{C}$ ($\pm 2.34\text{ }^{\circ}\text{F}$). They found that 4-point measurement is better than two and 1-point measuring. They recommended that the heat loss characteristic (λ) of the setup ought to be under $2.18 \times 10^{-6}\text{ s}^{-1}$.

2.5. Thermal Conductivity

Many researchers (K.-H. Kim et al. 2003; Davraz, Koru, and Akdağ 2015) have studied the effect of different mix properties (water-cement ratio, aggregate volume fraction, moisture conditions) on the thermal conductivity of concrete. There are few studies available that studied the effect of different types of inclusions (steel reinforcement, fiber) on the overall thermal conductivity of a composite. Fraternali et al. (Fraternali et al. 2011) investigated the thermal conductivity of recycled PET fiber reinforced concrete (RPETFRC), making use of PET filaments with a variety of mechanical properties and profiles and contrasting 1% volumetric fiber inclusion to unreinforced concrete monitored enhancements in thermal resistance. Kanbur et al. (Kanbur et al. 2013) studied the thermal conductivity of concrete and reinforced concrete experimentally and numerically using two types of reinforcements with a diameter of 10-mm (0.39-in) and 12-mm (0.47-in) in three different cross-sectional geometries and acquired the thermal conductivities of these cases exemplifying the remarkable effect of rebars. Agrawal and Satapathy (Agrawal and Satapathy 2015) developed a mathematical model for evaluating the thermal conductivity of polymer composites with hybrid fillers experimentally and theoretically using two types of each

matrix materials, conductive fillers, and insulative fillers and found their model prognosticated the measurements closely.

Noh et al. (Noh et al. 2018) studied the effective thermal conductivity of reinforced concrete containing multiple layers of reinforcements; they developed a mathematical model to calculate the effective thermal conductivity of reinforced concrete, which was validated using a finite-volume method. The effective thermal conductivity (k_{ef}) reduced when rebar quantity increased, which was perpendicularly placed to the heat flow; however, k_{ef} was increased when rebar quantity increased, which was in parallel placing while both types of placings having steel volume ratios fixed.

Kim et al. (Kim, Jeon, & Lee, 2012) studied the thermal conductivity of lightweight aggregate concrete with a high volume of entrained air using a heat flow meter. They found that using lightweight aggregates reduced thermal conductivity, and it kept being reduced by air-entraining agent inclusion. Furthermore, porosity showed inverse proportionality with thermal conductivity.

Yun et al. (Yun et al. 2013) evaluated the thermal conductivity of lightweight concrete materials with various lightweight aggregate and glass bubbles using linear and plane heat source methods. They found that glass bubbles were less effective than lightweight aggregates in terms of reducing thermal conductivity.

In this study, the CRD-C 36-73 method (US Army Corps of Engineers 1973) was used to measure the thermal conductivity of mortar samples with and without steel reinforcement. Then, the experimental results were compared to numerical calculations using ABAQUS finite element analysis (FEA) software. Finally, the FEA software was used to simulate reinforced concrete with

various volumetric reinforcement ratios, and an empirical equation was developed to calculate the effective thermal conductivity.

Chapter 3 : Properties of Concrete Used in the Study

Two sets of experiments were performed. These concrete mix designs contain ground granulated blast furnace slag (GGBFS) grade 100, and Class F fly ash. ASTM C989 (AASHTO M 302-19) stated that GGBFS Grade 100 has a moderate activity index. ASTM C618 (2019) stated that Class F fly ash has pozzolanic properties. The mix design is shown in Table 3-1. The Portland cement used in the mixes is Type I/II. The coarse aggregate used is crushed no. 57 limestone aggregate and the fine aggregate is river sand. The absorption rate of coarse aggregate and fine aggregate are 0.5% and 1.4%. To be consistent, the coarse aggregate and sand were dried in an industrial oven for about 24 hours at approximately 221 °F (105 °C).

Table 3-1: Concrete mix design.

Materials	50% slag		30% fly ash	
	(lb./yd ³)	(kg/m ³)	(lb./yd ³)	(kg/m ³)
Cement	254	150.7	340	201.7
Slag	254	150.7	-	
Fly ash	-		168	99.7
Water	233.7	138.6	233.5	138.5
Limestone aggregate	1795	1064.9	1780	1056
Sand	1364	809.2	1360	806.9
Water cementitious (w/cem) ratio	0.46		0.459	

The required amounts of materials were measured based on the volume of each batch and mixed inside an electric concrete mixer. Thirty 4-in (10.2-cm diameter) by 8-in (20.3-cm height) cylinders were made for compressive and splitting tensile strength tests. Four 6-in (15.2-cm diameter) by 12-in (30.5-cm height) cylinders were made for static elastic modulus, thermal conductivity, and compressive creep tests. Two of the 6-in by 12-in (15.2-cm by 30.9-cm) cylinders were used for static elastic modulus test up to 28 days of curing then were drilled to be

used for thermal conductivity testing. The other 6-in by 12-in (15.2-cm by 30.9-cm) cylinders both fully wrapped were used for compressive creep testing, and the other was placed in the same room for shrinkage compensation consideration. Six 3-in by 3-in by 11.25-in (7.62-cm by 7.62-cm by 28.58-cm) rectangle prisms were made for shrinkage tests. Two of the shrinkage prisms were fully wrapped using plastic sheet wrap. Four of the shrinkage prisms were used to measure the drying shrinkage. Two of the drying shrinkage specimens were half wrapped, and the other two were exposed to the ambient humidity and air temperature.

All the specimens were batched at the same time then unmold after 24 hours. The compressive and tensile cylinders were cured in lime water for 28 days at the curing temperature of 73 °F (23 °C). The compressive creep and sealed (fully wrapped) cylinders for shrinkage were cured in lime water for seven days at the curing temperature of 73 °F (23 °C). After mixing, the slump, air percentage, w/cem ratio of the fresh concrete were measured and are shown in Table 3-2.

Table 3-2: Measured properties of concrete mixes.

Date	Batch	Slump [in(cm)]	Air %	Measured w/cem ratio	Target w/cem
04/01/2019	50% slag 1	6.5 (16.51)	5.2	0.468	0.46
04/01/2019	50% slag 2	6 (15.24)	5	0.478	0.46
06/25/2019	30% fly ash 1	6.5 (16.51)	6	0.466	0.459
07/17/2019	30% fly ash 2	6.5 (16.51)	4.75	0.508	0.459

The w/cem ratio was measured using the microwave oven method (Mardmomen, Chen, and Leon 2019). The results of compressive strength are shown in Figure 3-1, and Figure 3-2 and the results of splitting tensile strength are shown in Figure 3-3 and Figure 3-4. Due to having a higher w/cem ratio in 50% slag 2 and 30% fly ash 2, both compressive and splitting tensile strength are shown lower results. The compressive and tensile strength of 28 days are compared in Table 3-3.

Figure 3-1: Comparison of 50% slag 4-in (10.2-cm diameter) by 8-in (20.3-cm height) cylinders compressive strength results.

Figure 3-2: Comparison of 30% fly ash 4-in (10.2-cm diameter) by 8-in (20.3-cm height) cylinders compressive strength results.

Figure 3-3 Comparison 50% slag 4-in (10.2-cm diameter) by 8-in (20.3-cm height) cylinders splitting tensile results.

Figure 3-4: Comparison of 30% fly ash 4-in (10.2-cm diameter) by 8-in (20.3-cm height) cylinders splitting tensile results.

Table 3-3: Comparison of 28 days compressive and splitting tensile strength.

Batch	28 days compressive strength, psi (MPa)	28 days splitting tensile strength, psi (MPa)
50% slag 1	5252.1 (36.212)	606.6 (4.182)
50% slag 2	4734.9 (32.646)	560.5 (3.865)
30% fly ash 1	3644.6 (25.129)	462.7 (3.190)
30% fly ash 2	3541.2 (24.416)	373.5 (2.575)

The static elastic modulus was measured at 1, 3, 7, 14, and 28 days using two 6-in (15.2-cm diameter) by 12-inch (30.5-cm height) cylinders. The cylinders were loaded up to 35% of the compressive strength. The loading frame and the setup that was used is shown in Figure 3-5. The dial gauge is used to measure the deflection in the axial direction with an accuracy of 0.0001-in (0.0025-mm). The average modulus from the two specimens are shown in Figure 3-6 and Figure 3-7.

Figure 3-5: The static elastic modulus loading frame and the setup.

Figure 3-6: Static elastic modulus of 50% slag 1 mix using 6-in (15.2-cm diameter) by 12-in (30.5-cm height) cylinders.

Figure 3-7: Static elastic modulus of 30% fly ash 2 mix using 6-in (15.2-cm diameter) by 12-in (30.5-cm height) cylinders.

Chapter 4 : Shrinkage and Compressive Creep Properties

4.1. Experimental Procedure

4.1.1. Shrinkage

Six prisms with the dimension of 3 x 3 x 12-in (7.62 x 7.62 x 30.5-in) were used in the shrinkage test based on ASTM C157 Method (Committee C09 2017). Figure 4-1 shows the shrinkage test setup. The length change is measured with an electronic gauge attached to the top of the setup. All the actions with the setup should be done very carefully because the device is susceptible. First, the Invar Bar placed on the apparatus to calibrate the digital indicator. The Invar Bar was made of a unique metal that has negligible temperature changes. The Invar Bar should be turned around 360°, and the digital gauge should be zeroed. The Invar Bar and the sample every time should be at the same point facing outwards before turned around placed on the apparatus. The calibration should be made twice before placing the first sample. When the first sample is placed, it should be turned around 360° and the number appearing on the digital screen should be written down. Before placing the sample again, calibration with the Invar Bar should be repeated at least one time. In total, three numbers that have proximity to each other at least the sensitivity of the digital indicator - 0.0001-in (0.0025-mm) - should be recorded for each specimen. Length change measurements should be recorded until 28 days. Starting with recording the time intervals every four hours. It might be prolonged, depending on the micro-strain changes. Three kinds of drying were done on the samples: full-wrapped (sealed shrinkage), half-wrapped, and no-wrapped (drying shrinkage).

Figure 4-1: Shrinkage apparatus. The leftmost figure demonstrates the Invar calibration bar.

Drying shrinkage (micro-strain) test results of the two different mix designs and ambient temperature-humidity recordings of 30% fly ash 1 were shown in Figure 4-2, Figure 4-3, Figure 4-4, and Figure 4-5, respectively. Ambient humidity recording for 50% slag was not performed and assumed as 40% in the prediction model input.

Figure 4-2: Shrinkage test results of 50% slag 1.

Figure 4-3: Shrinkage test results of 30% fly ash 1.

Figure 4-4: Ambient temperature recordings of two mix designs.

Figure 4-5: Ambient humidity recordings of 30% fly ash 1.

4.1.2. Compressive Creep

A 6-in (15.2-cm diameter) and 12-in (30.5-cm height) cylinder was used in the compressive creep test based on ASTM C512 Method (C09 Committee 2015). It specifies that, regardless of any dimension alteration of the specimen, a loading set-up should be able to utilize and sustain the requisite load on the specimen. The loaded specimen was held with steel header plates and a spring

as a load-preserving component to damp the response of the loaded set-up. A jack utilized the preliminary compression. The spring-loaded set-up is shown in Figure 4-6. A load cell was implanted in the set-up to measure the utilized load. An affixed device was employed to measure the specimen's longitudinal strain.

Figure 4-6: The loading set-up for the compressive creep test.

Shrinkage of a sealed (fully wrapped) cylinder was also measured to calculate the actual compressive creep because the results from the compressive loading setup include the shrinkage

of the sealed specimen as well as the compressive creep. Both specimens were wrapped with plastic sheets to ensure there was very limited drying shrinkage. The shrinkage of the sealed specimen was tested based on the ASTM C157 Method (Committee C09 2017). The shrinkage test set-up of the sealed specimen was shown in Figure 4-7.

Figure 4-7: The shrinkage test set-up of the sealed specimen.

The compressive creep test results without deducting the shrinkage of the sealed specimen, and the compressive creep with the shrinkage of the sealed specimen deducted test are shown in Figure 4-8, Figure 4-9, Figure 4-12, and Figure 4-13, respectively. The compressive creep loading over time of two tests are shown in Figure 4-14 and Figure 4-15. Comparison of 50% slag 1 shrinkage results of 3-in (7.62-cm) by 3-in (7.62-cm) by 11.25-in (28.58-cm) fully-wrapped prism and 6-in (15.2-cm) by 12-in (30.5-cm) cylinder is shown in Figure 4-10, and the difference between them was around 25 micro-strains around 58 days. Comparison of 30% fly ash 1 shrinkage results of 3-in (7.62-cm) by 3-in (7.62-cm) by 11.25-in (28.58-cm) fully-wrapped prism and 6-in (15.2-cm) by 12-in (30.5-cm) cylinder is shown in Figure 4-11 and difference between them was around 95 micro-strains around 23 days.

Figure 4-8: Compressive creep test result of 50% slag 1 without sealed shrinkage deducted.

Figure 4-9: Compressive creep test result of 30% fly ash 1 without sealed shrinkage deducted.

Figure 4-10: Comparison of 50% slag 1 shrinkage results of 3-in (7.62-cm) by 3-in (7.62-cm) by 11.25-in (28.58-cm) fully-wrapped prism and 6-in (15.2-cm) by 12-in (30.5-cm) cylinder.

Figure 4-11: Comparison of 30% fly ash 1 shrinkage results of 3-in (7.62-cm) by 3-in (7.62-cm) by 11.25-in (28.58-cm) fully-wrapped prism and 6-in (15.24-cm) by 12-in (30.5-cm) cylinder.

Figure 4-12: Compressive creep test result of 50% slag 1 with sealed shrinkage deducted.

Figure 4-13: Compressive creep test result of 30% fly ash 1 with sealed shrinkage deducted.

Figure 4-14: The compressive creep loading over time of 50% slag 1.

Figure 4-15: The compressive creep loading over time of 30% fly ash 1.

The prediction for shrinkage and compressive creep can be done with the following equations provided by Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete (ACI Committee 209 2008), Model B3 (Bažant and Baweja 2000), CEB MC 90, CEB MC90-99, and GL2000 (Gardner and Lockman 2001). Regular concrete mixes and mixes with fly ash up to 30% replacement were used to calibrate the prediction models (ACI Committee 209 2008). The range of the parameters of each model are shown in Table 4-1 (ACI Committee 209 2008), showing the ACI 209R-92 model (ACI Committee 209 2008) has no limitations of 28 days

concrete mean compressive cylinder strength (f_{cm28}), the aggregate-cement ratio by weight (a/c), the water-cement ratio by weight (w/c) and specimen material, the B3 model (Bažant and Baweja 2000) has no limitations of the steam curing time [t_c (steam cured)], the CEB MC 90 and the CEB MC90-99 models (fib Committee 1999) has no limitations of the aggregate-cement ratio by weight (a/c), the cement content by weight (c), the water-cement ratio by weight (w/c), the steam curing time [t_c (steam cured)] and specimen material, the GL2000 model (Gardner and Lockman 2001) has no limitations of the aggregate-cement ratio by weight (a/c), the cement content by weight (c), the steam curing time [t_c (steam cured)] and specimen material,

Table 4-1: Parameter ranges of each model (ACI Committee 209 2008).

Input variables	Model				
	ACI 209R-92	Bažant-Baweja B3	CEB MC90	CEB MC90-99	GL2000
f_{cm28} , MPa (psi)	-	17 to 70 (2500 to 10,000)	20 to 90 (2900 to 13,000)	15 to 120 (2175 to 17,400)	16 to 82 (2320 to 11,900)
a/c	-	2.5 to 13.5	-	-	-
c, kg/m ³ (lb./yd ³)	279 to 446 (470 to 752)	160 to 720 (270 to 1215)	-	-	-
w/c	-	0.35 to 0.85	-	-	0.4 to 0.6
Rh, %	40 to 100	40 to 100	40 to 100	40 to 100	20 to 100
Type of cement, European (U.S.)	R or RS (I or III)	R, SL, RS (I, II, III)	R, SL, RS (I, II, III)	R, SL, RS (I, II, III)	R, SL, RS (I, II, III)
t_c (moist cured)	≥ 1 day	≥ 1 day	< 14 days	< 14 days	≥ 1 day
t_c (steam cured)	1 to 3 days	-	-	-	-
t_o	≥ 7 days	$t_o \geq t_c$	> 1 day	> 1 day	$t_o \geq t_c \geq 1$ day
Other required parameters	s, ψ , α , mix proportions, and V/S or d	E_{cm28} , shape, V/S or d _{effective cross-section}	V/S	V/S	V/S, f_{cmto} , E_{cm28} , E_{cmto}
Specimen material	-	Portland cement concrete	-	-	-

where f_{cm28} is the 28 days concrete mean compressive cylinder strength, a/c is the aggregate-cement ratio by weight, c is the content of cement by weight, w/c is the water-cement ratio by weight, Rh is the ambient relative humidity, t_c is the curing time, t_o is the loading time, s is the

slump, ψ is the fine aggregate percentage, α is the air content, V/S is the volume-surface ratio, d is the average thickness, E_{cm28} is the 28 days concrete mean modulus of elasticity, $d_{\text{effective cross-section}}$ is the average thickness of the effective cross-section, f_{cmto} is the loading time concrete mean compressive strength and E_{cmto} is the loading time concrete mean modulus of elasticity (2008). In this study w/cem ratio were used in the calculation process of 50% slag 1 and 30% fly ash 1 shrinkage and compressive creep predictions instead of w/c ratio mentioned in the ACI 209.2R-08 (2008). Input values used in the calculation process of 50% slag 1 and 30% fly ash 1 shrinkage and compressive creep predictions is shown in Table 4-2.

Table 4-2: Input values used in the calculation process of 50% slag 1 and 30% fly ash 1 shrinkage and compressive creep predictions

Input values	50% slag 1		30% fly ash 1	
	Shrinkage	Compressive creep	Shrinkage	Compressive creep
f'_c (specified 28-day strength)	5252.1 psi (36.212 MPa)		3644.6 psi (25.129 MPa)	
h% (relative humidity)	0.4	1	0.5781	1
T (temperature)	71.27 °F (21.817 °C)	71.78 °F (22.1 °C)	73.12 °F (22.84 °C)	72 °F (22.22 °C)
V/S (volume-surface ratio)	0.657-in (16.7-mm)	1.2-in (30.5-mm)	0.657-in (16.7-mm)	1.2-in (30.5-mm)
t_c (curing time)	1 day	7 days	1 day	
t_o (loading time)	1 day	13.82 days	1 day	
Maximum aggregate size	1-in (25.4-mm)			
cem (cementitious material content by weight)	508 lb./yd ³ (301.384 kg/m ³)			
w/cem (water-cementitious material ratio by weight)	0.468		0.466	
a/cem (aggregate-cementitious material ratio by weight)	6.219		6.181	
Ψ (fine aggregate percentage)	43.2		43.3	
α % (air content)	5.2		6	
s (slump)	6.5-in (165.1-mm)			
γ_c (unit weight of concrete)	3900.7 lb./yd ³ (2314.19 kg/m ³)		3881.5 lb./yd ³ (2302.8 kg/m ³)	

4.2. Calculations

The following models are used to predict the shrinkage and compressive creep:

4.2.1. ACI 209R-92 model

ACI Committee 209 (2008) stated that their model for predicting shrinkage and creep included standard conditions, and correction factors were provided to estimate the ultimate values for other conditions.

4.2.1.1 Shrinkage

The $\varepsilon_{sh}(t, t_c)$ is the shrinkage strain might be calculated by Equation 4-1,

$$\varepsilon_{sh}(t, t_c) = \frac{(t-t_c)^\alpha}{f+(t-t_c)^\alpha} \varepsilon_{shu}, \quad (4-1)$$

where t is the concrete age (days), t_c is curing time (days), ε_{shu} is the ultimate shrinkage strain might be calculated by Equation 4-3, and in Equation 4-1, through substituting $\alpha = 1.0$, along with f where V/S is the volume-surface ratio in inches or millimeters as given by Equation 4-2, the shape and size effect might be utterly assumed to be on the time-ratio,

$$f = 26.0e^{[0.36(V/S)]} \text{ (in.)}, f = 26.0e^{[1.42 \times 10^{-2}(V/S)]} \text{ (mm)}, \quad (4-2)$$

$$\varepsilon_{shu} = 780\gamma_{sh} \times 10^{-6} \geq 150 \times 10^{-6} \text{ (in./in.)} (\text{state of continuous drying}), \quad (4-3)$$

$$\gamma_{sh} = \gamma_{sh,tc} \gamma_{sh,RH} \gamma_{sh,vs} \gamma_{sh,s} \gamma_{sh,\psi} \gamma_{sh,c} \gamma_{sh,\alpha} \leq 0.2, \quad (4-4)$$

where γ_{sh} calculated by Equation 4-4 is the cumulative outcome of the applicable adjustment components, $\gamma_{sh,tc}$ calculated by Equation 4-5 is the preliminary moist curing coefficient, $\gamma_{sh,RH}$ calculated by Equation 4-6 is the ambient relative humidity coefficient, $\gamma_{sh,vs}$ calculated by

Equation 4-7 is the coefficient to reflect the member dimensions effect in terms of V/S , $\gamma_{sh,s}$ calculated by Equation 4-8 is the factor of the slump, $\gamma_{sh,\Psi}$ calculated by Equation 4-9 is the factor of fine aggregate, $\gamma_{sh,c}$ calculated by Equation 4-10 is the factor of the content of cement, $\gamma_{sh,\alpha}$ calculated by Equation 4-11 is the factor of the content of air,

$$\gamma_{sh,tc} = 1.202 - 0.2337 \log(t_c), R^2 = 0.9987, \quad (4-5)$$

$$\gamma_{sh,RH} = \begin{cases} 1.4 - 1.02h, & 0.4 \leq h < 0.8 \\ 3 - 3h, & 0.8 \leq h \leq 1 \end{cases}, \quad (4-6)$$

where h is the relative humidity in percentage,

$$\gamma_{sh,vs} = 1.2e^{[-0.12(V/S)]}, \quad (4-7)$$

$$\gamma_{sh,s} = 0.89 + 0.041s, \quad (4-8)$$

where s is the slump (in.),

$$\gamma_{sh,\Psi} = \begin{cases} 0.3 + 0.014\Psi, & \Psi < 50\% \\ 0.9 + 0.002\Psi, & \Psi \geq 50\% \end{cases}, \quad (4-9)$$

where Ψ is the fine/total aggregate weight ratio,

$$\gamma_{sh,c} = 0.75 + 0.00036c, \quad (4-10)$$

where c is the content of cement (lb/yd^3),

$$\gamma_{sh,\alpha} = 0.95 + 0.008\alpha \geq 1, \quad (4-11)$$

where α is the content of air (%).

4.2.1.2 Creep Compliance

The $J(t, t_0)$ is the compliance function was calculated by Equation 4-12,

$$J(t, t_0) = \frac{1 + \phi(t, t_0)}{E_{cmto}}, \quad (4-12)$$

where E_{cmto} calculated by Equation 4-13 is the loading time modulus of elasticity (psi), $\phi(t, t_0)$ calculated by Equation 4-15 is the loading time creep/elastic strain coefficient of creep,

$$E_{cmto} = 33\gamma_c^{1.5}\sqrt{f_{cmto}}, \quad (4-13)$$

where γ_c is the concrete unit weight (lb/ft^3), f_{cmto} calculated by Equation 4-14 is the loading time mean compressive strength of concrete (psi),

$$f_{cmto} = \left(\frac{t}{a+bt}\right) f_{cm28}, \quad (4-14)$$

where f_{cm28} is the 28 days mean compressive strength of concrete, t is the concrete age, a and b are recommended as 4 and 0.85 in the standard, respectively,

$$\phi(t, t_0) = \frac{(t-t_0)^\Psi}{d+(t-t_0)^\Psi} \phi_u, \quad (4-15)$$

where ϕ_u calculated by Equation 4-16 is the coefficient of ultimate creep, d and Ψ are recommended as f and 1 in standard, respectively,

$$\phi_u = 2.35\gamma_c, \quad (4-16)$$

where γ_c calculated by Equation 4-17 is the applicable improvement factors' cumulative outcome,

$$\gamma_c = \gamma_{c,to} \gamma_{c,RH} \gamma_{c,vs} \gamma_{c,s} \gamma_{c,\Psi} \gamma_{c,\alpha}, \quad (4-17)$$

where $\gamma_{c,to}$ calculated by Equation 4-18 is the creep loading time factor, $\gamma_{c,RH}$ calculated by Equation 4-19 is the factor of the relative humidity of ambient, $\gamma_{c,vs}$ calculated by Equation 4-20

is the coefficient to reflect the member dimensions effect in terms of V/S , $\gamma_{c,s}$ calculated by Equation 4-21 is the factor of the slump, $\gamma_{c,\psi}$ calculated by Equation 4-22 is the factor of fine aggregate, $\gamma_{c,\alpha}$ calculated by Equation 4-23 is the factor of the content of air.

$$\gamma_{c,to} = 1.25t_0^{-1.118} \quad (4-18)$$

$$\gamma_{c,RH} = 1.27 - 0.67h \quad (4-19)$$

$$\gamma_{c,vs} = \frac{2}{3} [1 + 1.13e^{(-0.54\{V/S\})}] \quad (4-20)$$

$$\gamma_{c,s} = 0.82 + 0.067s \quad (4-21)$$

$$\gamma_{c,\psi} = 0.88 + 0.0024\Psi \quad (4-22)$$

$$\gamma_{c,\alpha} = 0.46 + 0.09\alpha \geq 1 \quad (4-23)$$

4.2.2. Bažant-Baweja B3 model

Bažant and Baweja (2000) stated that the B3 model is calibrated by a data bank, including test data obtained in many laboratories.

4.2.2.1 Shrinkage

From Equation 4-24, the cross-section mean shrinkage strain $\varepsilon_{sh}(t, t_c)$ might be calculated,

$$\varepsilon_{sh}(t, t_c) = -\varepsilon_{sh\infty} k_h S(t - t_c), \quad (4-24)$$

where $\varepsilon_{sh\infty}$ calculated by Equation 4-25 is the ultimate shrinkage strain, k_h calculated by Equation 4-26 is the factor of the dependence of humidity, and $S(t - t_c)$ calculated by Equation 4-27 is the curve of time,

$$\varepsilon_{sh\infty} = -\varepsilon_{s\infty} \frac{E_{cm607}}{E_{cm(t_c + \tau_{sh})}}, \quad (4-25)$$

where $\varepsilon_{s\infty}$ calculated by Equation 4-28 is a constant, and $\frac{E_{cm607}}{E_{cm(t_c + \tau_{sh})}}$ ultimate shrinkage time dependence interpretation factor,

$$k_h = 1 - h^3, \quad (4-26)$$

$$S(t - t_c) = \tanh \sqrt{\frac{(t - t_c)}{\tau_{sh}}}, \quad (4-27)$$

where τ_{sh} calculated by Equation 4-30 is the half-time shrinkage (days),

$$\varepsilon_{s\infty} = -\alpha_1 \alpha_2 (0.02565 w^{2.1} f_{cm28}^{-0.28} + 270) \times 10^{-6}, \quad (4-28)$$

where w is the content of water (lb/yd^3), α_1 and α_2 are recommended by the standard as 0.85 and 1.2, respectively,

$$E_{cmt} = E_{cm28} \left(\frac{t}{4 + 0.85t} \right)^{0.5}, \quad (4-29)$$

$$\tau_{sh} = 190.8 t_c^{-0.08} f_{cm28}^{-0.25} (2k_s[V/S])^2, \quad (4-30)$$

where k_s is the factor of shape-correction of the cross-section is recommended by the standard as 1.25 for this study.

4.2.2.2 Creep Compliances

The function of median compliance $J(t, t_o)$ might be calculated from Equation 4-31,

$$J(t, t_o) = q_1 + C_o(t, t_o), \quad (4-31)$$

where q_1 calculated by Equation 4-32 is the unit stress caused immediate strain, $C_o(t, t_o)$ calculated by Equation 4-34 is the basic creep function of compliance,

$$q_1 = 0.6/E_{cm28}, \quad (4-32)$$

where,

$$E_{cm28} = 57,000 \sqrt{f_{cm28}}, \quad (4-33)$$

$$C_o(t, t_o) = q_2 Q(t, t_o) + q_3 \ln[1 + (t - t_o)^n] + q_4 \ln(t/t_o), \quad (4-34)$$

where $q_2 Q(t, t_o)$ calculated by multiplying Equations 4-35 and 4-36 is the term for aging viscoelastic compliance, q_3 calculated by Equation 4-40 is the parameter of nonaging viscoelastic compliance, and q_4 calculated by Equation 4-41 is the term for aging flow compliance and n is recommended by the standard as 0.1,

$$q_2 = 86.814 \times 10^{-6} c^{0.5} f_{cm28}^{-0.9}, \quad (4-35)$$

$$Q(t, t_0) = Q_f(t_0) \left\{ 1 + \left[\frac{Q_f(t_0)}{Z(t, t_0)} \right]^{r(t_0)} \right\}^{-1/r(t_0)}, \quad (4-36)$$

where,

$$Q_f(t_0) = [0.086(t_0)^{2/9} + 1.21(t_0)^{4/9}]^{-1}, \quad (4-37)$$

$$Z(t, t_0) = t_0^{-m} \ln[1 + (t - t_0)^n], \quad (4-38)$$

$$r(t_0) = 1.7(t_0)^{0.12} + 8, \quad (4-39)$$

$$q_3 = 0.29(w/c)^4 q_2, \quad (4-40)$$

$$q_4 = 0.14 \times 10^{-6} (a/c)^{-0.7}, \quad (4-41)$$

where a/c is the aggregate/cement in weight.

4.2.3. CEB MC90-99 model

The CEB MC90 model (1993) for creep and shrinkage predicts a hyperbolic change with time, and it also takes mixture proportion and environmental conditions into consideration when applying an ultimate value. The CEB MC90-99 model (1999) was intended to predict shrinkage and creep for normal and high-strength concrete, and it calculates autogenous and drying shrinkage.

4.2.3.1 Shrinkage (CEB MC90)

The $\varepsilon_{sh}(t, t_c)$ was calculated by Equation 4-42 is the concrete total shrinkage strains,

$$\varepsilon_{sh}(t, t_c) = \varepsilon_{cso} \beta_s(t - t_c), \quad (4-42)$$

where ε_{cso} calculated by Equation 4-43 is the coefficient of estimated shrinkage, $\beta_s(t - t_c)$ calculated by Equation 4-46 is the shrinkage progress coefficient,

$$\varepsilon_{cso} = \varepsilon_s(f_{cm28}) \beta_{RH}(h), \quad (4-43)$$

$$\varepsilon_s(f_{cm28}) = [160 + 10\beta_{sc}(9 - f_{cm28}/f_{cmo})] \times 10^{-6}, \quad (4-44)$$

where f_{cmo} (psi) and β_{sc} are recommended in the standard as 1450 and 5, respectively,

$$\beta_{RH}(h) = -1.55 \left[1 - \left(\frac{h}{h_o} \right)^3 \right], \quad (4-45)$$

where h_o is recommended in the standard as 1,

$$\beta_s(t - t_c) = \left\{ \frac{(t - t_c)/t_1}{350[(V/S)/(V/S)_o]^2 + (t - t_c)/t_1} \right\}^{0.5}, \quad (4-46)$$

where t_1 (days) and $(V/S)_o$ (in.) are recommended by the standard as 1 and 2, respectively.

4.2.3.2 Shrinkage (CEB MC90-99)

The $\varepsilon_{sh}(t, t_c)$ was calculated by Equation 4-47 is the concrete total shrinkage,

$$\varepsilon_{sh}(t, t_c) = \varepsilon_{cas}(t) + \varepsilon_{cds}(t, t_c), \quad (4-47)$$

where $\varepsilon_{cas}(t)$ calculated by Equation 4-48 is the autogenous shrinkage and $\varepsilon_{cds}(t, t_c)$ calculated by Equation 4-51 is the drying shrinkage,

$$\varepsilon_{cas}(t) = \varepsilon_{caso}(f_{cm28})\beta_{as}(t), \quad (4-48)$$

where $\varepsilon_{caso}(f_{cm28})$ calculated by Equation 4-49 is the coefficient of estimated autogenous shrinkage and $\beta_{as}(t)$ calculated by Equation 4-50 is the autogenous shrinkage time progress function,

$$\varepsilon_{caso}(f_{cm28}) = -\alpha_{as} \left(\frac{f_{cm28}/f_{cmo}}{6+f_{cm28}/f_{cmo}} \right)^{2.5} \times 10^{-6}, \quad (4-49)$$

where α_{as} is recommended by the standard as 700 for this study,

$$\beta_{as}(t) = 1 - \exp \left[-0.2 \left(\frac{t}{t_1} \right)^{0.5} \right], \quad (4-50)$$

$$\varepsilon_{cds}(t, t_c) = \varepsilon_{cdso}(f_{cm28})\beta_{RH}(h)\beta_{ds}(t - t_c), \quad (4-51)$$

where $\varepsilon_{cdso}(f_{cm28})$ calculated by Equation 4-52 is the coefficient of estimated drying shrinkage, $\beta_{RH}(h)$ calculated by Equation 4-53 is the drying shrinkage relative humidity effect coefficient, and $\beta_{ds}(t - t_c)$ calculated by Equation 4-46 is the drying shrinkage time progress function,

$$\varepsilon_{cdso}(f_{cm28}) = [(220 + 110\alpha_{ds1})\exp(-\alpha_{ds2}f_{cm28}/f_{cmo})] \times 10^{-6}, \quad (4-52)$$

where α_{ds1} and α_{ds2} recommended by the standard as 4 and 0.12, respectively,

$$\beta_{RH}(h) = \begin{cases} -1.55 \left[1 - \left(\frac{h}{h_o} \right)^3 \right], & 0.4 \leq h < 0.99\beta_{s1}, \\ 0.25, & h \geq 0.99\beta_{s1} \end{cases} \quad (4-53)$$

where h_o is recommended by the standard as 1, β_{s1} calculated by Equation 4-54 is the high-performance self-desiccation coefficient.

$$\beta_{s1} = \left(\frac{3.5f_{cmo}}{f_{cm28}} \right)^{0.1} \leq 1 \quad (4-54)$$

4.2.3.3 Creep Compliance

The $J(t, t_o)$ was calculated by Equation 4-55 is the total stress-dependent strain by unit stress,

$$J(t, t_o) = \frac{1}{E_{cm28}} [\eta(t_o) + \Phi_{28}(t, t_o)] = \frac{1}{E_{cmto}} + \frac{\Phi_{28}(t, t_o)}{E_{cm28}}, \quad (4-55)$$

where $\Phi_{28}(t, t_o)$ calculated by Equation 4-57 is the coefficient of 28-day creep exempt from dimension, $\eta(t_o)$ calculated by Equation 4-56 is the 28 days concrete mean modulus of elasticity,

$$\eta(t_o) = \frac{E_{cm28}}{E_{cmto}}, \quad (4-56)$$

$$\Phi_{28}(t, t_o) = \Phi_o \beta_c(t - t_o), \quad (4-57)$$

where Φ_o calculated by Equation 4-58 is the estimated creep coefficient and $\beta_c(t - t_o)$ calculated by Equation 4-64 is the creep progress coefficient,

$$\Phi_o = \Phi_{RH}(h) \beta(f_{cm28}) \beta(t_o), \quad (4-58)$$

where,

$$\Phi_{RH}(h) = \left[1 + \frac{1-h/h_o}{\sqrt[3]{0.1\{(V/S)/(V/S)_o\}}} \alpha_1 \right] \alpha_2, \quad (4-59)$$

$$\beta(f_{cm28}) = \frac{5.3}{\sqrt{f_{cm28}/f_{cmo}}}, \quad (4-60)$$

$$\beta(t_o) = \frac{1}{0.1 + (t_o/t_1)^{0.2}}, \quad (4-61)$$

$$\alpha_1 = \left(\frac{3.5f_{cmo}}{f_{cm28}} \right)^{0.7}, \quad (4-62)$$

$$\alpha_2 = \left(\frac{3.5f_{cmo}}{f_{cm28}} \right)^{0.2}, \quad (4-63)$$

$$\beta_c(t - t_o) = \left[\frac{(t - t_o)/t_1}{\beta_h + (t - t_o)/t_1} \right]^{0.3}, \quad (4-64)$$

where,

$$\beta_h = 150[1 + (1.2h/h_o)^{18}] (V/S)/(V/S)_o + 250\alpha_3 \leq 1500\alpha_3, \quad (4-65)$$

where,

$$\alpha_3 = \left(\frac{3.5f_{cmo}}{f_{cm28}} \right)^{0.5}. \quad (4-66)$$

If the elasticity modulus at an age other than 28 days E_{cmt} needed, it might be calculated by Equation 4-67,

$$E_{cmt} = E_{cm28} \exp \left[\frac{s}{2} \left(1 - \sqrt{\frac{28}{t/t_1}} \right) \right], \quad (4-67)$$

where s is recommended by the standard as 0.25 for this study,

$$E_{cm28} = 3,118,310 \alpha_E^3 \sqrt{\frac{f_{cm28}}{f_{cmo}}}, \quad (4-68)$$

where α_E is recommended by the standard as 0.9 for limestone aggregates,

$$f_{cm28} = f'_c + 1160, \quad (4-69)$$

where f'_c is the characteristic compressive cylinder strength (psi).

4.2.4. GL2000 model

The GL2000 model (2011) proposes a design-office procedure for predicting normal strength concrete's shrinkage and creep.

4.2.4.1 Shrinkage

The f_{cm28} was calculated by Equation 4-70 is the 28 days concrete mean compressive strength,

$$f_{cm28} = 1.1f'_c + 700, \quad (4-70)$$

where f'_c is the characteristic compressive strength. E_{cmt} calculated by Equation 4-71 is the modulus of elasticity,

$$E_{cmt} = 500,000 + 52,000\sqrt{f_{cmt}}, \quad (4-71)$$

where f_{cmt} calculated by Equation 4-72 is the strength development with time,

$$f_{cmt} = \beta_e^2 f_{cm28}, \quad (4-72)$$

where β_e calculated by Equation 4-73 relates strength development to cement type,

$$\beta_e = \exp\left[\frac{s}{2}\left(1 - \sqrt{\frac{28}{t}}\right)\right], \quad (4-73)$$

where s is recommended by the standard as 0.4 for this study. $\varepsilon_{sh}(t, t_c)$ calculated by Equation 4-74 is the shrinkage strain,

$$\varepsilon_{sh}(t, t_c) = \varepsilon_{shu}\beta(h)\beta(t - t_c), \quad (4-74)$$

where ε_{shu} calculated by Equation 4-75 is the ultimate shrinkage strain, $\beta(h)$ calculated by Equation 4-76 is a correction term for the effect of humidity, $\beta(t - t_c)$ calculated by Equation 4-77 is a correction term for the drying time effect,

$$\varepsilon_{shu} = 900k \left(\frac{4350}{f_{cm28}} \right)^{0.5} \times 10^{-6}, \quad (4-75)$$

where k is recommended by the standard as 0.75 for this study.

$$\beta(h) = (1 - 1.18h^4) \quad (4-76)$$

$$\beta(t - t_c) = \sqrt{\frac{(t - t_c)}{(t - t_c) + 77(V/S)^2}} \quad (4-77)$$

4.2.4.2 Creep Compliance:

$$J(t, t_o) = \frac{1}{E_{cmto}} + \frac{\Phi_{28}(t, t_o)}{E_{cm28}}, \quad (4-78)$$

where,

$$\Phi_{28}(t, t_o) = \Phi(t_c) \left[2 \frac{(t-t_o)^{0.3}}{(t-t_o)^{0.3}+14} + \left(\frac{7}{t_o} \right)^{0.5} \left(\frac{(t-t_o)}{(t-t_o)+7} \right)^{0.5} + 2.5(1 - 1.086h^2) \left(\frac{(t-t_o)}{(t-t_o)+77(V/S)^2} \right)^{0.5} \right]. \quad (4-79)$$

The numeric examples in ACI 209 (ACI Committee 209 2008) shown in Figure 4-16 and Figure 4-17 were built in Excel, shown in Figure 4-18 and Figure 4-19. Then, 50% slag 1, 30% fly ash 1 mix design, and specimen inputs were inserted to calculate predictions of the test results.

Comparisons of predictions and test results are illustrated in Figure 4-23, Figure 4-24, Figure 4-25, and Figure 4-26, respectively.

Figure 4-16: Authorized reprint of shrinkage strain predictions of numeric examples from ACI 209 (ACI Committee 209 2008).

Figure 4-17: Authorized reprint of compliance predictions of numeric examples from ACI 209 (ACI Committee 209 2008).

Figure 4-18: Shrinkage strain predictions of numeric examples of ACI 209 document (ACI Committee 209 2008) were built in the Excel.

Figure 4-19: Compliance predictions of numeric examples of ACI 209 document (ACI Committee 209 2008) were built in the Excel.

To check the validity of the models, they first compared to an OPC mix design, which was cast in the WVD0H RP312 research project. The OPC mix design properties and input parameters are shown in Table 4-3. Comparisons of the predictions, and the test results of the OPC mix are shown in Figure 4-20 and Figure 4-21, respectively. Comparison of the shrinkage test results of 50% slag 1, 30% fly ash 1 and the OPC mixes are shown in Figure 4-22.

Table 4-3: The OPC mix design properties and input parameters.

Materials	OPC				
	(lb./yd³)	(kg/m³)	Input parameters		
c	564	335	f _c '	3987 (psi)	27 (MPa)
w	282	167			
a/c	5.418		h	0.5	
ψ	41.9				
Measured w/c	0.5		s	5 (in)	127 (mm)
γ _c	3895.6	2311.2

Figure 4-20: Comparison of the OPC shrinkage test results and predictions.

Figure 4-21: Comparison of the OPC compressive creep test results and predictions.

Figure 4-22: Comparison of the shrinkage test results of 50% slag 1, 30% fly ash 1 and the OPC mixes.

Figure 4-23: Comparison of 50% slag 1 shrinkage test results and predictions.

Figure 4-24: Comparison of 50% slag 1 compressive creep test result and predictions.

Figure 4-25: Comparison of 30% fly ash 1 shrinkage test results and predictions.

Figure 4-26: Comparison of 30% fly ash 1 compressive creep test result and predictions.

4.3. Conclusions

50% slag 1 shrinkage test results appeared to fall within the range of the predictions of GL2000 and CEB MC 90-99 models. 50% slag 1 compressive creep test results appeared to fall within the range of the predictions of the B3 model.

30% fly ash 1 shrinkage test results appeared to fall within the range of predictions of the ACI 209 method; however, compressive creep test results failed to appear to fall within the range of any prediction, possibly due to first-day loading.

The OPC shrinkage test results appeared to fall within the range of predictions of the GL2000 model. The OPC shrinkage test results appeared to compare closely with 50% slag no-wrap 1's. The OPC compressive creep test results appeared to fall within the range of predictions of the B3 model.

Chapter 5 : Measurement of Adiabatic Temperature Rise

5.1. Experimental Setups

The first design of the semi-adiabatic device from inside consists of a 10-in by 10-in (25.4-cm by 25.4-cm) cylindrical specimen cast inside and covered by a plastic bag, an insulated plastic cooler jug, and six layers of 1-in (2.54-cm) thick extruded polystyrene (XPS) insulation. The gap between the insulated plastic cooler jug and the XPS insulation board was filled with thermal fiberglass insulation wool. The device was separated from the ground by 2-in by 4-in (5.08-cm by 10.16-cm) woods placed horizontally with another layer of 1-in (2.54-cm) thick XPS insulation to prevent heat loss to the ground. The plastic bag was sealed with duct tape. The casting was made with a K-type thermocouple inserted at the center of the specimen. Temperature time histories were recorded with an Omega RDXL4SD 4-Channel datalogger thermometer connecting which had an accuracy of $\pm (0.4\% + 1\text{ }^{\circ}\text{F})$ [$\pm (0.4\% + 0.5\text{ }^{\circ}\text{C})$]. The ambient temperature was measured with another K-type thermocouple connected to the digital thermometer. The temperatures of the two locations mentioned above were recorded for 7-days. The center and ambient temperatures were recorded at intervals of 5-minutes. The Omega RDXL4SD 4-Channel datalogger thermometer is shown in Figure 5-1. The device with the lid opened is shown in Figure 5-2.

Figure 5-1: Omega RDXL4SD 4-channel datalogger thermometer.

Figure 5-2: The first design of the semi-adiabatic device.

The second design of the semi-adiabatic device consists of a 20-in (50.8-cm) cube concrete specimen cast inside and covered by a double plastic wrap, three layers of 2-in (5.08-cm) thick extruded polystyrene (XPS) insulation, and 0.688-in (1.748-cm) thick plywood. The selected XPS insulation was Type-IV in the ASTM C578 classification. There were also 2-in by 4-in (5.08-cm by 10.16-cm) wooden belt supporting the system from the sides at the middle of the plywood cover. The device was separated from the ground with three 2-in by 4-in (5.08-cm by 10.16-cm)

placed under the system vertically and with 2-in (5.08-cm) thick XPS insulation in between to prevent heat loss to the ground. The plastic wraps were sealed with duct tape. The casting was made with three K-type thermocouples fixed at three locations: (1) center of specimen, (2) center of a side face, and (3) a top corner using metal frames and zip ties. XPS insulation was put together with construction adhesive to maintain stability and to have no air gaps between layers. Temperature time histories were recorded with an Omega RDXL4SD 4-Channel datalogger thermometer. The ambient temperature was measured with another K-type thermocouple connected to the digital thermometer. The temperatures of the four locations mentioned above were recorded for 14-days. The inside of the device and the schematic drawing from a vertical viewpoint is shown in Figure 5-3 and Figure 5-4, respectively.

Figure 5-3: The second semi-adiabatic device with the lid opened in the first two pictures and one side opened in the right picture.

Figure 5-4: Insulated mold for 20-in (50.8-cm) cube specimen.

5.2. Calculations

The analysis of the results from semi-adiabatic tests can be done with the following equations provided by Ng et al. (P. L. Ng 2008). $T_V - T_P$ calculated by Equation 5-1 is the difference between volumetric mean and placing temperatures,

$$T_V - T_P = T_G - \frac{H_L}{V\rho c}, \quad (5-1)$$

where V is the specimen volume, T_G calculated by Equation 5-2 is the adiabatic temperature rise, H_L calculated by Equation 5-4 with integrating Equation 5-3 and substituting it in Equation 5-1 is the heat loss, ρ is the mass density, and c is the specific heat capacity.

$$T_G = (T_V - T_P) + \frac{H_L}{V\rho c} \quad (5-2)$$

∂H_L calculated by Equation 5-3 is the heat loss rate determined by applying Fourier Law (Holman 2010),

$$\frac{\partial H_L}{\partial t} = k(T_S - T_A), \quad (5-3)$$

where k is the overall thermal conductivity of the insulating materials, T_S is the surface mean temperature of the specimen, T_A is the ambient temperature, and t is the time passed after the casting.

$$H_L = k \int_0^t (T_S - T_A) dt \quad (5-4)$$

T_G can be Equation 5-5 by substituting Equation 5-4 into Equation 5-2 and denominating $\frac{k}{V\rho c}$ into λ ,

$$T_G = (T_V - T_P) + \lambda \int_0^t (T_S - T_A) dt, \quad \lambda = \frac{k}{V\rho c}, \quad (5-5)$$

where λ calculated by Equation 5-6 is the heat loss characteristic (1/sec) because after very long-time, heat generation stops, and adiabatic temperature rise becomes stable so $\frac{\partial T_G}{\partial t}$ considered zero.

$$\frac{\partial T_G}{\partial t} = \frac{\partial T_V}{\partial t} + \lambda(T_S - T_A), \quad \lambda = \frac{\frac{\partial T_V}{\partial t}}{(T_S - T_A)} \quad (5-6)$$

Volumetric mean temperature and surface temperature (T_S) calculated by Equation 5-7,

$$T_V = \frac{1}{V} \int_V T(x, y, z) dV, \quad T_S = \frac{1}{A} \int_A T(x, y, z) dA, \quad (5-7)$$

where A is the concrete surface area, (x, y, z) is the spatial coordinates. After integrating Equation 5-5, T_G becomes Equation 5-8,

$$T_G = (T_V - T_P) + \lambda(\sum_{i=1}^n (T_S - T_A)_i \Delta t_i), \quad (5-8)$$

where, Δt_i is the time interval at time step i , and n is the number of time steps. The temperature-time histories of the 10-in (25.4-cm) cylinder specimen cast from 50% slag 1 are shown in Figure 5-5.

Figure 5-5: Temperature-time history of 10-in (25.4-cm) cylinder specimen cast from 50% slag 1.

The casting of 30% fly ash 2 in the second design of the semi-adiabatic device was made with three thermocouples inserted at locations center of specimen, the center of a side face, and a top corner to see if there are much difference or not. Test results show that the maximum difference happens between specimen center and corner, and the value was 3.8 °F (6.84 °C). Hence, it was a neglectable value; only the center (1-point) was used to calculate the heat loss compensation. Temperature-time histories of the 20-in (50.8-cm) cube specimen cast from 30% fly ash 2 are shown in Figure 5-6.

Figure 5-6: Temperature-time history of 20-in (50.8-cm) cube specimen cast from 30% fly ash 2.

The adiabatic temperature rise of concrete mixes was also measured using an adiabatic calorimeter. The adiabatic calorimeter (Lin and Chen 2015) was developed by using the concept described by (Gibbon, Ballim, and Grieve 1997), which limits the heat exchange between the concrete and the environment to determine the ATR of the specimen. The ATR measurements obtained from the adiabatic calorimeter and heat loss compensations calculated from the semi-adiabatic device are shown in Figure 5-7 and Figure 5-8. The average value of λ calculated within the time period of 120 to 144 hours after casting in the calculation of 50% slag 1 and 30% fly ash 2 heat loss compensation shown in Table 5-1. The heat loss characteristic can be calculated by Equation 5-9. The heat loss characteristic ranges and effecting values taken into consideration of both mixes shown in Table 5-2. As shown in Figure 5-7, the heat loss compensated ATR is much less than the ATR obtained from the calorimetry. Therefore, the first semi-adiabatic design using the 10-in (25.4-cm) cylindrical specimen is not acceptable. However, Figure 5-8 shows that both heat loss compensated ATR and the calorimetry compare well. Therefore, the second semi-adiabatic calorimetry design can be an acceptable method to measure the ATR of a concrete mix.

Table 5-1: Comparison of average λ 's calculated with literature recommendation.

Design No	Average λ (hr ⁻¹)	Max λ (hr ⁻¹) from Ng et al. (P. L. Ng 2008)
1 st	0.012481711	$> 7.848 \times 10^{-3}$
2 nd	0.004416667	$< 7.848 \times 10^{-3}$

$$\lambda = \left(\frac{T_{V_{n+1}} - T_{V_n}}{t_{n+1} - t_n} \right) / (T_{V_n} - T_{A_n}) \quad (5-9)$$

Table 5-2: The heat loss characteristic ranges and effecting values taken into consideration of both mixes.

Design No	Max λ	Min λ	$T_{V_{max}}$		$T_{V_{min}}$		$T_{A_{max}}$		$T_{A_{min}}$		t_{max}	t_{min}
	(h ⁻¹)		(°F)	(°C)	(°F)	(°C)	(°F)	(°C)	(°F)	(°C)	(h)	
1 st	3.607214	0	78.8	26	77	25	76.4	24.7	71.9	22.2	144	120
2 nd	0.042431	-0.035517	108.9	42.7	101.8	38.8	79.7	26.5	67.4	19.7	193	145

Figure 5-7: Comparison of the semi-adiabatic device and ATR calorimetry of 50% slag 1.

Figure 5-8: Comparison of the semi-adiabatic device and ATR calorimetry of 30% fly ash 2.

5.3. Conclusions

The second semi-adiabatic design compares better with the ATR calorimetry. The leading causes of the failure of the first semi-adiabatic design may be caused by the relatively small specimen size [10-in (25.4-cm) diameter cylinder] and using fragmental XPS with significant air gaps. Its heat loss characteristic value failed to stay below the recommended value. However, the second semi-adiabatic design's heat loss characteristic value was below the recommended value, possibly due to the larger specimen size [20-in (50.8-cm) cube] and more intact XPS leaving fewer air gaps.

Chapter 6 : Measurement of Thermal Conductivity

6.1. Thermal conductivity of concrete materials

After 28 days of curing, those two 6-inch (15.24-cm diameter) by 12-inch (30.48-cm height) cylinders that were used to measure the static elastic modulus were used to measure the thermal conductivity for either 50% slag or 30% fly ash mix. The cylinders were first drilled using a 3/8-in (0.9525-cm) drill bit with a height of 6-in (15.24-cm) with a hammer drill. Then, a thermocouple was placed in the hole and grouted by a cement paste mixture with a water-cement ratio of 0.5. The specimens were cured for one day to ensure the hardness of the cement paste then used to measure the temperature decays. CRD-C 36-73 (US Army Corps of Engineers 1973) was adopted to measure the thermal conductivities. Each specimen was placed in a hot water bath with a heating unit to reach 180 °F (82.2 °C). Then, each specimen was placed on top of two pieces of plywood and submerged in a cold-water bath. The temperature of the water was kept constant throughout each experiment by circulating the water. The temperature of the water was measured using a thermocouple placed 2-in (5.08-cm) from the side of the specimen. The concrete and water temperatures were recorded at an interval of 5-seconds until the concrete temperature reached the water temperature. Each test was repeated two times. Then the time that the temperature difference dropped from 60 °F (33.33 °C) to 20 °F (11.11 °C) was used to calculate the thermal conductivities using the following equations,

$$\alpha = \frac{M}{(t_2 - t_1)}, \quad \alpha = \frac{K}{\rho C_p}, \quad (6-1)$$

$$M = \frac{60 \ln\left(\frac{T_1}{T_2}\right)}{\left(\frac{5.783}{r^2} + \frac{\pi^2}{l^2}\right)}, \quad (6-2)$$

where α is thermal diffusivity (ft^2/h or m^2/h), M (ft^2 or m^2) is a factor depending on the size and shape of the specimen, t_1 and t_2 are times at which the center of the specimen reaches the specified temperature differences (min), T_1 and T_2 are temperature differences ($^{\circ}\text{F}$ or $^{\circ}\text{C}$) at times t_1 and t_2 , r is the radius of cylinder (ft or m), l is the length of cylinder (ft or m), K is thermal conductivity ($\text{BTU}/\text{h}/\text{ft}/^{\circ}\text{F}$ or $\text{W}/\text{m}/\text{K}$), ρ is weight density of the concrete (lb/ft^3 or kg/m^3), and C_p is specific heat capacity ($\text{BTU}/^{\circ}\text{F}/\text{lb}$ or $\text{J}/\text{kg}/\text{K}$). Based on the dimension of the cylinder and the $T_1 = 60^{\circ}\text{F}$ (33.33°C), $T_2 = 20^{\circ}\text{F}$ (11.11°C), the M factor can be calculated to be 0.643733198 (ft^2) [0.0598 (m^2)]. The specific heat was calculated using the following Equation 6-3 (Lin and Chen 2015; Van Breugel 1998),

$$C_p = \frac{(740\rho_c + 710\rho_s + 4184\rho_w + 840\rho_a)}{\rho}, \quad (6-3)$$

where ρ_c , ρ_s , ρ_w , and ρ_a are the weight densities (kg/m^3) of the cement, sand, water, and aggregate in the mix design, respectively. Two 50% slag 1 cylinder's temperature decays of center locations and ambient water are shown in Figure 6-1 and Figure 6-2. The comparison of the temperature differences between these two 50% slag 1 cylinders was shown in Figure 6-3, and they compare well. Two 30% fly ash 2 cylinders' temperature decays of center locations and water temperature were shown in Figure 6-4 and Figure 6-5. The comparison of the temperature differences between two 30% fly ash 2 cylinders was shown in Figure 6-6, and they show slight differences. The density and specific heat are shown in Table 6-1. The calculated thermal conductivities are shown in Table 6-2.

Figure 6-1: 50% slag 1 cylinder 1 temperature decay of center location and water temperature.

Figure 6-2: 50% slag 1 cylinder 2 temperature decay of center location and water temperature.

Figure 6-3: Comparison of the temperature differences between two 50% slag 1 cylinders.

Figure 6-4: 30% fly ash 2 cylinder 1 temperature decay of center location and ambient water.

Figure 6-5: 30% fly ash 2 cylinder 2 temperature decay of center location and ambient water.

Figure 6-6: Comparison of the temperature differences between two 30% fly ash 2 cylinders.

Table 6-1: Density and specific heat of 50% slag 1 and 30% fly ash 2 cylinders.

Batch	ρ		C_p	
	(lb/ft^3)	(kg/m^3)	$(BTU/^\circ F/lb)$	$(J/kg/K)$
50% slag 1	144.2	2309.9	0.23492	983.56
30% fly ash 2	144.5	2314.7	0.23868	999.31

Table 6-2: Thermal conductivities of 50% slag 1 and 30% fly ash 2 cylinders.

Batch	Cylinder 1				Cylinder 2			
	α		K		α		K	
	(ft^2/h)	(m^2/h)	$(BTU/h/ft/^\circ F)$	$(W/m/K)$	(ft^2/h)	(m^2/h)	$(BTU/h/ft/^\circ F)$	$(W/m/K)$
50% slag 1	0.03615	0.003358	1.23	2.13	0.03643	0.00338	1.24	2.14
30% fly ash 2	0.03622	0.003364	1.26	2.18	0.03346	0.00311	1.16	2.01

6.2. Thermal conductivity of reinforced concrete

Six 6-in (15.24-cm diameter) by 12-in (30.48-cm height) cylinders were prepared. The specimens were prepared using plain mortar, 4.24% steel rebars-mortar volumetric ratio (plain mortar plus six steel rebars), and 7.78% steel rebars-mortar volumetric ratio (plain mortar plus eleven steel rebars). The steel rebars were 18-in (45.72-cm long) and #4 [0.5-in (1.27-cm diameter)]. The area of #4 steel rebar cross-section was calculated as 0.2-in² (1.29-cm²). The cross-section areas of six and eleven steel rebars were calculated as 1.2-in² (7.74-cm²) and 2.2-in² (14.19-cm²), respectively. The cross-section area of the mortar specimen was calculated as 28.27- in² (182.39-cm²). The volumetric steel reinforcement-mortar ratios of the specimens containing six and eleven steel rebars were calculated by dividing the steel rebar cross-section areas with specimen cross-section areas resulting in 4.24% and 7.78%, respectively. The mortar mix design

is shown in Table 6-3. The locations of the steel rebars are shown in Figure 6-7. The unmolded specimens are shown in Figure 6-8.

Table 6-3: Mortar mix design.

Material	Weight density	
	(lb./yd ³)	(kg/m ³)
Cement	879.5	521.8
Sand	2550.6	1513.2
Water	432.8	256.8

Figure 6-7: Specimens layout of (a) 4.24% reinforcement ratio (mortar + 6 steel rebars) (b) 7.78% reinforcement ratio (mortar + 11 steel rebars) (dimensions are in inch).

Figure 6-8: 4.24% reinforcement ratio (mortar + 6 steel rebars) and 7.78% reinforcement ratio (mortar + 11 steel rebars).

Three K-type thermocouples were inserted in each cylinder. They were located at the center, 1.75-in (4.445-cm) from the center, and 2.75-in (6.985-cm) from the center. All the thermocouples were in the middle of the cylinders and fixed with multiple zip ties. The locations of the thermocouples are shown in Figure 6-9. In order to maintain the location and the spacing of the rebars, two pieces of plywood with holes at the locations of the steel rebar were placed on the top and the bottom of the plastic mold. The specimens were cured for 14-days with a curing temperature of 109 °F (42.7778 °C), which is more than 28-days of curing at a reference temperature of 73.4 °F (23 °C).

Figure 6-9: Location of thermocouples (dimensions are in inch).

6.3. Analysis of Results

Based on dimensions of the cylinder and assuming T_1 and T_2 are 60 °F (33.33 °C) and 20 °F (11.11 °C), M in Equation 6-2 can be calculated to be 0.64373 ft² (0.0598 m²). The specific heat of mortar can be calculated using Equation 6-4 (Lin and Chen 2015; Van Breugel 1998).

$$C_{p-mortar} = \frac{(740\rho_c + 710\rho_s + 4184\rho_w)}{\rho_M} \quad (6-4)$$

Therefore, the effective thermal conductivities can be obtained and are shown in Table 6-4. As shown in Table 6-4, the effective thermal conductivities of the cylinders with the steel reinforcement increased by 16.9% and 22.3%.

Finite element analysis (FEA) software (ABAQUS) was used to simulate the thermal behavior of the cylinders. For the cylinders with the steel rebar, the reinforcements were bonded with the mortar using a tie constraint (i.e., perfect bond). Hexahedral linear elements (DC3D8) with a size of 0.06-in (1.524-mm) were used for both the mortar and steel rebar. The FEA temperature distributions in the middle of the cylinders at 10 minutes are shown in Figure 6-10. The temperature-time histories were exported and compared with the experimental results in Figure 6-11. The convection boundary condition had a film coefficient (convection coefficient) of $800 \text{ BTU/h/ft}^2/\text{°F}$ ($4500 \text{ W/m}^2/\text{K}$) and was assumed to model the effect of the circulating water. As shown in Figure 6-10, the FEA results compare well with the experimental temperature measurement at different locations. The largest variation occurs with the sensors closest to the surface. The initial temperature and the water temperature were input individually for each case. Since the experimental initial temperature of the 1.75-in (4.445-cm) and 2.75-in (6.985-cm) thermocouples is not the same as the center temperature, due to the delay between taking the cylinders out of the hot water and submerging it in the cold-water bath, the FEA results were adjusted accordingly.

Table 6-4: Specific heat and thermal conductivity.

Material	Mass density		Specific heat		Thermal conductivity	
	(lb./ft ³)	(kg/m ³)	(BTU/°F/lb.)	(J/kg/K)	(BTU/h/ft/°F)	(W/m/K)
Steel	493	7897.1	0.12	502.4	30.64	53
Mortar	143	2291.8	0.264	1106	1.3	2.253
Mortar + 6 reinforcements	157.92	2529.7	0.245	1026	1.52	2.634
Mortar + 11 reinforcements	170.3	2727.9	0.232	970	1.59	2.756

Figure 6-10: Temperature distribution at 10 min for (a) mortar (b) 4.24% reinforcement ratio (mortar + 6 steel rebars) (c) 7.78% reinforcement ratio (mortar + 11 steel rebars) (temperatures are in °F).

(a)

(b)

(c)

Figure 6-11: Temperature time history comparisons of experiment and FEA at different locations: (a) mortar (b) 4.24% reinforcement ratio (mortar + 6 steel rebars) (c) 7.78% reinforcement ratio (mortar + 11 steel rebars).

6.4. Effective Thermal Conductivity

Another set of FEA models was used to develop an effective thermal conductivity equation. The effective thermal conductivity could be used to simulate the complicated steel reinforcement. In these analyses, the temperature-time history of the cylinders with different reinforcement ratios was calculated using the material properties mentioned in Table 6-4. The temperature decays were obtained for cylinders with 2.12%, 3.54%, 4.95%, 6.37% and 7.78% reinforcement ratios (3, 5, 7, 9 and 11 steel rebars) using the same spacing [1-in (2.54-cm) from the surface of the cylinder] for all the analyses. An initial temperature of 212 °F (100 °C) and a water temperature of 68 °F (20 °C) were chosen to calculate the effective thermal conductivities. The time period it took for the temperature difference between the center location of the cylinder and the water temperature to reach from 80 °F (44.44 °C) to 20 °F (11.11 °C) was used to calculate the effective thermal conductivities. An M of 0.8123 ft² (0.0755 m²) is calculated based on the mentioned temperature differences and the dimension of the cylinder using Equations 6-1 and 6-2. The linear relationships, of the natural logarithm of the temperature difference versus time shown in Figure 6-12 from FEA, indicate that the homogeneous solution in Equations 6-1 and 6-2 can still be a reasonable estimation in the calculation of the effective thermal conductivity with rebar. Using the FEA results, the normalized effective thermal conductivity (K_{eff}/K_0) versus the percentage of reinforcements (by volume) can be calculated, and the results are shown in Figure 6-12. The fitted trend line is shown in Equation 6-5. The value of K_0 was obtained to be 1.271 BTU/h/ft/°F (2.198 W/m/K) based on the temperature decay calculated by the FEA analysis which matches well with the measured value of 1.303 BTU/h/ft/°F (2.253 W/m/K). Using the proposed Equation 6-5, the effective thermal conductivities of 4.24% reinforcement ratio (mortar with six steel rebars) and 7.78% reinforcement ratio (11 steel rebars) are 1.472 and 1.645 BTU/h/ft/°F

(2.546 and 2.846 W/m/K), respectively. The calculated values are 5.3% and 2.9% different than the experimental values calculated using Equations 6-1 and 6-2, as shown in Table 6-4. The comparison of the calculated thermal conductivities and the measured data is also displayed in Figure 6-12.

$$\frac{K_{eff}}{K_0} = 0.0388 (\text{reinforcement ratio } (\%)) + 1 \quad (6-5)$$

Figure 6-12: (a) Natural logarithm of temperature difference vs. time, (b) normalized effective thermal conductivity vs. volumetric reinforcement ratio.

6.5. Conclusions

In this study, the thermal conductivity of mortar cylinders with 4.24% and 7.78% reinforcement ratios (embedded with 6 and 11 steel rebars) was experimentally measured. CRD-C 36-73 was used to calculate the thermal conductivities based on the temperature decay of the center. Besides the center temperature, two other locations, 1.75-in (4.445-cm) and 2.75-in (6.985-cm) from the center were monitored. FEA was conducted and successfully predicted the experimental measurements. The experimental measurements showed a thermal conductivity increase of 22.9% and 26.5% with 4.24% and 7.78% reinforcement ratios (by embedding 6 and 11

steel rebars in the cylinders). Then, the FEA was used to develop a linear effective thermal conductivity equation that can be used in the thermal analysis of reinforced concrete elements without complicated calculations or modeling.

Chapter 7 : Conclusion and Future Study

7.1. Conclusion

In this study, 50% slag 1 and 30% fly ash 1 replaced mix designs' shrinkage, and compressive creep test results were compared with the empirical models of ACI 209 (ACI Committee 209 2008), B3 (Bažant and Baweja 2000), CEB MC 90, CEB MC 90-99, and GL2000 (Gardner and Lockman 2001). 50% slag 1 and 30% fly ash 2 replaced mix designs' ATR were investigated, and semi-adiabatic calorimetry was designed. Steel reinforcements' effect on thermal conductivity was investigated, and a linear effective thermal conductivity equation was proposed.

50% slag 1 shrinkage test results appeared to fall within the range of the predictions of GL2000 and CEB MC 90-99 models (Gardner and Lockman 2001). 50% slag 1 compressive creep test results appeared to fall within the range of the predictions of the B3 model (Bažant and Baweja 2000). 30% fly ash 1 shrinkage test results appeared to fall within the range of predictions of the ACI 209 method (ACI Committee 209 2008); however, compressive creep test results of 30% fly ash 1 failed to appear to fall within the range of any prediction, possibly due to first-day loading.

The second design of the semi-adiabatic calorimetry proved that heat loss compensation methods could be applicable. More intact XPS material and less air gap in the design were crucial to match ATR calorimetry results, as well as, increasing the specimen size played a significant role in the back-calculation accuracy. As it might be seen in Chapter 6, reinforcements increased thermal conductivity of concretes significantly, such as 22.9% and 26.5% for insertion of 4.24%

reinforcement ratio (6 steel rebars) and 7.78% reinforcement ratio (11 steel rebars), respectively, and the proposed equation might reduce the need for complicated experiments or modeling.

In this thesis, only two mix designs, 50% slag, and 30% fly ash was investigated to determine the applicability of Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete (ACI Committee 209 2008), Model B3 (Bažant and Baweja 2000), CEB MC 90, CEB MC90-99, and GL2000 (Gardner and Lockman 2001) to predict drying shrinkage and compressive creep properties. Only a semi-adiabatic calorimetry result of 30% fly ash 2 mix was converged to ATR calorimetry results with heat loss compensation calculations. Temperature [59.3 °F (15.2 °C) ~ 82.4 °F (28 °C)] and humidity (43% ~ 66%) of the laboratory were the limitations of the study.

7.2. Future Study

The study was limited with only two different mix designs one for 50% slag and one for 30% fly ash replacement, and the comparison with their results to five shrinkage and four creep prediction models based on the ACI 209.2R-08 (2008). Investigating different slag or fly ash ratios in mix designs and comparing to those prediction methods in the future would be recommended. Slag and fly ash affecting ATR should be investigated with different mix designs; additionally, new semi-adiabatic designs using more quality insulation and smaller specimen size should be investigated.

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